Main Dam Preliminary Design Report

Prepared for

Minto Explorations Ltd.

Prepared by

SRK Consulting (Canada) Inc.
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July 2014

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1 Introduction

1.1 Background

The Minto Mine, owned and operated by Minto Explorations Ltd. (a wholly-owned subsidiary of Capstone Mining Corporation) is located 240 km north of Whitehorse in central Yukon, Canada as illustrated on Drawing SRK-MD-02. Mine development started in 2007 and at that time, tailings deposition consisted of filtered tailings deposited onto a dry-stack facility. The mine subsequently expanded and is currently operating under their Phase IV mine plan. When the Phase IV water licence was received, tailings deposition transitioned to thickened (50 to 60% solids by weight) hydraulic placement of tailings directly into the Main Pit (Drawing SRK-MD-03).

During the next planned phases of mining (Minto Phases V/VI), additional storage capacity will be needed for storage of tailings, water (operational and surge capacity) and waste rock with a neutralization potential: acid potential ratio value of less than 3 (NP:AP < 3). Since the total volume of tailings, water and NP:AP < 3 waste rock is expected to exceed the natural capacity of the Main Pit, a comprehensive waste rock and tailings storage alternatives assessment, including an evaluation of candidate tailings disposal sites, was carried out in 2012 (SRK 2013a). This assessment concluded that the preferred tailings disposal strategy is thickened (50 to 60% solids by weight) hydraulically placed in-pit storage within the Main Pit, Area 2 Pit and Ridgetop North Pit. However, since there is an insufficient natural containment capacity to store the planned waste volumes, operational volumes, and required surge capacity for site surface runoff, engineered interventions are required.

After consideration of options for increasing the in-pit storage capacity, Minto identified construction of a containment dam to increase the storage capacity of the Main Pit as the preferred solution. Following this decision, and in the absence of site specific field work or design analyses, SRK completed a conceptual level design for a tailings storage dam immediately adjacent to the Main Pit (SRK 2013b).

In the second half of 2013, in order to advance the design of the Main Dam, a dam breach assessment, complete with a dam hazard classification (SRK 2013c) and site specific field investigation (SRK 2014a) were undertaken based on the conceptual design. The Main Dam conceptual design, the dam classification based on the breach assessment and the findings from the site investigation were presented to the Yukon Environmental and Socio-economic Assessment Board (YESAB), the Selkirk First Nation (SFN) and Dr. Richard Dawson of Norwest Corporation, the independent third party reviewer appointed by Minto and SFN in January 2014.

By February 2014, SRK had commenced the preliminary design of the Main Dam, taking into account issues and design requirements identified during the development of the conceptual design and comments from YESAB, SFN and the independent third party reviewer. This report describes the preliminary design of the Main Dam.

1.2 Scope of Work

SRK Consulting (Canada) Inc. was contracted by Minto to carry out the preliminary design of the Main Dam. The design and related information provided in this report has been prepared in accordance with YESAB's "*Dam Guide: Design Expectation and Required Information*" (Yukon Environment 2012) in order to fulfil regulatory assessment requirements. In accordance with best practice, the design has been carried out in accordance with the Canadian Dam Safety Guidelines, as documented by the Canadian Dam Association (CDA 2007, Revised 2013).

The design as presented, is considered preliminary engineering. In the context of this report, this refers to a design that is sufficiently defined to ensure that there are no fatal flaws, and that a cost estimate to an accuracy of ±30% can be developed.

1.3 Report Layout

Section 2 of this report provides an overview of the general site conditions that are relevant to the preliminary Main Dam design. Design criteria and assumptions are presented in Section 3. Section 4 summarizes the preliminary dam design as well as supporting geotechnical and hydrotechnical analyses. Section 5 highlights the construction approach for the preliminary Main Dam design. A discussion of the dam operation, maintenance, and monitoring is presented in Section 6 while Section 7 provides preliminary closure assumptions associated with the Main Dam.

2 Site Characterization and Background Data

The general conditions at the Minto Mine site are discussed in a number of public documents related to permitting of the development, operation, expansion and closure of the mine. As a result, this section of the report focuses primarily on those topics which are of direct relevance to the preliminary design of the Main Dam. This includes the following:

- Bedrock characterization and local faults;
- Overburden characterization and permafrost;
- Hydrogeology;
- History of ground instability; and
- Tailings properties.

A discussion of each of these topics, and their relevance to the preliminary design of the Main Dam, is provided below.

2.1 Bedrock

2.1.1 Interpreted Bedrock Surface

Appendix A contains information related to the shape of the bedrock surface at the Minto Mine site. In particular, Figure 1 in Appendix A shows a contour plan of the bedrock surface that has been prepared following a review of all available information, including but not limited to the results of various resource and geotechnical drilling campaigns as well as bedrock outcrop mapping. The bedrock surface shows a deep erosional feature, or paleochannel, that is “fed” by two tributaries, one of which extends beneath the south side of the Main Pit (and corresponds to the south wall overburden failure that is discussed in Section 2.4) and the other which extends below the south side of the dry stack tailings storage facility (DSTSF). The two tributary channels coalesce beneath the DSTSF and then continue as a single paleochannel down the Minto Creek valley.

2.1.2 Weathered Bedrock

The upper 5 to 15 m of bedrock is generally moderately to heavily weathered. Characterized by higher fracture frequencies, consistently low intact rock strength and zones of heavy alteration and oxidation, the weathered bedrock (or residuum) is of significantly lower quality than the unweathered (or fresh) rock below. Mean rock quality designation (RQD) and rock mass rating (RMR) values of 73 and 54, respectively, have been measured for the weathered rock, which, based on the Bieniawski RMR classification system, translates to fair to good quality rock mass (SRK 2007). Drilling and blasting of the weathered rock is not required when pit development is carried out.

2.1.3 Fresh Bedrock

The fresh or unweathered bedrock at the Minto Mine is a much more competent rock mass than the weathered rock, possessing lower fracture frequencies and significantly higher intact rock strengths. The fresh rock encountered is relatively massive and shows fewer signs of alteration and weathering when compared to the weathered rock. The fresh bedrock also demonstrates higher overall RMR values with a mean value of 64. The fresh rock would be described as a fair to good quality rock mass based on the Bieniawski RMR system. RQD values for the fresh rock were also higher than the weathered rock with a mean value of 85%.

Several zones of foliated granodiorite were encountered in the fresh rock but exhibited similar intact rock strengths and rock mass properties as did samples of non-foliated granodiorite (SRK 2007).

2.1.4 Faults

Three significant fault structures intercept the Main Pit (Figure 1). The DEF fault is at the northern end of the Main Pit and strikes nearly east-west and dips north-northeast. This fault has been characterized with high gouge content that, based on water pressures measured across the fault, acts as a barrier to groundwater flow (SRK 2010).

The Minto Creek fault (or Fault 2) strikes almost perfectly east-west where it bisects the Main Pit and is believed to be north dipping (SRK 2008). Horizontal and lateral displacements have been observed, but are not clearly defined. Although surface expression of this fault exists for horizontal and vertical displacement, the magnitude of movement is not well characterized.

The third fault lies to the south of the previous two and strikes south-east to north-west, and possibly intersects the other two faults towards the western edge of the pit. This fault is believed to dip north-northeast.

2.2 Overburden

2.2.1 General

Overburden thickness across the site is closely correlated with geomorphological features. Near topographic highs, there is little to no overburden, with increasing overburden thickness down valley slopes and generally the thickest deposits in valley bottoms (with the exception of the paleochannel described in Section 2.1.1). Overburden thicknesses range from very shallow (less than 5 m) close to bedrock outcrop zones to about 100 m along the paleochannel, and 30 to 50 m in the Minto Creek valley adjacent to the paleochannel. An overburden isopach is provided in Figure 5 of Appendix A, based on the bedrock surface described in Section 2.1.1, and the January 2014 ground surface topographical survey.

Generally, ridge tops are dominated by sandy, residual soils grading to weathered bedrock. Fine weathered products have been washed downslope, which means within the valleys, finer sandy silts and clays are found. Layers of sandy gravel with cobbles are occasionally found within

the overburden unit; however, the overburden appears sufficiently heterogeneous that continuous high conductivity zones are not expected (with the possible exception of the paleochannel).

More information on the overburden characteristics in the vicinity of the Main Dam is provided in Section 2.4.

2.2.2 Permafrost

Minto mine is located in a discontinuous permafrost region. In general, the south facing slopes are unfrozen, while north facing slopes contain permafrost. Where permafrost is present it is generally warm, with temperatures close to -0.5^{o}C. The active layer is variable ranging from less than 1 m to about 3 m thick. The general distribution of permafrost on the site is illustrated on Figure 1.

Ice contents within the overburden range from non-visible to 30% excess ice, although ice lenses are common and massive ice up to 4 m thick occurs, particularly near the bedrock surface within the paleochannel.

Total depth of permafrost is variable, but, in certain locations is known to extend down to the base of the paleochannel.

2.3 Hydrogeology

At Minto, groundwater flow direction is dominated by topography, with groundwater flowing from the upland areas towards Minto Creek. Within the Minto Creek valley, the flow follows the trend of the valley, which affords ample opportunities to monitor groundwater flow and quality. This means that it will be possible to track the possible effects of seepage from the Main Dam whether it emerges as toe seepage, passes through fill, or reports to deeper groundwater.

2.4 History of Ground Instability

The challenging overburden conditions as described in the preceding sections have, in part, contributed to significant ground instability at three locations on the Minto Mine site:

- Creep displacement of the Dry Stack Tailings Storage Facility (DSTSF);
- The Main Pit south wall failure; and
- Foundation movements at the Southwest Waste Dump.

The DSTSF is constructed over the paleochannel. Detailed field characterization has confirmed that movement can be attributed to a shear surface occurring within a deep (26 to 64 mbgs), ice rich clay layer located approximately 7 m from the bedrock contact. The geometry of the slip surface is well defined beneath the facility, but the exit of the surface has not been located. It is believed that the onset or increase in shearing beneath the facility was most likely a result of warming of the ice-rich clay layer caused by placement of the tailings and possibly the ore stockpiles to the west. By raising the temperature in the ice-rich clay layer, the resistance to movement has been lowered likely more so than the actual mass of the added fill has increased

the driving force (SRK 2012a). A buttress, referred to as the Mill Valley Fill Extension, was constructed adjacent to the DSTSF. Since the installation of the buttress, movement of the DSTSF has slowed down but not ceased. An expansion of the buttress is proposed; the preliminary design of which is documented in SRK (2014b).

Geotechnical drilling, testing, and instrumentation indicated that displacements associated with the Main Pit's south wall failure and foundation movement of the Southwest Waste Dump were linked and are associated with an ice-rich, plastic clay layer(s) generally within the lower portion of the paleochannel materials (i.e., essentially the same foundation conditions as underneath the DSTSF). In contrast with the displacements associated with the DSTSF, displacements of the Main Pit south wall and the Southwest Waste Dump foundation are believed to have been initiated primarily by excavation of the Main Pit, which simply created a void for the ice-rich overburden soils to move towards due to gravitational forces. The buttress, which is being constructed, has slowed down the movement and is expected to completely stop movement when it has reached its design height (SRK 2012b).

The overburden slide block that comprised the Main Pit south wall failure daylighted in the open pit and is not kinematically connected to overburden on the east side of the Main Pit, i.e. where the south portion of the Main Dam will overlie overburden.

2.5 Tailings

2.5.1 Physical Properties

At least three physical tailings characterization campaigns have been carried out specifically focused towards understanding the tailings performance as it relates to the design of tailings deposition at the site (EBA 1997, Klohn Crippen Berger 2013, SRK 2012a and 2012b). This includes indicator properties (grain size distribution and Atterberg Limits), consolidation tests, settling tests, specific gravity, and strength and saturated hydraulic conductivity tests. In addition, there has also been characterization of the in-situ dry-stack tailings specifically to evaluate strength characteristics of the material (SRK 2012a and 2012b). The tailings can be classified as sand and silt with a trace of low plasticity clay.

Although the tailings deposition changed from dry-stack placement to hydraulically placed tailings into the Main Pit over the period of operations to date, the actual tailings physical properties have not changed and therefore the available database of information remains valid.

2.5.2 Geochemical Properties

Tailings geochemistry data suggests that metal leaching and acid generation potential are low; therefore, unsaturated storage of tailings is acceptable, but should be minimized where possible to minimize low-level leaching of metals (SRK 2013d).

3 Design Criteria

3.1 Selection of Design Cross Section

A comprehensive dam alternative assessment was completed (Appendix B) to determine the optimal design cross section to carry forward to preliminary engineering. The selection process included compilation of a list of possible dam alternatives, evaluation of each alternative and subsequently selection of the preferred option.

The initial screening was based on a comparison of advantages and disadvantages of each dam concept and assigning a preference rating of low, medium or high to each. Options assigned the low preference rating were considered not suitable for further consideration. Dam alternatives assigned a medium or high preference ratings were evaluated further. These included geosynthetic liner and fine grained core type dams, as well as combinations of these features.

The liner is effective in mitigating against seepage but its success requires a high level of confidence in the liner integrity. Likewise, seepage through a fine grained core is dependent on the permeability and integrity of the core. While both are likely acceptable design methods, a combination liner and fine grained core dam was selected as the preferred option as it provides redundancy.

The preferred dam cross section is illustrated in Drawing SRK-MD-06. Seepage containment will be provided by means of a liner upstream of a fine grained core clad in run of mine rock. Appropriate filter and bedding layers will ensure the liner and core is adequately protected. Due to potential deformations calculated through thaw settlement (Appendix C) and consolidation (Appendix D), an LLDPE liner has been selected as the preferred liner as it has a higher strain capacity than other geosynthetics.

3.2 Storage Requirements

During the conceptual design, it was determined that there was a need for 1.7 Mm^3 additional mine rock and tailings storage over and above the capacity provided by the mined-out pits within the Minto Creek catchment (SRK 2013e). It was determined that a dam on the east side of the Main Pit, with storage capacity up to elevation 804 m (at full supply level (FSL)), was the best option for gaining the necessary additional capacity. This design increased the Main Pit storage capacity by nearly 2.2 Mm^3. Figure 2 presents the staged storage of the Main Pit provided by Minto in 2013.

In the conceptual design, the issue of ice entrainment during beach deposition of slurry tailings under winter conditions was raised, along with the attendant risk of loss of tailings storage volume. At the time it was decided that no allowance for ice entrainment would be designed for, and the rationale was that appropriate deposition practices could be adopted. This was revisited during the preliminary engineering phase to ensure the design tailings storage capacity would be achieved even in the event that substantial ice entrainment would occur. Experience elsewhere with operation of tailings beaches in northern environments has shown that up to 20% ice entrainment (by volume) can occur (BGC 2003). However, tailings deposited subaqueously are

not subject to ice entrainment. Based on production records, it is estimated that 2 Mt tailings have been deposited subaqueously into the Main Pit as of March 2014, which would account for between 1.5 Mm^3 to 1.8 Mm^3 of the original storage capacity (depending on the achieved density of the placed tailings which range between 1.1 and 1.3 dry t/m^3).

The dam design was subsequently modified to increase the FSL to 809 m. This resulted in an increase of approximately 0.9 Mm^3 storage capacity, or an overall allowance for ice entrainment of 13%. However, when discounting for the tailings already deposited as well as waste rock placement beneath the full supply level, which are not subject to ice entrainment, the actual available capacity of ice entrainment is about 25%. This represents about 25% which is more than adequate to allow for ice entrainment volumes. This level of conservatism was considered appropriate, at this level of design, since if the engineering analyses show that the larger dam will satisfy the stability-related design criteria, , later optimization (and possible lowering of the final design dam crest) will also allow the design criteria to be met.

3.3 Beach Slope

Tailings placement will result in beach development, which means the assumption of struck levels as discussed in Section 3.2 is not entirely accurate. No specific tailings physical characterization has been carried out to confirm the expected tailings beach slope angle at Minto; however, typical copper tailings deposited sub-aerially are known to have beach slope angles that range between 0.5 and 2% (Vick 1990). Considering the Main Pit layout that allows for beach development around most of the perimeter of the pit, storage capacity loss is not expected to be significant and, therefore, is not likely to affect the overall selection of dam height. This will, however, be confirmed during the final design stage.

3.4 Design Life

Minto Mine is currently expected to be in operation until about 2022, with subsequent closure taking about 3 years. Therefore, the Main Dam operational life (i.e. the period prior to full closure) is about 10 years. To conservatively account for possible unforeseen conditions, the operational design life was extended to 20 years. Furthermore, since this dam will contain both tailings solids and water during its operational design life, it has been assumed that the Main Dam must be able to perform as a full water retaining structure for its design life.

At closure, the Main Dam will only contain tailings solids; however, the actual structure will remain in perpetuity.

3.5 Dam Hazard Classification

The design, construction, operation and monitoring of dams in Canada have to be completed in accordance with appropriate territorial, provincial, and federal regulations and industry best practices. One of the foremost guidance documents in this regard is the Canadian Dam Safety Guidelines (CDA 2007, Revised in 2013) published by the Canadian Dam Association (CDA). The CDA recognizes that tailings dams are different than water dams and, as a result, is in the process of developing a technical bulletin to provide guidance on how the CDA guidelines may be

tailored to tailings dams; however, until this technical bulletin is published, all mining dams should be evaluated in accordance with the guidelines currently available.

A key component of the guidelines is classifying the dam(s) in question into hazard categories (dam class) that establish appropriate geotechnical and hydro-technical design criteria. Table 1 is a reproduction of the recommended dam classifications as presented in the CDA guidelines. This classification is based on the Incremental Consequence of a dam failure (as opposed to Total Consequences). The Incremental Consequences of failure are defined as the total damage from an event with dam failure, less the damage that would have resulted from the same event (e.g., a large earthquake or a large flood event) had the dam not failed.

Table 1: Dam Classifications as per CDA (2007, Revised 2013)

Dam Class	Population at Risk[1]	Incremental losses		
		Loss of Life[2]	Environmental and Cultural Values	Infrastructure and Economics
Low	None	0	Minimal short-term loss No long-term loss	Low economic losses; area contains limited infrastructure or services
Significant	Temporary only	Unspecified	No significant loss or deterioration of fish or wildlife habitat Loss of marginal habitat only Restoration or compensation in kind highly possible	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	Significant loss or deterioration of *important* fish or wildlife habitat Restoration or compensation in kind highly possible	High economic losses affection infrastructure, public transportation, and commercial facilities
Very high	Permanent	100 or fewer	Significant loss or deterioration of *critical* fish or wildlife habitat Restoration or compensation in kind possible but impractical	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of *critical* fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses affecting critical infrastructure or services (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)

[1] Definitions for population at risk:

None—There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.

Temporary—People are only temporarily in the dam-breach inundation zone (e.g., seasonal cottage use, passing through on transportation routes, participating in recreational activities).

Permanent—The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is carried out).

[2] Implications for loss of life:

Unspecified – The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season.

Determination of the appropriate hazard rating is often subjective and is dependent on site-specific circumstances that may require an agreement between the proponent, regulators, and

stakeholders. During the dam classification process, each of the four hazard rating components, in Table 1 (i.e., population at risk, loss of life, environmental and cultural values, and infrastructure and economics) is considered individually and the overall dam hazard rating is defined by the component with the highest (i.e., most severe) rating. It is important to note that the hazard rating refers to the downstream consequences in the inundation zone of a dam breach.

A preliminary dam hazard classification for the Main Dam has been carried out by SRK after completion of a dam breach analysis and confirms that the appropriate hazard category for the dam is VERY HIGH (SRK 2013c).

3.6 Design Earthquake

Based on the preliminary dam hazard classification of VERY HIGH assigned to the Main Dam, the Annual Exceedance Probability earthquake design ground motion will be the 1/5,000 year event (CDA 2007, Revised 2013).

3.7 Design Precipitation Event

For hydrotechnical design, the CDA guidelines indicate that the design precipitation event for a VERY HIGH dam classification should be equivalent of the 1/5,000 year annual exceedance probability for storm events. However, it also indicates that extrapolation of storm events beyond 1/1,000 year should be discouraged since the outcome would only provide an order of magnitude estimate. Instead, storm events are extrapolated between the 1/1,000 year storm event and the probable maximum flood PMF. For design purposes, it was decided to move forward with the design precipitation event that triggers the PMF. This is the highest standard outlined in the CDA design guidelines.

3.8 Inflow Design Flood

The detailed assessment of the probable maximum flood (PMF) associated with the Main Dam is described in Appendix E. The flood was adjusted based on storm duration for the catchment size, and confirmed that the inflow design flood (IDF) is the 1 hour PMF with a magnitude of 55 m^3/s.

3.9 Design Freeboard

The IDF depth, based on the spillway design is 1 m (Appendix F), and the maximum wind induced wind setup and wave run-up is also 1 m (Appendix G). Freeboard is defined as the maximum of either normal or minimum freeboard.

The minimum freeboard for the Main Dam is defined as the vertical distance between the IDF (which must be equal to or higher than the top of the core) and the crest of the dam. This freeboard height is to ensure that the dam does not overtop due to wind set-up and wave run-up, and is therefore set at 1 m.

The normal freeboard for the Main Dam is defined as the vertical distance between the crest of the dam and the Full Supply Level (FSL). The normal freeboard must ensure that the top of the

core is at or above the maximum sustained wind induced wind set-up and wave run-up, even when the maximum expected core deformation has occurred. In addition the core must be sufficiently protected from damage due to frost or potentially damaging action. The maximum expected deformation over a small section of the dam is less than 1 m (see Appendix H); and therefore, a settlement allowance in the freeboard of 1 m has been allowed for. The core will be protected with a 1 m thick protective cover, bringing the normal freeboard for the Main Dam to 3 m. The normal freeboard therefore becomes the final freeboard design criteria. Table 2 summarize the key elements.

Table 2: Design Freeboard Elements

Component	Elevation (masl)
Full Supply Level (FSL)	809
Inflow Design Flood (IDF)	810
Top of Core	811
Top of Crest	812

4 Preliminary Design

4.1 General

The stage-capacity curve for the main dam preliminary design is presented in Figure 2. The current design FSL of 809 masl provides a storage capacity of 6.2 Mm^3.

Two cases were considered for dam design; a dam with FSL of 804 m, which provides the required increase in capacity and a dam with FSL of 809 m to provide the required capacity as well as some contingency for capacity reduction due to ice entrainment as presented in Section 3.2. The analyses presented further in Section 4 were completed based on dams with crest heights of 807 m and 812 m which correspond to FSL levels of 804 m and 809 m respectively. Although geotechnical and hydrotechnical analyses were carried forward for both cases, the preliminary design presented in this report is based on a FSL of 809 m, with 3 m freeboard, yielding a final dam crest of 812 m.

Drawing SRK-MD-03 provides an overview of the Main Pit area showing the locations of the dam and spillway in relation to the South Wall Buttress and Main Pit Dump. The Main Pit Tailings Management Facility (MPTMF) will be operated such that a tailings beach is developed along the upstream side of the dam at the east side of the facility, in order to minimize the potential for seepage to the east. Further details of the MPTMF are provided in the tailings management plan (Minto 2013a).

A permanent spillway is located south of the dam to direct water towards the Area 2 Pit in accordance with the site wide water management plan (Minto 2013b).

4.2 Dam Components

A typical section of the Main Dam is provided in Drawing SRK-MD-06. The dam has a crest width of 14.5 m, a 2.5H:1V upstream slope and a 3H:1V downstream slope. The maximum dam height is expected to be approximately 28 m with crest length of approximately 430 m based on an 812 m crest elevation.

The dam is a rockfill structure with an internal geomembrane liner and a fine-grained core system that is keyed into fine grained overburden and/or bedrock. The key-trench has a typical 10.5 m base width and variable depths up to approximately 10 m. The upstream face of the dam will include a 1 m thick rip-rap layer for erosion protection.

Dam seepage control is based on full redundancy though a combination of a geomembrane and a low permeability compacted core. The liner-fine grained core system consists of a 2.25 m thick (or 6 m horizontal width) layer of low hydraulic conductivity material that is overlain by a LLDPE liner. A bedding layer is to be placed over top of the liner followed by 2.5 m of waste rock. Two filter layers, each 0.5 m thick, are located downstream of the fine-grained core (Drawing SRK-MD-06)

The following sections describe the individual dam components in greater detail.

4.2.1 Core

The dam core is to consist of fine grained, low hydraulic conductivity material. Locally available overburden materials were evaluated to assess their suitability for use, and the results are detailed in Appendix I.

For the purposes of the preliminary design, the core material is assumed to originate from the Reclamation Overburden Dump (ROD). Samples collected from the ROD contain significant fines and its hydraulic conductivity was estimated to be approximately $2x10^{-7}$ m/s. A summary of the grain size distribution used in the design is provided in Table 3.

Table 3: Core Grain Size Distribution Used in Design

Diameter (% Passing)	Minimum Diameter (mm)	Maximum Diameter (mm)
D_{15}	0.0010	0.043
D_{85}	0.032	0.73
D_{100}	0.060	1.3

4.2.2 Geosynthetic Liner

The geosynthetic liner is to consist of a 60 mil low linear density polyethylene (LLDPE). This liner type is recommended based on its flexibility due to the potential for some differential settlement. The liner is also recommended to be textured both sides for overall stability and construction safety purposes.

4.2.3 Filter Zones

Two filter zones (interchangeably referred to as bedding layers), a fine filter and a coarse filter, are required between the core and run-of-mine rock fill zones to support the fine-grained core material and to prevent it from being eroded by seepage, as well as to allow water to drain and prevent build-up of pore water pressures. Requirements for these filters were assessed based on the available particle size distributions of the core materials described in Section 4.2.1. Details of the filter design are provided in Appendix I. A summary of the filter particle size requirements is provided in Table 4.

Table 4: Core and Filter Design Particle Sizes

Diameter (% Passing)	Minimum Diameter (mm)	Maximum Diameter (mm)
Fine Filter		
D_{15}	0.0080	0.18
D_{85}	0.10	4.0
D_{100}	0.16	8.4
Coarse Filter		
D_{15}	0.025	0.99
D_{85}	0.30	17
D_{100}	0.50	47

4.2.4 Shell

The outer shell of the dam will consist of select mine waste rock. Throughout this report, as well as the supporting Drawings and Appendixes the shell is referred to as rock fill or waste rock interchangeably.

4.2.5 Erosion Protection

Erosion protection for the dam and the spillway will be appropriately sized rip-rap sourced from mine waste rock.

4.3 Stability Analyses

4.3.1 Slope Stability

A geotechnical slope stability assessment was completed in accordance with the guidelines set by CDA (2007, Revised 2013). Material properties were selected based on a review of material properties at the Minto site (Appendix J), as well as site specific testing completed during the Minto Main Dam Phase 1 Field Investigation (SRK 2014a). Complete details of the stability assessment are presented in Appendix K. The analyses focused on both surface failures (skin failures) and global mechanisms (slip surfaces greater than 5 m depth). Static and pseudo-static stability analyses were completed in the upstream and downstream movement direction under various pore water conditions and dam geometries. The analyses indicate that the dam meets CDA requirements in all cases.

A portion of the dam foundation contains ice-rich clays similar to those encountered in the south wall of the Main Pit and beneath the Dry Stack Tailings Storage Facility. Deep seated down-valley analyses were completed to assess stability in the event that a shear zone is present in the warm permafrost clay. The analysis suggests that such down-valley movement of the Main Dam is extremely unlikely, and furthermore the difference in FOS between the current conditions with the Mill Valley Fill Buttress in place and after the Main Dam has been constructed is insignificant. It should be noted that as discussed in Section 2, the potential shear zone beneath the Main Dam is no longer connected to the shear zone in the Main Pit south wall, which further minimized the likelihood of this potential failure mechanism.

4.3.2 Deformation

A 2-dimensional finite element (FE) deformation analysis was completed to better assess the maximum expected movements and shear stresses in, under and immediately surrounding the dam (Appendix H).

Settlement of the foundation is expected as a result of consolidation, as well as from thaw. The largest magnitudes of deformation are expected beneath the southern portion of the dam where ice-rich fine grained overburden is present.

The deformation analysis was undertaken using the same primary section as was used for the thermal analysis (Appendix C) that is expected to represent the least favourable foundation

conditions as it has significant fine grained materials close to surface and contains the most excess ice found during the 2013 geotechnical drilling investigation (SRK 2014a).

The models were developed for static analysis for a base case scenario consisting of a fully developed tailings beach as well as a worst case scenario consisting of the Main Pit flooding and remaining at FSL for the life of the structure.

Deformations from thaw settlement (due to melting of excess ice in the foundation soils) were computed based on the results of the thermal analysis (as described in Section 4.5). Zero degree isotherms from the thermal models were imported into the deformation model and used to define the frozen and unfrozen zones such that thaw settlement could be incorporated into the deformation modeling.

Overall displacements for the base case where a fully developed tailings beach exists were found to be around 0.2 m. The worst case pit lake scenario indicated displacements less than 0.8 m. The analysis confirmed that during the operational period of the dam, displacements are sufficiently low that core and liner integrity will not be compromised.

4.3.3 Liquefaction Analysis

A discussion based liquefaction assessment was completed to investigate the effect on potential dam performance during earthquake loading and/or possible tailings liquefaction. In early stages of design, it was determined that the design should allow for a possible pit flooding as a contingency in the event a tailings beach is not developed.

The tailings gradation and current deposition strategy (low density hydraulic deposition) make the tailings susceptible to liquefaction; however, the case of liquefied tailings ponded against the dam face represents a more conservative case than ponded water. This is due to the fact that both materials (ponded water or liquefied tailings) are expected to have no shear strength, but the liquefied tailings would have a higher density and would be applying a greater resisting pressure to the upstream face than ponded water.

Instead of assessing the potential depth to which liquefaction may penetrate, it is assumed that tailings behind the dam liquefy and pseudo-static analyses were carried out as part of the stability analysis. The results of the stability analysis, as discussed in Section 4.3.1, indicate that dam performance during a seismic event meet the design criteria outlined by CDA event for the case of fully liquefied tailings. Details of this assessment are presented in Appendix L.

4.4 Thermal Analysis

Thermal analysis was performed to estimate the amount of thaw and related settlement that may occur beneath the foundation during operations. Complete details are provided in Appendix C.

The thermal model was calibrated for site specific conditions, using site ground temperature cable data. The material properties were estimated based on soil index properties. The results of the thermal model calibration, detailed in Appendix C, showed good agreement between the predicted and measured ground temperature profiles at the site.

The thermal analysis was carried out for the dam cross section that was found to contain the most ice during the 2013 Main Dam field investigation (SRK 2014a). The overburden at this location is 40 m thick and generally consists of frozen fine grained silts and clays and is considered to be the least favourable foundation condition along the dam alignment from a thaw settlement perspective.

Global warming was accounted for in all prediction models and was applied at a rate of +4.1°C per 100 years. This value is the 50 year (1958-2008) warming rate for the Yukon and Mackenzie Valley region published in the CSA technical guide “Infrastructure in Permafrost: A guideline for climate change adaption”.

The analyses considered a tailings beach scenario where tailings are placed up to the maximum storage capacity and a worst case scenario that assumed that the Main Pit fills with water up to the spillway elevation during the first freshet and remains there indefinitely. Additional analyses were also completed to evaluate the sensitivity of the models to changes in the foundation soil volumetric water contents, and the applied surface boundary condition.

Complete figures demonstrating the ground temperature distributions during the late summer in Years 0, 3, 8, 30 and 50, respectively following construction are presented in Appendix C, for both the tailings beach and pit lake scenarios.

The tailings beach scenario results show that during the 20 year operational design life, the active layer beneath the dam generally decreases due to the thermal insulation provided by the tailings and fill placement. At the downstream toe of the dam where there is no fill placement, the active layer was predicted to increase by 0.4 m during the first 20 years.

As the water in the pit lake scenario provides a year round heat source upstream of the dam, it results in a higher rate of thaw on the upstream half of the dam compared to the tailings beach scenario. At the upstream toe, the active layer is predicted to increase by approximately 20 m during the operational design life. At the dam centerline, the performance during operations is similar to the tailings beach scenario and where there is no significant change in the thaw depth during operations.

The thermal analyses were completed using many conservative assumptions that would increase the rate of thawing, and as a result, the actual thawing during operations is expected to be much less. These conservative assumptions include: lower bound soil moisture contents used for the foundation soils, omission of ice layers and the fine grained dam core in the thermal model, complete removal of the boulder laydown area, and immediate thermal loading on the upstream side of the dam with tailings at +15°C or water up to the spillway elevation.

4.5 Settlement Analysis

Settlement of the foundation is expected as a result of conventional consolidation, as well as thaw consolidation associated with ice lenses present beneath the southern portion of the dam where fine grained overburden is present. No significant settlement is expected over the northern portion of the dam alignment that is founded on bedrock and weathered bedrock.

Thaw and consolidation settlement were estimated based on the results of the thermal analysis described in Section 4.4. The ultimate thaw settlement of the dam in this area will be in the order of meters; however, the thermal modelling shows that the rate of settlement will be slow, and the ultimate settlement will require multiple decades to centuries. The precise time to ultimate settlement is difficult to predict due to future climate uncertainties.

Table 5 provides a summary of the expected settlement for the tailings beach and pit lake scenarios at the end of the operational design life as well as following complete melting of the permafrost foundation. Details of the settlement predictions are provided in the following sections.

Table 5: Summary of Settlement Results

Case	Consolidation Settlement (m)	Thaw Settlement (m)	Total Settlement (m)
Pit Lake - End of design life (20 years)	0.45	0.9	1.3
Tailings Beach End of design life (20 years)	0	0	0
Ultimate settlement (overburden completely thawed) [2]	1.5	5.1	6.6

Note(s):

(1) Settlement in the tailings beach and ultimate settlement cases are estimated at the dam centerline. Settlement in the pit lake case is estimated near the upstream toe where the settlement is expected to be the largest.

(2) Thawing of the overburden will take multiple decades to centuries. The preliminary thermal modelling was completed over a project 50 year duration, at which time the top 3 to 7 m (out of ~ 40m had thawed)

(3) The thaw settlement values are based on an 807 m crest elevation dam, results for an 812 m crest dam would likely be lower.

4.5.1 Thaw Settlement

Results from the thermal models were used to estimate the amount of thaw settlement that could occur as a result of the phase change from ice to water, and as a result of the flow of excess water out of the soil. The analysis estimated the thaw settlement at the dam centerline, and at the toe of the upstream and downstream slopes. The settlement was based on the excess ice content estimated at borehole 13-MPD-06, the borehole found to contain the most ice during the Main Dam field investigation (SRK 2014a). Further details on the thaw estimate methodology and results are provided in Appendix C.

In the tailings beach scenario, the foundation is expected to remain frozen throughout the operational period. As a result, no settlement is expected at the upstream toe and at the centerline, with minor settlement of 0.1 m predicted at the downstream toe. In the worst case pit lake scenario, approximately 20 m of the permafrost foundation thaws by year 20 at the upstream toe, corresponding to a settlement of 0.9 m as listed in Table 5. Settlement at the centerline and downstream toe is similar to the tailings beach scenario.

4.5.2 Consolidation Settlement

Estimates of the consolidation settlement were completed using Terzaghi's 1-D consolidation theory for normally consolidated soils. Secondary consolidations are small in comparison to primary consolidations and were assumed to be negligible. Consolidation was completed for

thawed fine grained soils only, where “thawed soils” were assumed to be any material near surface with temperature above the zero degree isotherm, as presented in the thermal modelling analysis (Appendix C). The fine grained soil thicknesses were discretized into 1 m intervals when determining change of stress in the foundation soils.

Maximum settlement during the design life of 20 years was determined by assessing the critical section fill thickness and thawed fine grained foundation thickness. The maximum settlement calculated for the design life of 20 years was 0.45 m not including thaw settlement calculated in the thermal modelling analysis (Appendix C). Total consolidation settlements after full foundation thawing were calculated to be 1.5 m settlement for the design with crest elevation at 812 m.

Further details of the consolidation settlement calculations can be found in Appendix D.

4.6 Tailings Consolidation Analysis

Consolidation analysis for the Main Pit tailings was carried out to demonstrate the extent primary consolidation and potential settlement within the tailings facility which will be carried forward to detailed design (if applicable), deposition strategy and closure planning. Details of the tailings consolidation analysis can be found in Appendix M.

Tailings consolidation was modelled based on finite strain consolidation theory using data from settlement and consolidation tests (KCB, 2013). The consolidation test results used in the model were selected for the lowest solids content, which represent a more conservative estimate. Modelling was conservatively carried out for the deepest point in the pit, i.e. 93 m depth. At this point, the tailings surface is expected to settle 3.7 m, 40 years after deposition ceases, and up to 8.5 m after 100 years. Areas near the edge of the pit or the Main Dam where tailings fill thickness is less, the consolidation will naturally be less. Notwithstanding these apparent large settlements, tailings consolidation will in no way affect the integrity of the dam.

4.7 Seepage Analysis

The dam is to retain water and the total seepage loss through the dam should be limited to the upper bound calculated and used in the inputs to the water quality prediction model of 83 m^3/d (SRK 2013d).

Multiple cases were analyzed using finite-element modelling represent a wide range of scenarios for seepage. Cases included a fully developed tailings beach scenario and a worst case pit lake scenario. All of these cases conservatively assumed that a geosynthetic liner is not present. Variants of each case evaluated the effect of a fully thawed foundation. Separate calculations were completed to assess the amount of potential leakage through the geomembrane liner of the dam. Appendix N provides complete details of the seepage and liner leakage analysis.

A summary of the key seepage analysis results is provided Table 6. With a liner in place, a seepage rate through the dam of 1 m^3/day is estimated. With no liner in place, results with a fully developed tailings beach in place ranged from 10 to 30 m^3/day. With no tailings beach and no liner, seepage rates are significantly higher. These results are well below the target value of

83 m^3/d, which implies that no further seepage control measure are required for the deeper bedrock foundation.

Table 6: Summary of Seepage Analysis Results

Model Case ID	Case Description	Dam Seepage (m^3/day)	Groundwater Flow (m^3/day.	Total Flow (m^3/day)
3	812 m dam with tailings beach and no liner	29	6	35
6	807 m dam with tailings beach and no liner	11	11	22
9	807 m dam with pit lake, frozen foundation, and no liner	330	117	447
7	807 m dam with pit lake, thawed foundation and no liner	342	169	551
L1	812 m lined dam	1	36	37
L6	807 m lined dam	1	36	37

Note(s):

(1) Frozen materials were assigned a permeability of $1.0x10^{-8}$ m/s. This value is lower than other modeled materials, but still allowed for some groundwater flow to occur in the frozen foundation cases.

(2) The cases with a liner assume one defect per acre with a hole radius of 0.1 m.

(3) Model Case ID corresponds to selected cases described in Appendix N.

4.8 Seepage Collection

Instrumentation in the dam (see Section 4.10) will determine how much seepage flows through the dam, and monitoring wells immediately downstream of the dam will confirm the volume of deeper ground water seepage. The geometry and stratigraphy downstream of the dam consists of run-of-mine waste rock placed on surface topographic lows from the Main Pit down to the water storage pond. This waste rock is expected to be continuous and highly permeable; therefore, any seepage from the dam will likely flow through the waste rock rather than percolate to the surface at the toe of the dam. Conventional toe drains are therefore not expected to be useful to control and capture seepage from the dam. Seepage interception and management, if deemed necessary will be based on an Adaptive Management Plan which will feed from the monitoring data.

4.9 Spillway

The rip-rap lined spillway, designed for the IDF, will be constructed between the Main Pit and Area 2 Pit, but will not be connected to the Main Dam in any way (Drawing SRK-MD-04). The spillway invert will be at the Main Dam FSL of 809 m. Given the large elevation difference between the Main Pit FSL and the Area 2 Pit FSL of 797, and the variable foundation conditions, the spillway will be a compound spillway consisting of three distinct sections as follows:

- Upper section founded on permafrost overburden with a longitudinal slope of 6%, a width of 25 m and rip-rap with a D_{50} of 0.3 m;
- Middle section founded on permafrost overburden, with a slope transitioning from 100% to 50%, a width of 35 m and rip-rap with a D_{50} of 1m; and

- Bottom section, founded on bedrock which will act as a stilling basin. The stilling basin will have a slope of 0%, a width of 35 m and the rip-rap D_{50} will transition from 1 m down to 0.3 m.

The transverse side slopes of the banks for each section of the spillway are 3H:1V. The maximum flow depth in the spillway will be 1 m under the IDF. Complete spillway design details are included in Appendix F.

4.10 Monitoring Instrumentation

An instrumentation plan was designed in order to monitor internal pore pressures, settlement, deformation and foundation thaw. Knowledge of the internal pore pressure will allow for real-time assessments of the dam stability. An indication of the amount of foundation thawing will enable an assessment of the accuracy of the thermal modelling and settlement predictions. Knowledge of the amount of settlement and deformation will allow for estimates of strain on the geomembrane liner. Table 7 displays the type of instrumentation suggested in order to monitor pore pressures, settlement, deformation and foundation thaw. Instrumentation plans can be viewed in Drawings SRK-MD-16, SRK-MD-17 and SRK-MD-18. A full discussion of the instrumentation plan can be found in Appendix O.

Table 7: Instrumentation Plan

Type of Instrumentation	Number of Instruments	Process Monitored	Reason for Instrumentation
Inclinometers	4	Deformation	Assess strain on geomembrane liner
Vibrating Wire Piezometers	2	Internal pore pressures	Assess dam stability, accuracy of seepage predictions
Ground Temperature Cables	5	Foundation thaw	Assess dam stability, settlement, thermal model accuracy
Survey Prisms	38	Deformation, settlement	Assess strain on geomembrane liner, dam stability, settlement

4.11 Material Quantities

Preliminary material quantities are presented in Table 8. The quantities are in-place, neat line quantities and do not account for bulking during excavation or shrinkage during placement and compaction.

Table 8: Material Quantities for Dam Height at 812 m

Material	Quantity
Rip-rap (m^3)	17,700
Bedding (m^3)	10,700
Fine grained core (m^3)	39,200
Transition (Coarse and Fine) Filters (m^3)	17,300
Run-of-mine waste rock (m^3)	321,800
LLDPE liner (m^2)	18,900
Foundation excavation (m^3)	123,800
Key-trench excavation (m^3)	80,800
Total dam material placement(excluding excavation) (m^3)	406,800

5 Construction Activities

5.1 Methodology

Prior to construction, all non-native fill material will be removed from the dam footprint. Next the key-trench will be excavated until low permeability natural soils are intercepted. The key-trench excavation will need to include a small horizontal bench on the upstream side in order to assist with keying the liner into natural ground. Following excavation of the key-trench, the bulk fill run of mine rock on the downstream side of the dam will be constructed to 1 m less than the maximum dam height. The core and transition materials will then be constructed from the base of the key trench up to the height of the constructed bulk fill. The excavation surface is presented in drawing SRK-MD-05 while the constructed dam is presented in drawing SRK-MD-04.

Bedding material will be placed on the upstream side of the core in preparation for liner placement. A trench will be then be cut into natural ground adjacent the toe of the core material where the end of the liner will be keyed-in. The liner will be deployed up the entire face of the dam with no horizontal welds and run over the crest beyond the core to an anchor trench. Typical section and details are presented in drawing SRK-MD-06.

The remaining fill, in front of the liner will be constructed in lifts up the front to maintain a 6 m wide minimum working bench for vehicle traffic.

Sample technical specifications for the dam construction are provided in Appendix P. These specifications will be revised as required during detailed design.

5.2 Borrow Sources

Run-of-mine material will be sourced from existing stockpiles of construction grade waste rock stored on site. Rip-rap will be sourced from the same area as the run-of-mine and sorted for gradation prior to transport.

The fine grained core will be sourced from the Reclamation Overburden Dump on the west side of the pit.

Further exploratory borrow characterization may be required to delineate sources for the filter materials. Other geotechnical field investigations have shown that there are areas where coarse grained soils may be found, such as the Ridgetop area near the Area 2 Pit. It is thought that many of the hill-top areas have reasonably large amounts of colluvium, or weathered bedrock, which would have high sand content that may be suitable for the filter materials. Processing of these filter materials will likely be required to meet gradation requirements.

5.3 Construction Equipment

The regulatory body for safe work procedures in the Yukon is Yukon Workers' Compensation Health and Safety Board (YWCHSB, 2010). It specifies that the minimum road width for mining or construction trucks is 1.5 times the effective width of the vehicle. For this reason, construction will be carried out with a small civil/mining fleet. The current construction method involves building

roadways with widths of 6 m. The construction fleet is expected to consist of 30-40 tonne trucks with widths of less than 4 m. The proposed construction equipment fleet is listed below:

- Articulated Trucks (30-40 tonne)
- Excavator (Mid-sized; approximately 150 hp)
- Loader
- Vibratory Compactors (10 tonne)
- Dozer (mid-sized; 30 tonne)

Equipment from the mining fleet can be used for foundation excavation and bulk fill, while the specified fleet above would be used for all other construction that requires greater attention to detail (core, transition, bedding, etc.).

6 Dam Operation, Maintenance and Monitoring

6.1 Tailings Deposition (Operational) Plan

With the containment dam in place, tailings deposition can continue using the existing slurry tailings deposition strategy. To improve seepage performance and overall long-term stability of the dam, tailings discharge should be done using end point spigots from the crest of the dam and from the southern slope dividing Main Pit from Area 2 Pit (Figure 3). That will create a significant beach along the upstream slope of the area consisting of the coarsest tailings and ultimately pushing the pond at least 100 m from the upstream slopes. Once sufficient beach has been developed spigotting can be moved to other areas within the pit. Spigot points should be moved around to over time create a final tailings landform that is conducive to closure.

This deposition method is simple, not susceptible to process upsets/changes in the mill and is suited to mitigation of potential deformation of the Main Dam that might occur during the operational life of the Main Dam. Furthermore, this method is well understood on-site, and is expected to be easily executed by site operational staff. Based on performance elsewhere, the deposition method has also been proven to be manageable under the harsh climatic conditions experienced at the Minto site.

6.2 Water Management Plan

Provided the containment freeboard is maintained, there is ample potential to allow for surge capacity in Main Pit or Area 2 Pit. Practically speaking, based on the design concept proposed for the containment dam, it is recommended that any excess water storage be managed with beach development in mind to minimize dam seepage. If the Main Pit is used to contain excess water it may be necessary to construct a permanent spillway prior to it reaching FSL. It would however be possible to defer spillway construction if the water level in Main Pit is controlled through the use of Area 2 Pit via active pumping for a period of time.

7 Preliminary Closure Design

Closure of the Main Pit Tailings Management Facility (MPTMF) is described in detail in the Minto Mine Phase V/VI Reclamation and Closure Plan (ACG 2014). Closure measures specific to the Main Dam are discussed below, along with brief descriptions of the closure measures from ACG (2014) that relate to conveyance through the MPTMF and to treatment of the final tailings surface within the facility.

7.1 Main Dam Closure

It is expected that the Main Dam will ultimately be declassified as a dam. Monitoring instrumentation will be left in place and subject to scheduled monitoring until declassification is complete. Following declassification, closure measures for the exposed surface of the dam will be similar to those employed in reclaiming other areas on site.

7.2 Tailings Cover

The final tailings surface will be graded to direct surface runoff towards the spillway, which will remain in place for post-closure water conveyance from Main Pit (and contributing catchment) to the Area 2 Pit. A composite cover will be applied to the tailings surface; the cover will consist of a layer of waste rock placed directly on the tailings surface to allow trafficking by construction equipment, and a layer of fine grained overburden placed over the waste rock as final cover material. Revegetation of the final cover will be carried out as described in the closure plan (ACG 2014).

7.3 Water Management

Surface runoff from the Main Pit catchment, including the covered tailings, will be directed towards the spillway and will join conveyed flows from upgradient catchment areas from the Main Pit to the Area 2 Pit. The post-closure conveyance network is described in the closure plan (ACG 2014).

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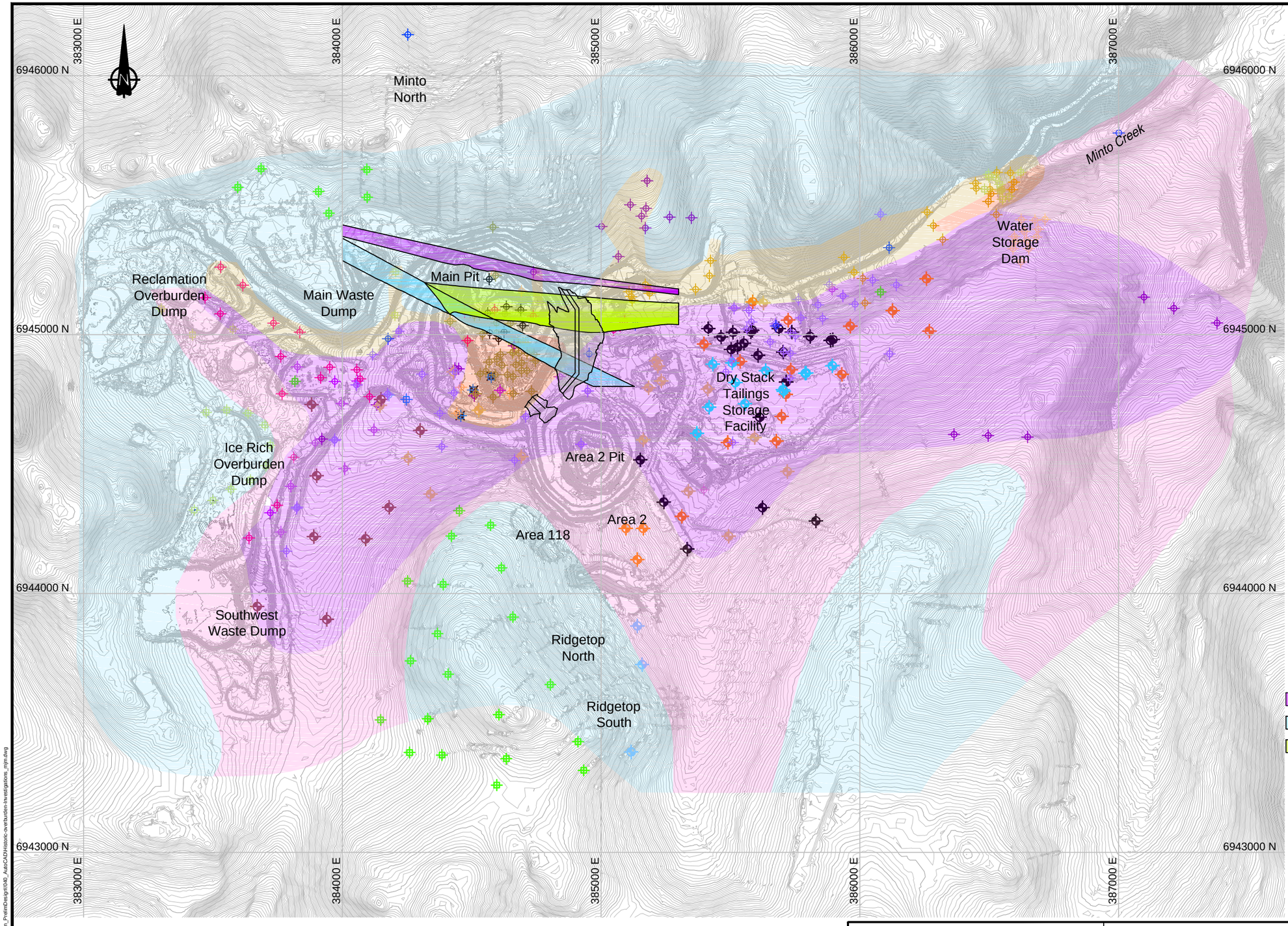
Figures



LEGEND

- SRK 2013 Drill Holes
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- SRK (boreholes) - 1993
- EBA (boreholes) - Preliminary Geotechnical Design, 1994
- EBA (boreholes) - Preliminary Geotechnical Design, Tailings/Water Retention Dam (July 1995, December 1995), 0201-11509
- EBA (boreholes) - 1996 Geotechnical Drilling Program (January 1997), 0201-96-11509, 5 BH
- EBA (boreholes) - 1997 Geotechnical Program and Construction Inspection Reports (February 2008), 0201-97-11509
- EBA (boreholes) - Preliminary Geotechnical Design, 2005
- DSSH14, ASH06
- SRK (boreholes) - 2008 Geotechnical program (April 2008)
- SRK (boreholes) - 2008 Overburden Drill holes
- EBA (boreholes) - 2010 Geotechnical Program (February 2010)
- EBA (boreholes) - 2011 Geotechnical Program (Winter and Fall 2011)
- ConeTec CPTs - 2010 Geotechnical Program (October 2010)
- SRK (test pits) - 2013
- EBA (test pits) - 2010
- EBA (test pits) - 2010 Geotechnical Program (February 2010)
- EBA (test pits) - 1997 Geotechnical Program & Construction Inspection reports (February 1998), 0201-97-11509
- EBA (test pits) - Proposed Reclamation Overburden dump (February 2008), W14101068.004

- Indicates zones containing permafrost and/or ice
- Indicates zones containing no permafrost and/or ice
- Indicates zones possibly containing permafrost and/or ice
- Indicates zones possibly containing no permafrost and/or ice
- Indicates zones containing thawed permafrost and/or ice

- Inferred DEF Fault Plane
- Inferred Fault 1 Plane
- Inferred Fault 2 Plane

0 200 400 600 800 1000
Scale in Metres

NOTES

1. Data presented in NAD 1983 UTM Zone 8N
2. Topographic information provided by Minto Mine, April 2013
3. Permafrost information adapted from EBA, October 2011

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SRK JOB NO.: 1CM002.018

FILE NAME: Historic-overburden-Investigations_mjm.dwg

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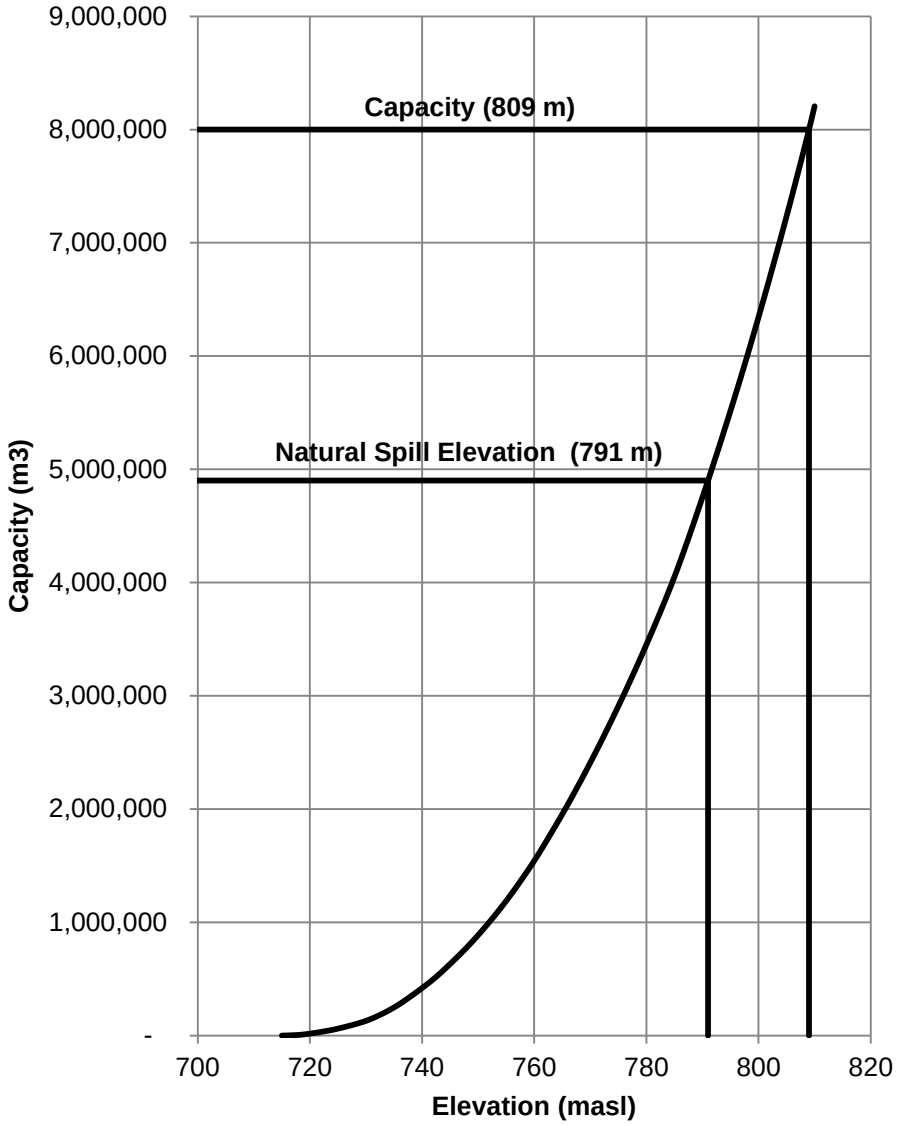
Minto Mine

Main Dam Preliminary Design

Permafrost Extent, Major Features and Historic Overburden Investigations

DATE:	APPROVED:	FIGURE:
May 2014	EMR	1

J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\040_AutoCAD\Historic-overburden-Investigations_mjm.dwg



Elevation	Volume (m3)	Elevation	Volume (m3)
715	56	762	1,702,614
716	1,410	763	1,785,134
717	3,430	764	1,868,975
718	5,924	765	1,954,154
719	10,572	766	2,041,393
720	17,101	767	2,130,616
721	24,486	768	2,221,757
722	32,496	769	2,314,613
723	41,106	770	2,409,258
724	50,406	771	2,505,664
725	61,359	772	2,603,819
726	73,295	773	2,703,671
727	85,866	774	2,805,205
728	99,306	775	2,908,425
729	113,668	776	3,013,316
730	129,671	777	3,119,864
731	149,251	778	3,228,341
732	170,830	779	3,338,983
733	194,649	780	3,451,613
734	220,359	781	3,566,123
735	247,945	782	3,682,665
736	278,251	783	3,801,550
737	311,887	784	3,923,509
738	346,774	785	4,050,150
739	382,533	786	4,183,015
740	419,275	787	4,320,592
741	457,291	788	4,461,936
742	497,007	789	4,605,998
743	539,846	790	4,752,430
744	585,246	**791**	**4,901,026**
745	631,931	792	5,051,705
746	679,739	793	5,204,488
747	728,699	794	5,359,367
748	779,103	795	5,516,396
749	832,134	796	5,675,899
750	887,105	797	5,838,492
751	943,845	798	6,004,032
752	1,002,258	799	6,172,243
753	1,062,287	800	6,343,273
754	1,124,301	801	6,516,842
755	1,188,907	802	6,692,757
756	1,255,845	803	6,870,930
757	1,324,791	804	7,051,384
758	1,395,328	805	7,234,243
759	1,467,699	806	7,420,670
760	1,542,825	807	7,610,333
761	1,621,544	808	7,802,829
762	1,702,614	**809**	**8,000,326**
763	1,785,134	810	8,206,034

Source: Main Pit Storage Capacity - 2014-07-11_mjm.xlsx

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Main Dam Preliminary Design

Capacity vs. Stage Elevation

Job No: 1CM002.018

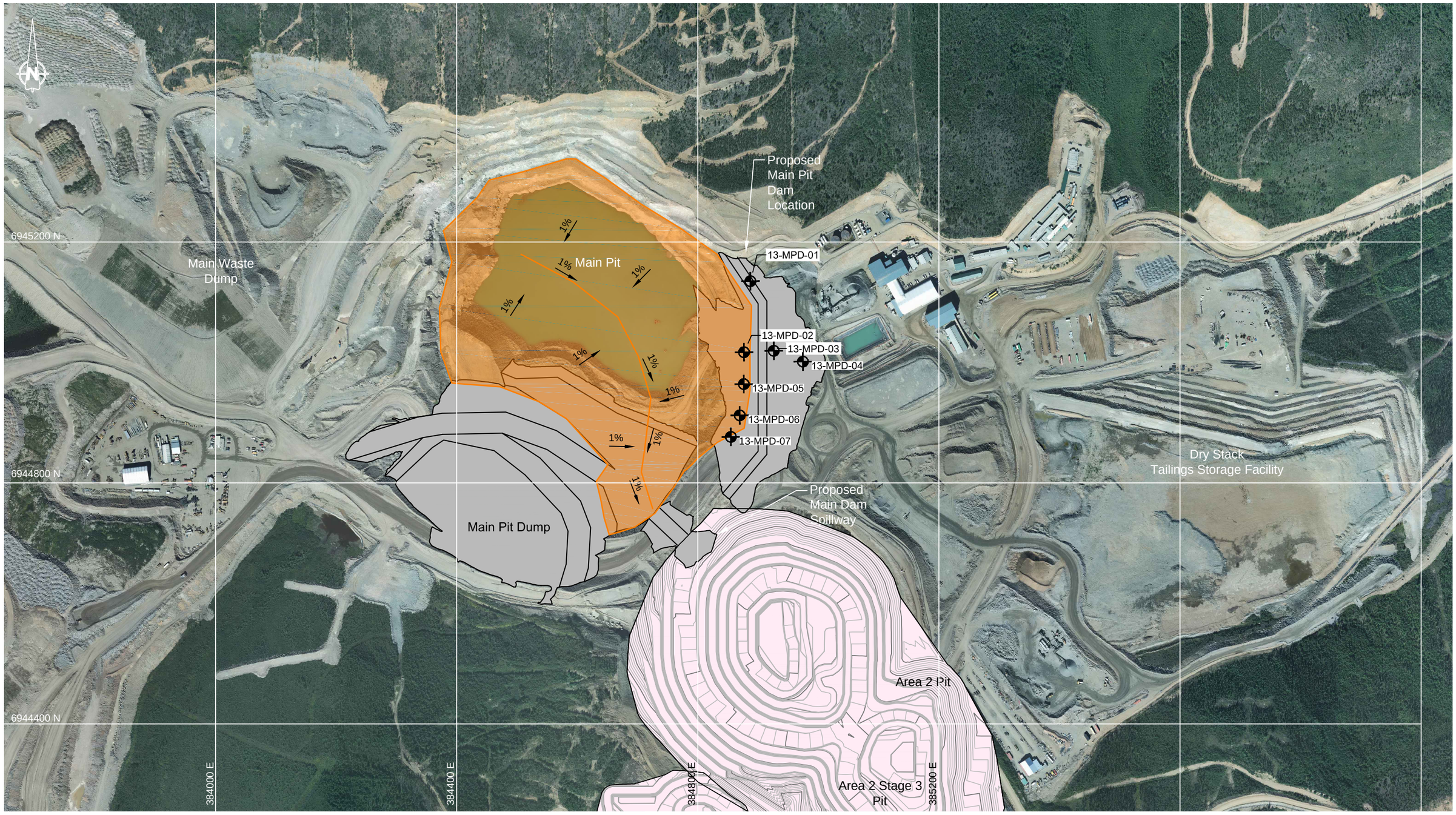
Filename: StagedStorageMainPit_20140714_mjm.pptx

Minto Mine

Date: July 2014

Approved: MJM

Figure: **2**



PLAN LEGEND

- Phase IV/V/VI Pit Shell
- Phase V/VI Waste Dump
- Approximate Tailings Area
- 2013 Main Dam Drillholes

NOTES

1. Data presented in NAD 1983 UTM Zone 8N
2. Orthophoto: August 2013
3. Phase V/VI pit shells provided by Minto Mine, March 2013
4. Phase V/VI Main Pit Dump, re-designed September 2013. Note no ramp modeled on dump at this time.

0 60 120 180 240 300
Profile Section Scale in Metres

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SRK JOB NO.: 1CM002.012.012

FILE NAME: Site Layout–Main Pit Tailings Management Facility_IR_B–dj_mjm2.dwg

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Minto Mine

Response To IR(B)-3

Main Dam Plan

DATE:	APPROVED:	FIGURE:
July 2014	MJM	3

J:\01_SITES\Minto\1CM002.018_MainDam_PreliminDesign\1040_AutoCAD\Site Layout-Main Pit Tailings Management Facility_IR_B-dj_mjm2.dwg

Drawings

Engineering Drawings for the Preliminary Main Dam Design, Minto Mine, Yukon, Canada

ACTIVE DRAWING STATUS

DWG NUMBER	DRAWING TITLE	REVISION	DATE	STATUS
SRK-MD-01	Engineering Drawings for the Preliminary Main Dam Design, Minto Mine, Yukon, Canada	B	July 18, 2014	Issued For Licensing
SRK-MD-02	Site Location	B	July 18, 2014	Issued For Licensing
SRK-MD-03	General Arrangement	B	July 18, 2014	Issued For Licensing
SRK-MD-04	Main Dam and Spillway	B	July 18, 2014	Issued For Licensing
SRK-MD-05	Main Dam Foundation Excavation with Keytrench	B	July 18, 2014	Issued For Licensing
SRK-MD-06	Typical Dam Section and Details	B	July 18, 2014	Issued For Licensing
SRK-MD-07	Dam Centerline Profile	B	July 18, 2014	Issued For Licensing
SRK-MD-08	Geological Sections	B	July 18, 2014	Issued For Licensing
SRK-MD-09	Dam Sections (1 of 5)	B	July 18, 2014	Issued For Licensing
SRK-MD-10	Dam Sections (2 of 5)	B	July 18, 2014	Issued For Licensing
SRK-MD-11	Dam Sections (3 of 5)	B	July 18, 2014	Issued For Licensing
SRK-MD-12	Dam Sections (4 of 5)	B	July 18, 2014	Issued For Licensing
SRK-MD-13	Dam Sections (5 of 5)	B	July 18, 2014	Issued For Licensing
SRK-MD-14	Spillway Profiles	B	July 18, 2014	Issued For Licensing
SRK-MD-15	Spillway Sections	B	July 18, 2014	Issued For Licensing
SRK-MD-16	Main Dam (812m) Instrumentation Plan	B	July 18, 2014	Issued For Licensing
SRK-MD-17	Instrumentation Shown on Dam Centerline Profile	B	July 18, 2014	Issued For Licensing
SRK-MD-18	Instrumentation Section and Details	B	July 18, 2014	Issued For Licensing

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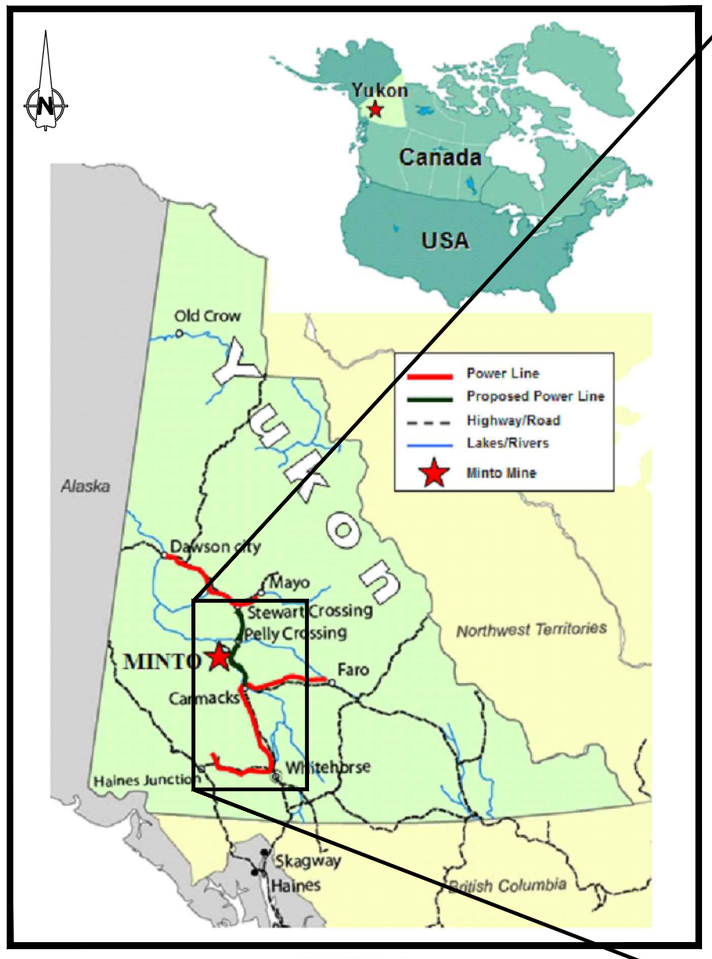
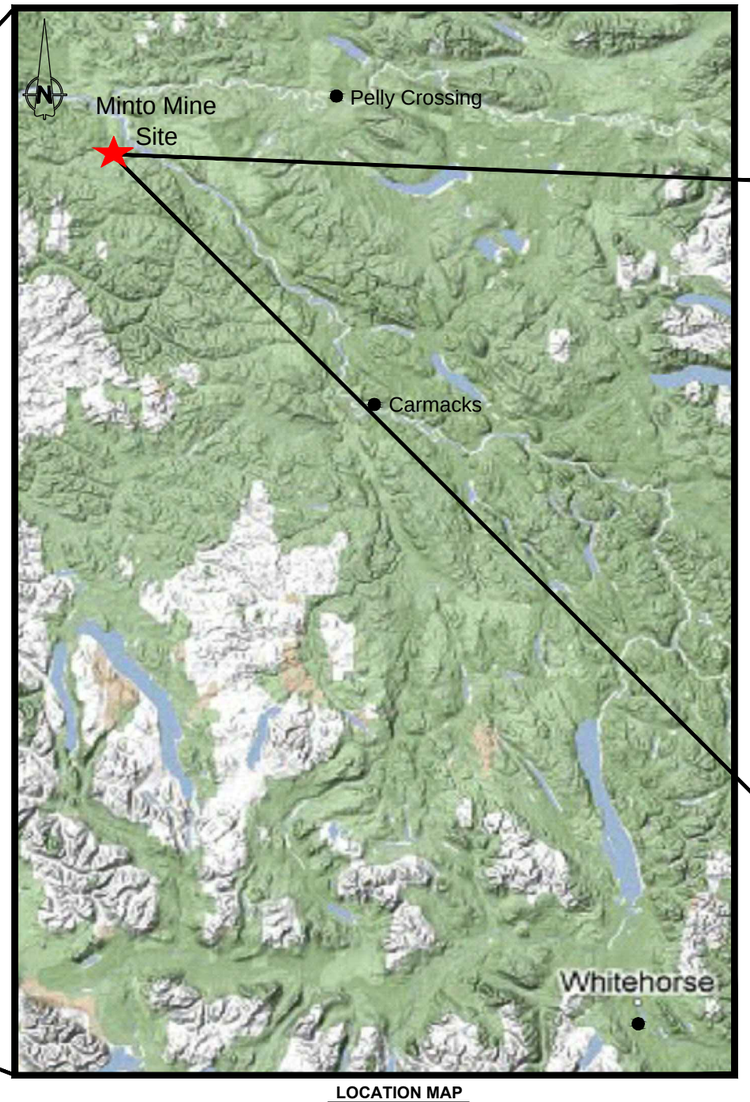
PROJECT NO: 1CM002.018.710
Revision B - Issued for Licensing
July 18, 2014
Drawing SRK-MD-01



VICINITY MAP

0 40 80 120 160 200

Scale in Metres



LOCATION MAP

0 10 20 30 40 50

Scale in Metres

MINE SITE

0 100 200 300 400 500

Scale in Metres

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REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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A	Issued for Review	MM	–	30May14

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CHECKED:	APPROVED:	DATE:
EMR	EMR	July 2014

FILE NAME: 1CM002_018_710-1_RevB.dwg

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MINTO MINE

SRK JOB NO.: 1CM002.018.710

Preliminary Dam Design

DRAWING TITLE:

Site Location

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-02	2 OF 18	B

C:\01_SITES\Minto\Dam Design\1CM002_018_710-1_RevB.dwg

REFERENCE

1. Orthophoto provided by mine site. Orthophoto dated August, 2013.

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0 50 100 150 200 250
Scale in Metres

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE

REFERENCE DRAWINGS

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MINTO MINE

SRK JOB NO.: 1CM002.018.710

Preliminary Dam Design

DRAWING TITLE: General Arrangement

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-03	3 OF 18	B

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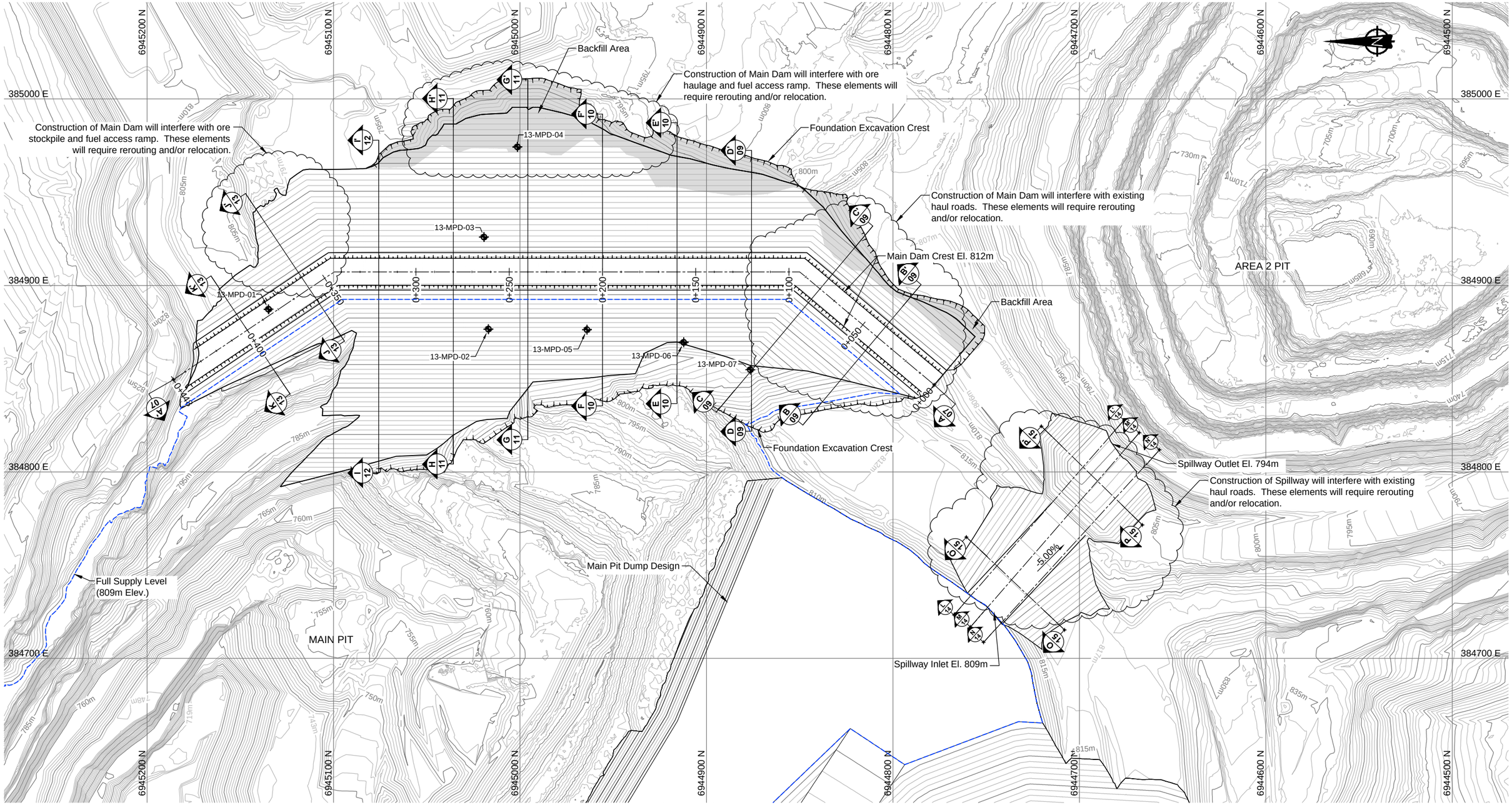

LEGEND

- Borehole Location
- Full Supply Level

NOTES

1. Contour interval 1m.
2. Topography received from mine site January 2014.
3. Coordinate system shown is UTM Zone 8, NAD83.

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0 10 20 30 40 50
Scale in Metres

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
REFERENCE DRAWINGS			

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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MINTO MINE

SRK JOB NO.: 1CM002.018.710

Preliminary Dam Design

DRAWING TITLE: Main Dam and Spillway

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-04	4 of 18	B

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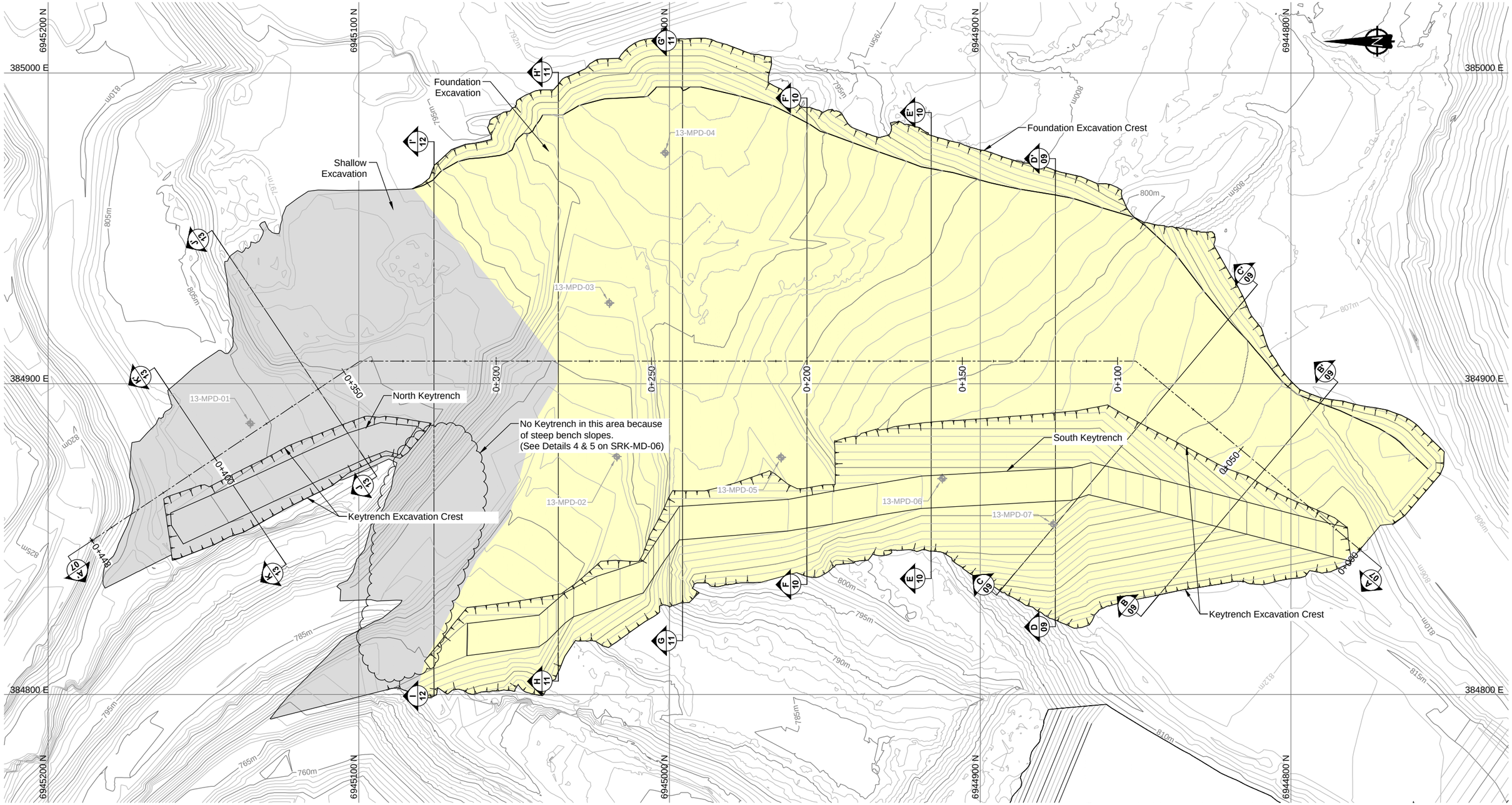

NOTES

1. Contour interval 1m.
2. Topography received from mine site January 2014.
3. Coordinate system shown is UTM Zone 8, NAD83.

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0 10 20 30 40 50
Scale in Metres

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				A	Issued for Review	MM	–	30May14

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EMR	EMR	July 2014

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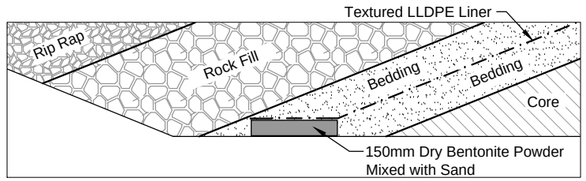
MINTO MINE

SRK JOB NO.: 1CM002.018.710

Preliminary Dam Design

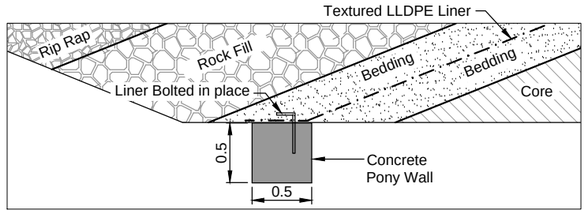
DRAWING TITLE:
Main Dam Foundation Excavation with Keytrench

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-05	5 of 18	B



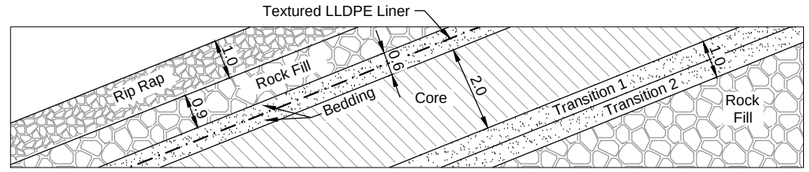
DETAIL 1
1 LINER TIE-IN TO SOIL FOUNDATION

0 0.2 0.4 0.6 0.8 1
Scale in Metres



DETAIL 2
2 LINER TIE-IN TO ROCK

0 0.2 0.4 0.6 0.8 1
Scale in Metres

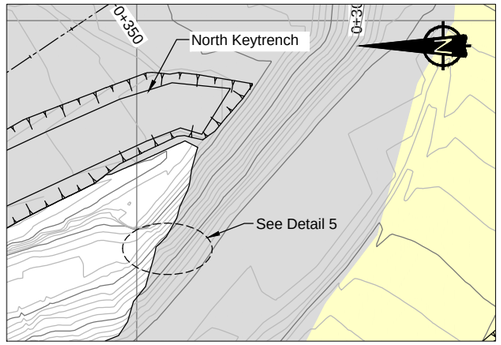


DETAIL 3 - LAYER SYSTEM
3

0 1 2 3 4 5
Scale in Metres

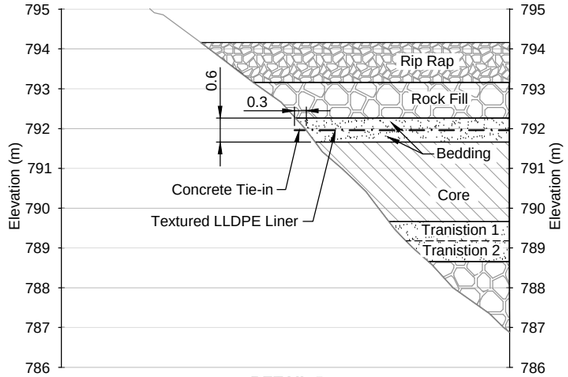
LEGEND

- Existing Ground (Jan. 2014)
- Top of Bedrock
- Liner
- Core Material
- Transition Material
- Rock Fill
- Rip Rap



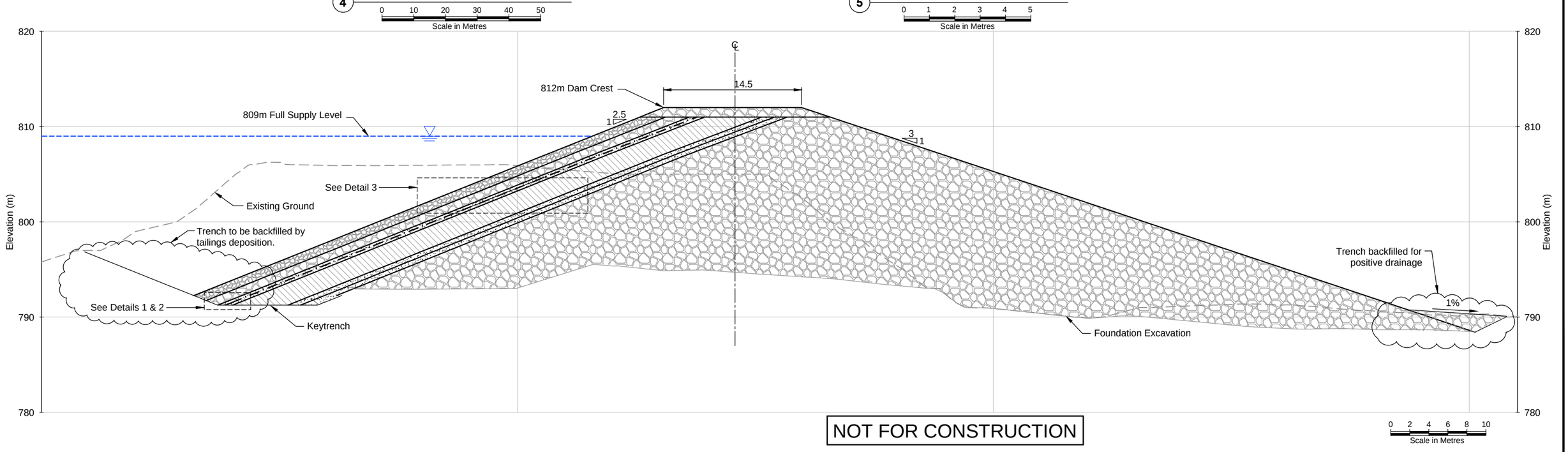
DETAIL 4
4 PLAN VIEW - BENCH WALL LINER TIE-IN

0 10 20 30 40 50
Scale in Metres



DETAIL 5
5 SECTION VIEW - BENCH WALL TIE-IN

0 1 2 3 4 5
Scale in Metres



0 2 4 6 8 10
Scale in Metres

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MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:
Typical Dam Section and Details

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-06	6 of 18	B

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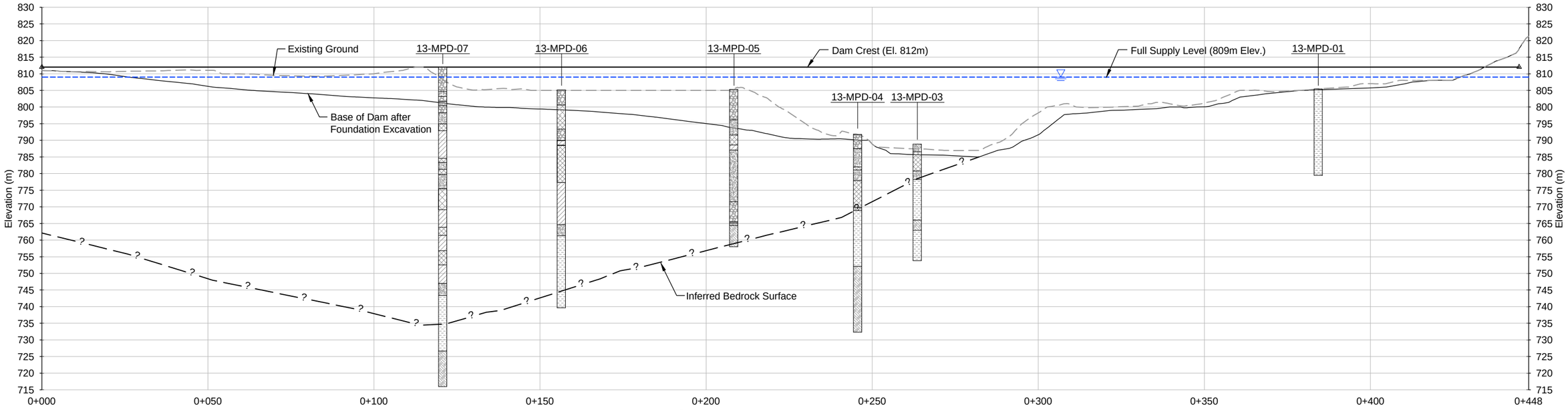

A 04 DAM CENTERLINE PROFILE

0 4 8 12 16 20
Scale in Metres

LEGEND

- Existing Ground (Jan. 2014)
- Inferred Bedrock Surface
- Full Supply Level

Borehole Stratigraphy:

- Gravel
- Sand
- Clay
- Silt
- Weathered Bedrock
- Bedrock

NOTE

1. Bedrock surface shown on profile was interpolated from borehole bedrock data. Borehole bedrock layers shown are projected onto the profile and may not match the interpolated bedrock surface shown.
2. For detailed information on boreholes refer to Geotechnical Report.

NOT FOR CONSTRUCTION

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
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REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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A	Issued for Review	MM	–	30May14

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EMR	EMR	July 2014

FILE NAME: 1CM002_018_710-3_RevB.dwg

capstone MINTO MINE
OPERATED BY MINTO EXPLORATIONS LTD.

MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE: Dam Centerline Profile

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-07	7 of 18	B

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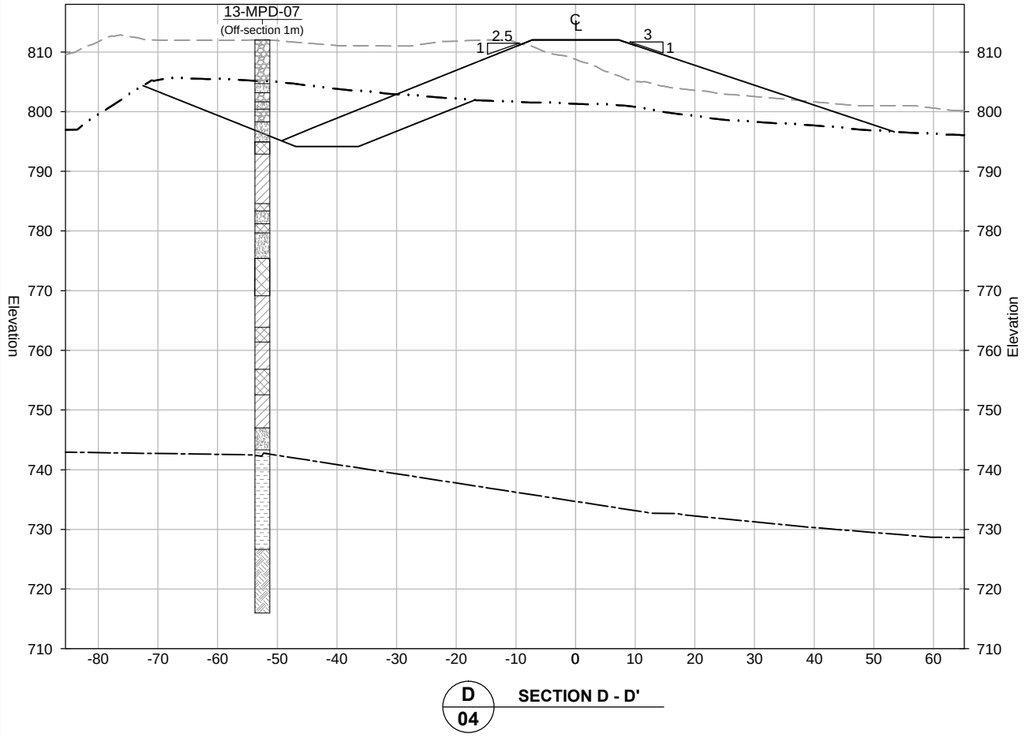
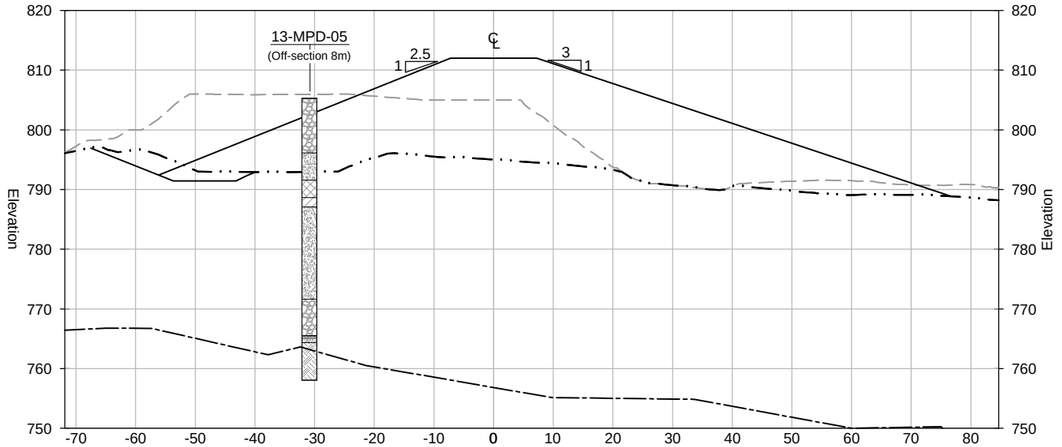
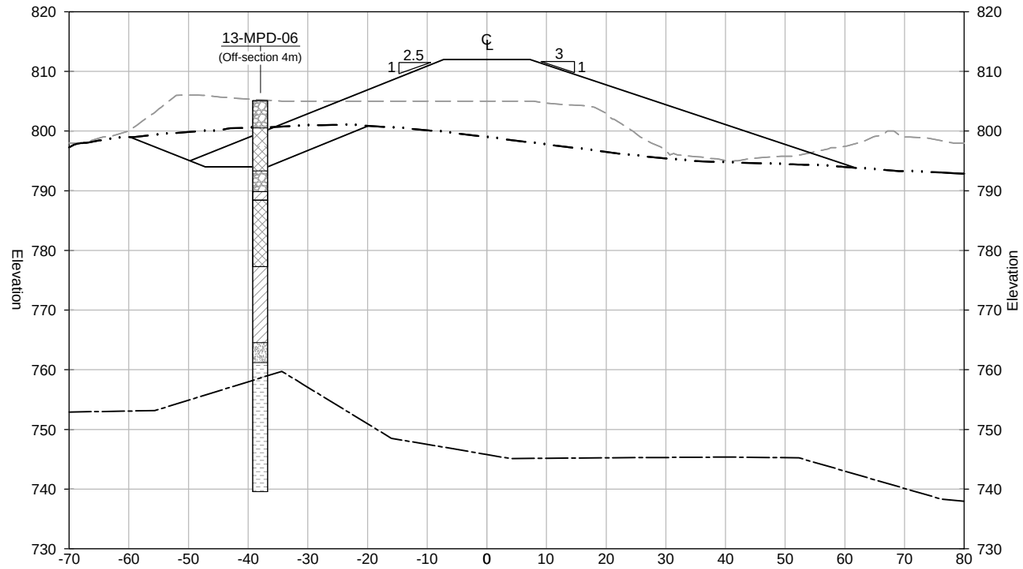


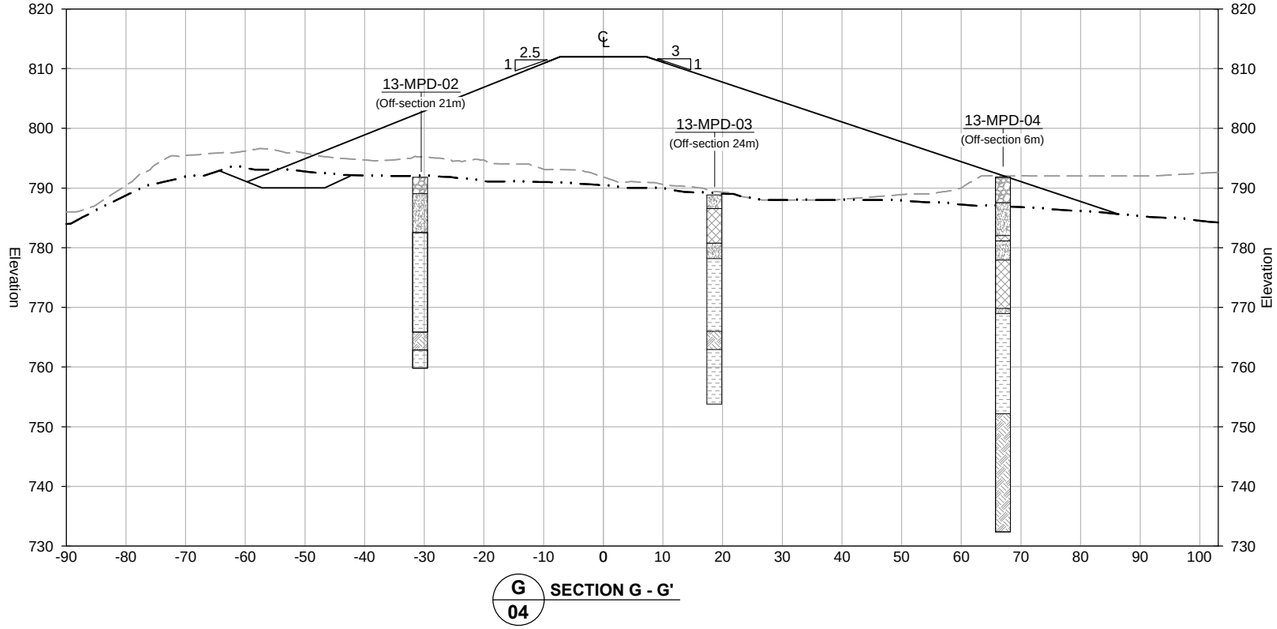
D 04 SECTION D - D'



F 04 SECTION F - F'



E 04 SECTION E - E'



G 04 SECTION G - G'

LEGEND

Borehole Stratigraphy:

- Gravel
- Sand
- Clay
- Silt
- Weathered Bedrock
- Bedrock

Surfaces:

- Existing Ground
- Estimated Natural Ground
- Dam Shell / Keytrench
- Estimated Bedrock

NOTES

1. Existing ground represents latest survey received (Jan. 2014)
2. Natural ground line indicates the estimated surface in which in-situ material has not been disturbed below.

NOT FOR CONSTRUCTION

Scale in Metres: 0 5 10 15 20 25

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
–	–		

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EMR	EMR	May 2014

FILE NAME: 1CM002_018_710-3_RevB.dwg

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MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE: Geological Sections

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-08	8 of 18	B

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LEGEND

- Existing Ground (Jan. 2014)
- Estimated Natural Ground
- Full Supply Level (El.809m)
- Liner

Dam Hatches:

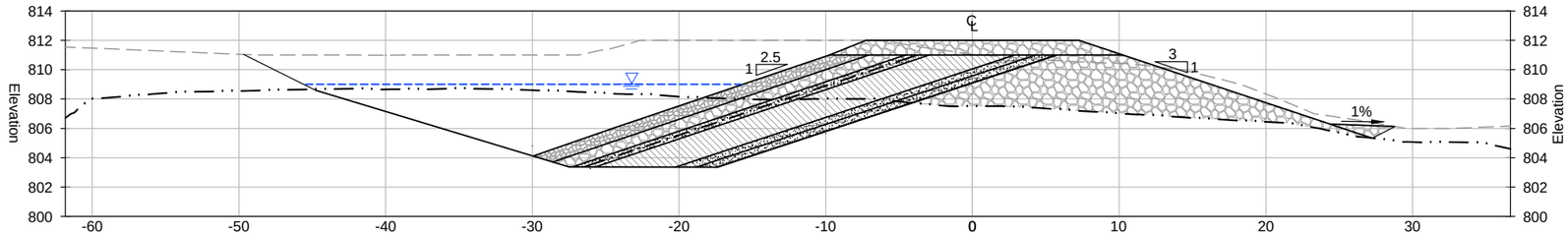
- Core Material
- Transition Material
- Rock Fill
- Rip Rap

Borehole Stratigraphy:

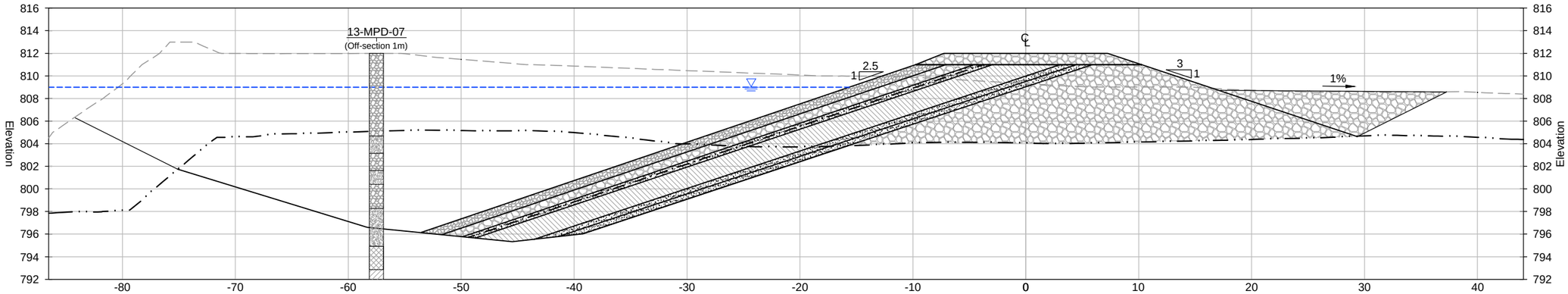
- Gravel
- Sand
- Clay
- Silt
- Weathered Bedrock
- Bedrock

NOTES

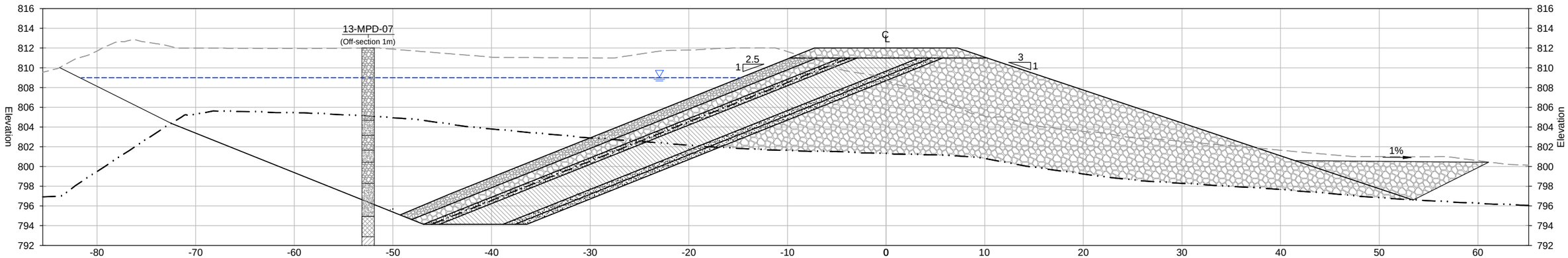
1. Downstream excavation backfilled for positive drainage.
2. Upstream excavation to be backfilled by tailings during initial operations.



B 04 SECTION B - B'



C 04 SECTION C - C'



D 04 SECTION D - D'

NOT FOR CONSTRUCTION

0 2 4 6 8 10
Scale in Metres

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
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FILE NAME: 1CM002_018_710-3_RevB.dwg

capstone MINTO MINE
OPERATED BY MINTO EXPLORATIONS LTD.

MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE: Dam Sections (1 of 5)

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-09	9 of 18	B

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LEGEND

- Existing Ground (Jan. 2014)
- Estimated Natural Ground
- Full Supply Level (El.809m)
- Liner

Dam Hatches:

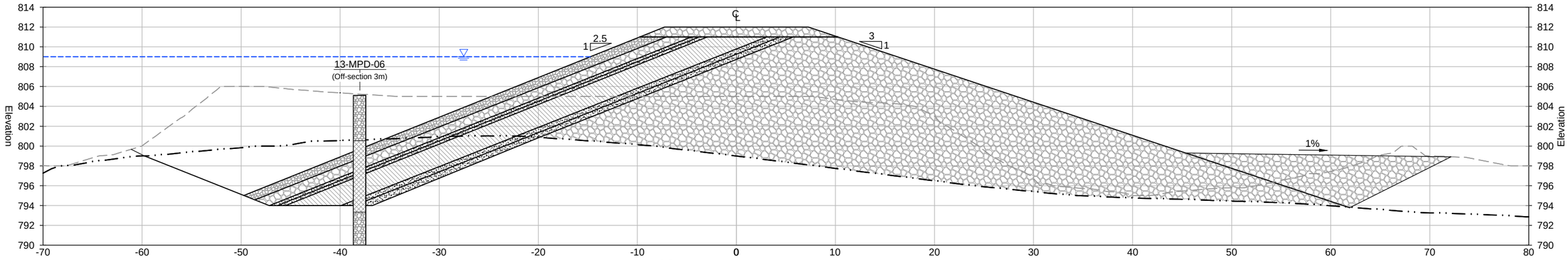
- Core Material
- Transition Material
- Rock Fill
- Rip Rap

Borehole Stratigraphy:

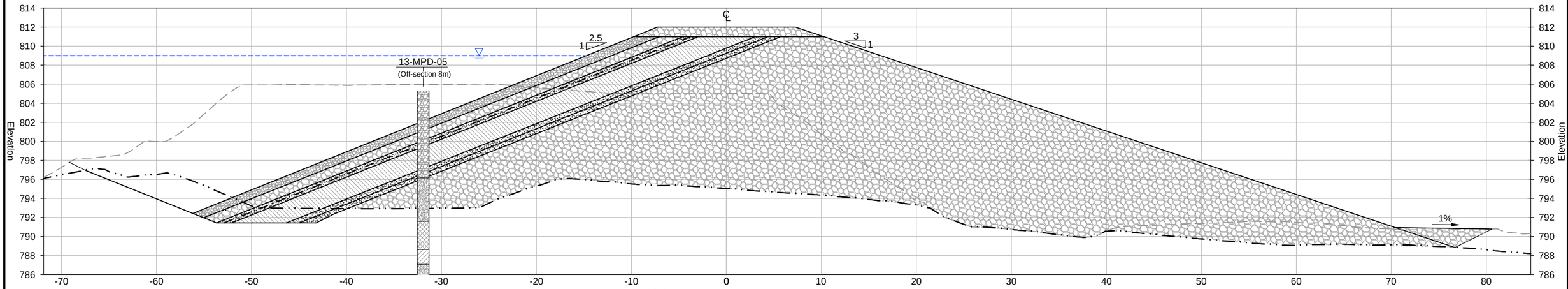
- Gravel
- Sand
- Clay
- Silt
- Weathered Bedrock
- Bedrock

NOTES

1. Downstream excavation backfilled for positive drainage.
2. Upstream excavation to be backfilled by tailings during initial operations.



E 04 SECTION E - E'



F 04 SECTION F - F'

NOT FOR CONSTRUCTION

0 2 4 6 8 10
Scale in Metres

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EMR	EMR	July 2014

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MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:
Dam Sections
(2 of 5)

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-10	10 of 18	B

LEGEND

- Existing Ground (Jan. 2014)
- Estimated Natural Ground
- Full Supply Level (El.809m)
- Liner
- Estimated Bedrock

Dam Hatches:

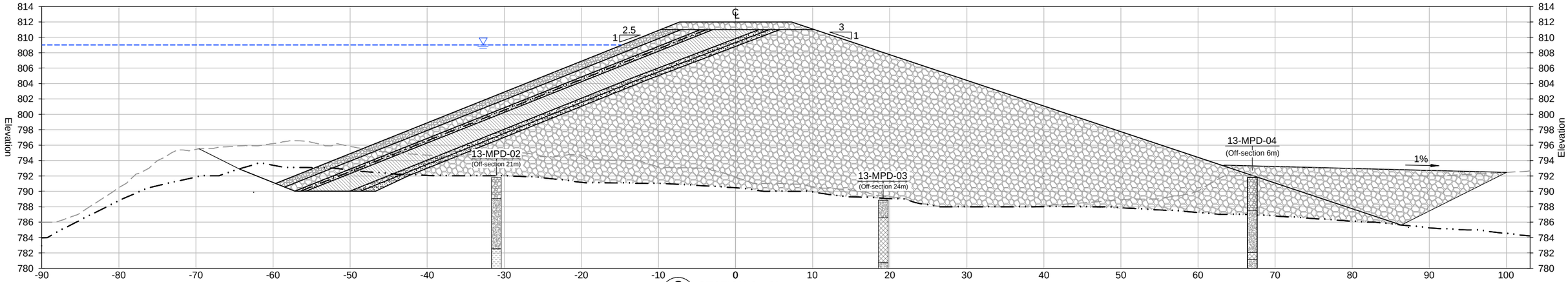
- Core Material
- Transition Material
- Rock Fill
- Rip Rap

Borehole Stratigraphy:

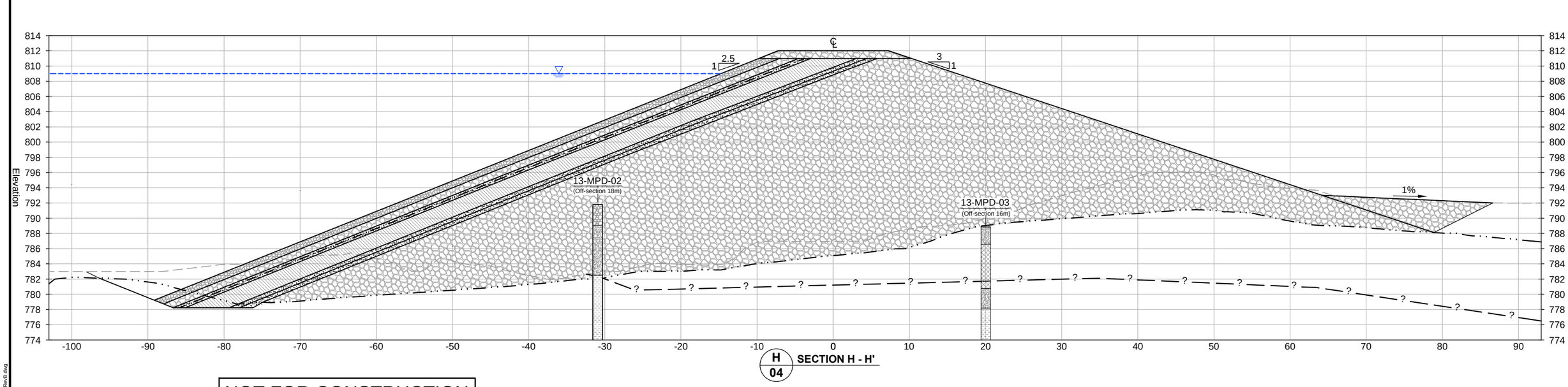
- Gravel
- Sand
- Clay
- Silt
- Weathered Bedrock
- Bedrock

NOTES

1. Downstream excavation backfilled for positive drainage.
2. Upstream excavation to be backfilled by tailings during initial operations.



G / 04 SECTION G - G'



H / 04 SECTION H - H'

NOT FOR CONSTRUCTION

0 2 4 6 8 10
Scale in Metres

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DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
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FILE NAME: 1CM002_018_710-3_RevB.dwg

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MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:

Dam Sections
(3 of 5)

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-11	11 of 18	B

LEGEND

- Existing Ground (Jan. 2014)
- Estimated Natural Ground
- Full Supply Level (El.809m)
- Liner
- Estimated Bedrock

Dam Hatches:

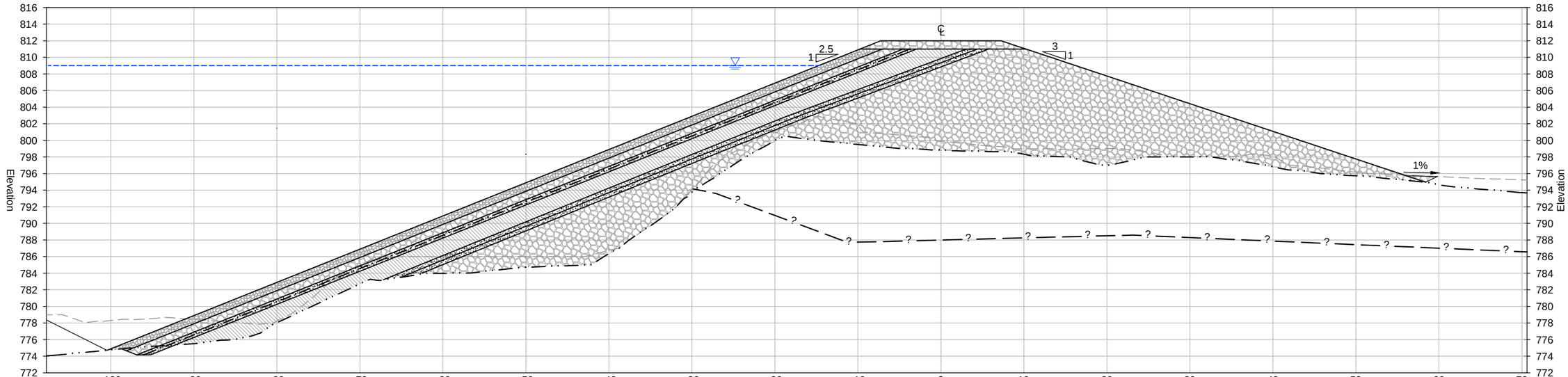
- Core Material
- Transition Material
- Rock Fill
- Rip Rap

Borehole Stratigraphy:

- Gravel
- Sand
- Clay
- Silt
- Weathered Bedrock
- Bedrock

NOTES

1. Downstream excavation backfilled for positive drainage.
2. Upstream excavation to be backfilled by tailings during initial operations.



I / 04 **SECTION I - I'**

NOT FOR CONSTRUCTION

0 2 4 6 8 10
Scale in Metres

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
–	–		

REFERENCE DRAWINGS

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A	Issued for Review	MM	–	30May14

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MINTO MINE

SRK JOB NO.: 1CM002.018

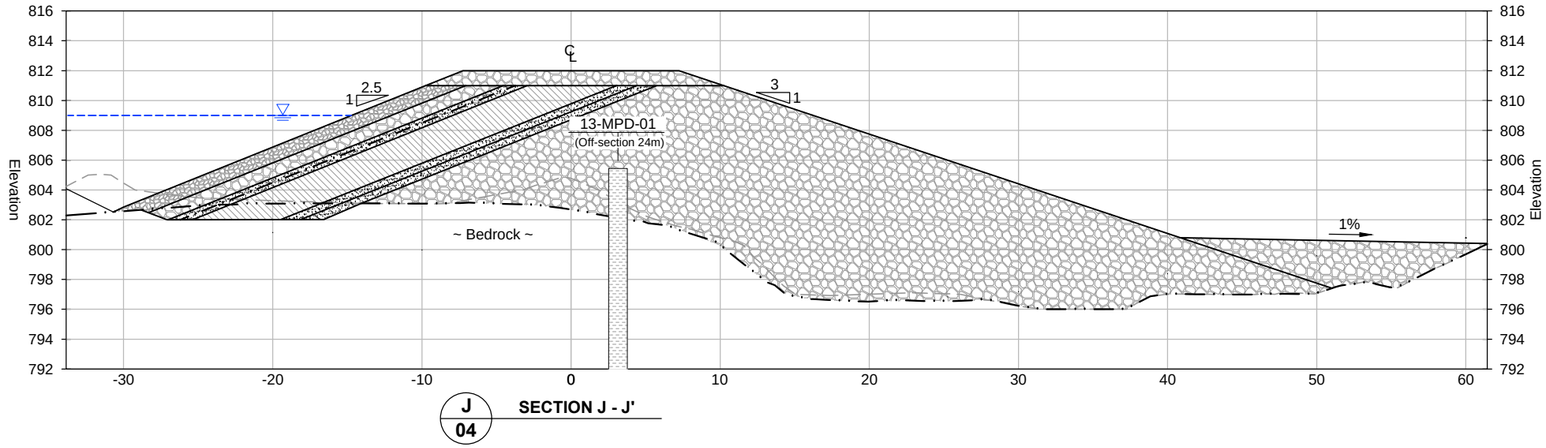
Preliminary Dam Design

DRAWING TITLE:

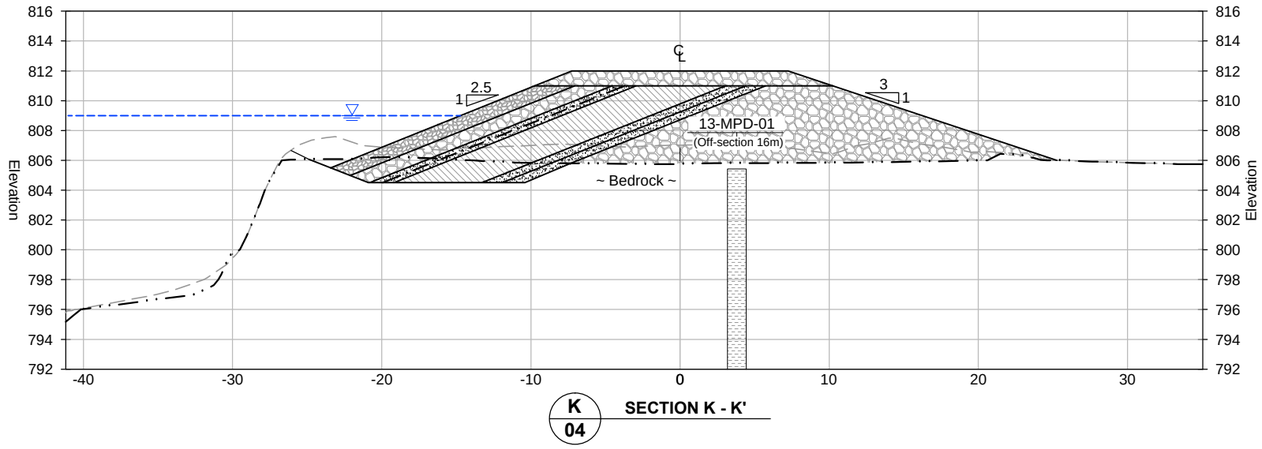
Dam Sections
(4 of 5)

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-12	12 of 18	B

C:\01_SITES\Minto\Dam Design\1CM002_018_710-3_RevB.dwg



J
04
SECTION J - J'



K
04
SECTION K - K'

LEGEND

- Existing Ground (Jan. 2014)
- Estimated Natural Ground
- Full Supply Level (El.809m)
- Liner

Dam Hatches:
- Core Material
- Transition Material
- Rock Fill
- Rip Rap

Borehole Stratigraphy:
- Gravel
- Sand
- Clay
- Silt
- Weathered Bedrock
- Bedrock

NOTES

1. Downstream excavation backfilled for positive drainage.
2. Upstream excavation to be backfilled by tailings during initial operations.

NOTE: Natural ground is bedrock in sections above.

NOT FOR CONSTRUCTION

0 2 4 6 8 10
Scale in Metres

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DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
–	–		

REFERENCE DRAWINGS

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EMR	EMR	July 2014

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OPERATED BY MINTO EXPLORATIONS LTD.

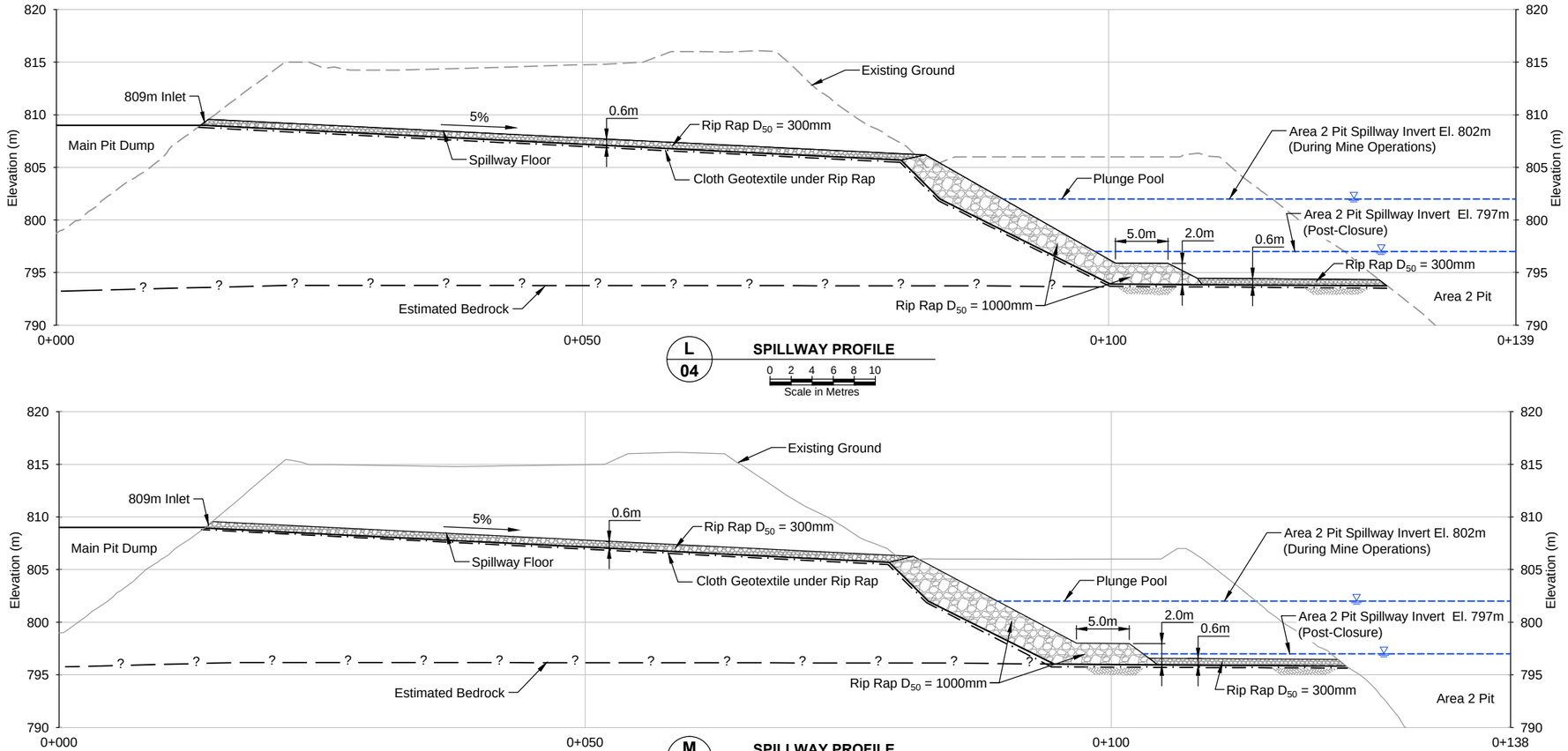
MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:
Dam Sections
(5 of 5)

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-13	13 of 18	B

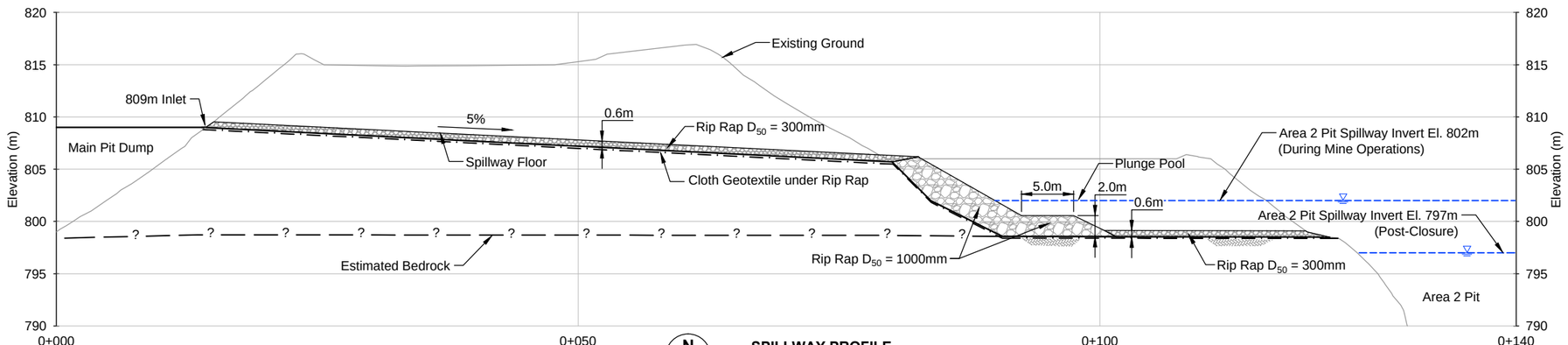


L 04 **SPILLWAY PROFILE**

0 2 4 6 8 10
Scale in Metres

M 04 **SPILLWAY PROFILE**

0 2 4 6 8 10
Scale in Metres



N 04 **SPILLWAY PROFILE**

0 2 4 6 8 10
Scale in Metres

NOT FOR CONSTRUCTION

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
–	–		

REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:

Spillway Profiles

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-14	14 of 18	B

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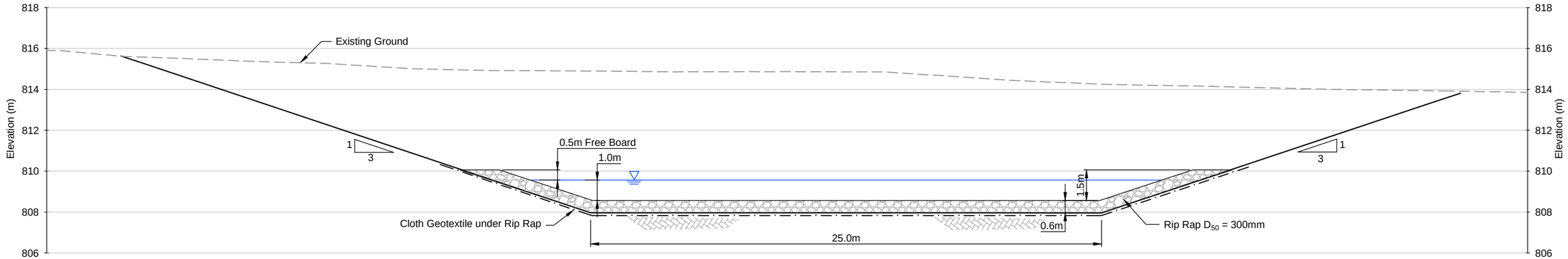
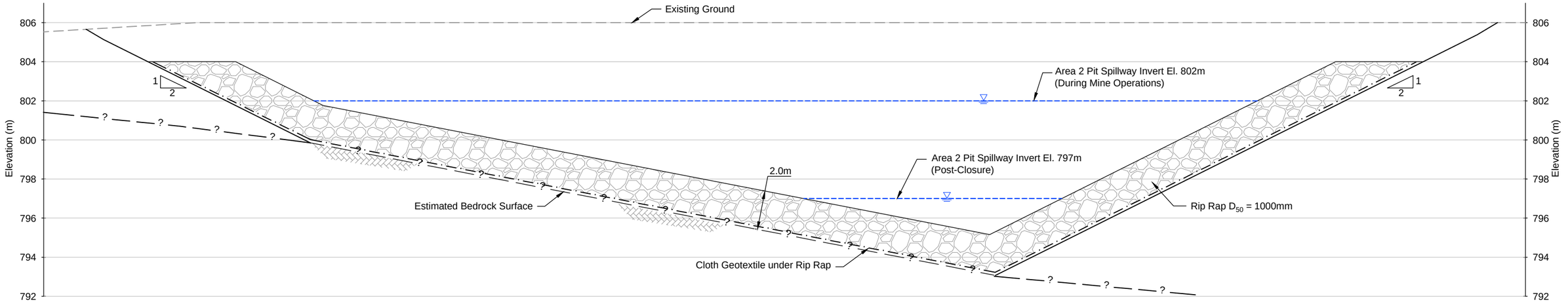


O 04 UPPER SPILLWAY CROSS SECTION

0 2 4 6 8 10
Scale in Metres



P 04 LOWER SPILLWAY CROSS SECTION

0 2 4 6 8 10
Scale in Metres

NOT FOR CONSTRUCTION

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
–	–		

REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:
Spillway Sections

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-15	15 of 18	B

C:\01_SITES\Minto\Dam Design\1CM002_018_710-3_RevB.dwg

Foundation Excavation Crest

Main Dam Crest El. 812m

Foundation Excavation Crest

Full Supply Level (El. 809m)

Main Pit Dump Design

385000 E

384900 E

384800 E

6945200 N

6945100 N

6945000 N

6944900 N

6944800 N

13-MPD-01

13-MPD-02

13-MPD-03

13-MPD-04

13-MPD-05

13-MPD-06

13-MPD-07

MD-SP-01

MD-SP-02

MD-SP-03

MD-SP-04

MD-SP-05

MD-SP-06

MD-SP-07

MD-SP-08

MD-SP-09

MD-SP-10

MD-SP-11

MD-SP-12

MD-SP-13

MD-SP-14

MD-SP-15

MD-SP-16

MD-SP-17

MD-SP-18

MD-SP-19

MD-SP-20

MD-SP-21

MD-SP-22

MD-SP-23

MD-SP-24

MD-SP-25

MD-SP-26

MD-SP-27

MD-SP-28

MD-SP-29

MD-SP-30

MD-SP-31

MD-SP-32

MD-SP-33

MD-SP-34

MD-SP-35

MD-SP-36

MD-SP-37

MD-SP-38

MD-TH-1

MD-TH-2

MD-TH

MD-TH-4

MD-TH-5

MD-INC-1

MD-INC-2

MD-INC-3

MD-INC-4

MD-VW-1

MD-VW-2

0+050

0+100

0+150

0+200

0+250

0+300

0+350

0+400

0+450

14.5m

LEGEND

- Surface Monitoring Location
- Vibrating Wire Piezometer
- Inclinometer Location
- Thermistor String Location
- Borehole Location

NOTES

1. Contour interval 1m.
2. Topography received from mine site January 2014.
3. Coordinate system shown is UTM Zone 8, NAD83.

NOT FOR CONSTRUCTION

0 5 10 15 20 25
Scale in Metres

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
REFERENCE DRAWINGS			

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MINTO MINE

SRK JOB NO.: 1CM002.018.710

Preliminary Dam Design

DRAWING TITLE:

Main Dam (812m)
Instrumentation Plan

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-16	16 of 18	B

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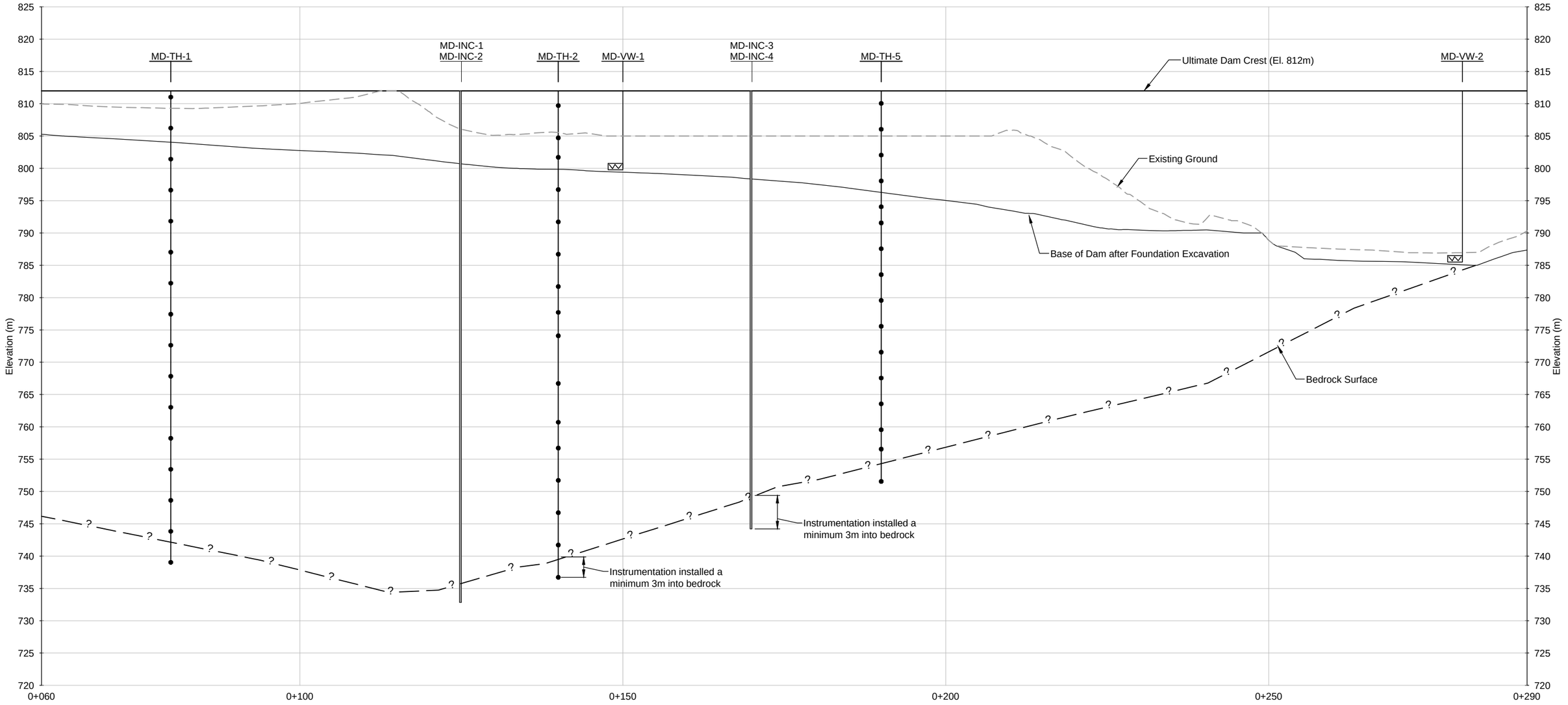

INSTRUMENTATION SHOWN ALONG DAM CENTERLINE PROFILE

0 2 4 6 8 10
Scale in Metres

LEGEND

- Existing Ground (Jan. 2014)
- Estimated Bedrock
- Thermistor Location
- Vibrating Wire Piezometer Location

NOT FOR CONSTRUCTION

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
–	–		

REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:
Instrumentation Shown on Dam Centerline Profile

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-17	17 of 18	B

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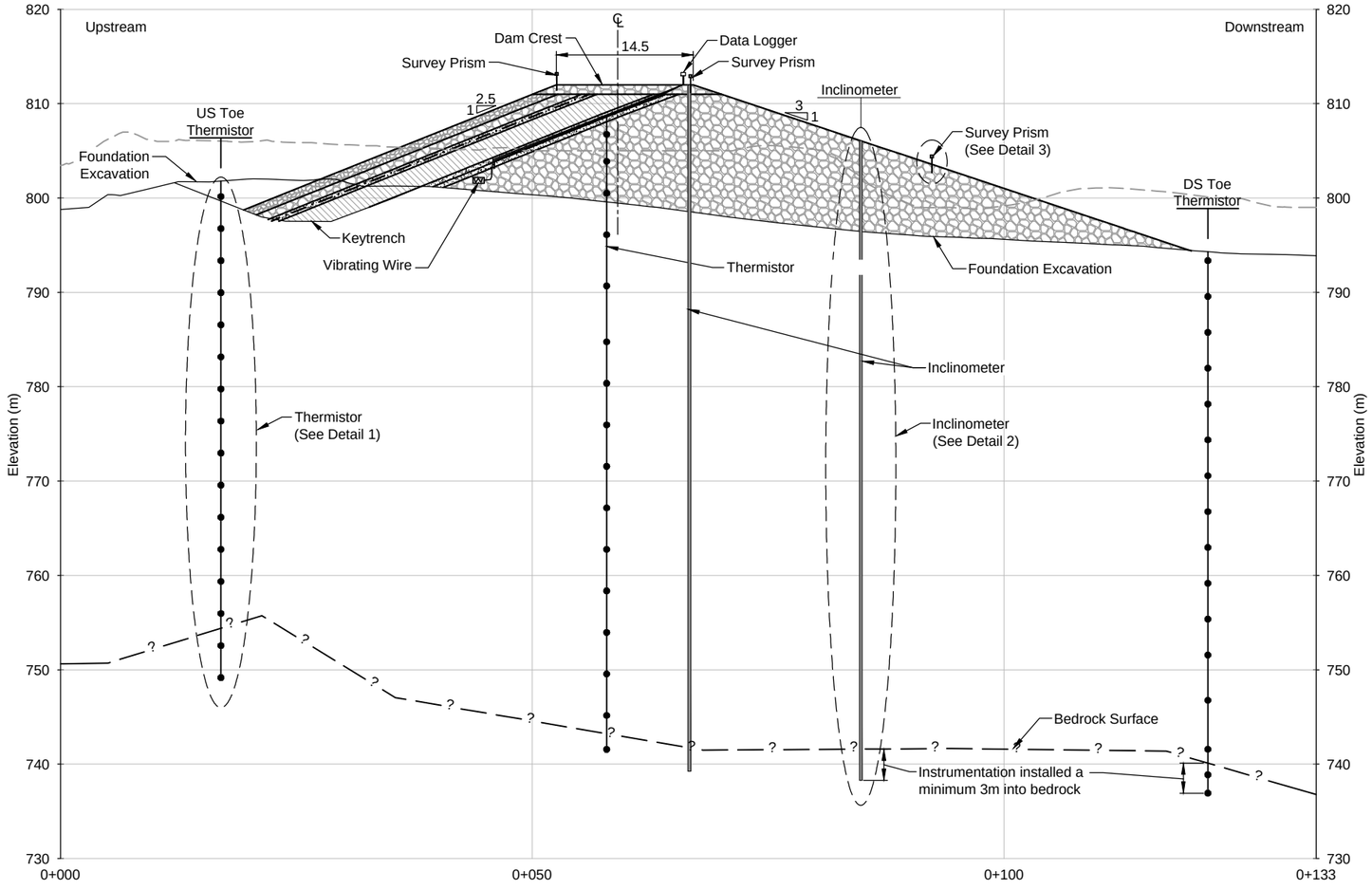
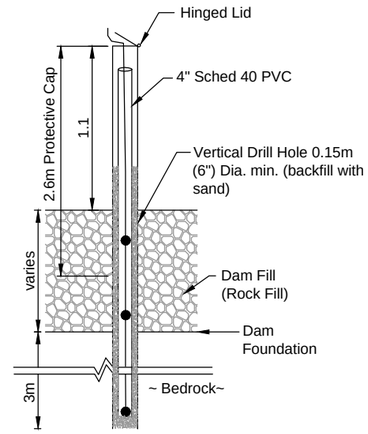


INSTRUMENTATION SECTION

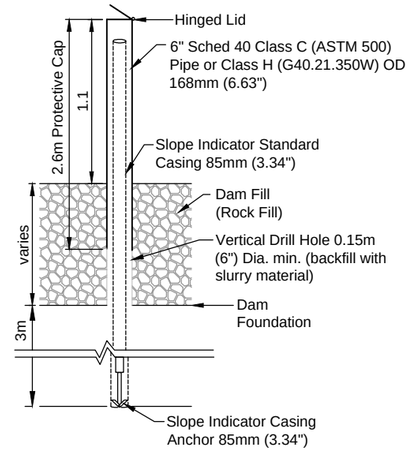
0 2 4 6 8 10
Scale in Metres

LEGEND

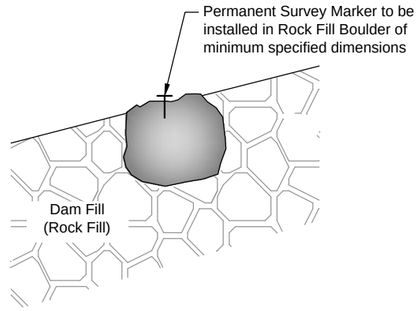
- Existing Ground (Jan. 2014)
- Estimated Bedrock
- Liner
- Core Material
- Transition Material
- Rock Fill
- Rip Rap



DETAIL 1
1 THERMISTOR INSTALLATION
Not to Scale



DETAIL 2
2 INCLINOMETER INSTALLATION
Not to Scale



DETAIL 3
3 SURFICIAL SURVEY POINT INSTALLATION
Not to Scale

NOT FOR CONSTRUCTION

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FILE NAME: 1CM002_018_710-3_RevB.dwg

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OPERATED BY MINTO EXPLORATIONS LTD.

MINTO MINE

SRK JOB NO.: 1CM002.018

Preliminary Dam Design

DRAWING TITLE:
Instrumentation Section and Details

DRAWING NO.	SHEET	REVISION NO.
SRK-MD-18	18 of 18	B

Appendices

Appendix A: Interpreted Bedrock Surface at the Minto Site

SRK Consulting (Canada) Inc.
2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

T: +1.604.681.4196
F: +1.604.687.5532

vancouver@srk.com
www.srk.com

Memo

To:	File	**Client:**	Minto Exploration Ltd.
From:	Kerry Ko	**Project No:**	1CM002.027
Cc:	Iozsef Miskolczi	**Date:**	July 17, 2014
Subject:	Interpreted Bedrock Surface at the Minto Mine Site		

1 Introduction

A bedrock surface for the Minto Mine was generated to improve understanding of the subsurface conditions and facilitate geotechnical analyses. This memo describes the methodology used to generate the bedrock surface and overburden isopach, and discusses the limitations of each.

2 Methodology

Borehole logs were reviewed and the top of bedrock was defined as the top of the weathered bedrock layer. In many instances, the weathered bedrock was described in the logs as residuum. Where data regarding residuum or weathered bedrock was not available, intact bedrock was used.

Site-wide bedrock elevation data was reviewed including Minto's exploration drillhole database, as well as geotechnical drilling and test pit program reports from 1976 to 2013. Geotechnical drilling reports reviewed included the following:

- Golder Associates, 1976. Geotechnical Investigation Minto Project Feasibility Study.
- EBA Engineering Consultants Ltd. 1994. Geotechnical Evaluation Mill and Camp Site.
- EBA Engineering Consultants Ltd., 1995. Geotechnical Design Tailings/Water Dam.
- EBA Engineering Consultants Ltd., 1996. Geotechnical Drilling Program.
- EBA Engineering Consultants Ltd., 1997. Geotechnical Program and Construction Inspection Reports.
- SRK Consulting (Canada) Inc., 2008. Waste Dump Overburden Drilling.
- SRK Consulting (Canada) Inc., 2010. Goundwater Baseline Conditions.
- ConeTec Investigations Ltd, 2010. Field Data Report.
- EBA Engineering Consultants Ltd., 2011. Summer of 2010 and Winter 2011 Drilling Services.

- EBA Engineering Consultants Ltd., 2012. Fall 2011 Drilling Services Results.
- SRK Consulting (Canada) Inc., 2013. Minto 2013 DSTSF Geotechnical Drilling Program Report.
- SRK Consulting (Canada) Inc., 2013. Ridgetop and Main Waste Dump Expansion Test Pit Investigation Results.
- SRK Consulting (Canada) Inc., 2014. Minto Main Dam Phase 1 Field Investigation.

For geotechnical drillholes or test pits where bedrock was encountered, UTM coordinates and depth to bedrock were compiled and primarily used to determine the bedrock surface. This data was supplemented with depth to bedrock data from Minto's exploration drilling database where the exploration drillholes provided improved resolution. Geotechnical drillholes were given priority in situations of conflicting data because the drilling methods provided better interpretation of stratigraphy at the bedrock contact.

Drillholes or test pits which were terminated before reaching bedrock were not used. In some cases, select drill holes were removed from the database where data showed significant inconsistency with surrounding drill holes. The compiled information was then used to create a 3D model in GEMS 6.5 (Dassault, 2013). The surface was created by method of triangulation with linear interpolation between data points.

Areas further away from current development and infrastructure typically have fewer drill holes; and therefore, provide less confidence in the modelled bedrock surface. Several areas had to be excluded because the confidence in the bedrock surface was low. As shown in Figure 1, the area 500 m east and southeast of the DSTSF is one of the low-confidence areas having few data points to confirm bedrock elevation.

3 Surfaces

3.1 Bedrock

The 3D bedrock surface generated in GEMS was exported to AutoCAD and a bedrock contour map was produced. The contour map (Figure 1) consists of a 5 m contour interval and shows the interpreted bedrock elevation. Figures 2 through 4 present the borehole data used to create the bedrock surface. The bedrock surface generally follows the pre-mining topography in the DSTSF area, with the bedrock trough slightly offset south of the valley bottom.

3.2 Overburden Isopach

The bedrock isopach was calculated as the thickness between the current topography (January 2014) and bedrock surface. The overburden isopach includes fill thickness as well as in-situ overburden. In the pit areas (Main Pit and Area 2 Pit), the maximum excavation surface was merged to topography. The pit areas are indicated to have no overburden and excludes the large amount of waste rock that have been disposed in the pit.

Several areas exhibit bedrock visible from the surface. These areas may not have been exposed naturally, but are now exposed or at shallow depth due to road surfacing. These outcrops were set to zero thickness for the purposes of displaying overburden isopach. Overburden isopach is presented in Figure 5.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

4 References

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Dassault Systèmes, 2013. GEMS integrated geology software package. Dassault Systèmes Vélizy-Villacoublay Cedex – France.

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SRK Consulting (Canada) Inc., 2008. Waste Dump Overburden Drilling. Report submitted to Minto Explorations LTD. SRK project number 2CM022.003.

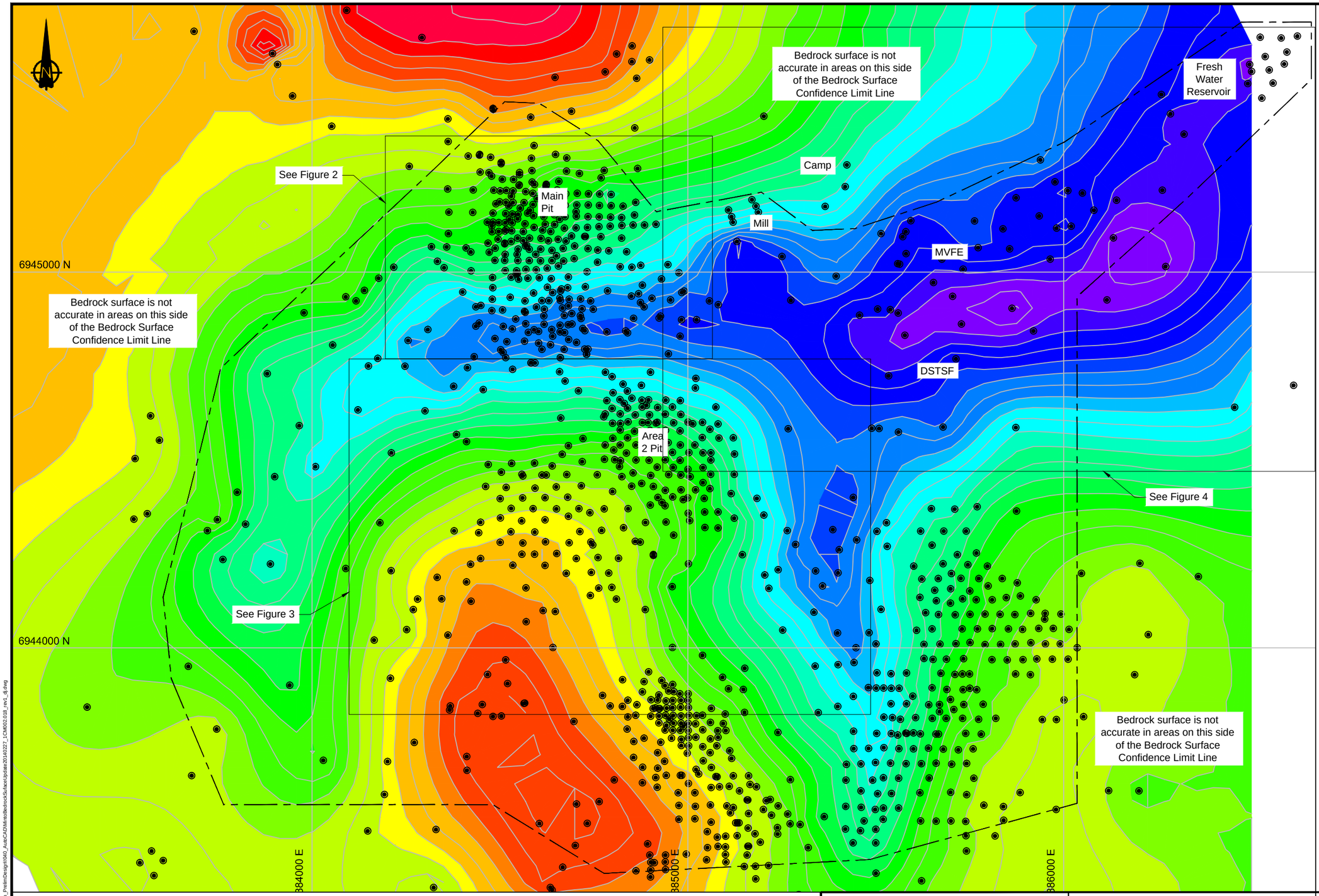
SRK Consulting (Canada) Inc., 2010. Groundwater Baseline Conditions. Letter submitted to Minto Explorations LTD. SRK project number 2CM022.017.

SRK Consulting (Canada) Inc., 2013a. Minto 2013 DSTSF Geotechnical Drilling Program Report. Report submitted to Minto Explorations LTD. SRK project number 1CM002.012.

SRK Consulting (Canada) Inc., 2013b. Ridgetop and Main Waste Dump Expansion Test Pit Investigation Results. Report submitted to Minto Explorations LTD. SRK project number 1CM002.012.

SRK Consulting (Canada) Inc., 2014. Minto Main Dam Phase 1 Field Investigation. Report submitted to Minto Explorations LTD. SRK project number 1CM002.013.

Figures



LEGEND

- Drillhole data used for bedrock surface
- Boundary of Higher Confidence Bedrock Surface Confidence Limi

Bedrock Elevation Range (m)	Color
690 to 680	
700 to 690	
710 to 700	
720 to 710	
730 to 720	
740 to 730	
750 to 740	
760 to 750	
770 to 760	
780 to 770	
790 to 780	
800 to 790	
810 to 800	
820 to 810	
830 to 820	
840 to 830	
850 to 840	
860 to 850	
870 to 860	
880 to 870	
890 to 880	
900 to 890	
910 to 900	
920 to 910	
930 to 920	
940 to 930	
950 to 940	
960 to 950	
970 to 960	
980 to 970	

NOTE

Bedrock surface created in Surfer 8.0 using Kriging gridding method.

0 100 200 300 400 500
Scale in Metres

srk consulting

SRK JOB NO.: 1CM002.027

FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

capstone MINTO MINE
OPERATED BY MINTO EXPLORATIONS LTD.

Minto Mine

Bedrock Surface Review

Bedrock Contour Map

DATE:	APPROVED:	FIGURE:
May 2014		1

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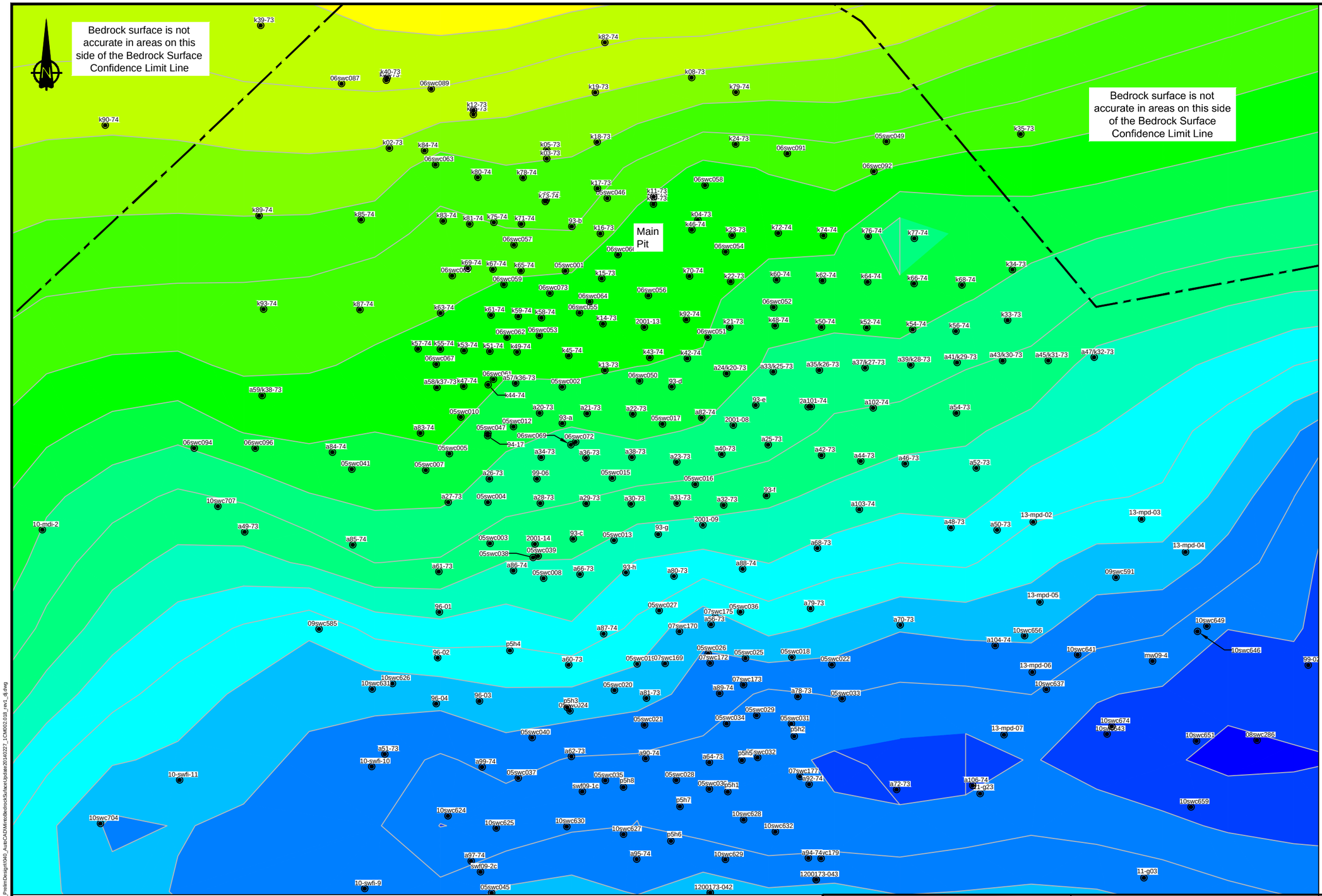

LEGEND

- Drillhole data used for bedrock surface
- Boundary of Higher Confidence Bedrock Surface Confidence Limit

Bedrock Elevation Range (m)	Color
690 to 680	
700 to 690	
710 to 700	
720 to 710	
730 to 720	
740 to 730	
750 to 740	
760 to 750	
770 to 760	
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910 to 900	
920 to 910	
930 to 920	
940 to 930	
950 to 940	
960 to 950	
970 to 960	
980 to 970	

NOTE

Bedrock surface created in Surfer 8.0 using Kriging gridding method.

0 20 40 60 80 100

Scale in Metres

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SRK JOB NO.: 1CM002.027

FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

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Minto Mine

Bedrock Surface Review

Bedrock Contour Map Main Pit Area

DATE:	APPROVED:	FIGURE:
May 2014		2

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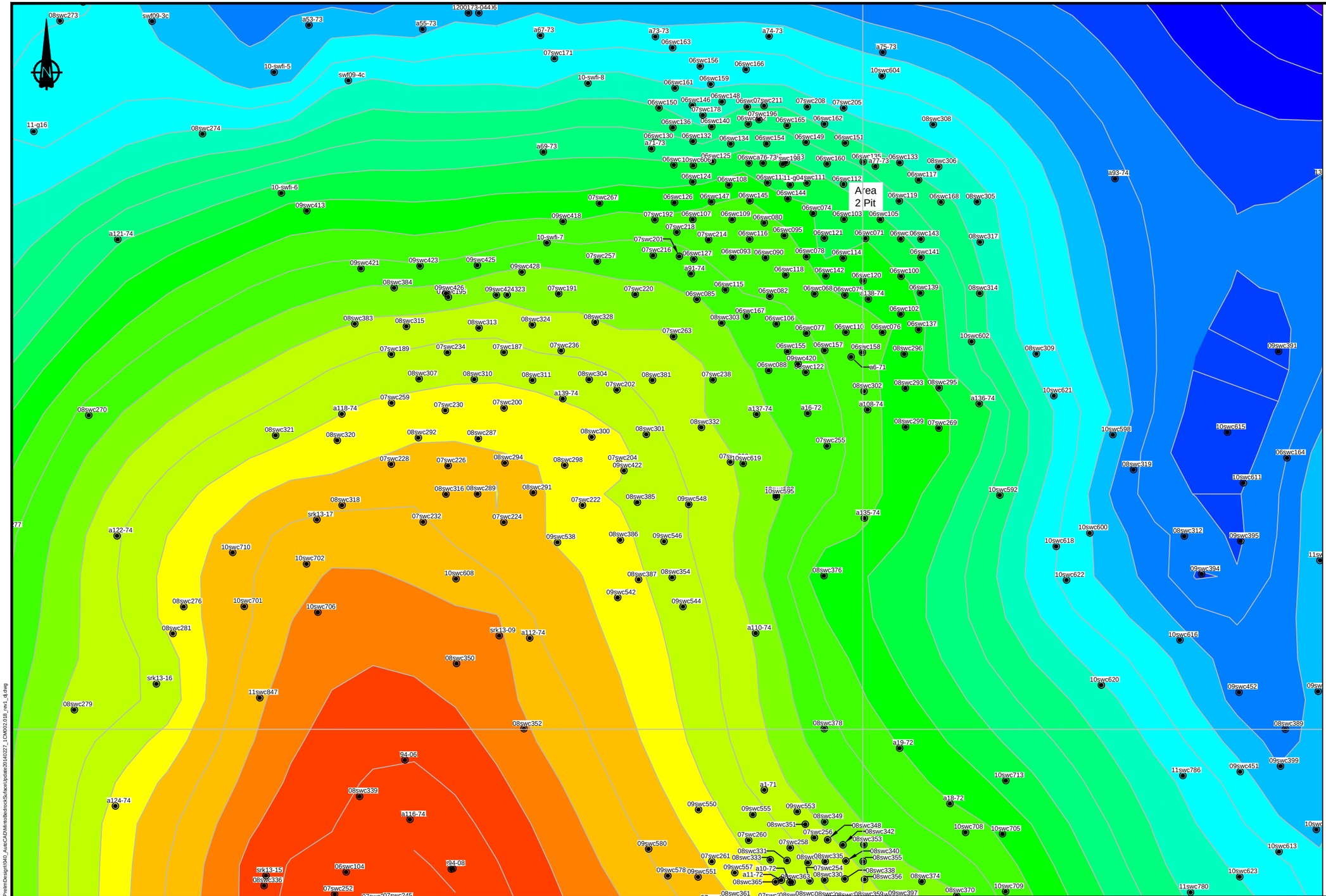

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- Drillhole data used for bedrock surface
- Boundary of Higher Confidence Bedrock Surface Confidence Limit

Bedrock Elevation Range (m)	Color
690 to 680	
700 to 690	
710 to 700	
720 to 710	
730 to 720	
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910 to 900	
920 to 910	
930 to 920	
940 to 930	
950 to 940	
960 to 950	
970 to 960	
980 to 970	

NOTE

Bedrock surface created in Surfer 8.0 using Kriging gridding method.

0 40 80 120 160 200

Scale in Metres

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SRK JOB NO.: 1CM002.027

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Minto Mine

Bedrock Surface Review

Bedrock Contour Map Area 2 Pit Area

DATE:	APPROVED:	FIGURE:
May 2014		3

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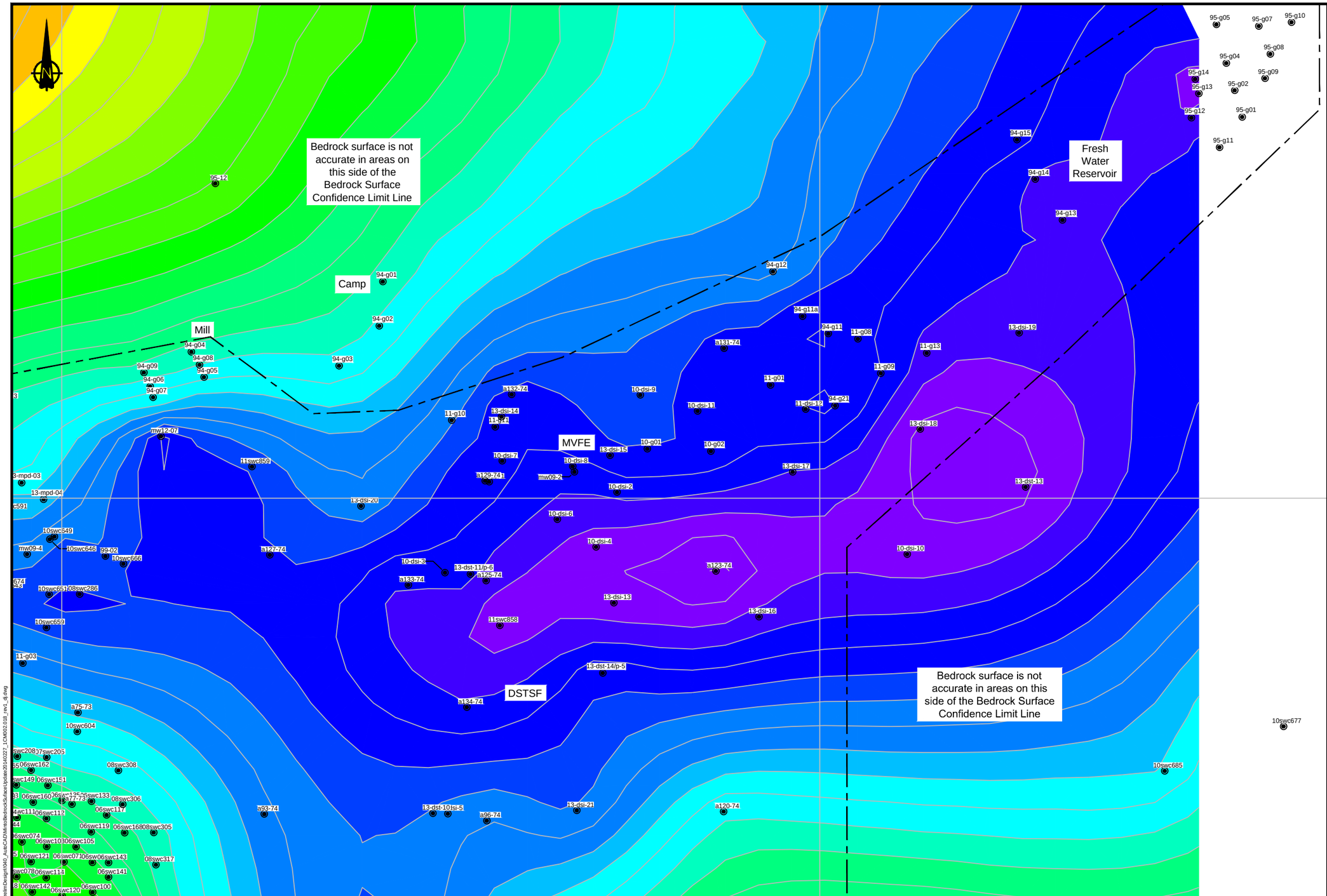

LEGEND

- Drillhole data used for bedrock surface
- Boundary of Higher Confidence Bedrock Surface Confidence Limit

Bedrock Elevation Range (m)	Color
690 to 680	
700 to 690	
710 to 700	
720 to 710	
730 to 720	
740 to 730	
750 to 740	
760 to 750	
770 to 760	
780 to 770	
790 to 780	
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910 to 900	
920 to 910	
930 to 920	
940 to 930	
950 to 940	
960 to 950	
970 to 960	
980 to 970	

NOTE

Bedrock surface created in Surfer 8.0 using Kriging gridding method.

0 50 100 150 200 250

Scale in Metres

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SRK JOB NO.: 1CM002.027

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Minto Mine

Bedrock Surface Review

Bedrock Contour Map DSTSF Area

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May 2014		4

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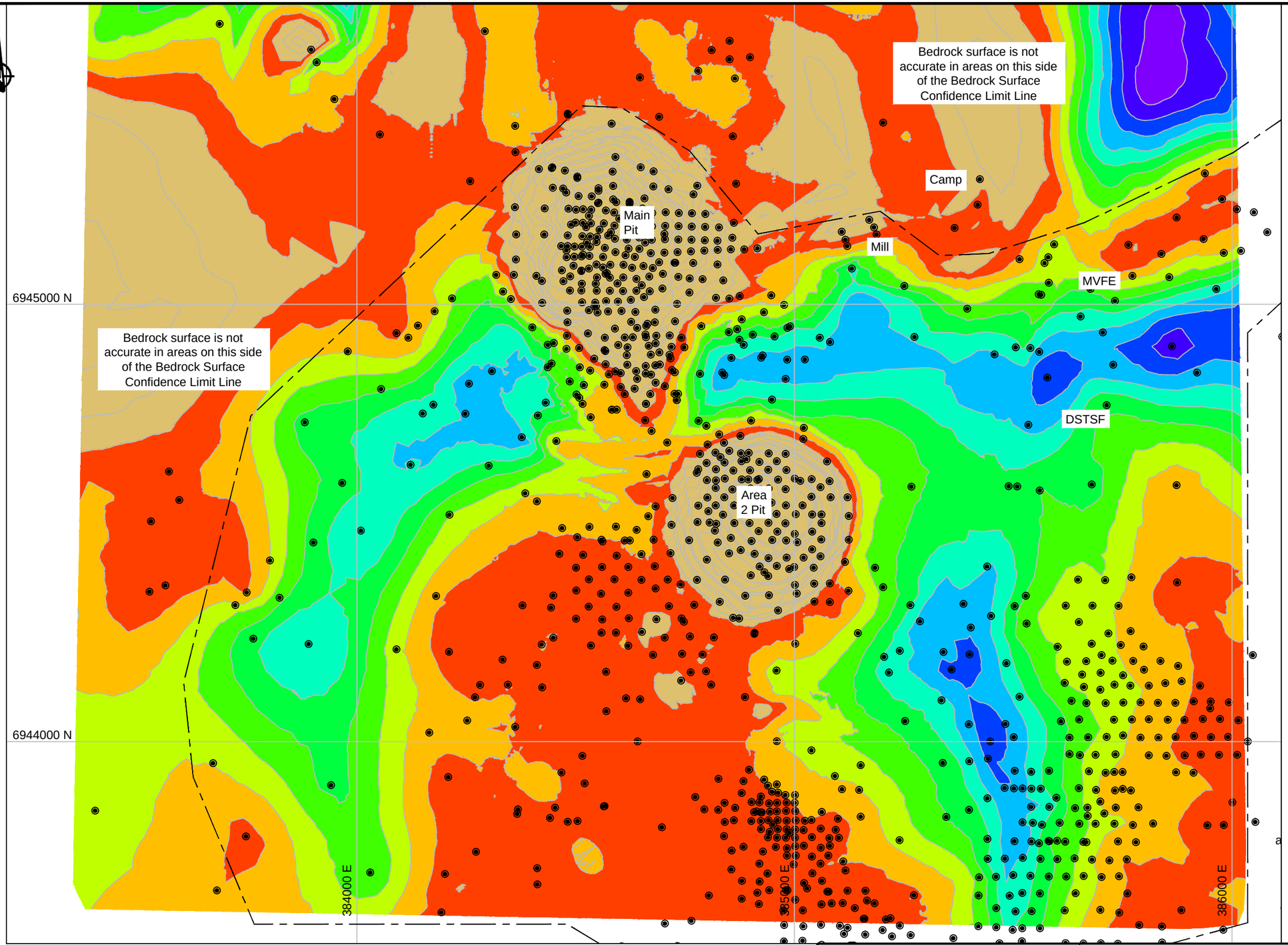

LEGEND

- Drillhole data used for bedrock surface
- Boundary of Higher Confidence Bedrock Surface Confidence Limit

Overburden Thickness Range (m)	Color
90 to 100	
80 to 90	
70 to 80	
60 to 70	
50 to 60	
40 to 50	
30 to 40	
20 to 30	
10 to 20	
0 to 10	
0	

NOTE

Bedrock surface created in Surfer 8.0 using Kriging gridding method.

0 100 200 300 400 500

Scale in Metres

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SRK JOB NO.: 1CM002.027

FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

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Bedrock Surface Review

Overburden Thickness Map (Excavated Pit and As-built)

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May 2014		5

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Appendix B: Main Dam Preliminary Design Alternatives

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Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Peter Luedke, Murray McGregor	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 15, 2014
Subject:	Main Dam Preliminary Design Alternatives		

1 Introduction

During the next phases of mining (Minto Phase V/VI), additional storage capacity will be needed for tailings and waste rock. The preferred solution to increase storage capacity is the construction of a containment dam in the Main Pit.

The conceptual design for the Main Dam was presented in SRK (2013a). The proposed dam is approximately 440 m long with a maximum crest height of 28 m and consists of a conventional earth fill design with a low-permeability core for seepage control. The Main Dam will retain both tailings solids and free water during its operational phase. At closure, the dam will not retain free water; however, surface runoff flow-through facilities will remain in place via a dedicated engineered spillway. A tailings beach on the upstream face of the Main Dam is expected to limit seepage during the operational phase; however, the ability to act as a water storage facility was set as a design criteria as a contingency.

The proposed dam design must also consider the challenging foundation conditions. The northern portion of the dam will be founded on weathered bedrock. The southern portion of the dam will be founded on warm permafrost consisting of fine-grained overburden with some ice-rich zones. The dam design must consider foundation settlement of the overburden, as well as consolidation associated with ice lenses and massive ice that may be present beneath the dam footprint.

In order to advance the preliminary design, an alternatives assessment was completed to determine the best dam design concept moving forward. The assessment began with the development of a comprehensive initial list of options based on SRK's experience at other sites. From this list, unsuitable or infeasible methods were eliminated from further consideration. Options passing this initial stage were then evaluated based on the design that will best suit the complex site foundation conditions, site material availability, constructability, and cost.

This memo outlines the selection process. The complete list of alternatives considered is presented in Section 2. The initial assessment results are presented in Section 3, and the preferred alternatives evaluation is presented in Section 4.

2 Considered Alternatives

The following sections provide a brief description of each alternative considered as part of this assessment.

2.1 Frozen Core

A frozen core dam typical section is provided in Figure 1. The concept of frozen core dams is to maintain both the core of the dam and the underlying foundation in a frozen state, which will provide an impervious barrier to water. They require that the core be constructed with soil containing sufficient fines to retain water and enable the pore water to freeze in a quasi-saturated state. Once saturated, the frozen pore water provides two benefits: strength and an impervious media.

The performance of a frozen core dam is dependent on the sustainability of the frozen conditions in the long term, and as a result, they are typically used in stable continuous permafrost regions. Passive (or active) freezing techniques with the use of thermosyphons are commonly needed to ensure frozen conditions are maintained.

2.2 Liner Systems

A liner system is placed on the upstream side of the dam to provide a low permeability layer and reduce seepage. The liner options consist of a geosynthetic (LLDPE, HDPE, PVC, GCL, EPDM, etc.), a bentonite amended sand layer, a bituminous geomembrane, or a concrete liner. If the liner is the only seepage control measure, the dam performance would be strongly dependent on the liner integrity and careful and diligent construction procedures, and QA/QC would be required.

Two examples of typical sections of a lined dam are provided in Figure 2. The top section includes a liner on the upstream face of the dam, while the liner in the bottom section has a chevron-shaped configuration.

2.3 Cut-off Wall System

An example cut-off wall typical section is provided in Figure 1. The purpose of a cut-off wall system would be to reduce seepage through the foundation materials. Different variants of the cut-off wall system include:

- Slurry Wall: A vertical trench is excavated and replaced with a slurry of low permeability material typically consisting of a mixture of the cement or bentonite with a portion of the excavated material. The trench would be located along the centerline of the dam and is installed after the bulk dam fill has been placed.

- Sheet Piles: Similar to the slurry wall option, steel sheet piles would be driven through the dam fill and foundation to provide a hydraulic barrier.

2.4 Fine Grained Core

A fine grained core dam is constructed with a low-permeability core of fine grained material to reduce seepage. Filter and drain rock would be required on the downstream side of the core to prevent migration of the fine grained core, to reduce pore pressure on the downstream slope and improve stability. The core is typically located in the center of the dam section for stability concerns since these materials typically have lower strength.

An example of a fine grained core dam typical section is provided in Figure 3.

2.5 Cycloned Tailings

A cycloned tailings dam would be constructed using coarse sand tailings that is produced through a cycloning process. Removal of the fines material allows the sand to be compacted and used as the embankment material. The fine underflow tailings from the cycloning process would be pumped into the Main Pit behind the retaining structure. Cycloned tailings dams often require small starter dams in order to streamline the construction process. Success of cycloned tailings retaining structures is strongly dependant on the tailings gradation coming from the mill.

An example cyclone tailings dam typical section is provided in Figure 3.

2.6 Waste Rock Shell

This option consists of a simple waste rock dam constructed with compacted waste rock with no additional seepage mitigation techniques. An example of a typical section is provided in Figure 4. Seepage through the dam would be controlled by the tailings beach on the upstream slope of the dam as well as a seepage pump back system (if required).

3 Initial Assessment

The advantages and disadvantages of each alternative is presented in Table 1. Combinations of select alternatives were also considered. Each alternative was assigned a preference rating of high, medium, or low.

The following alternatives have been classified as low and were not carried forward for further evaluation:

- A frozen core dam was eliminated as the permafrost is not stable and the warm climate would require significant supplemental ground freezing to be effective.
- Slurry wall or sheet pile cut off systems were eliminated as they are difficult to implement when the overburden depth is substantial and can be very costly.
- A concrete lined earth dam was eliminated due to the potential for differential settlement resulting in cracking of the liner.

- A cycloned tailings dam was eliminated due to the significant operational challenges. A cyclone process was assessed in the early stages of tailings alternatives assessment and it was indicated that obtaining sufficient underflow to construct a dam would be difficult (SRK 2013b).
- A waste rock dam with no seepage control was also eliminated due to the potential risk that storage of water may be required at an early stage of operation as a contingency.

Alternatives with a medium or high rating were carried through for further analysis and compared based on integrity, material requirements and relative cost.

Table 1: Comparison of Dam Design Alternatives

Dam Type	Advantages	Disadvantages	Preference
Frozen core	• Low permeability • Increased internal dam stability • Promotes frozen condition in soil foundation (no settlement issues while frozen conditions are maintained)	• Weather risks: warmer weather may result in thawing, and increased seepage/settlement • Thermosyphons (or an active freeze system) would be required • Additional instrumentation required to monitor and ensure frozen conditions are maintained • Inclusion of an active system would be costly	Low
Geosynthetic liner	• Wider range of bulk dam material can be used including run of mine rock • Liner is relatively low cost compared to dams with cores • Multiple liner options available depending on application requirements	• Liner integrity is key to dam performance • Long term liner integrity difficult to predict • Significant strain due to settlement can lead to poor seepage performance	Medium
Concrete lined	• Durable Liner • Simple construction • Concrete protects dam from wave run-up	• High cost • Highly susceptible to settlement resulting in cracking	Low
Slurry cut-off wall	• Simple design • Fast construction of bulk fill	• Significant installation costs • Difficult to assess key-trench requirements	Low
Sheet piles	• Good seepage characteristics • Improves both seepage and stability	• High cost • Constructability issues (It may be difficult to drive piles through run-of-mine materials, as a result, a finer grained core may be required).	Low
Fine grained core	• Commonly accepted design • Core design can be adjusted depending on seepage requirements • Robust. Allows for some settlement and maintains integrity over larger strains that liner systems	• Requires careful construction quality control due to core and filter grain size specification requirements • Increased construction duration due to core placement	Medium
Cyclone tailings embankment	• Use of tailings in dam construction reduces storage requirements • Less material needed for dam construction • Potential cost savings if operated correctly	• Additional operational support needed • Difficult operation in winter months • Increased liquefaction concerns • Starter dam using another alternative required • Freeboard needs to be modified throughout the design life since the structure height is changing	Low
Waste rock dam (no core/no liner)	• Low cost • Simple construction • Favourable stability under drained conditions	• Not suitable as a water retaining structure	Low
Liner and fine grained core	• Redundancy with liner and core material increases seepage performance over liner-only alternative • Can be conducive to downstream construction methods • Fine grained core acts as filter for the liner system	• More costly than a single retaining alternative • More complex to construct	High

4 Preferred Alternative Selection

A geosynthetic liner in combination with a fine-grained core was selected as the preferred dam alternative that best suits the complex foundation conditions while providing the ability to act as a water storage facility during the operational stage of the project. An example of this design is included in Figure 4.

The fine-grained core is required to provide robustness to allow for settlement. A thermal analysis of the conceptual main dam design estimated that if the Main Pit fills with water, there was a potential for significant settlement to occur at the upstream toe (SRK 2014). As a result, the dam must have the capability to tolerate a large amount of strain.

The addition of a geosynthetic liner would prevent significant seepage assuming liner integrity is maintained and provide redundancy in seepage protection. The combined system also helps mitigate potential piping effects caused by localized loss of integrity in the liner since the core would act as a filter to the liner system. The geosynthetic liner will marginally increase construction costs, but will substantially improve the robustness of the design.

5 Recommended Design

A flexible geomembrane such as a low density polyethylene (LLDPE) liner is recommended for the Main Dam due to the potential for some differential settlement to occur. The selected dam design should include the buried liner on the upstream face of the facility with a fine grained core underlying the liner system. Inclusion of the liner on the upstream slope simplifies construction compared to tie-ins associated with chevron type liner designs. The bulk of the dam would be constructed with run-of-mine rock. Transition zones between the fine grained core and run-of-mine will be required. The liner is to be keyed into bedrock or native fine grained overburden ground to limit seepage.

The geometry should be re-evaluated in conjunction with deformation and thermal analysis as the design is advanced.

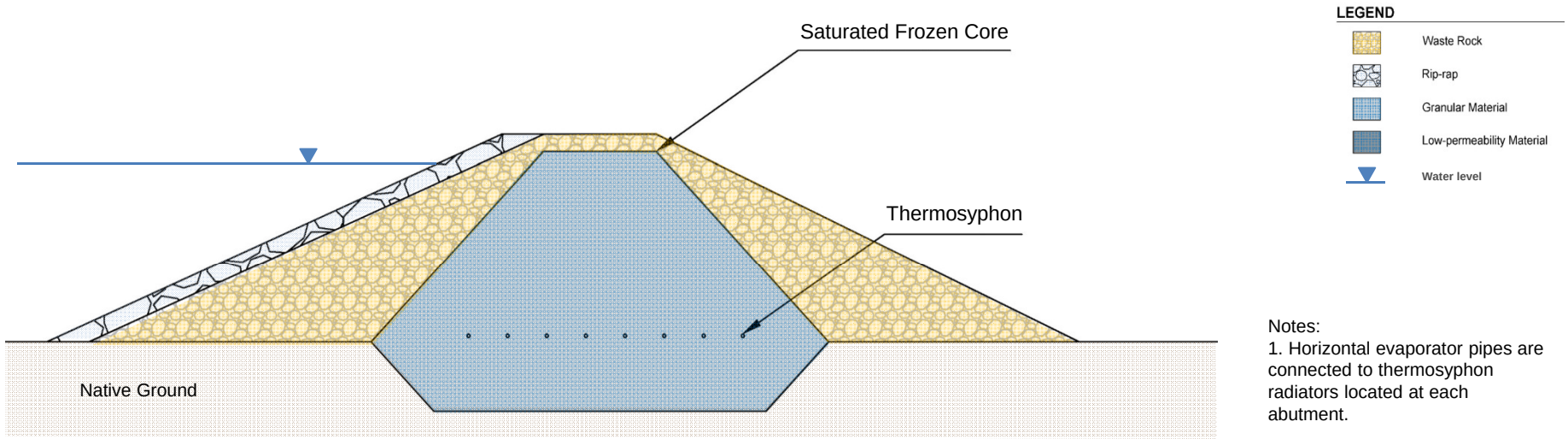
6 References

SRK Consulting (Canada) Ltd., 2013a. Main Dam Conceptual Design Report. Prepared for Minto Explorations Ltd. SRK Project Number 1CM002.010. June, 2013.

SRK Consulting (Canada) Ltd., 2013b. Alternate Tailings Disposal Methods for Minto Mine Phase V/VI Tailings Disposal in Main Pit - DRAFT. Prepared for Minto Explorations Ltd. SRK Project Number 1CM002.010. March 14, 2013.

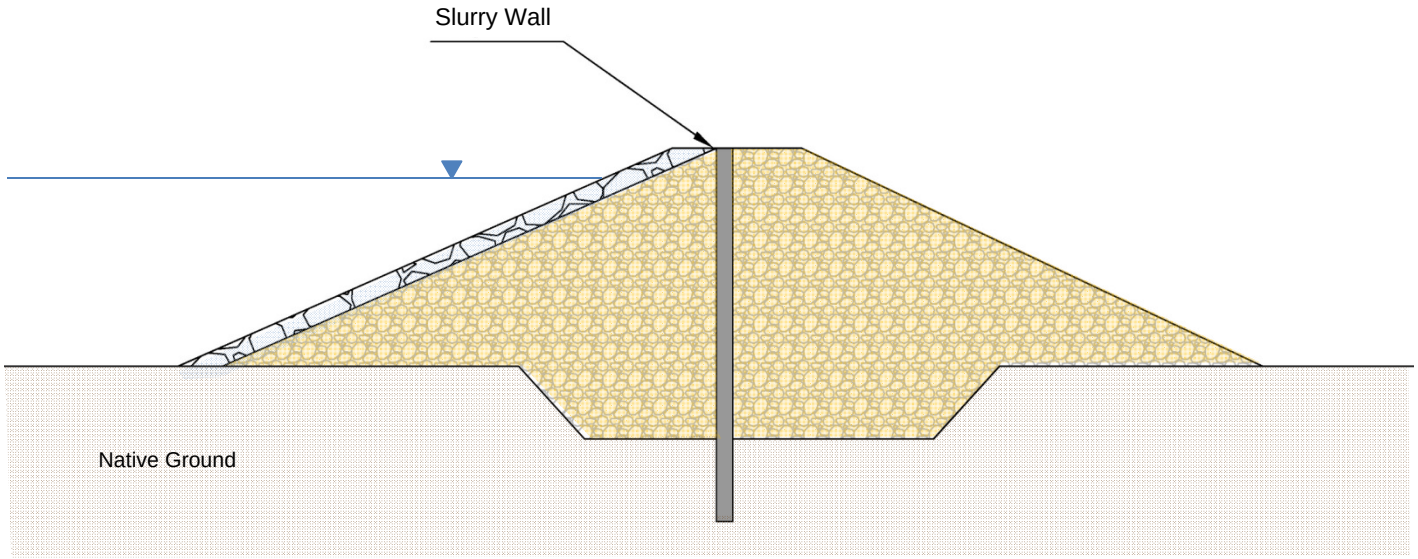
SRK Consulting (Canada) Ltd., 2014. Minto Main Dam Thermal Analysis and Settlement Predictions. Prepared for Minto Explorations Ltd. SRK Project Number 1CM002.018. July, 2014.

Figures



Notes:
1. Horizontal evaporator pipes are connected to thermosyphon radiators located at each abutment.

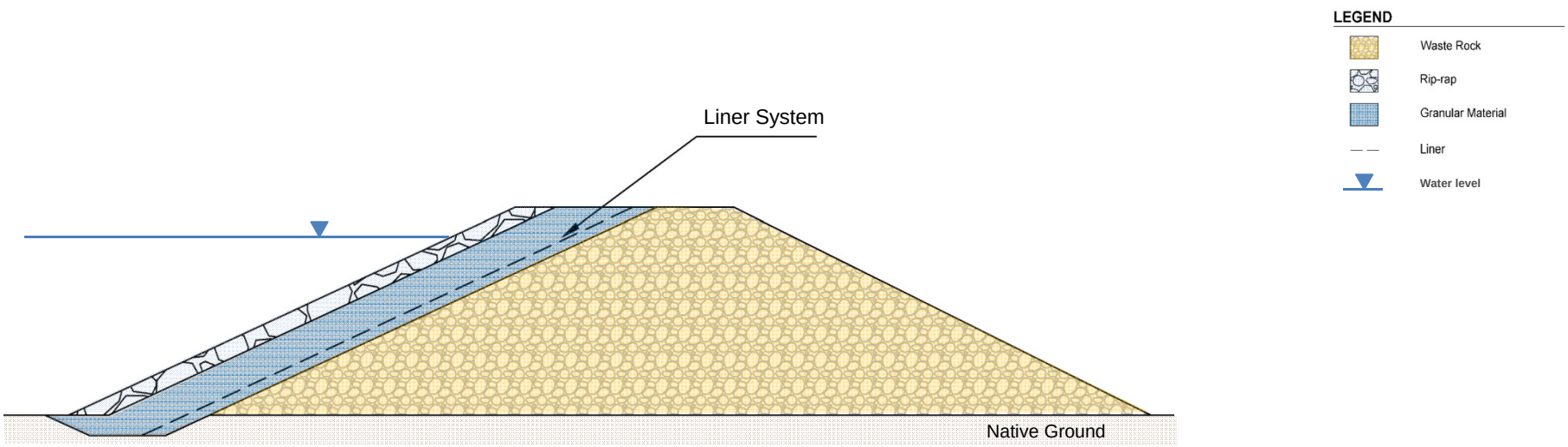
1) Frozen Core



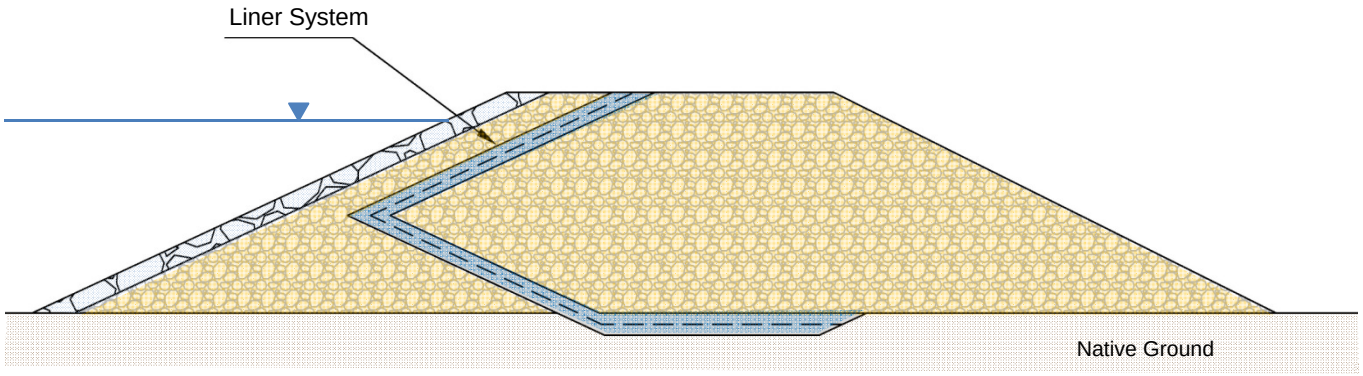
Notes:
1. A variant of this alternative is to use sheet piles in place of the slurry wall..

2) Slurry Wall

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Preliminary Design Alternatives
Job No: 1CM002.018 Filename: 1CM002.018_DamAlternativeFigures_rev06	Main Dam Preliminary Design	**Frozen Core and Slurry Wall Alternatives**
		Date: July 2014 / Approved: PM / Figure: **1**



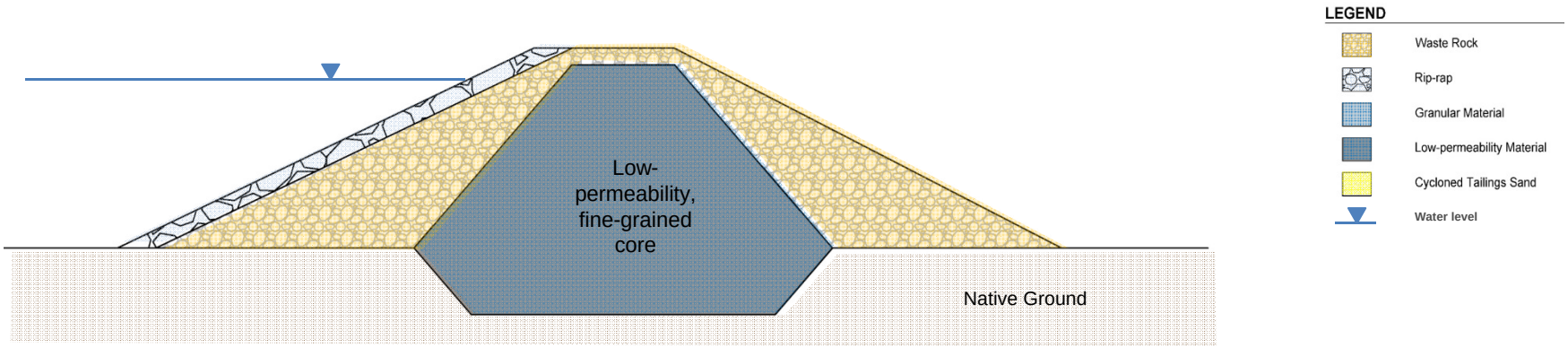
3a) Upstream Liner System



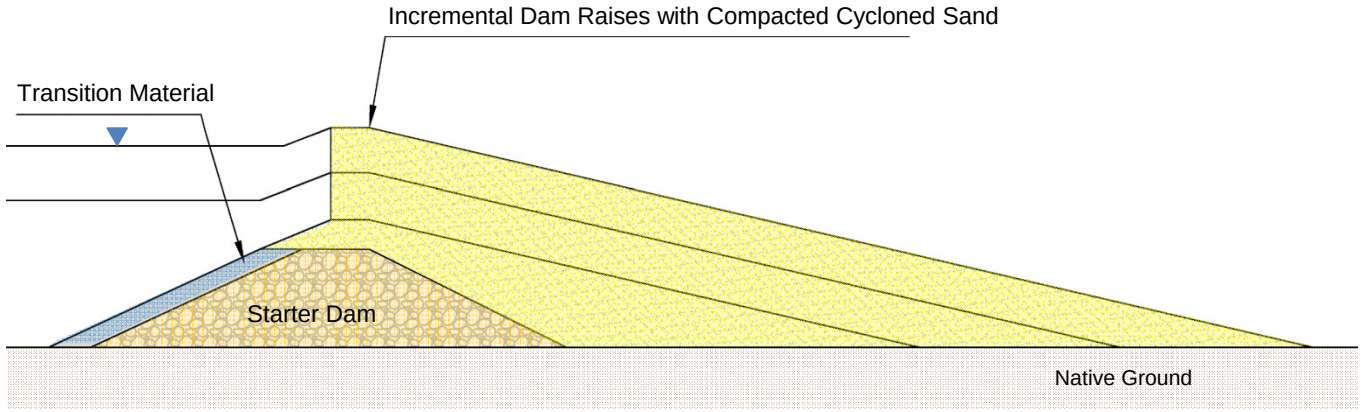
3b) Chevron-Shaped Liner

Notes:
1. Example liner variants include HDPE, LLDPE, GCL, bituminous geomembrane, PVC, etc.

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Preliminary Design Alternatives		
Job No: 1CM002.018 Filename: 1CM002.018_DamAlternativeFigures_rev06	Main Dam Preliminary Design	**Liner System Alternatives**		
		Date: July 2014	Approved: PM	Figure: **2**

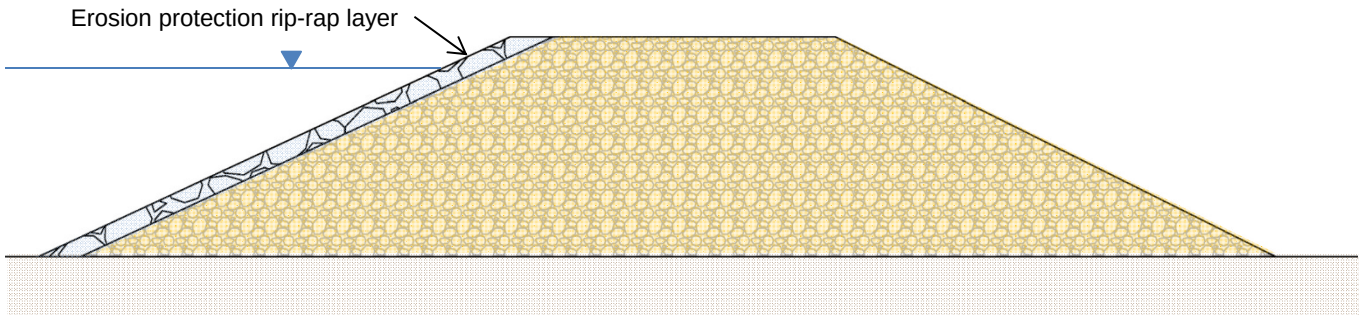


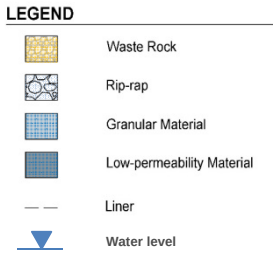
4) Fine Grained (low-permeability) Core



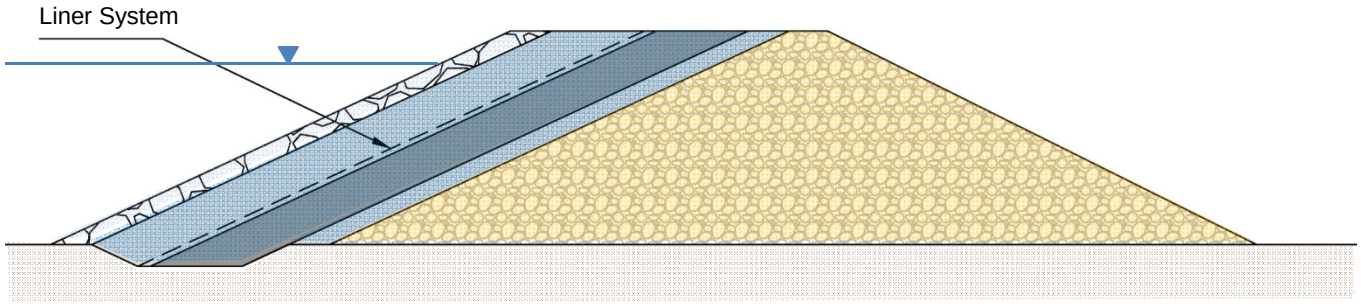
5) Cycloned Tailings

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Preliminary Design Alternatives
		Fine Grained Core and Cycloned Tailings Alternatives
Job No: 1CM002.018 Filename: 1CM002.018_DamAlternativeFigures_rev06	Main Dam Preliminary Design	Date: July 2014 / Approved: PM / Figure: **3**





7) Waste Rock Dam (No seepage control)



8) Liner and Low-permeability core combined

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Preliminary Design Alternatives
		Concrete Liner and Waste Rock Dam Alternatives
Job No: 1CM002.018 Filename: 1CM002.018_DamAlternativeFigures_rev06	Main Dam Preliminary Design	Date: July 2014 / Approved: PM / Figure: 4

Appendix C: Main Dam Thermal Analysis and Settlement Predictions

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Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Peter Mikes	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 15, 2014
Subject:	Main Dam Thermal Analysis and Thaw Settlement Predictions		

1 Introduction

During the next planned phases of mining (Minto Phase V/VI), additional storage capacity is required for storage of tailings, water (operational and surge capacity), and waste rock. Minto has identified the construction of a containment dam to increase the storage capacity of the Main Pit as the preferred solution.

The conceptual design of the proposed Main Dam was presented in SRK (2013). The dam will be located on the east side of the Main Pit and will retain both tailings and water during operations (Figure 1). At closure, the dam will not retain free water. The actual operational life is expected to be up to 10 years, and the operational design life is set at 20 years. After closure, the structure will remain in perpetuity.

Settlement of the foundation is expected as a result of conventional consolidation, as well as thaw consolidation associated with ice lenses beneath the southern portion of the dam where fine grained overburden is present. The ultimate thaw settlement is expected to be in the order of meters; however, this settlement is not expected to occur for a very long time.

This memo presents the thermal modelling analysis completed to predict the amount of thaw that may develop beneath the Main Dam foundation during its operational life and use that information to estimate the amount of thaw settlement that may be expected.

2 Methodology

2.1 Thermal Modelling Method

The thermal modelling was completed using the finite element model SVHeat version 6 developed by SoilVision Systems Ltd. and FlexPDE Version 6.32 developed by PDE Solutions Inc. Details of the model are available in the User's Manual of SVHeat (SoilVision Systems, 2004).

2.2 Thermal Model Input Parameters

This section presents details of the material properties and boundary conditions that were used in the analysis.

2.2.1 Soil Properties

The material properties used in the analyses are summarized in Table 1. The thermal properties are the same as those reported in EBA (2011) and were estimated based on soil index properties. These properties were found to be reasonable for use in the model calibration analysis presented in Section 2.3. The volumetric water contents were updated for this analysis and were representative of the soil conditions at the Main Dam (based on test results at bore hole 13-MPD-06; SRK (2014)).

Table 1: Material Properties

Material	Volumetric Water Content (%)	Bulk Density ($Mg.m^{-3}$)	Thermal Conductivity ($W.m^{-1}\ °C^{-1}$)		Specific Heat Capacity ($kJ.kg^{-1}\ °C^{-1}$)	
			Frozen	Unfrozen	Frozen	Unfrozen
Organics	47.0	0.94	0.70	0.42	1.89	2.94
Sand and Gravel	15.0	2.32	2.67	2.04	0.86	1.05
Silt	30.0	2.01	2.24	1.32	1.09	1.33
Silt and Clay	33.0	1.98	2.31	1.66	1.03	1.31
Weathered Bedrock	12.0	2.32	2.67	2.04	0.86	1.05
Bedrock	1.2	2.63	3.00	3.00	0.75	0.77
Waste Rock	6.3	2.12	1.41	1.38	0.77	0.83
Tailings	58.0	1.74	2.51	1.15	1.19	1.88

Source: EBA (2011)

Freezing point depression was not considered in this analysis and the freezing point for all materials was assumed to be 0°C. Pore water salinity tests of the foundation soils provided in SRK (2014a) resulted in low salinities ranging from 1.5 to 3.0 parts per thousand. As the thermal model defined the phase change for each material to occur between temperatures of -0.5 and -0.01 °C, this assumption is not expected to significantly influence the results.

2.2.2 Climatic Data

Estimated monthly air temperatures at the Minto site, for the period of 1971-2000, are presented in EBA (2011). The mean annual air temperature at site was estimated at -3.3°C. Based on the USACE (1998) methodology, this data results in an estimated mean annual ground surface temperature of -2.3°C with a temperature amplitude of approximately 6°C. A review of ground temperature measurements from site, at locations with sensors near the surface, found that site temperatures were generally warmer than those obtained by the USACE (1998) method. As a result, onsite ground temperatures were used to curve fit surface boundary conditions into the model.

The surface boundary condition was applied as a sinusoidal function of the form:

$$T_{surface} = n_i \times (MST + A_T \sin(2\pi \frac{t}{365})) \quad \text{(Equation 1)}$$

In Equation 1, MST is the median surface temperature (°C), A_T is an amplitude parameter (°C), and n_i is the surface index. The surface index is the thawing index (n_t) when ground temperatures are above zero, and the freezing index (n_f) when ground temperatures are below zero. The climate parameters are listed in Table 2 and are plotted in Figure 2a.

Table 2: Surface Boundary Condition Inputs

Surface Boundary Condition	Median ground surface temperature, MST (°C)	temperature amplitude, A_t (°C)	Surface thawing index, n_t	Surface freezing index, n_f	Average Annual Ground Temperature (°C)
1	-0.5	8	1.0	1.0	+0.3
2	-0.5	9	1.1	0.5	+2.1
3	-0.5	10	2.0	1.0	+3.9

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Each surface boundary condition was fit to provide good agreement with different ground temperature cables at different locations around the site. The locations of the ground temperature cables are provided in Figure 1.

Surface boundary condition 1 was found during model calibration (Section 2.3) to provide the best agreement to ground temperature cable 10-SDT-1 and good agreement with 08-SWC-271. These cables are located near the Southwest Waste Dump, and are generally less exposed to direct sunlight and are suspected to have significant snow cover during the winter.

Surface boundary condition 2 provides good agreement with ground temperature cables MWPT1 and MWPT2 located near the Mill Water Pond. This boundary condition was used as the base case in all model runs. These locations have waste rock fill at the surface and based on the ground temperatures are suspected to have a snow cover during the winter months. These locations may be partially protected from the wind and sunlight exposure from the nearby ore stockpiles, fill placement, and the mill.

Surface boundary condition 3 is based on two ground temperature cables (DST-4 and DST-5), located at the Dry Stack Tailings Storage Facility (DSTSF), that represent exposed conditions. Based on the low temperature readings from the cables during the winter months, it is suspected that the temperature sensors may be located above the ground surface and as a result, overestimate the summer ground temperature. This boundary condition was used during the sensitivity analyses to represent the worst-case condition.

Global warming was accounted for in all prediction models and was applied at a rate of +4.1°C per 100 years. This value is the 50 year (1958-2008) warming rate for the Yukon and Mackenzie

Valley regions published in CSA technical guide “Infrastructure in Permafrost: A guideline for climate change adaption”.

2.2.3 Geothermal Heat Flux

Geothermal heat flux is the flow of heat from within the earth to the surface. It was used in the thermal modelling as the boundary condition at the bottom of the model domain. The geothermal gradient is the change in ground temperature with depth and is related to the geothermal heat flux through the following formula:

$$q_{gt} = k\, i \qquad \text{(Equation 2)}$$

where q_{gt} is the geothermal heat flux (W m-2), k is the thermal conductivity (W m^{-1} °C^{-1}), and i is the temperature gradient (°C/km).

The geothermal heat flux was estimated to be 99 mW m^{-2} based on a geothermal gradient of 33°C/km and a bedrock thermal conductivity of 3.0 W m^{-1} °C^{-1}. The geothermal gradient was measured at four locations at site where ground temperature cables extend below the bottom of the permafrost. The ground temperature profiles used to derive the geothermal gradient are provided in Figure 3.

2.3 Thermal Model Calibration

Calibration analyses were completed to confirm that the material properties and surface boundary conditions were appropriate and that model results were in general agreement. The results are reported in EBA (2011).

Calibration was completed for the same two soil profiles used in EBA (2011) using ground temperature cables 08-SWC-271 and 10-SDT-1 located near the South Waste Dump. These cables have two of the longer data sets available from the site. 08-SWC-271 has available data between March 2008 and February 2010; 10-SDT-1 has available data between February 2010 and June 2013.

One dimensional soil profiles were established for each location based on the borehole logs. Mean ground surface temperatures were applied over a 30 year period with the surface -n factors adjusted until a good agreement was reached between the measured and predicted ground temperatures.

A number of analyses were also completed during calibration to determine the sensitivity of different material properties. The analysis found that the results were generally most sensitive to changes in the water content and were insensitive to changes in thermal conductivity and specific heat capacity. As a result, water contents for each material were updated and selected to represent a low range estimate of the moisture contents found at 13-MPD-06.

The results of the model calibration are provided in Figure 4. The figures also show the simplified soil profile used in each analyses. The models show very good agreement below depths of 5 m where there is no seasonable variability. Near the surface, mean annual surface temperature of

-0.5°C with an amplitude of 8°C, and freezing and thawing indices of 1.0 were found to provide the best agreement. Both ground temperature cables are located at the toe of South West Dump, and are located in an area where the conditions are expected to receive less direct sunlight, wind, and greater snow accumulation compared to the Main Dam. As a result, the thermal simulations at the Main Dam used slightly different surface boundary conditions. The Main Dam surface boundary condition details are described in Section 2.2.2.

2.4 Dam Geometry

The locations of the proposed Main Dam and the section used in the thermal analysis are shown in Figure 1. The dam is a rockfill structure with an internal geomembrane liner system with a fine-grained core that is keyed into fine grained overburden and/or bedrock. The dam has a crest width of 14.5 m, a 2.5H:1V upstream slope and a 3H:1V downstream slope. The dam has a crest elevation of 807 m, with a provision to raise the crest height to 812 m, if required. The key trench has a variable depth with a base width of 7 m and 2H:1V side-slopes. During operations, a tailings deposition strategy will be implemented to develop a tailings beach upstream of the dam.

A geotechnical field investigation was completed in 2013 to characterize the foundation soils (SRK, 2014a). The dam is located at the edge of a paleochannel, with overburden thicknesses ranging from 0 m to 60 m along the southern abutment. Borehole logs have confirmed the presence of ice lenses and permafrost. A thick zone of unconsolidated fill (operationally referred to as the “Boulder Laydown”) covers most of the southern abutment. This material was placed during pit development and consists of oversized waste rock and random fill. The fill material will be excavated prior to the dam construction.

The thermal analysis was completed through a section that intersects borehole 13-MPD-06, where the most ice was found during the 2013 investigation. The overburden at this location is 40 m thick and generally consists of frozen fine grained silts and clays with a 4 m sand and gravel layer located approximately 7 m below the original surface. The ice is typically described as containing randomly oriented ice lenses with 0 to 10 percent excess ice, and ice lenses that are up to 0.55 m thick. Moisture contents ranged from 12 to 45 percent.

Figures 5 and 6 show the thermal section stratigraphy and geometry at three points in time, a) prior to mining activities, b) during mining of the Minto Pit, and c) following construction of the Main Dam. Several assumptions and simplifications were made to the geometries:

- Fine grained soils were grouped into major soil units based on their ice and moisture contents.
- The dam fill excluded the fine-grained core and was modelled as waste rock. This assumption is expected to overestimate the thaw depth as, during the initial years, a portion of the dam fill is expected to freeze and subsequently, the fine-grained core would thaw slower compared to the waste rock.
- During mining of the Main Pit (between 2008 and 2015), the south wall buttress and waste rock fill in the pit was omitted for simplicity. This simplification would lead to warmer

temperatures along the pit face during this period, but as seen in Figure 7, it was found to have no impact to the thermal regime beneath the dam area.

- During dam construction, the boulder laydown was assumed to be completely removed. This assumption is expected to overestimate the thaw depth as the additional fill upstream and downstream of the dam would be expected to provide thermal insulation.

2.5 Initial Conditions

The initial conditions for the thermal analyses were completed in two stages to approximate the ground temperature conditions at the time of the dam construction.

A model of the pre-mining stratigraphy was simulated for a period of 30 years until no year to year variations were observed. Figure 7a provides the ground temperature contours following this period. The initial conditions for this model specified a temperature -0.8°C at elevations above 764 m with temperatures increasing at a rate of 33°C/km with depth below this elevation. These conditions were selected to match the typical permafrost temperature and thickness to the base of permafrost measured at site.

The second stage modelled the excavated Main Pit with the current ground surface that includes the 'boulder laydown'. This stage ran for a period of 7 years to be representative since the pit was excavated (2008 to 2015). Figures 7b and 7c provide the ground temperature contours at the start and end of this simulation. The fill placement over the original ground was assumed to be placed instantaneously with an initial temperature of +5C. For simplification, waste rock placement in the pit (e.g. the South Wall Buttress) was not included. Based on Figure 7c, the omission of this waste rock did not impact the temperature contours near the dam location.

2.6 Modelled Scenarios

Table 3 lists the Main Dam scenarios analyzed. For all cases, the dam was assumed to be constructed in May 2015 with the fill placed at an initial temperature of +5°C.

The base case tailings beach scenario assumes that tailings are placed up to the maximum storage capacity with an initial temperature of +15°C immediately following construction. This simplification applies a conservatively large thermal load upstream of the dam.

The pit lake scenario is considered to be the worst-case condition. It assumes that the Main Pit fills with water up to the spillway elevation of 804 m during the first freshet in May 2015 and remains there indefinitely (the design freeboard is 3 m). The assumed water temperature boundary condition is provided in Figure 4. The water temperature was modelled to be +4°C during the fall, winter and spring seasons with temperatures up to +14°C during the summer. This scenario applies a year round heat source that would result in the highest rate of thaw possible.

Table 3: Main Dam Scenarios

Scenario	Description
Tailings Beach	807 m crest dam with a tailings beach developed up to 805 m.
Pit Lake	807 m crest dam with the Main Pit filled with water up to the spillway elevation of 804 m.
Volumetric Water Content Sensitivity	Sensitivity Analysis – 807 m crest dam with a lower rage estimate of volumetric water content in the foundation soils (silt volumetric water content lowered from 0.3 to 0.24).
Surface Boundary Condition Sensitivity	Sensitivity Analysis – 807 m crest dam with warmer, worst case surface boundary conditions (Surface Boundary Condition #3).
812 m Dam Sensitivity	Sensitivity Analysis – 812 m crest dam with a tailings beach developed up to elevation 810 m.

Three sensitivity analyses were completed on: the height of the dam, the volumetric water content (VWC) of the foundation soils, and the surface boundary condition.

The first sensitivity analysis raised the crest height of the dam to an elevation of 812 m. This case was completed in the event that ice entrainment in the tailings results in the need to raise the dam to expand the storage capacity. The 812 m dam geometry maintained the key trench in the same location and a downstream construction method was used. The dam was assumed to be constructed to the full height in 2015.

The VWC sensitivity analysis lowered the water content in the silt layer immediately below the dam to determine its effect on the rate of thaw. During the 2013 geotechnical field investigation (SRK 2014a), moisture contents on the silt in borehole 13-MPD-06 were found to be 27% and 45%. The borehole log for the 27% sample noted no excess ice, and the 45% sample estimated 10% excess ice content. The base case tailings beach model conservatively selected a moisture content of 30% to represent the silt unit based on the test results and borehole log descriptions of the ice content and lenses. Lower moisture contents would result in higher thaw rates as less energy would be required to thaw the soil. This sensitivity analysis used a VWC of 24%, which was one of the lowest moisture contents measured from the silt and clay samples from the field investigation.

This surface boundary condition sensitivity analysis applied a warmer ground surface boundary condition to the tailings beach scenario. The boundary condition (surface boundary condition #3) was based on data collected from a ground temperature cable located in the DSTSF area. The sensors that this boundary condition is based on are suspected to be located above the ground surface as the readings suggest that there is no snow cover and very warm summer temperatures[1]. As a result, this boundary condition is expected to be conservative and will result in an overestimation of the rate of thaw.

[1] Surface temperature data is typically collected 1 to 2.5 cm below the ground surface.

2.7 Thaw Settlement Estimation Methodology

Settlement caused by thaw will occur as a result of the phase change from ice to water, and as a result of the flow of excess water out of the soil. Additional settlement may also occur due to the consolidation of the thawed soil as a result of additional loading on the thawed soil due to the dam fill and tailings placement. This analysis estimated the thaw settlement at the dam centerline, and at the toe of upstream and downstream slopes. Settlement caused by consolidation is addressed separately (SRK 2014b).

The thaw settlement was estimated based on the excess ice content estimated at borehole 13-MPD-06 (Figure 1). This borehole was found to contain 5.1 m of total excess ice within the 40 m thick overburden. This borehole intercepts the middle of the anchor trench in the analyzed section and is approximately midway between the upstream toe and centerline for the 807 m crest dam.

Based on the 13-MPD-06 borehole log, the thickness of excess ice per m (m/m) was estimated at 0.1 m increments. The cumulative ice content (in meters) with depth was then tabulated. The thaw settlement for a given year was then calculated to be equal to the cumulative ice content of the unfrozen zone provided by the thermal model.

As the original ground at the downstream toe is lower in elevation compared to 13-MPD-06, the depths of the ice lenses and descriptions were adjusted downwards to match the top of each major soil unit. For conservatism, the 4 m thick sand and gravel layer in 13-MPD-06 was not assumed to be present at the downstream toe. This layer was not identified in the borehole log of MW-09 that is located approximately 75 m to the east of 13-MPD-06.

This analysis methodology provides a rough estimate of settlement and rates of thaw as, for simplicity, soil units were grouped together into major units and distinct ice lenses were ignored in the thermal models. This is a source of error in the estimates as the rate of thaw will be less in ice layers due to the latent heat of water. However, as the water contents specified in the model were conservatively low, the modelled thaw depths are expected to be overestimated, and the calculated settlements will therefore be conservatively high.

3 Results

Tables 4 and 5 summarize the results of the thermal analysis and thaw settlement estimates. Further details on the results are presented in the following sections.

Table 4 presents the changes to the active layer thickness (relative to the base of the dam) 20 and 50 years following dam construction. The maximum thaw depths for a given year were measured at three locations: at the upstream toe, the dam centerline, and at the downstream toe. The thawed ground was defined by the zero degree contour.

Table 4: Predicted Foundation Thaw Depth Change

Scenario	Description	Upstream Toe (m)		Dam Centerline (m)		Downstream Toe (m)	
		Year 20	Year 50	Year 20	Year 50	Year 20	Year 50
1	Tailings Beach	-2.3	-0.6	-1.5	3.2	0.4	4.2
2	Pit Lake	10.2	14.7	-0.3	4.7	0.8	4.2
3	Sensitivity Analysis 1 – 812 m crest dam, tailings beach	-2.2	-2.2	-1.3	0.0	1.1	4.4
4	Sensitivity Analysis 2 – Lower VWC	-2.3	0.4	-1.4	3.7	1.1	4.7
5	Sensitivity Analysis 3 – Surface Boundary Condition	-2.3	0.0	-0.1	4.8	1.5	6.3

Source: J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Model Spreadsheets\NewPrelimDam Section_Settlement.rev02.xlsx

Table 5 presents the predicted thaw settlement 20 and 50 years following construction based on the thaw depths highlighted in Table 4. Where fill and/or tailings are placed as part of dam construction (i.e. the upstream and centerline locations), the calculated thaw settlements are relative to the top of the foundation soils. The active layer that freezes at a result of the fill placement, is assumed to develop the same ice conditions as 13-MPD-06.

Table 5: Estimated Thaw Settlement

Scenario	Description	Upstream Toe (m)		Dam Centerline (m)		Downstream Toe (m)	
		Year 20	Year 50	Year 20	Year 50	Year 20	Year 50
1	Tailings Beach	0.0	0.2	0.0	0.5	0.1	0.6
2	Pit Lake	0.9	2.3	0.0	0.5	0.1	0.4
3	Sensitivity Analysis 1 – 812 m crest dam, tailings beach	0.0	0.0	0.1	0.3	0.0	0.5
4	Sensitivity Analysis 2 – Lower VWC	0.0	0.3	0.0	0.5	0.2	0.7
5	Sensitivity Analysis 3 – Surface Boundary Condition	0.0	0.2	0.3	0.5	0.2	1.0

Source: J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Model Spreadsheets\NewPrelimDam Section_Settlement.rev02.xlsx

3.1 Tailing Beach Scenario

Figure 8 provides the predicted temperature distribution for the tailings beach scenario during the late summer in years 0, 3, 8, 30 and 50 following placement of the dam and tailings. During the initial years of the simulation, the tailings freeze to the surface upstream of the dam. As surface temperatures warm due to the applied climate change, the frozen tailings act to slow the rate of thaw on the upstream slope due to the high ice content in the tailings and latent heat effects.

Figure 9 provides a summary of the yearly active layer depth and settlement prediction for the tailings beach scenario. During the 20 year design operation life of the dam, the active layer (relative to the base of the dam) decreases due to the thermal insulation provided by the tailings

and fill placement. At the downstream toe, the active layer increases by 0.4 m during the first 20 years, this is mostly caused by no fill placement at the toe and the removal of the boulder laydown fill. After 50 years, the active layer increases by 4.2 m at the downstream toe.

The thawing of the foundation soils is not predicted to result in any settlement during operations at the upstream toe and dam centerline. Minor settlement of 0.1 m is predicted at the downstream toe where no fill placement has occurred. By year 50, the predicted thaw settlement ranges from 0.2 to 0.6 m.

3.2 Pit Lake Scenario

Figure 10 provides the predicted temperature distribution for the pit lake scenario during the late summer in years 0, 3, 8, 30, and 50. The pit water, directly upstream of the dam, provides a year round heat source and results in a higher rate of thaw on the upstream half of the dam, compared to the tailings beach scenario.

Figure 11 provides a summary of the yearly active layer depth and settlement prediction for the pit lake scenario. At the upstream toe, the active layer is predicted to increase by approximately 10 m during the 20 year operational design life. This corresponds to a settlement of 0.9 m. At the dam centerline, the performance during operations is similar to the tailings beach scenario and there would be no significant change in the active layer during operations. After 50 years, the active layer would increase by 5 m (approximately 1.5 m greater depth compared to the tailings beach scenario). At the downstream toe, there is no significant difference in the rate of thaw compared to the tailings beach scenario.

3.3 Sensitivity Analysis Results

3.3.1 812 m Crest Dam

Figure 12 provides the results of the 812 m crest dam scenario. During the initial two years, the active layer deepens due to the thermal loading provided by the tailings. Similar to the 807 m crest dam, the tailings upstream of the dam was predicted to refreeze with the majority of the dam foundation frozen by the sixth year. The performance at the downstream toe is similar to the 807 m crest dam, with approximately 1 m increase in the active layer within the first 20 years, and 4.4 m in 50 years.

3.3.2 Volumetric Water Content Sensitivity

Figure 13 compares the zero degree contours and predicted settlement with 24% and 30% VWC in the silt material. As expected, the lower volumetric water content case generally has a higher rate of thaw. During the first 20 years, there is very little difference between the two cases at the upstream toe and centerline. At the downstream toe, the increase in the active layer is 0.7 m greater with the lower volumetric water content.

3.3.3 Surface Boundary Condition Sensitivity

Figure 13 compares the zero degree contours and predicted settlement with two different surface boundary conditions. As expected, the warmer surface boundary condition results in a higher rate of thaw. However, even under the extreme condition using surface boundary condition #3, there is no increase to the active layer depth beneath the dam centerline during the operational period.

4 Summary

Thaw settlement of the foundation materials is expected to occur in the southern section of the dam that is founded over ice-rich fine grained overburden. Approximately 100 m of the dam is founded over these materials with the overburden thickness ranging from 0 m to 60 m. No thaw settlement is expected in the northern two-thirds of the dam.

In the areas of thick fine grained overburden, the ultimate thaw settlement of the Main Dam, once complete thaw has occurred, will be in the order of meters. At 13-MPD-06, where the most ice was encountered during the 2013 geotechnical investigation, it is estimated to be 5.1 m. Based on the thermal modelling, the rate of the thaw settlement will be slow, and the ultimate settlement will require multiple decades, or more likely centuries. The time to ultimate settlement is difficult to predict due to climate uncertainties. Under the assumed warming rate of 0.41°C per decade, the thaw settlement during the operational period is predicted to be in the order of 0.1 to 0.3 m. In the worst case pit lake scenario, settlement up to 0.9 m was predicted at the upstream toe. The settlement at the dam centerline is insignificant in both the tailings beach and pit lake scenarios.

The thermal analyses and thaw settlement estimates were completed using many conservative assumptions that would increase the rate of thawing, and as a result, the actual settlement during operations is expected to be much less. These conservative assumptions include: lower bound soil moisture contents used for the foundation soils, omission of ice layers and the fine grained dam core in the thermal model, complete removal of the boulder laydown area, and immediate thermal loading on the upstream side of the dam with tailings at +15°C or water up to the spillway elevation.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

5 References

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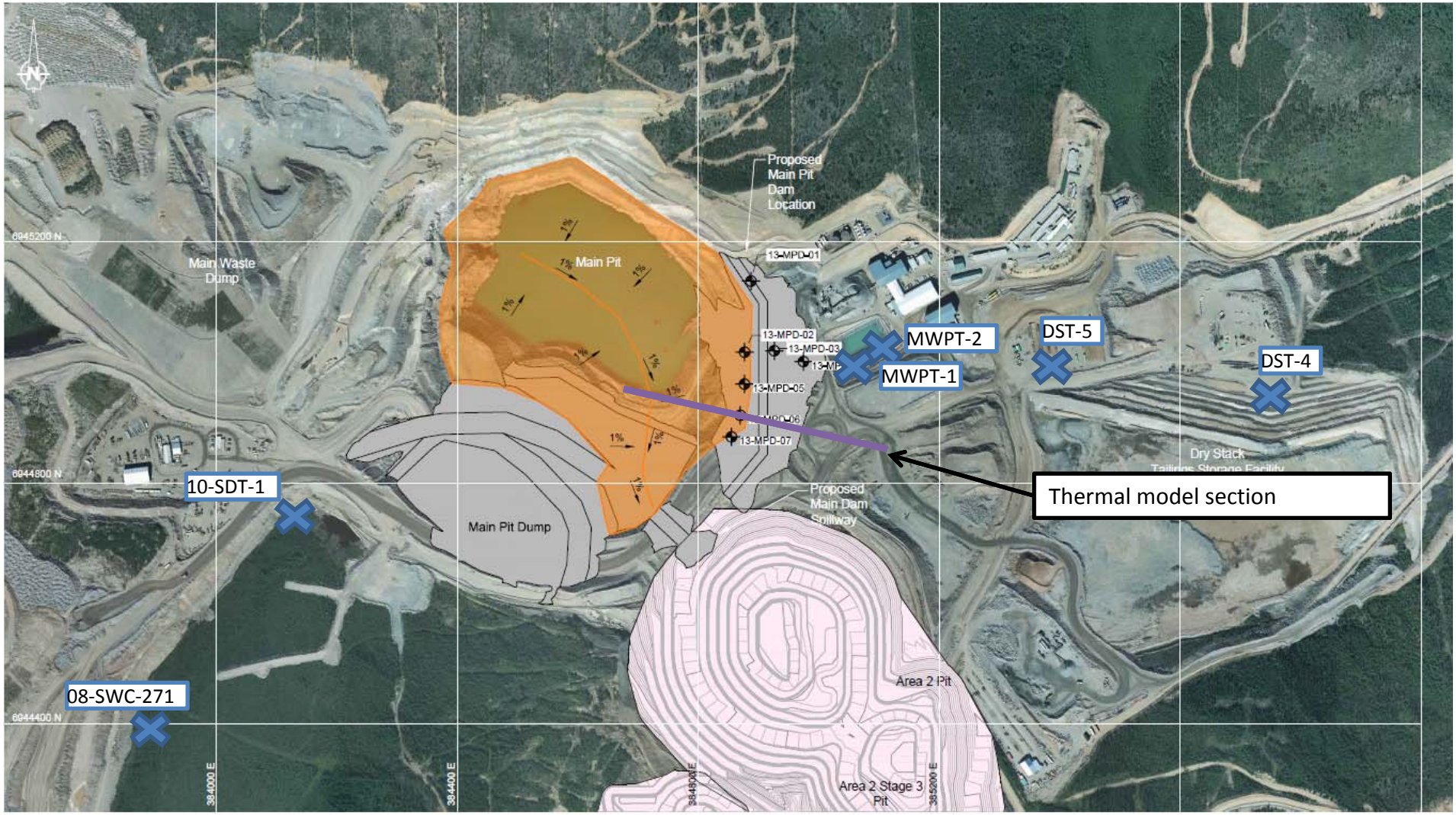
[USACE] US Army Corps of Engineers, 1988. Technical Manual, Arctic and Subarctic Construction Methods for Determination of Depths of Freeze and Thaw in Soils. January 1998.

Figures



PLAN LEGEND

- Phase IV/V/VI Pit Shell
- Phase V/VI Waste Dump
- Approximate Tailings Area
- Approximate limits of South Wall Failure
- 2013 Main Dam Drillholes
- Ground temperature cable locations

NOTES

1. Data presented in NAD 1983 UTM Zone 8N
2. Orthophoto: August 2013

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Main Dam Thermal Analysis

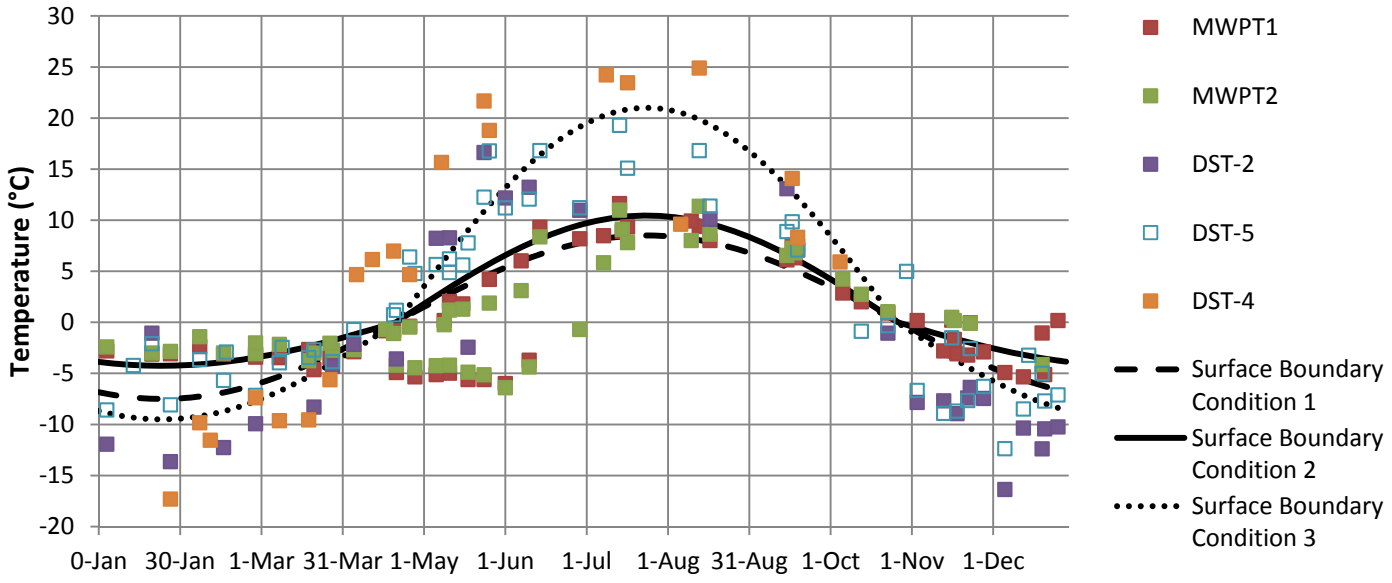
Site Plan

Job No: 1CM002.018.325

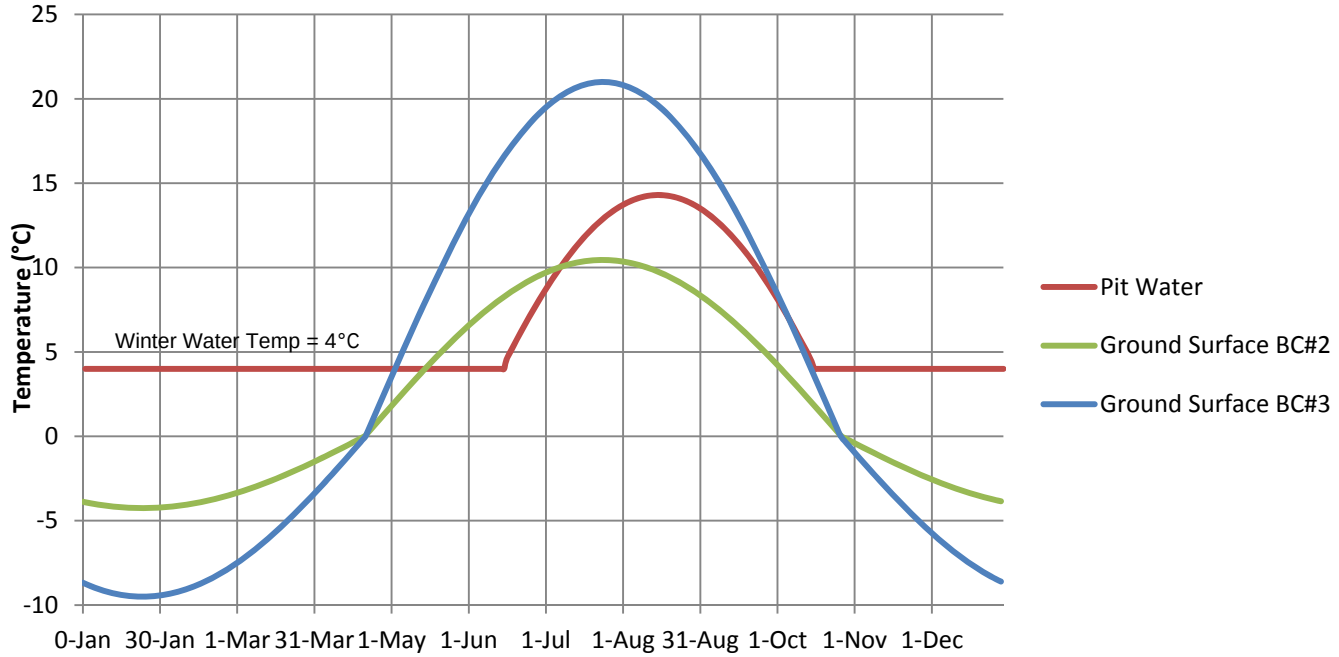
Filename: Fig_MainDamThermal_landsc_mjm.pptx

Minto Mine

Date:	Approved:	Figure:
July 2014	MJM	1



a) Ground surface temperature boundary conditions



b) Main Pit water temperature boundary condition

Notes:

1. In plot (a), surface boundary condition 1 was found to provide the best agreement with the thermal model calibration at SDT-1 and 08-SWC-271.
2. Surface boundary condition 2 was used in all main dam thermal model simulations based on ground temperature date collected from the mill water pond ground temperature cables (MWPT1, and MWPT2)
3. Surface boundary condition 3 was used in sensitivity runs to account for potentially greater sunlight exposure and less snow cover at the dam location.
4. Global warming was applied to the surface boundary conditions at a rate of +4.1°C/100yrs.

Source: J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Model Spreadsheets\Climate data.xlsx

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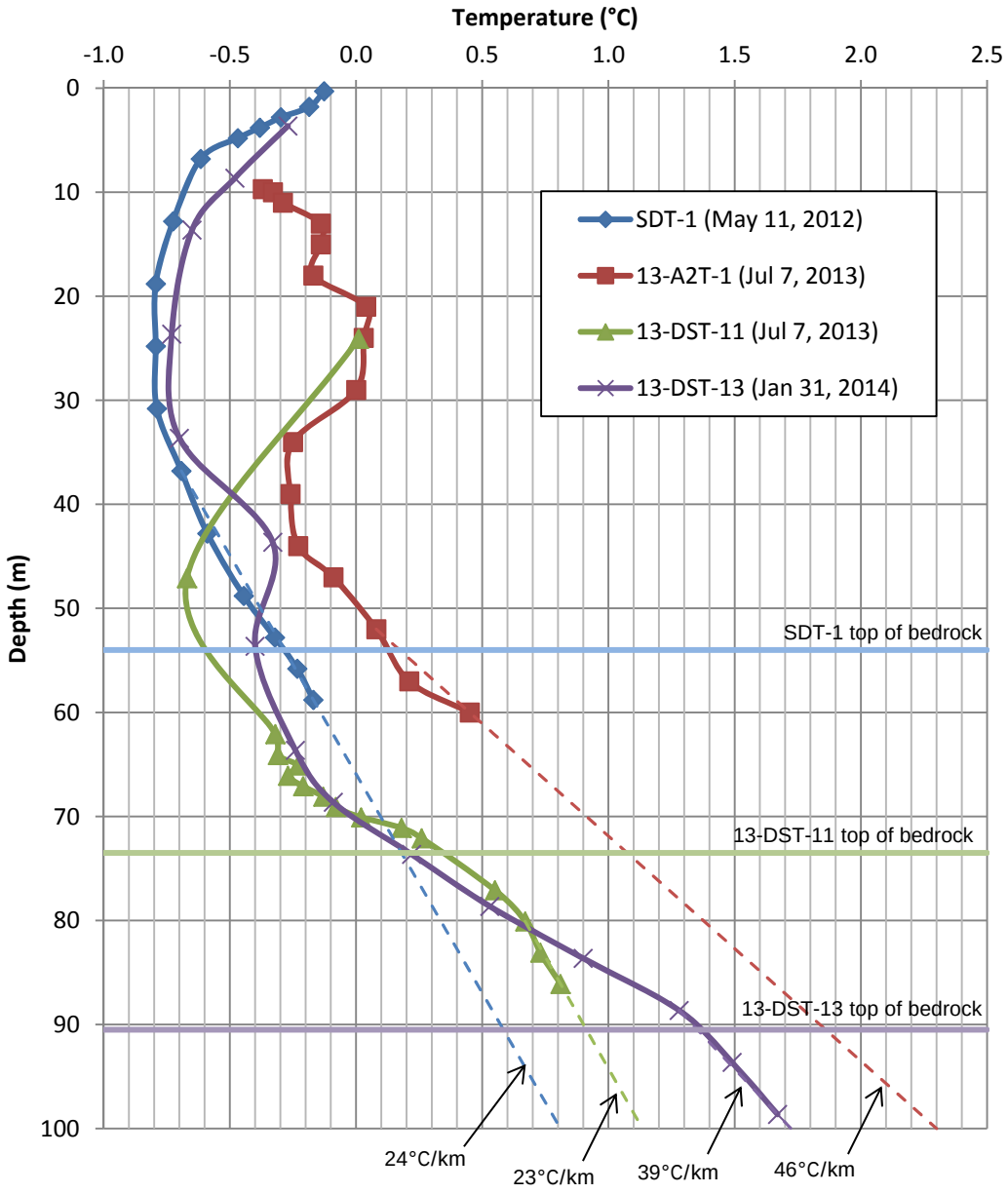
Job No: 1CM002.018.325 Filename: Fig_MainDamThermal_Portrait.pptx	Minto Mine	Date: March 2014 / Approved: PM / Figure: **2**

Ground temperature profiles



Notes:
1. Where possible, only sensors located in bedrock were used to estimated the temperature gradient.
2. The ground temperature cable at 13-A2T-1 did not encounter bedrock.
3. The graph plots the last reading obtained from each cable at the time of analysis.
4. Cables installed at 13-DST-11, 13-DST-13, and 13-A2T-1 were installed in the spring of 2013 and may not be in equilibrium with the surrounding soil, although the temperature slope is not expected to change significantly from the graph shown.
5. Average temperature gradient of 33°C/km was used to estimate the geothermal heat flux in all model simulations.

Source: J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Model Spreadsheets\Geothermal Gradient.xlsx

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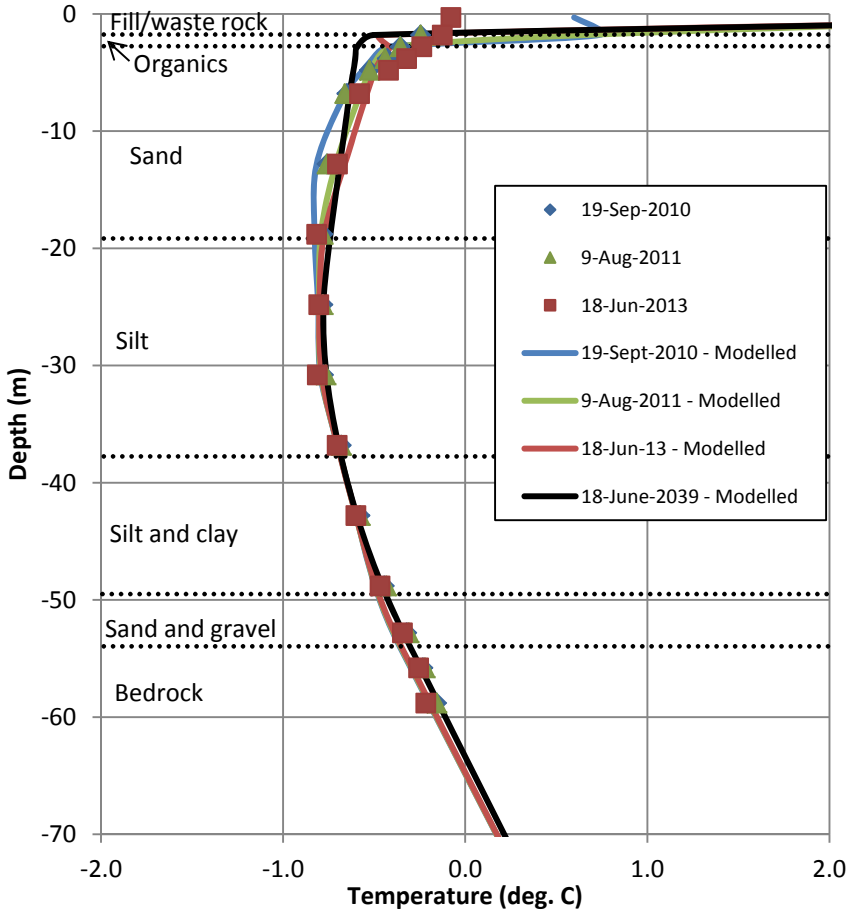
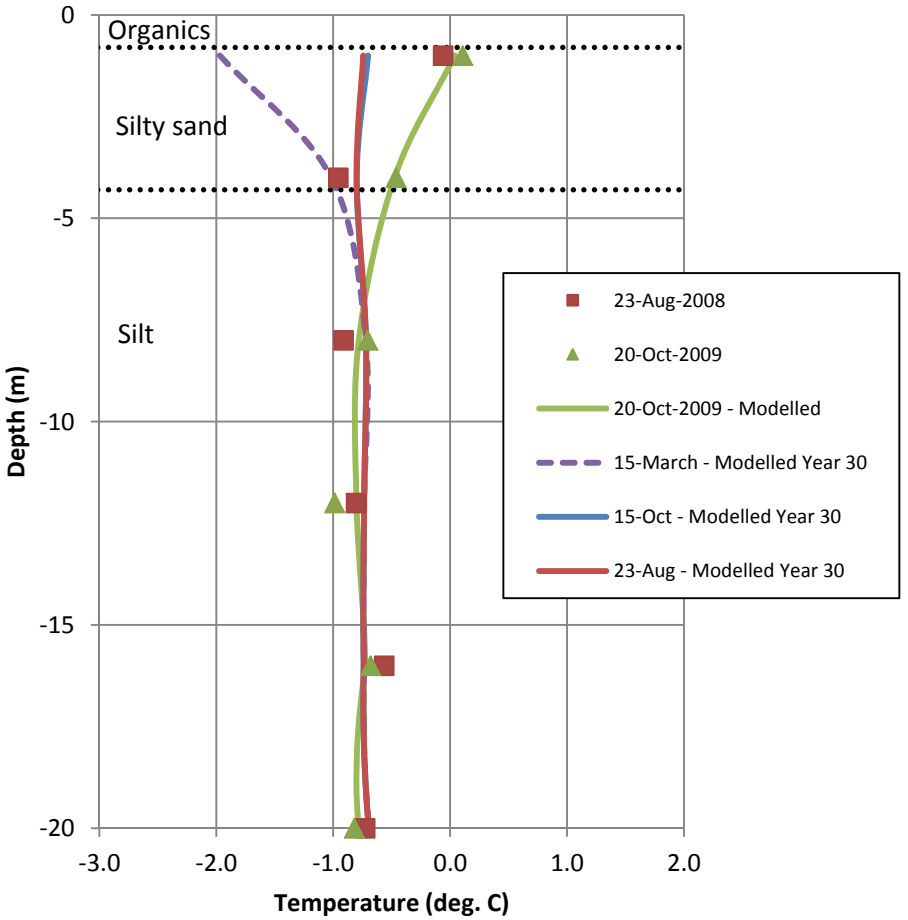
Job No: 1CM002.018.325 Filename: Fig_MainDamThermal_Portrait.pptx	Minto Mine	Date: March 2014 / Approved: PM / Figure: **3**

a) SDT-1



b) 08-SWC-271



Notes:

1. Symbols = Measured; Lines = modelled.
2. Surface modeled with a mean annual surface temp of -0.5 °C and an amplitude of 8 °C; climate warming was not considered.

Sources:

a) J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Model Spreadsheets\SDT-1 Calibration Results.rev02.xlsx

b) J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Model Spreadsheets\08-SWC-271 Calibration Results.xlsx

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Main Dam Thermal Analysis

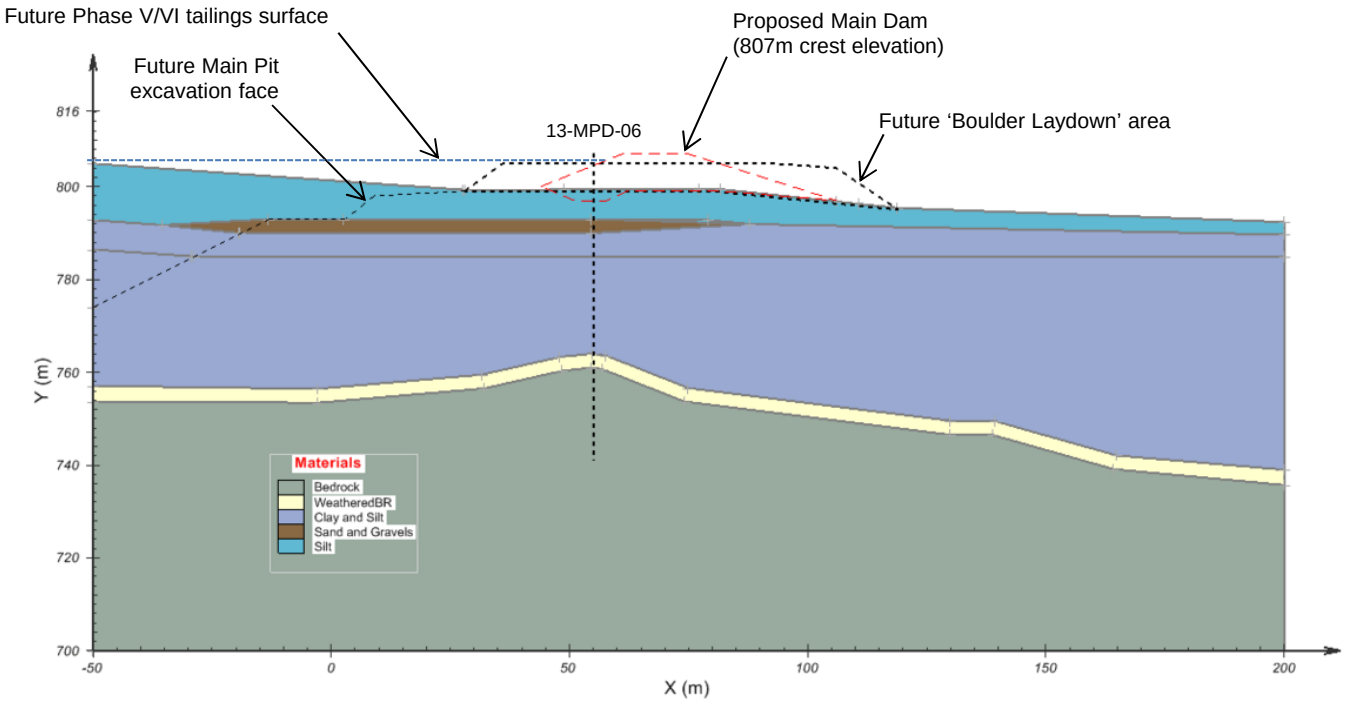
Thermal Model Calibration

Job No: 1CM002.018.325

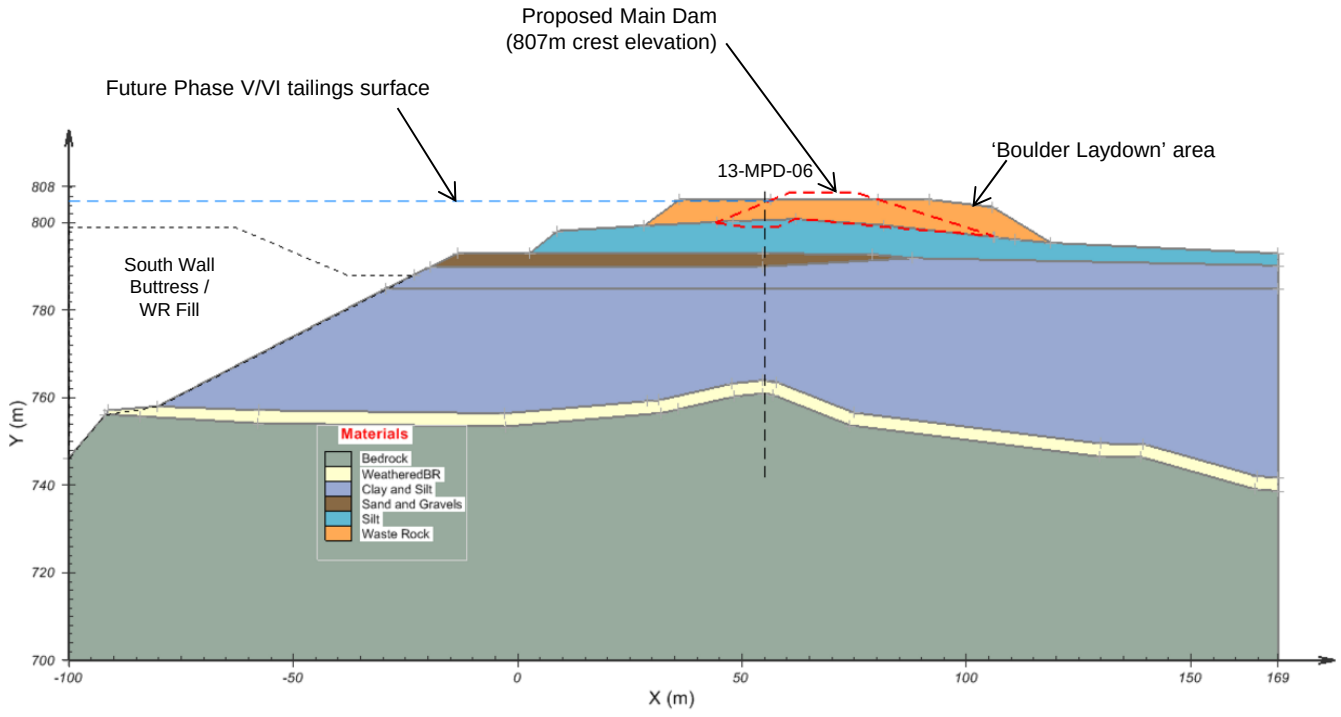
Filename: Fig_MainDamThermal_landsc.pptx

Minto Mine

Date:	Approved:	Figure:
March 2014	PM	4



a) Pre-mining model geometry



b) Mining model geometry (2007-2015)

Sources:
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b) \\MNOEL\C Drive\svs\Model_Files2009\MintoMainDam\2D\Transient\MineGeometry\MineGeometry.svm

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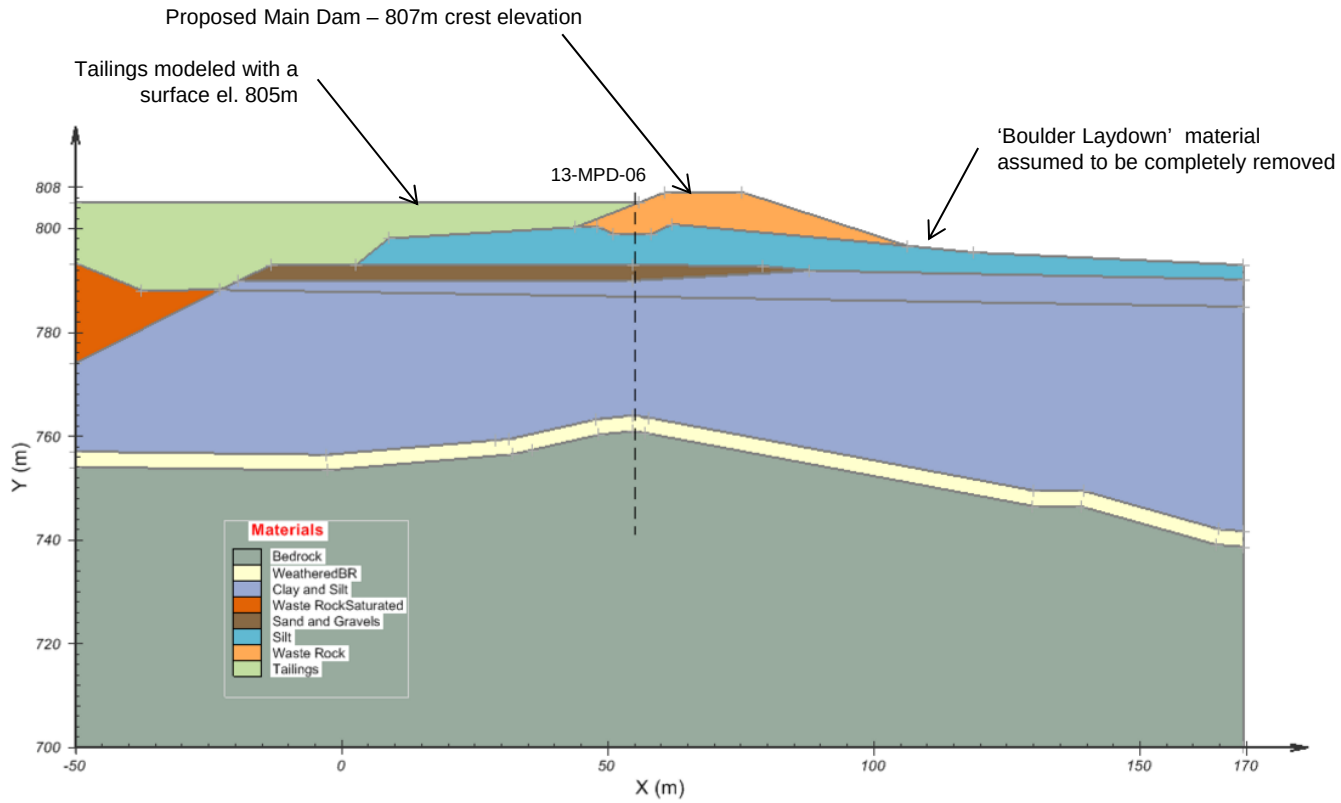
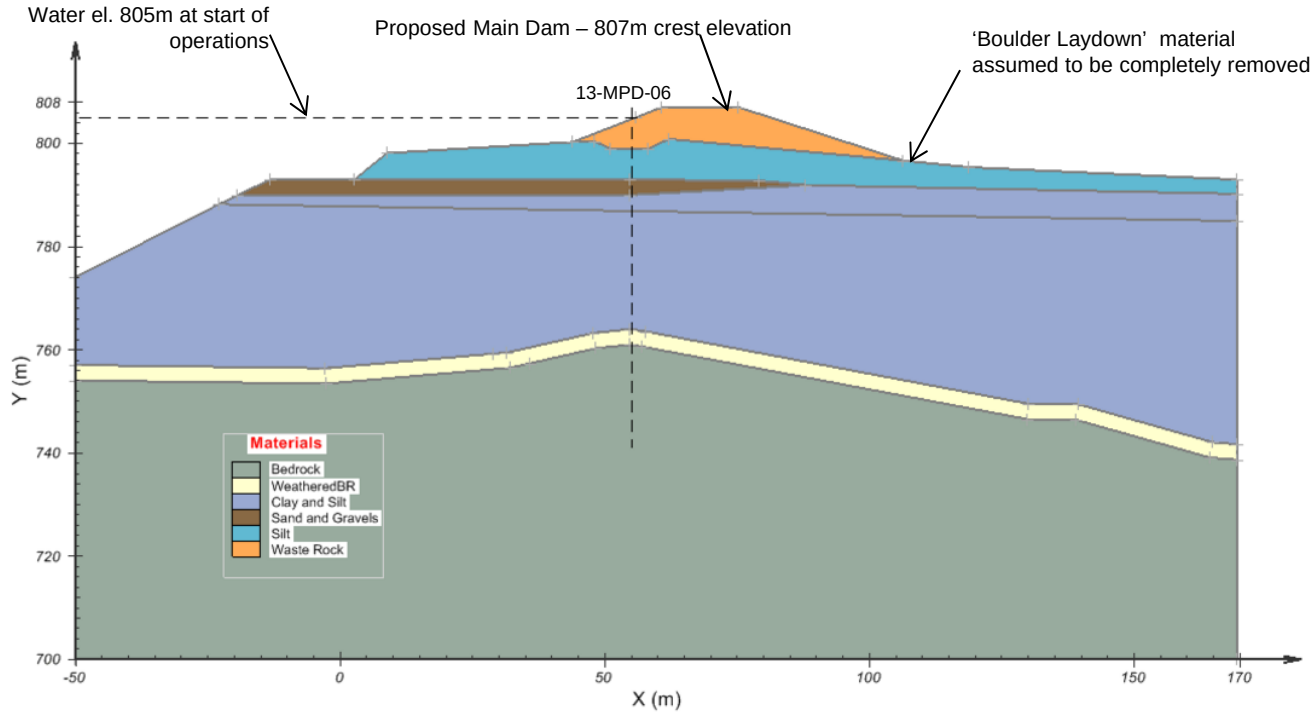
Job No: 1CM002.018.325 Filename: Fig_MainDamThermal_Portrait.pptx	Minto Mine	Date: March 2014 | Approved: PM | Figure: **5**



c) Main Dam model geometry with tailings on upstream face



d) Main Dam model geometry with water on upstream face

Sources:
c & d) \\MNOEL\C Drive\svs\Model_Files2009\MintoMainDam\2D\Transient\beachgeometry\beachgeometry.svm

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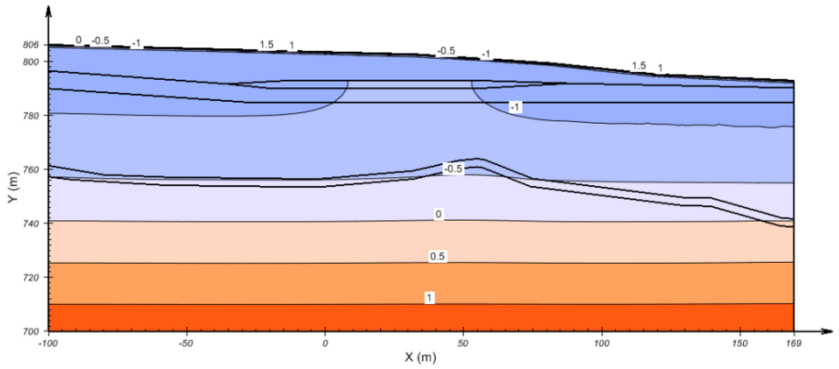
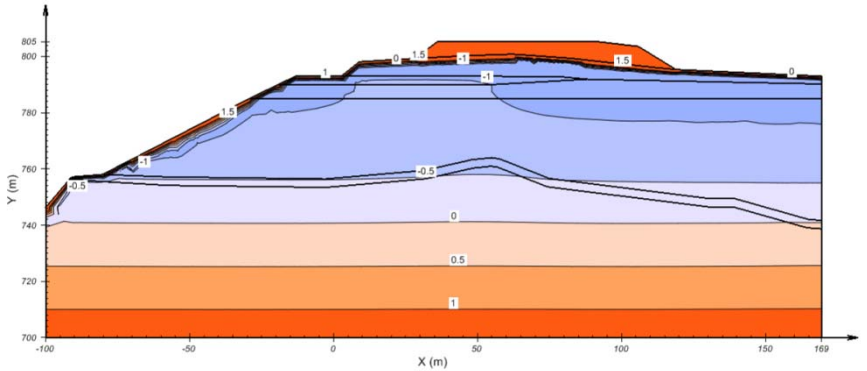
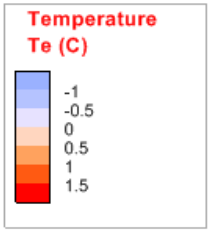
Job No: 1CM002.018.325 Filename: Fig_MainDamThermal_Portrait.pptx	Minto Mine	Date: March 2014 / Approved: PM / Figure: **6**

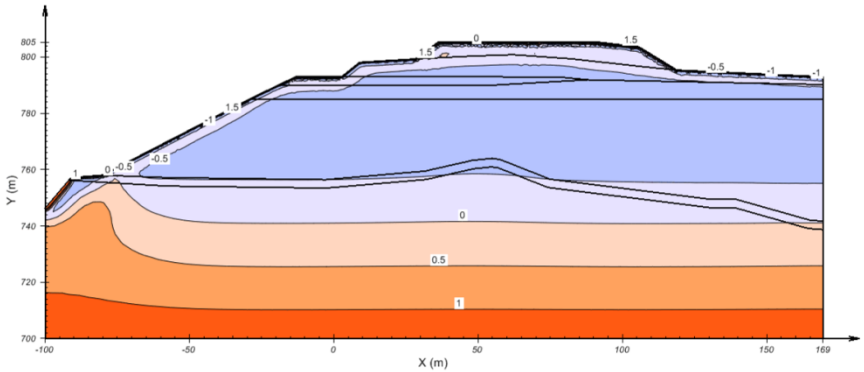


a) Pre-mining Conditions following 30 year simulation



b) Initial Conditions following excavation of the Main Pit and placement of boulder laydown





c) Initial Conditions prior to tailings and dam construction.

Notes:

1. Temperatures below -1°C and above 1.5 °C were assigned the same temperature contour in each plot.
2. The waste rock fill placed in plot (b) had an initial temperature of +5°C.
3. Time 0 in plots (a) and (b) occurs in May.
4. Plot (c) is 7 years following (b) with May temperatures contours plotted.

Sources:
a) \\MNOEL\C Drive\svs\Model_Files2009\MintoMainDam\2D\Transient\MD06SectionPreMine_6\MD06SectionPreMine_6.svm
b & c) \\MNOEL\C_Drive\svs\Model_Files2009\MintoMainDam\2D\Transient\MD06SectionMining_4\MD06SectionMining_4.svm

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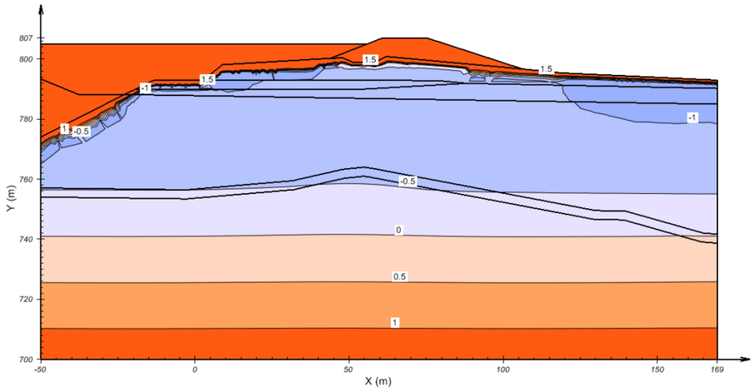
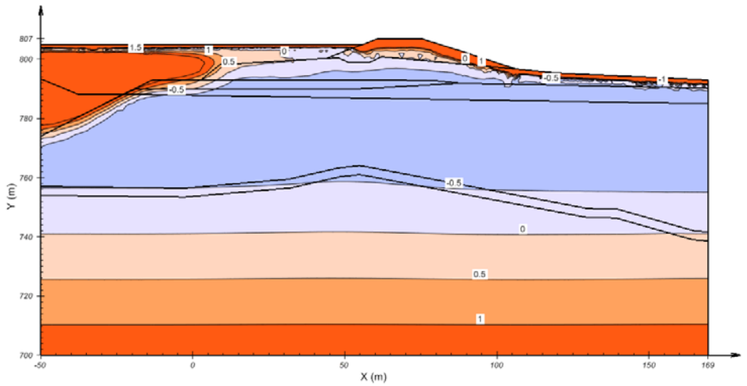
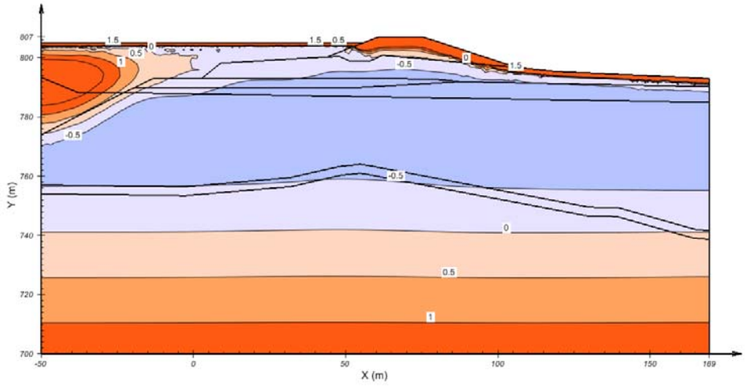
		Thermal Model Initial Conditions
Job No: 1CM002.018.325 Filename: Fig_MainDamThermal_landsc.pptx	Minto Mine	Date: March 2014 / Approved: PM / Figure: **7**



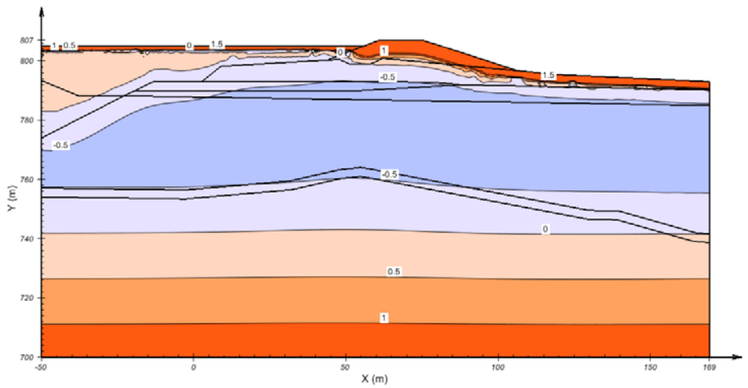
a) Summer of year 0.



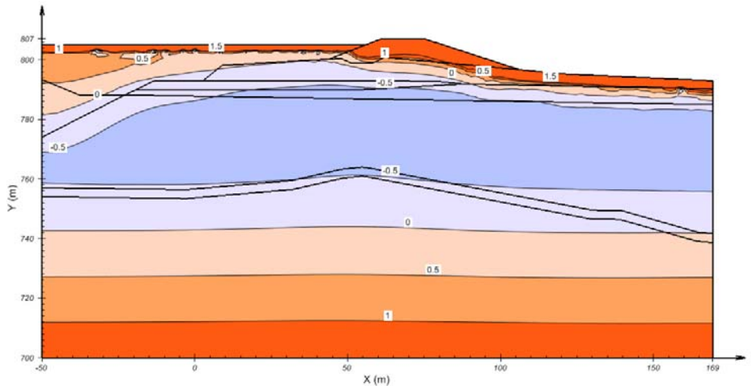
b) Summer of year 3.



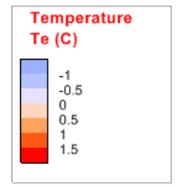
c) Summer of year 8.



d) Summer of year 30.



e) Summer of year 50.



Notes:
1. Temperatures below -1°C and above 1.5 °C were assigned the same temperature colour in each plot.
2. The dam fill was placed with an initial temperature of +5°C applied on day 0
3. The tailings were assigned an initial temperature of +15°C, applied on day 0.

Source: \\MNOEL\C Drive\svs\Model_Files2009\MintoMainDam\2D\Transient\807PitLake-1\807PitLake-1.svm

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Filename: Fig_DamThermal_11x17land.pptx

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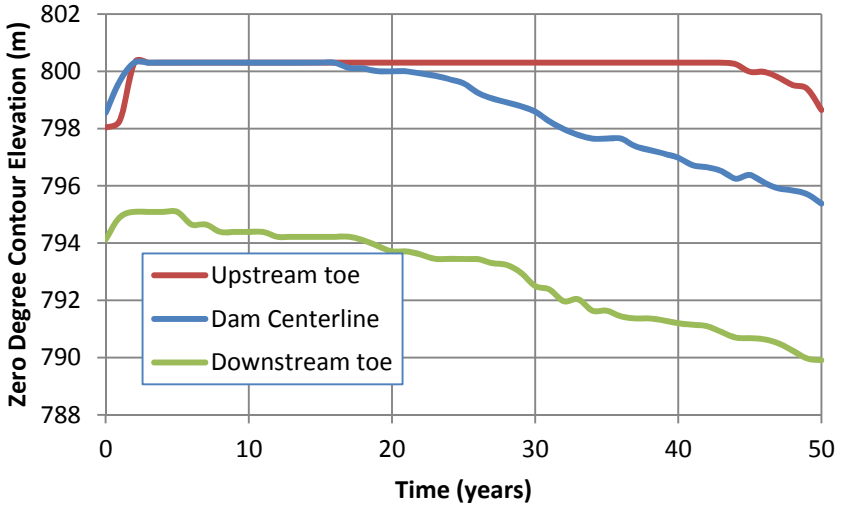
Minto Mine

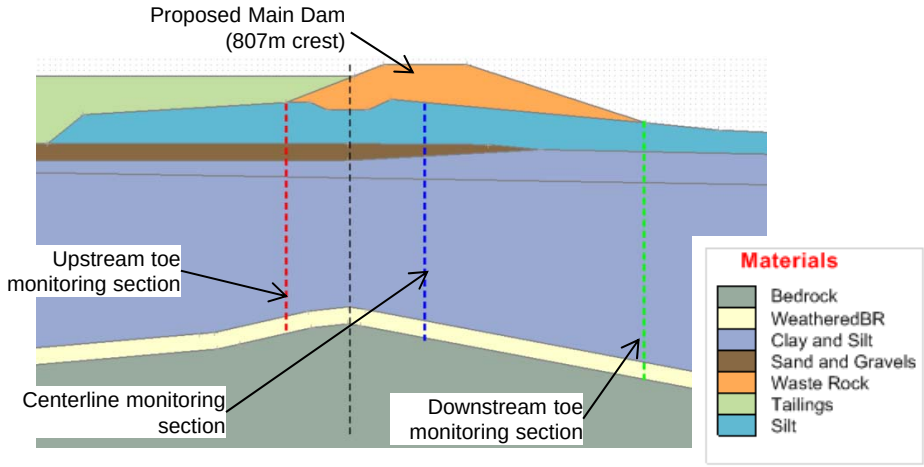
Main Dam Thermal Analysis

Predicted Temperature Distribution – Tailings Beach

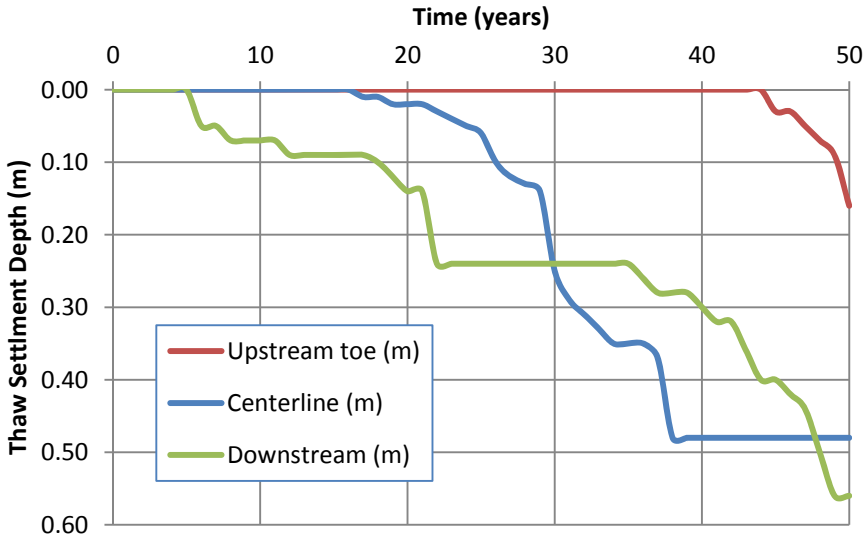
Date:	Approved:	Figure:
March 2014	PM	8



a) 0°C contour elevation versus time



c) Model Geometry and materials



b) Thaw settlement versus time

Notes:

1. The thaw depth defined by the 0°C temperature contour was monitored along three sections lines below the dam on an annual basis in the summer/fall. The progress of the 0°C contour is shown in plot (a).
2. The thaw settlement depth assumes that all excess ice above the 0°C contour elevation has melted.
3. The thaw settlement depth does not consider the initial thawing during dam construction.
4. The excess ice content is based on the 13-MPD-06 borehole log (SRK 2014).

Source:
J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Settlement Calcs\MD06Dam Section_Settlement.xlsx

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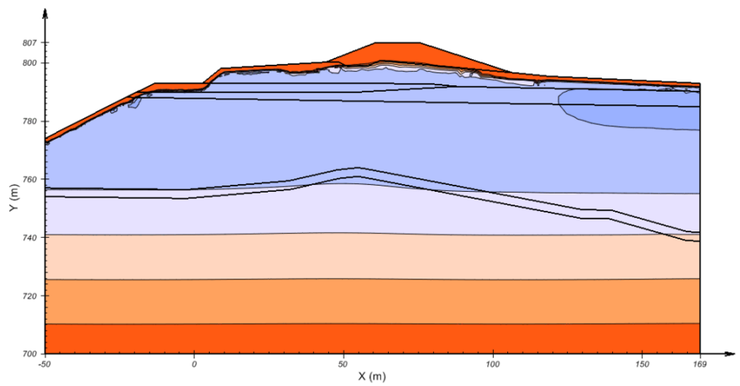
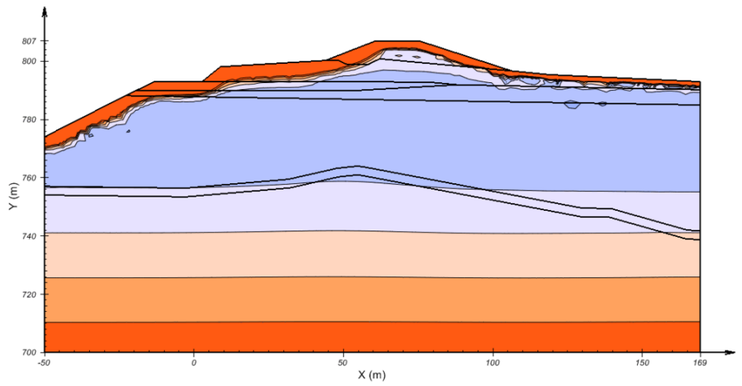
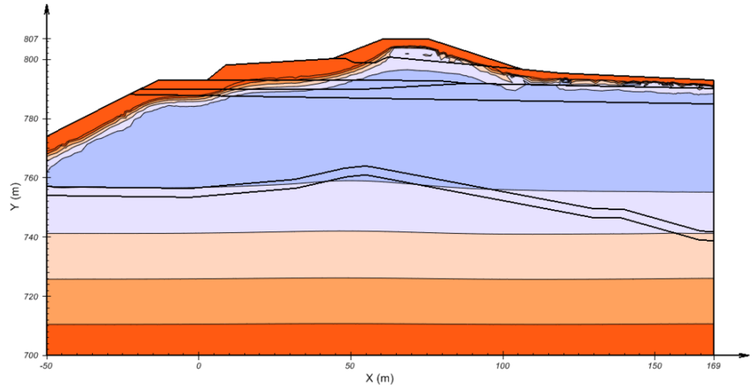
Job No: 1CM002.018.325 Filename: Fig_MainDamThermal_landsc.pptx	Minto Mine	**Predicted Settlement– Tailings Beach**		
		Date: March 2014	Approved: PM	Figure: **9**



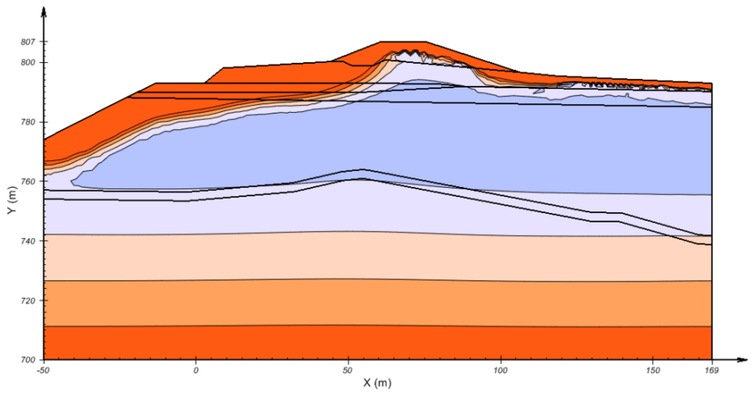
a) Summer of year 0.



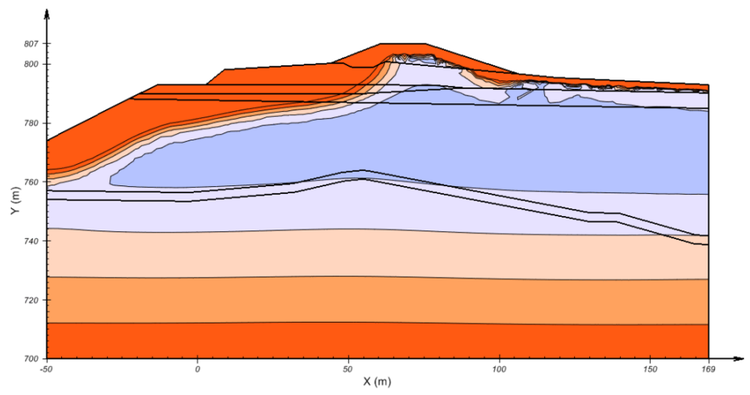
b) Summer of year 3.



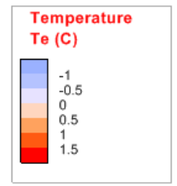
c) Summer of year 8.



d) Summer of year 30.



e) Summer of year 50.



Notes:

1. Temperatures below -1°C and above 1.5 °C were assigned the same temperature colour in each plot.
2. The dam fill was placed with an initial temperature of +5°C applied on day 0
3. The Main Pit was assumed to fill to an elevation of 804m on day 0 of the simulation.

Source: \\MNOEL\C Drive\svs\Model_Files2009\MintoMainDam\2D\Transient\MD06SectionFuture_9\MD06SectionFuture_9.svm

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Job No: 1CM002.018.325

Filename: Fig_DamThermal_11x17land.pptx

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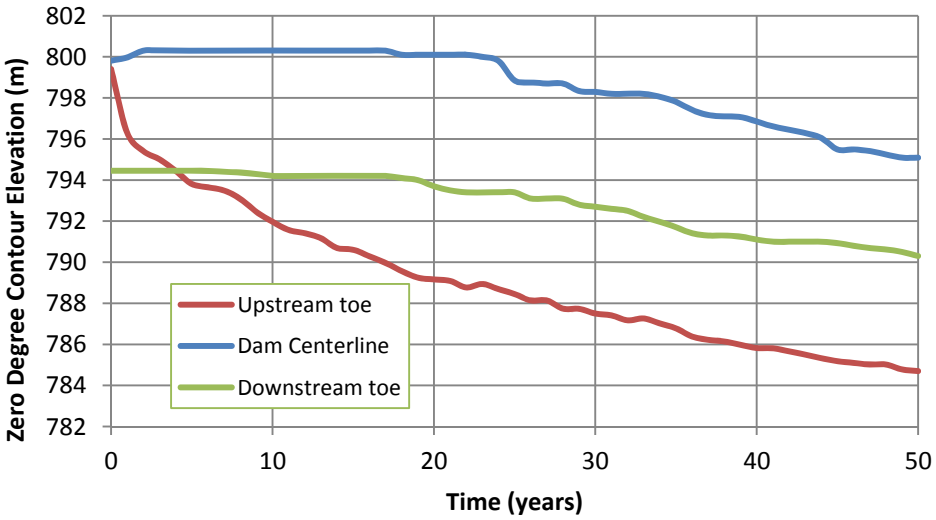
Minto Mine

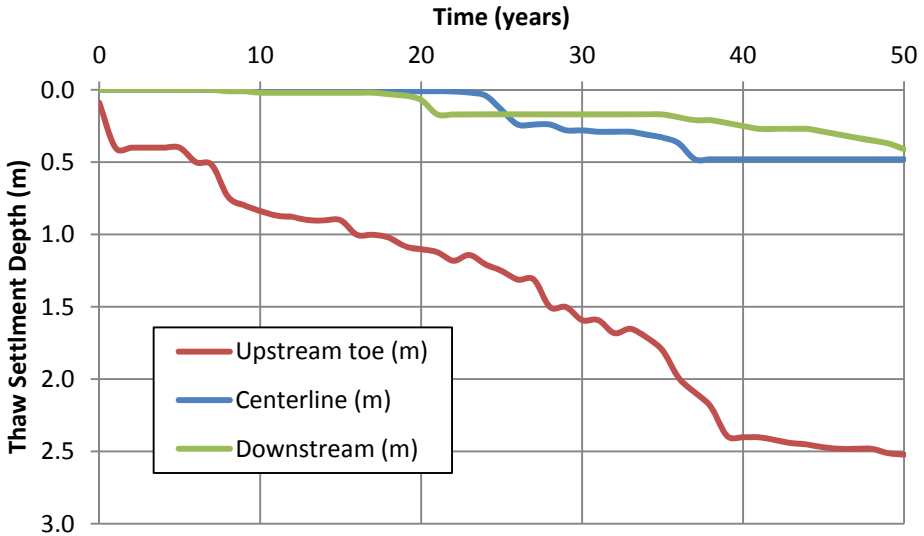
Main Dam Thermal Analysis

Predicted Temperature Distribution – Pit Lake

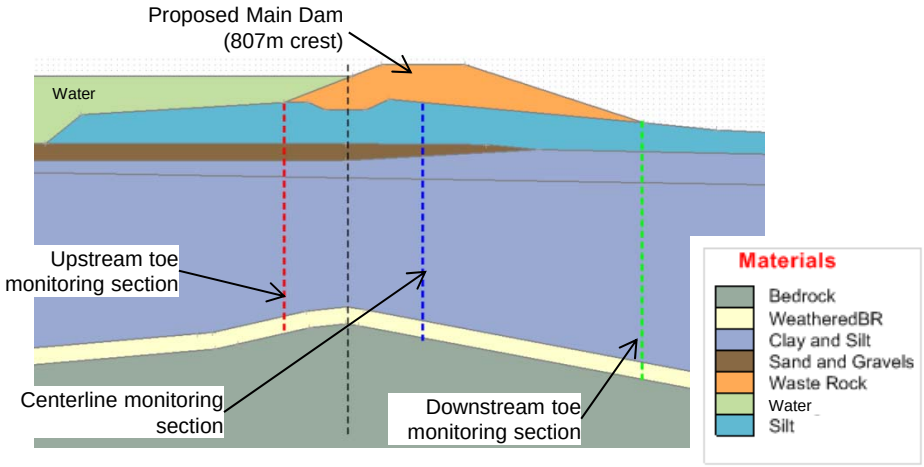
Date:	Approved:	Figure:
March 2014	PM	**10**



a) 0°C contour elevation versus time



b) Thaw settlement versus time



c) Model Geometry and materials

Notes:

1. The water in the pit was assumed to fill to the dam spillway elevation (804m) at the start of the simulation.
2. The thaw depth defined by the 0°C temperature contour was monitored along three sections lines below the dam on an annual basis in the summer/fall. The progress of the 0°C contour is shown in plot (a).
3. The thaw settlement depth assumes that all excess ice above the 0°C contour elevation has melted.
4. The thaw settlement depth does not consider the initial thawing during dam construction.
5. The excess ice content is based on the 13-MPD-06 borehole log (SRK 2014).

Source:
J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Settlement Calcs\MD06Dam Section_Settlement.xlsx

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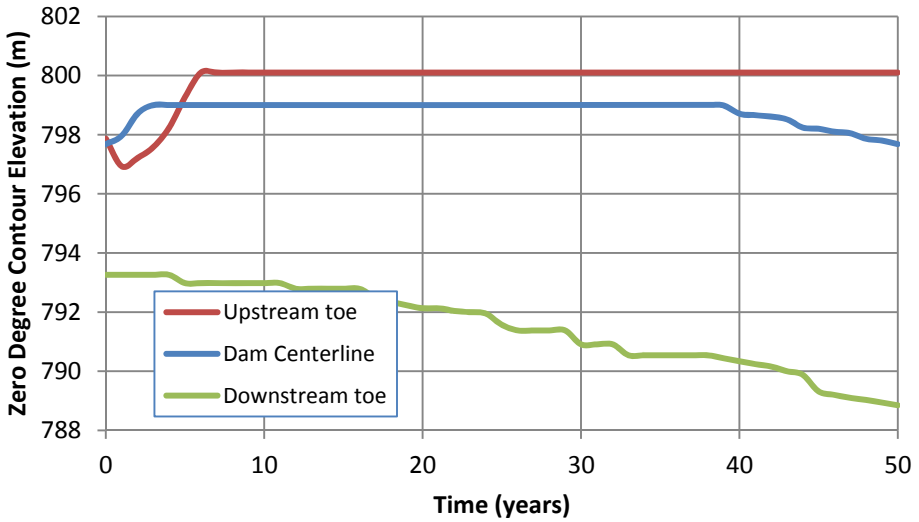
Job No: 1CM002.018.325
Filename: Fig_MainDamThermal_landsc.pptx

Minto Mine

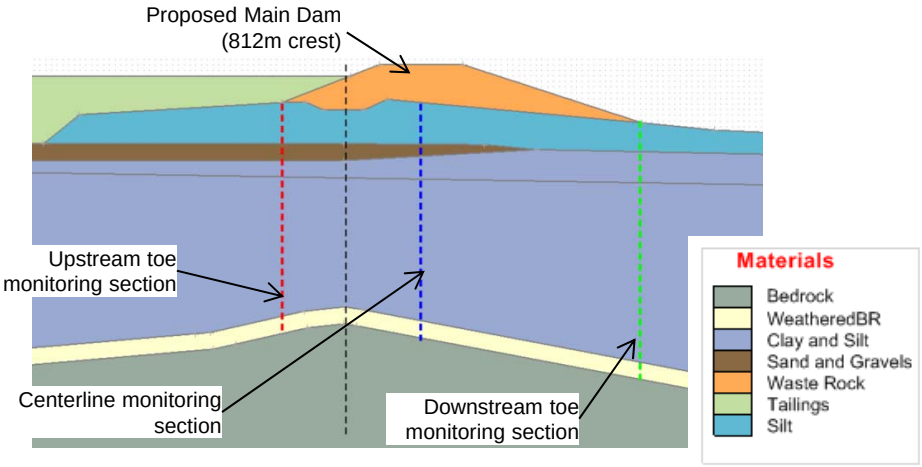
Main Dam Thermal Analysis

Predicted Settlement– Pit Lake Scenario

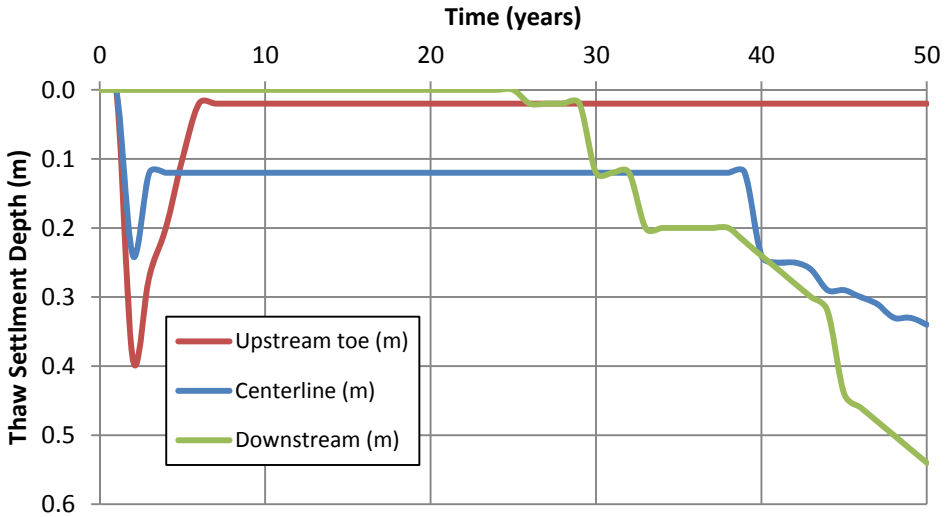
Date:	Approved:	Figure:
March 2014	PM	**11**



a) 0°C contour elevation versus time



c) Model Geometry and materials



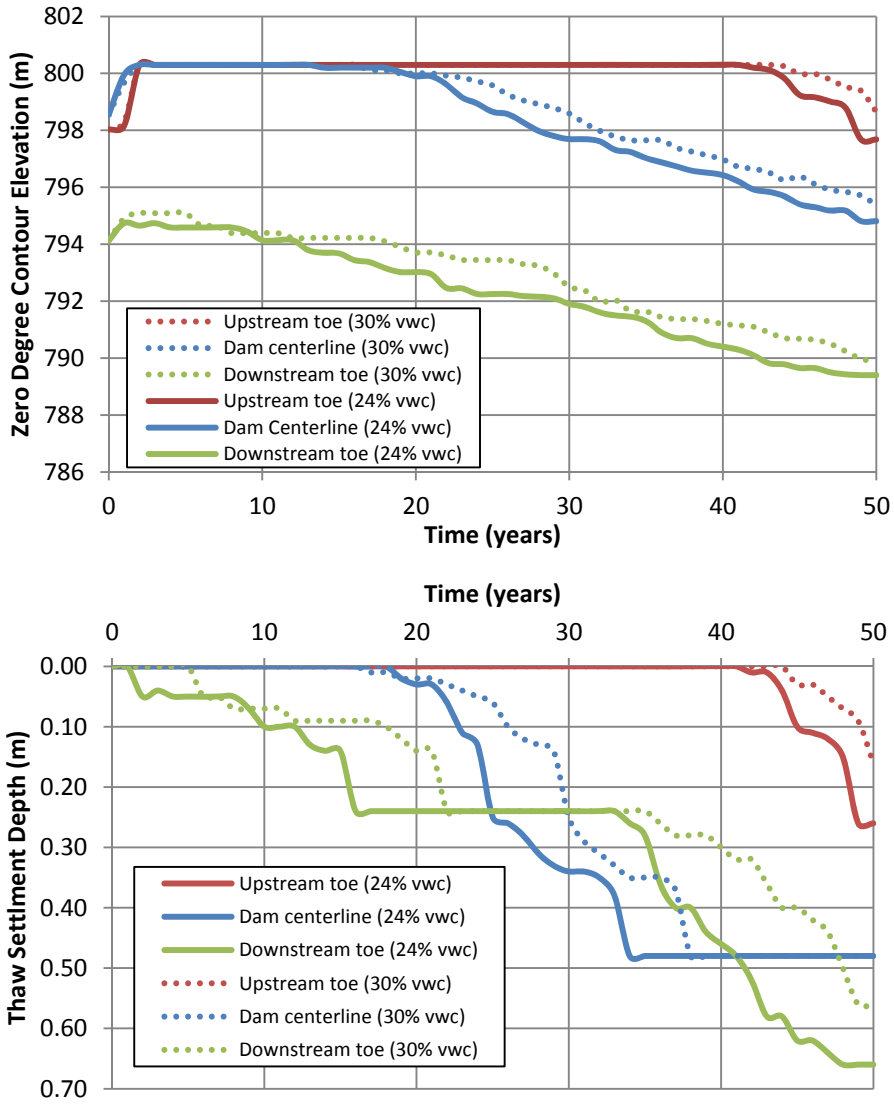
b) Thaw settlement versus time

Notes:

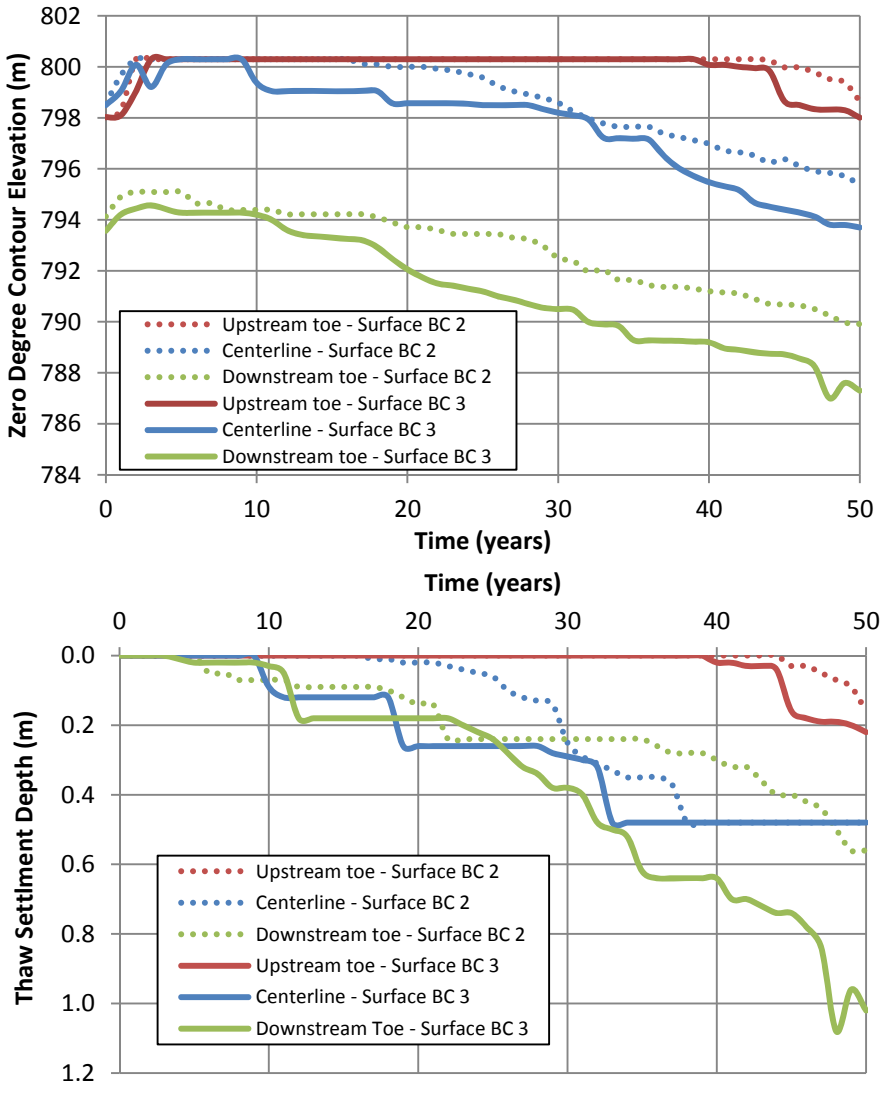
1. The thaw depth defined by the 0°C temperature contour was monitored along three sections lines below the dam on an annual basis in the summer/fall. The progress of the 0°C contour is shown in plot (a).
2. The thaw settlement depth assumes that all excess ice above the 0°C contour elevation has melted.
3. The thaw settlement depth does not consider the initial thawing during dam construction.
4. The excess ice content is based on the 13-MPD-06 borehole log (SRK 2014).

Source:
J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Settlement Calcs\MD06Dam Section_Settlement.xlsx

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Thermal Analysis		
Job No: 1CM002.018.325 Filename: Fig_MainDamThermal_landsc.pptx	Minto Mine	**Predicted Settlement– 812m Crest Dam**		
		Date: March 2014	Approved: PM	Figure: **12**



a) Silt volumetric water content sensitivity



b) Surface boundary condition sensitivity

Notes:
1. See figure 4 for surface boundary condition (bc) details

Source: J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Thermal Analysis\Settlement Calcs\MD06Dam Section_Settlement.xlsx

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Main Dam Thermal Analysis

Settlement Sensitivity

Job No: 1CM002.018.325	Minto Mine	Date: March 2014	Approved: PM	Figure: **13**
Filename: Fig_MainDamThermal_landsc.pptx				

Appendix D: Main Dam Consolidation Settlement Assessment

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2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

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vancouver@srk.com
www.srk.com

Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Peter Luedke	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 15, 2014
Subject:	Main Pit Tailings Consolidation		

1 Introduction

The proposed Phase V/VI Main Pit Tailings Management Facility (TMF) would contain tailings up to a maximum depth of 93 m at a full storage capacity, with an 812 m Main Dam crest elevation, and an 809 m maximum tailings elevation. This memo presents the results of conservative preliminary consolidation modelling of tailings in the Main Pit. Ice entrainment was not considered in this analysis.

2 Tailings Consolidation Modeling

2.1 Method of Analysis

The analysis was completed with FSConsol, created by GWP Software Inc. This software uses finite strain consolidation theory to describe large strain consolidation problems with non-linear soil properties (GWPP, 2007). FSConsol also allows varied tailings discharge rates and staged-storage capacities. As FSConsol is a one-dimensional model, the maximum consolidation was calculated at the surface of the tailings where the depth is the greatest (i.e. 93 m).

The FSConsol model predicts consolidation of uniform tailings discharged continuously at a specified rate which can be varied over time. As FSConsol analyzes a single column of soil (or tailings), the deposition rate in this column has an inversely proportional relationship with the surface area of the tailings facility at a given elevation. Updating the model with actual filling rates would allow for an improved predictive model; however, this was not deemed necessary at this stage of design.

2.2 Model Inputs

The soil properties were based on consolidation test results completed by Klohn Crippen Berger (KCB) on samples from underflow tailings in December 2012. The consolidation tests were completed on two different samples, after a 55% and 60% solids content settling test, respectively. The base case tailings properties used in the FSConsol model were the 55% solids content consolidation data; however, a sensitivity analysis was run using the 60% solids content

data. This was deemed more conservative input, since with lower solids content, the tailings will likely settle more after discharging at a higher rate. The tailings test results are provided in Attachment 1.

A stage-storage capacity curve was generated based on a TMF model at five meter intervals. For the purposes of modelling, the stage-storage capacity curve was simplified by averaging the change in area over approximately 20 m intervals. The simplified capacity curve is presented in Figure 1. An assumed tailings deposition rate of 7,445 metric tons per day was specified based on a mill throughput of 4,200 dry metric tonnes and a tailings solids content of 55%.

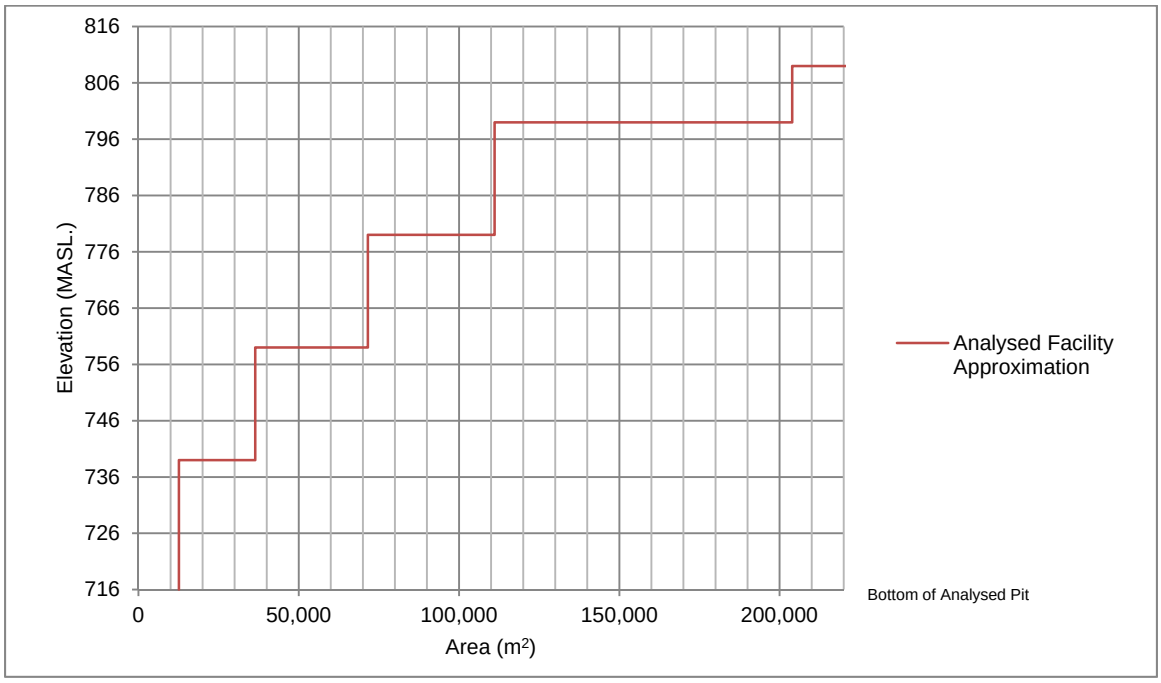

Figure 1: Approximated Stage-Storage Capacity Curve

Source: \\ van-svr0\Projects\01_SITES\Minto1CM002.018_MainDam_PrelimDesign\Stability_+_Seepage_Analysis\Tailings Consolidation\Minto_FSConsolCompiledData_1CM002.018_rev01_PL.xlsx

2.3 Results

The model results show the facility reaches an elevation of 809 m (maximum tailings storage capacity) in 6.5 years with a milling rate of 4,200 dry metric tonnes per day at 55% solids content. Deposition of hydraulic tailings was modelled to start on November 1, 2012. Following tailings deposition, the tailings are estimated to consolidate by about 3.7 m after 40 years (approximately 33 years after end of deposition) and 8.5 m after 100 years (approximately 93 years after end of deposition). The consolidation settlement is summarized in Table 1.

Table 1: Tailings Surface Elevation Over Time

Year	Tailings Surface Elevation (m)	Change in Tailing Surface Elevation (m) @ 55% Solids Content	Change in Tailing Surface Elevation (m) @ 60% Solids Content
0	0.0	-	-
6.4	809.0	-	-
40	-	-3.7	-4.9
100	-	-8.5	-7.6

The settlement results listed in Table 1 are predicted at the deepest portion of the facility with a full tailings depth of 93 m. Settlement will be proportionally less in shallower zones of the facility. The modelled Main Pit TMF filling curve and the consolidation is shown in Figures 2 and 3 for both 55% and 60% solids content tailings.p

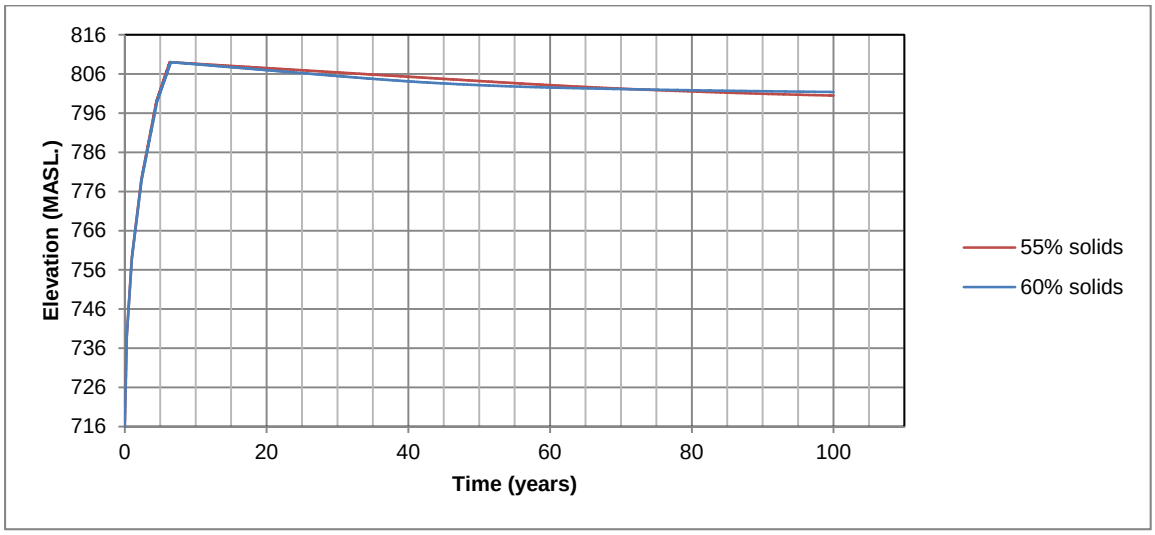

Figure 2: Predicted Tailings Surface Elevation

Source: \\VAN-SVR0\Projects\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Stability_+_Seepage_Analysis\Tailings Consolidation\FSConsol Output\Updated model_20140520\Minto_FSCdata_4200tpd_1CM002.018_rev02_20140520.xlsx

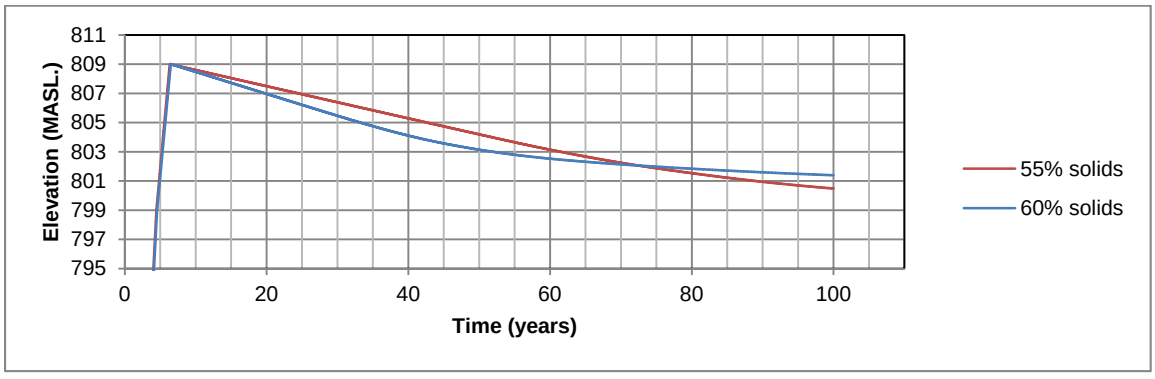

Figure 3: Predicted Tailings Surface Elevation (Close-up of Consolidation)

Source: \\VAN-SVR0\Projects\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Stability_+_Seepage_Analysis\Tailings Consolidation\FSConsol Output\Updated model_20140520\Minto_FSCdata_4200tpd_1CM002.018_rev02_20140520.xlsx

3 Summary

The numeric assessment of total tailings consolidation for the Main Pit tailings impoundment suggests long-term consolidation settlement of between 7.6 and 8.5 m for the maximum depth of tailings deposited in Main Pit of 93 m. Although this analysis is overly simplistic and inherently conservative, it provides an early indication of how the tailings surface will deform over time. This is important in assessing the long-term tailings closure planning, as well as taking into consideration the long-term thaw settlement of the Main Dam.

Prior to closing of the facility, additional detailed tailings characterization should be completed, and this assessment should be updated and refined to take into consideration actual 3D geometry of the impoundment, variability of tailings properties, as well as ice entrainment.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

4 References

GWP Software Inc. (2007). *FSConsol Slurry Consolidation Software Users Manual.* Edmonton, AB: GWP Software Inc.

Attachment 1 - Tailings Test Results

Klohn Crippen Berger

January 16, 2013

Capstone Mining Corp.
P.O. Box 33174
Whitehorse, Yukon
Y1A 6S1

Mr. Eamon Mauer
Minto Mine Geotech and Reclamation Engineer

Dear Mr. Mauer:

Minto Project
Tailings Lab Testing

Please find the results of the laboratory tailings testing attached to this cover letter. More specifically, this includes:

- Consolidation Test Results;
- Grain Size Distribution Results;
- Settling Test Results; and
- Specific Gravity of Soil Solids Results.

If you have any questions or comments about the attached laboratory tailings testing results, please feel free to contact me by phone (604-251-8438) or email (bchambers@klohn.com).

Thank you for the opportunity to complete the above scope of work, we look forward to continuing to provide our services to you in the future.

Yours truly,
KLOHN CRIPPEN BERGER LTD.

Robert W. Chambers, P.Eng.
Staff Consultant

DH:cd

130116L-Tailings Lab Testing .docx
M09638A03.732

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APPENDIX I

Consolidation Test Results

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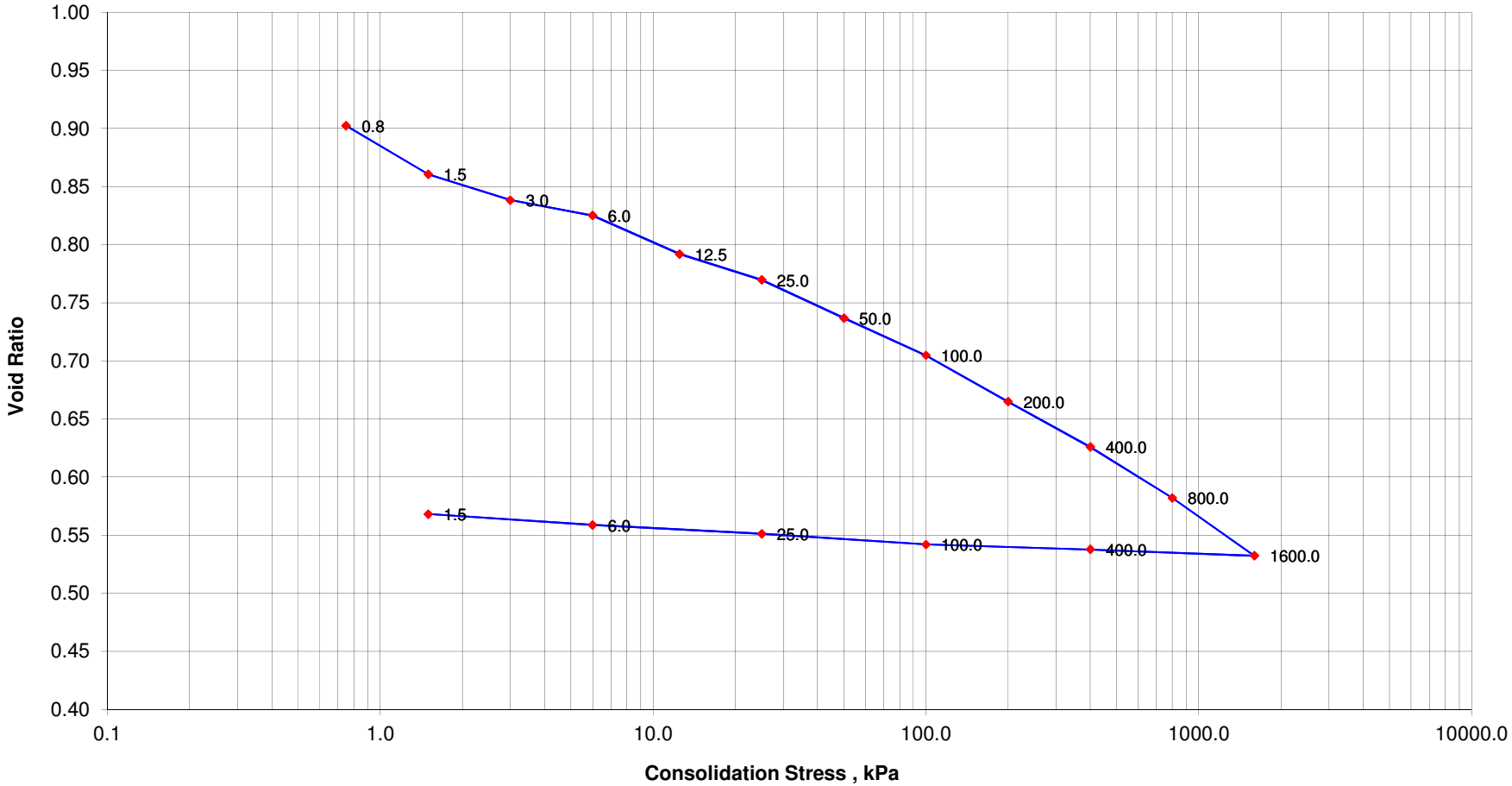
Consolidation Test - Tailings after 55% Solids Content Settling Test

e - log(p)



Klohn Crippen Berger	PROJECT NO.:	M09638A03		
	PROJECT:	Minto Mine		
	LOCATION:	YK	DATE TESTED:	15-Dec-2012
	SAMPLE NO.:	Underflow Tailings	DEPTH:	After 55% solids content settling test
	TESTED BY:	BY	CHECKED BY:	JG

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CONSOLIDATION

PROJECT NO.:	M09638A03
PROJECT:	Minto Mine
SAMPLE NO.:	Underflow Tailings
DETAILS	After 55% solids content settling test
	Supernatant removed
TEST NO.:	T1262CONS02R
LOADING MACHINE NO.:	OED1 / ID80

Test Specimen Information:

Initial water content:	59.15 % (based on trimmings)
Final water content:	23.17 % (based on final dry weight)
Dry mass:	84.01 g
Diameter	63.53 mm
Area	31.699 cm^2
Specific Gravity	2.72

Initial Specimen Height (mm):	25.33
Height of Solid (mm):	9.743
Initial void ratio:	1.600
Void Ratio Factor	0.1026

* Calibration to be done after test

** Estimated t_{50}

Pressure (kPa)		*Change in Height	Final	Change in	Change in	Void	t_{50**}	Cv	Mv	k	Cc
From	To	Corrected (mm)	Height (mm)	Void Ratio	Void Ratio Acc	Ratio	(min)	(cm^2/sec)	(cm^2/N)	(cm/sec)	
0.0	**0.8**	6.794	18.536	0.6973	0.6973	**0.902**					
0.8	**1.5**	0.407	18.129	0.0418	0.7391	**0.861**					
1.5	**3.0**	0.217	17.912	0.0223	0.7614	**0.838**					
3.0	**6.0**	0.129	17.783	0.0132	0.7746	**0.825**					
6.0	**12.5**	0.324	17.459	0.0333	0.8078	**0.792**	0.56	4.6E-03	6.1E-02	2.7E-06	**0.104**
12.5	**25.0**	0.217	17.242	0.0223	0.8301	**0.770**	0.20	1.2E-02	1.9E-02	2.3E-06	**0.074**
25.0	**50.0**	0.321	16.922	0.0329	0.8630	**0.737**	0.12	2.0E-02	1.5E-02	2.9E-06	**0.109**
50.0	**100.0**	0.312	16.610	0.0320	0.8950	**0.705**	0.09	2.6E-02	7.4E-03	1.9E-06	**0.106**
100.0	**200.0**	0.387	16.223	0.0397	0.9347	**0.665**	0.07	3.2E-02	4.7E-03	1.4E-06	**0.132**
200.0	**400.0**	0.382	15.841	0.0392	0.9739	**0.626**	0.05	4.2E-02	2.4E-03	9.7E-07	**0.130**
400.0	**800.0**	0.428	15.413	0.0439	1.0178	**0.582**	0.05	4.5E-02	1.3E-03	5.9E-07	**0.146**
800.0	**1600.0**	0.485	14.929	0.0497	1.0675	**0.532**	0.04	4.7E-02	7.9E-04	3.6E-07	**0.165**
1600.0	**400.0**	-0.052	14.981	-0.0054	1.0621	**0.538**					
400.0	**100.0**	-0.043	15.024	-0.0044	1.0577	**0.542**					
100.0	**25.0**	-0.089	15.113	-0.0091	1.0486	**0.551**					
25.0	**6.0**	-0.074	15.187	-0.0076	1.0410	**0.559**					
6.0	**1.5**	-0.092	15.279	-0.0094	1.0316	**0.568**					

Klohn Crippen Berger

PROJECT NO.:	M09638A03		
PROJECT:	Minto Mine		
LOCATION:	YK	DATE TESTED:	15-Dec-12
SAMPLE NO.:	Underflow Tailings	DETAILS	After 55% solids content settling test
TESTED BY:	BY	CHECKED BY:	JG

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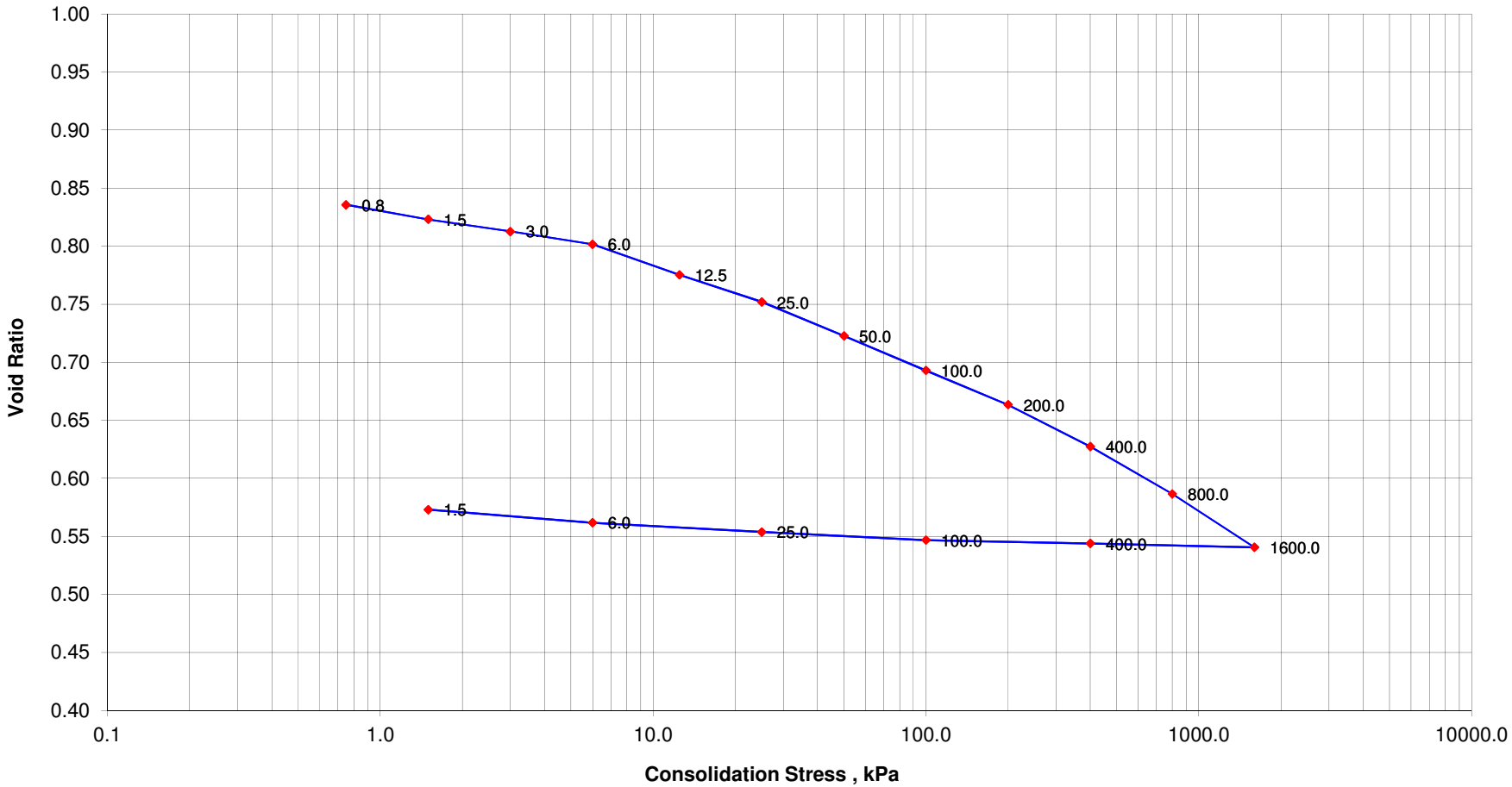
Consolidation Test - Tailings after 60% Solids Content Settling Test

e - log(p)



Klohn Crippen Berger	PROJECT NO.:	M09638A03		
	PROJECT:	Minto Mine		
	LOCATION:	YK	DATE TESTED:	7-Dec-2012
	SAMPLE NO.:	Underflow Tailings	DEPTH:	After 60% solids content settling test
	TESTED BY:	BY	CHECKED BY:	JG

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CONSOLIDATION

PROJECT NO.:	M09638A03
PROJECT:	Minto Mine
SAMPLE NO.:	Underflow Tailings
DETAILS	After 60% solids content settling test
	Supernatant removed
TEST NO.:	T1262CONS01
LOADING MACHINE NO.:	OED1 / ID80

Test Specimen Information:

Initial water content:	46.80 % (based on trimmings)
Final water content:	21.83 % (based on final dry weight)
Dry mass:	94.16 g
Diameter	63.53 mm
Area	31.699 cm^2
Specific Gravity	2.72

Initial Specimen Height (mm):	24.71
Height of Solid (mm):	10.921
Initial void ratio:	1.263
Void Ratio Factor	0.0916

* Calibration to be done after test

** Estimated t_{50}

Pressure (kPa)		*Change in Height	Final	Change in	Change in	Void	t_{50}**	Cv	Mv	k	Cc
From	To	Corrected (mm)	Height (mm)	Void Ratio	Void Ratio Acc	Ratio	(min)	(cm^2/sec)	(cm^2/N)	(cm/sec)	
0.0	**0.8**	4.664	20.047	0.4270	0.4270	**0.836**					
0.8	**1.5**	0.138	19.909	0.0126	0.4397	**0.823**					
1.5	**3.0**	0.113	19.796	0.0103	0.4500	**0.813**					
3.0	**6.0**	0.121	19.675	0.0111	0.4611	**0.802**					
6.0	**12.5**	0.287	19.388	0.0263	0.4874	**0.775**	0.40	7.8E-03	4.9E-02	3.7E-06	**0.082**
12.5	**25.0**	0.256	19.132	0.0234	0.5108	**0.752**	0.20	1.5E-02	2.0E-02	3.0E-06	**0.078**
25.0	**50.0**	0.320	18.812	0.0293	0.5401	**0.723**	0.14	2.1E-02	1.3E-02	2.8E-06	**0.097**
50.0	**100.0**	0.325	18.487	0.0297	0.5698	**0.693**	0.15	1.9E-02	6.9E-03	1.3E-06	**0.099**
100.0	**200.0**	0.323	18.164	0.0296	0.5994	**0.663**	0.08	3.7E-02	3.5E-03	1.3E-06	**0.098**
200.0	**400.0**	0.393	17.771	0.0360	0.6354	**0.627**	0.06	4.7E-02	2.2E-03	1.0E-06	**0.119**
400.0	**800.0**	0.446	17.326	0.0408	0.6762	**0.587**	0.05	5.5E-02	1.3E-03	6.8E-07	**0.136**
800.0	**1600.0**	0.503	16.823	0.0460	0.7222	**0.541**	0.05	5.3E-02	7.3E-04	3.8E-07	**0.153**
1600.0	**400.0**	-0.036	16.860	-0.0033	0.7188	**0.544**					
400.0	**100.0**	-0.032	16.892	-0.0029	0.7159	**0.547**					
100.0	**25.0**	-0.077	16.969	-0.0070	0.7089	**0.554**					
25.0	**6.0**	-0.086	17.055	-0.0079	0.7010	**0.562**					
6.0	**1.5**	-0.123	17.178	-0.0112	0.6897	**0.573**					

Klohn Crippen Berger

PROJECT NO.:	M09638A03		
PROJECT:	Minto Mine		
LOCATION:	YK	DATE TESTED:	7-Dec-12
SAMPLE NO.:	Underflow Tailings	DETAILS	After 60% solids content settling test
TESTED BY:	BY	CHECKED BY:	JG

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APPENDIX II

Grain Size Distribution Results

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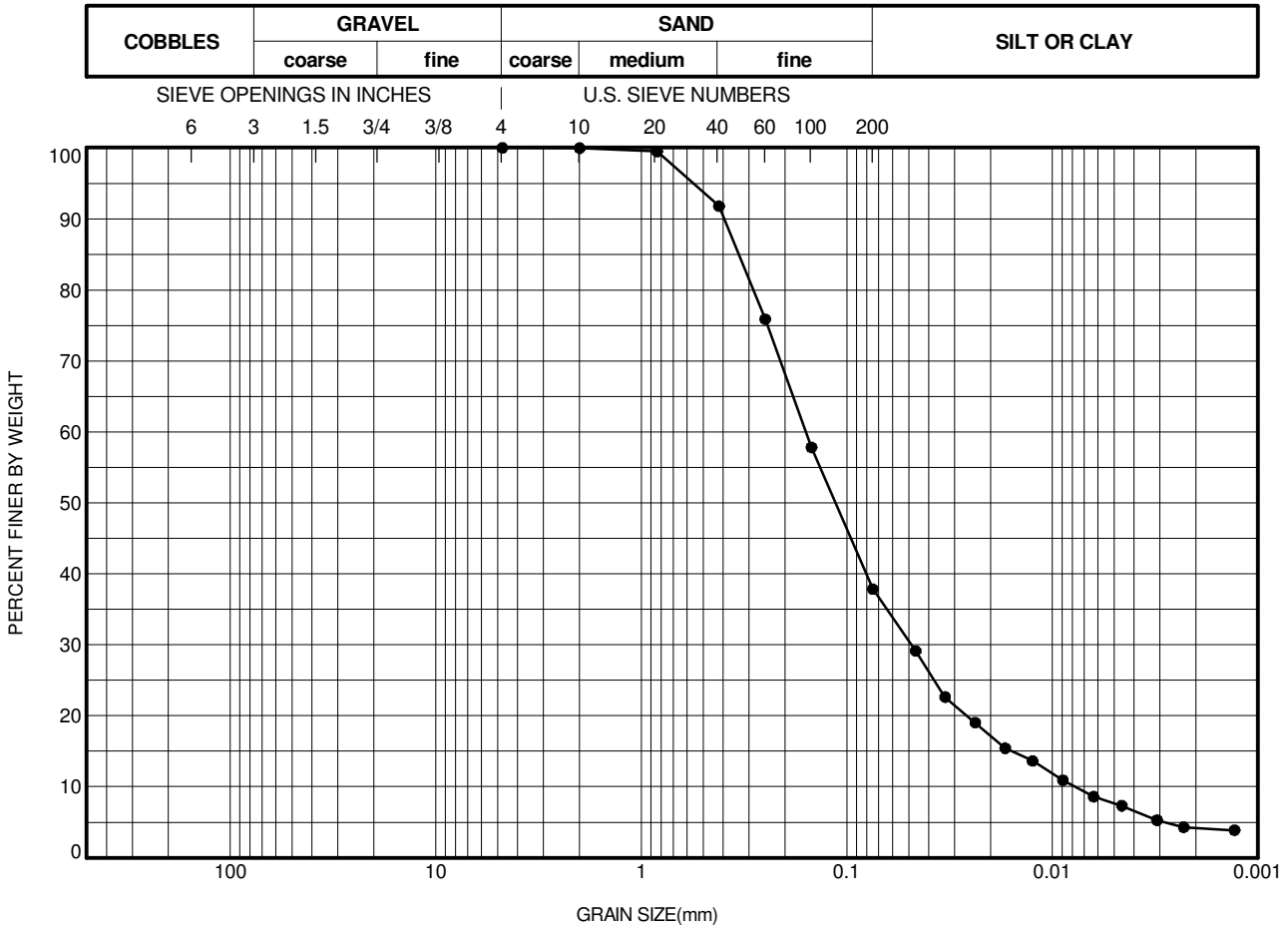
GRAIN SIZE DISTRIBUTION



	HOLE	DEPTH (m)	D85	D60	D50	D15	D10	CU	%GRAVEL	%SAND	%FINES
●	Tailings	0.00	0.337	0.159	0.114				0.0	62.1	37.9

	HOLE	SAMPLE	DEPTH (m)	W%	W_L	W_P	PI	REMARKS / SAMPLE DESCRIPTION
●	Tailings	Underflow	0.00					

CU = COEFFICIENT OF UNIFORMITY = D60/D10 **PARTICLE SIZES, e.g. D85, in mm** **Tested by Wet Sieving Method (ASTM D1140 & D422)**

Klohn Crippen Berger

PROJECT NO.: M09638A03
PROJECT: Minto Mine
LOCATION: Yukon
FIGURE:
DRAWN BY: FG **CHECKED BY:** BY

KCB_GRAIN_SIZE-SI M09638A03 - MINTO MINE T1262.GPJ SIEVE.GDT 12/18/12

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APPENDIX III

Setting Test Results

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SETTLING TEST (using 2L graduated standard beaker)

Beaker #	ST-20-2000
Beaker Tare (g)	509.66
Proposed Solid Content (%)	55%
Supernatant Density (g/cm^3)	1
Start Date	16-Dec-12
Start Time	4:25PM
Temperature (°C)	24.0

Initial Weight + Tare (g)	3718.37	Final Weight + Tare (g)	3715.98
Intial Dry weight (g)	1748.75	Final Dry weight (g)	1748.75
Initial Solid Volume (cm^3)	2091.25	Final Solid Volume (cm^3)	1613.14
Initial Solid Content (%)	54.50%	Final Solid Content (%)	64.04%
Initial Water Content (%)	83.49%	Final Water Content (%)	56.15%
Initial Dry Density (g/cm^3)	0.836	Final Dry Density (g/cm^3)	1.084
Initial Wet Density (g/cm^3)	1.534	Final Wet Density (g/cm^3)	1.693

		Readings											
Elapsed Time (min)	Temp. (°C)	Top Supernatant Readings	Top Solids Readings	Top Settlement Readings	*Height Supernatant (mm)	Volume Supernatant (cm^3)	Volume Supernatant Variation (cm^3)	*Height Solids (settlement) (mm)	Volume Solids (settlement) (cm^3)	Wet Density (g/cm^3)	Dry Density (g/cm^3)	Water Content (%)	Solid Content (%)
0	24.0	254.0	254.0	254.0	0.00	0.00	2091.25	164.77	2091.25	1.534	0.836	83.49%	54.50%
4		254.0	254.0	254.0	0.00	0.00	2091.25	164.77	2091.25	1.534	0.836	83.49%	54.50%
5		254.0	253.0	253.0	0.97	12.42	2078.83	163.80	2078.83	1.538	0.841	82.78%	54.71%
7		254.0	252.0	252.0	1.95	24.84	2066.41	162.82	2066.41	1.541	0.846	82.07%	54.93%
8		254.0	251.5	251.5	2.43	31.05	2060.20	162.34	2060.20	1.542	0.849	81.71%	55.03%
9		254.0	251.0	251.0	2.92	37.25	2053.99	161.85	2053.99	1.544	0.851	81.36%	55.14%
10		254.0	250.5	250.5	3.40	43.46	2047.78	161.37	2047.78	1.546	0.854	81.00%	55.25%
11		254.0	250.0	250.0	3.89	49.67	2041.57	160.88	2041.57	1.547	0.857	80.65%	55.36%
12		254.0	249.5	249.5	4.38	55.88	2035.36	160.39	2035.36	1.549	0.859	80.29%	55.47%
13		254.0	249.0	249.0	4.86	62.09	2029.15	159.91	2029.15	1.551	0.862	79.94%	55.58%
14		254.0	248.5	248.5	5.35	68.30	2022.94	159.42	2022.94	1.552	0.864	79.58%	55.69%
15		254.0	248.0	248.0	5.84	74.51	2016.74	158.93	2016.74	1.554	0.867	79.23%	55.80%
16		254.0	247.5	247.5	6.32	80.72	2010.53	158.45	2010.53	1.556	0.870	78.87%	55.91%
17		254.0	247.0	247.0	6.81	86.93	2004.32	157.96	2004.32	1.558	0.872	78.52%	56.02%
18		254.0	246.5	246.5	7.29	93.14	1998.11	157.48	1998.11	1.559	0.875	78.16%	56.13%
19		254.0	246.0	246.0	7.78	99.35	1991.90	156.99	1991.90	1.561	0.878	77.81%	56.24%
20		254.0	245.5	245.5	8.27	105.56	1985.69	156.50	1985.69	1.563	0.881	77.45%	56.35%
21		254.0	245.0	245.0	8.75	111.76	1979.48	156.02	1979.48	1.565	0.883	77.10%	56.47%
22		254.0	244.5	244.5	9.24	117.97	1973.27	155.53	1973.27	1.566	0.886	76.74%	56.58%
23		254.0	244.0	244.0	9.73	124.18	1967.06	155.04	1967.06	1.568	0.889	76.38%	56.69%
24		254.0	243.5	243.5	10.21	130.39	1960.85	154.56	1960.85	1.570	0.892	76.03%	56.81%
25		254.0	243.0	243.0	10.70	136.60	1954.64	154.07	1954.64	1.572	0.895	75.67%	56.92%
26		254.0	242.5	242.5	11.18	142.81	1948.43	153.59	1948.43	1.574	0.898	75.32%	57.04%
27		254.0	242.0	242.0	11.67	149.02	1942.23	153.10	1942.23	1.575	0.900	74.96%	57.15%
28		254.0	241.0	241.0	12.64	161.44	1929.81	152.13	1929.81	1.579	0.906	74.25%	57.39%
29		254.0	240.0	240.0	13.62	173.86	1917.39	151.15	1917.39	1.583	0.912	73.54%	57.62%
30		254.0	239.0	239.0	14.59	186.27	1904.97	150.18	1904.97	1.587	0.918	72.83%	57.86%
31		254.0	238.0	238.0	15.56	198.69	1892.55	149.21	1892.55	1.590	0.924	72.12%	58.10%
32		254.0	237.0	237.0	16.53	211.11	1880.13	148.24	1880.13	1.594	0.930	71.41%	58.34%
33		254.0	236.0	236.0	17.51	223.53	1867.72	147.26	1867.72	1.598	0.936	70.70%	58.58%
34		254.0	234.5	234.5	18.96	242.16	1849.09	145.81	1849.09	1.604	0.946	69.64%	58.95%
35		254.0	233.5	233.5	19.94	254.58	1836.67	144.83	1836.67	1.608	0.952	68.93%	59.20%
36		254.0	232.5	232.5	20.91	266.99	1824.25	143.86	1824.25	1.613	0.959	68.22%	59.45%
37		254.0	232.0	232.0	21.40	273.20	1818.04	143.37	1818.04	1.615	0.962	67.86%	59.57%
39		254.0	231.0	231.0	22.37	285.62	1805.62	142.40	1805.62	1.619	0.969	67.15%	59.83%
41		254.0	230.0	230.0	23.34	298.04	1793.21	141.43	1793.21	1.623	0.975	66.44%	60.08%
44		254.0	229.0	229.0	24.31	310.46	1780.79	140.46	1780.79	1.628	0.982	65.73%	60.34%
47		254.0	228.0	228.0	25.29	322.88	1768.37	139.48	1768.37	1.632	0.989	65.02%	60.60%
52		254.0	227.0	227.0	26.26	335.29	1755.95	138.51	1755.95	1.636	0.996	64.31%	60.86%
57		254.0	226.0	226.0	27.23	347.71	1743.53	137.54	1743.53	1.641	1.003	63.60%	61.12%
64		254.0	225.0	225.0	28.20	360.13	1731.11	136.57	1731.11	1.646	1.010	62.89%	61.39%
70		254.0	224.0	224.0	29.18	372.55	1718.70	135.59	1718.70	1.650	1.017	62.18%	61.66%
78		254.0	223.0	223.0	30.15	384.97	1706.28	134.62	1706.28	1.655	1.025	61.47%	61.93%
88		254.0	222.0	222.0	31.12	397.39	1693.86	133.65	1693.86	1.660	1.032	60.76%	62.20%
102		254.0	221.0	221.0	32.09	409.80	1681.44	132.68	1681.44	1.665	1.040	60.05%	62.48%
115		254.0	220.0	220.0	33.07	422.22	1669.02	131.70	1669.02	1.670	1.048	59.34%	62.76%
138		254.0	219.0	219.0	34.04	434.64	1656.60	130.73	1656.60	1.675	1.056	58.63%	63.04%
155		254.0	218.0	218.0	35.01	447.06	1644.19	129.76	1644.19	1.680	1.064	57.92%	63.32%
207		254.0	217.0	217.0	35.98	459.48	1631.77	128.79	1631.77	1.685	1.072	57.21%	63.61%
253		254.0	216.0	216.0	36.96	471.90	1619.35	127.81	1619.35	1.690	1.080	56.50%	63.90%
313		254.0	215.7	215.7	37.25	475.62	1615.62	127.52	1615.62	1.692	1.082	56.29%	63.98%
373		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%
480		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%
960		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%
1440		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%

Klohn Crippen Berger

PROJECT NO.:	M9638A03	DATE:	16-Dec-12
PROJECT:	Minto Mine	LOCATION:	Yukon
DETAIL:	55% Solids Content		
DESCRIPTION:	Underflow Tailings		
TESTED BY:	FG	CHECKED BY:	BY

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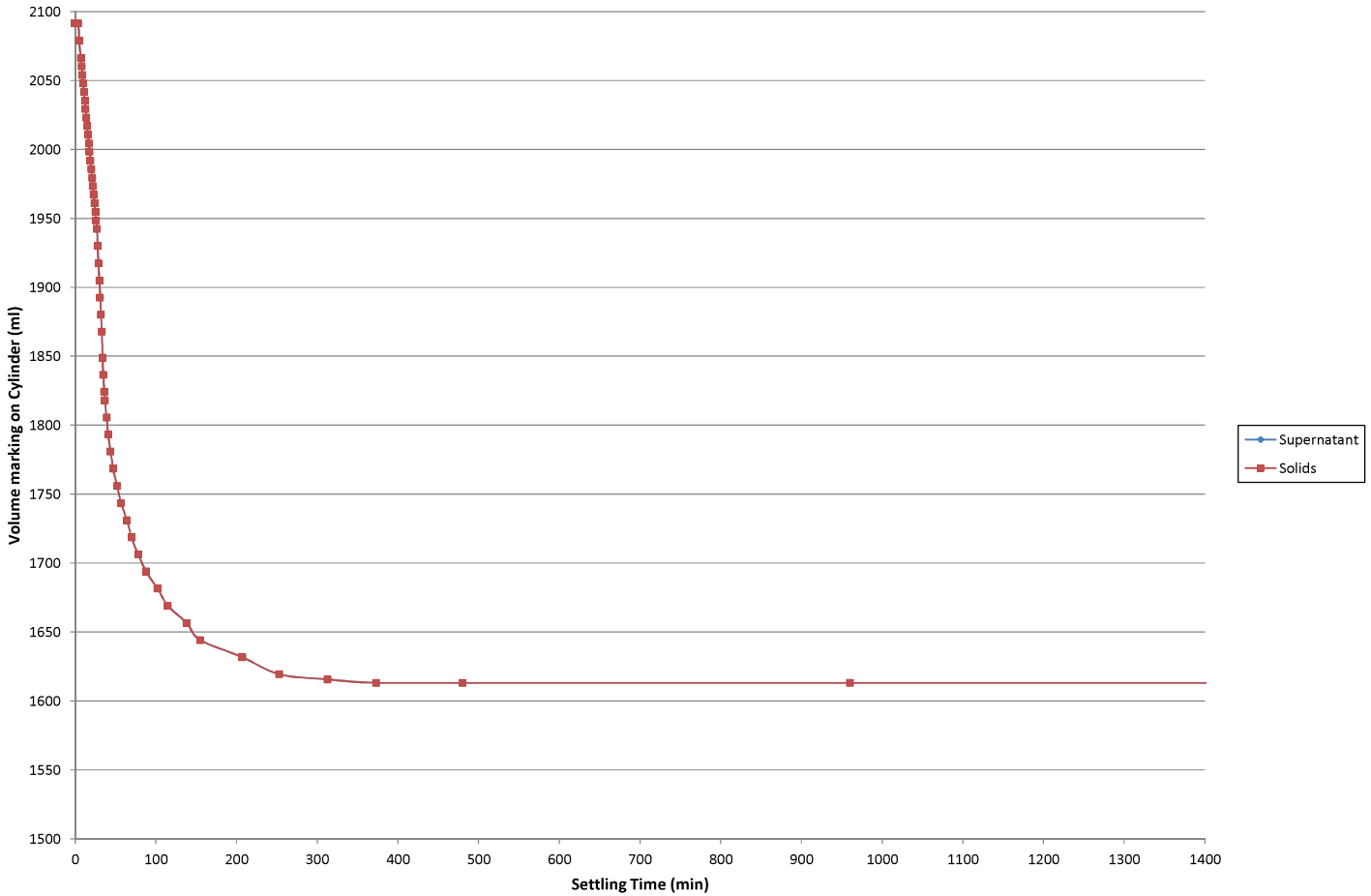
Settlement Test for Underflow Tailings at 55% Solids Content



Klohn Crippen Berger	PROJECT NO.:	M9638A03		DATE	16-Dec-12
	PROJECT:	Minto Mine		LOCATION:	Yukon
	DETAIL:	55% Solids Content			
	DESCRIPTION:	Underflow Tailings			
	TESTED BY:	FG	CHECKED BY BY	SHEET	2 of 2

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SETTLING TEST (using 2L graduated standard beaker)

Beaker #	ST-06-2000
Beaker Tare (g)	535.93
Proposed Solid Content (%)	60%
Supernatant Density (g/cm^3)	1
Start Date	3-Dec-12
Start Time	1:34 PM
Temperature (°C)	23.5

Initial Weight + Tare (g)	3650.82	Final Weight + Tare (g)	3644.20
Intial Dry weight (g)	1853.36	Final Dry weight (g)	1853.36
Initial Solid Volume (cm^3)	1976.18	Final Solid Volume (cm^3)	1521.47
Initial Solid Content (%)	59.50%	Final Solid Content (%)	69.67%
Initial Water Content (%)	68.07%	Final Water Content (%)	43.53%
Initial Dry Density (g/cm^3)	0.938	Final Dry Density (g/cm^3)	1.218
Initial Wet Density (g/cm^3)	1.576	Final Wet Density (g/cm^3)	1.748

		Readings											
Elapsed Time (min)	Temp. (°C)	Top Supernatant Readings	Top Solids Readings	Top Settlement Readings	*Height Supernatant (mm)	Volume Supernatant (cm^3)	Volume Supernatant Variation (cm^3)	*Height Solids (settlement) (mm)	Volume Solids (settlement) (cm^3)	Wet Density (g/cm^3)	Dry Density (g/cm^3)	Water Content (%)	Solid Content (%)
0	23.5	248.0	248.0	248.0	0.00	0.00	1976.18	157.38	1976.18	1.576	0.938	68.07%	59.50%
1		248.0	247.5	247.5	0.49	6.14	1970.04	156.89	1970.04	1.578	0.941	67.74%	59.62%
2		248.0	247.0	247.0	0.97	12.29	1963.89	156.41	1963.89	1.580	0.944	67.40%	59.74%
3		248.0	246.0	246.0	1.95	24.58	1951.60	155.43	1951.60	1.583	0.950	66.74%	59.97%
4		248.0	245.0	245.0	2.92	36.87	1939.31	154.46	1939.31	1.587	0.956	66.08%	60.21%
5		248.0	244.0	244.0	3.89	49.16	1927.02	153.49	1927.02	1.591	0.962	65.41%	60.45%
6		248.0	243.0	243.0	4.86	61.45	1914.74	152.52	1914.74	1.595	0.968	64.75%	60.70%
7		248.0	241.5	241.5	6.32	79.88	1896.30	151.06	1896.30	1.600	0.977	63.76%	61.07%
8		248.0	240.0	240.0	7.78	98.32	1877.87	149.60	1877.87	1.606	0.987	62.76%	61.44%
9		248.0	238.0	238.0	9.73	122.90	1853.29	147.65	1853.29	1.614	1.000	61.44%	61.94%
10		248.0	237.0	237.0	10.70	135.18	1841.00	146.68	1841.00	1.619	1.007	60.77%	62.20%
11		248.0	236.0	236.0	11.67	147.47	1828.71	145.71	1828.71	1.623	1.013	60.11%	62.46%
12		248.0	235.0	235.0	12.64	159.76	1816.42	144.74	1816.42	1.627	1.020	59.45%	62.72%
13		248.0	233.5	233.5	14.10	178.20	1797.98	143.28	1797.98	1.633	1.031	58.45%	63.11%
14		248.0	232.0	232.0	15.56	196.63	1779.55	141.82	1779.55	1.640	1.041	57.46%	63.51%
15		248.0	230.5	230.5	17.02	215.07	1761.12	140.36	1761.12	1.647	1.052	56.46%	63.91%
16		248.0	228.5	228.5	18.96	239.65	1736.54	138.42	1736.54	1.656	1.067	55.14%	64.46%
17		248.0	226.0	226.0	21.40	270.37	1705.81	135.98	1705.81	1.668	1.086	53.48%	65.16%
18		248.0	225.0	225.0	22.37	282.66	1693.52	135.01	1693.52	1.672	1.094	52.82%	65.44%
19		248.0	224.0	224.0	23.34	294.95	1681.23	134.04	1681.23	1.677	1.102	52.15%	65.72%
20		248.0	223.0	223.0	24.31	307.24	1668.94	133.07	1668.94	1.682	1.110	51.49%	66.01%
22		248.0	222.0	222.0	25.29	319.53	1656.65	132.09	1656.65	1.687	1.119	50.83%	66.30%
24		248.0	221.0	221.0	26.26	331.82	1644.37	131.12	1644.37	1.692	1.127	50.16%	66.59%
28		248.0	220.0	220.0	27.23	344.11	1632.08	130.15	1632.08	1.698	1.136	49.50%	66.89%
32		248.0	219.0	219.0	28.20	356.40	1619.79	129.18	1619.79	1.703	1.144	48.84%	67.19%
37		248.0	218.0	218.0	29.18	368.69	1607.50	128.20	1607.50	1.708	1.153	48.17%	67.49%
42		248.0	217.0	217.0	30.15	380.98	1595.21	127.23	1595.21	1.714	1.162	47.51%	67.79%
53		248.0	216.0	216.0	31.12	393.27	1582.92	126.26	1582.92	1.719	1.171	46.85%	68.10%
65		248.0	215.0	215.0	32.09	405.55	1570.63	125.29	1570.63	1.725	1.180	46.19%	68.41%
82		248.0	214.0	214.0	33.07	417.84	1558.34	124.31	1558.34	1.731	1.189	45.52%	68.72%
110		248.0	213.0	213.0	34.04	430.13	1546.05	123.34	1546.05	1.737	1.199	44.86%	69.03%
170		248.0	212.0	212.0	35.01	442.42	1533.76	122.37	1533.76	1.742	1.208	44.20%	69.35%
240		248.0	211.5	211.5	35.50	448.57	1527.61	121.88	1527.61	1.745	1.213	43.86%	69.51%
470		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1140		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1230		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1320		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1390		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1440		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%

Klohn Crippen Berger

PROJECT NO:	M09638A03			DATE:	3-Dec-12
PROJECT:	Minto Mine			LOCATION:	Yukon
DETAIL:	60% Solids Content				
DESCRIPTION:	Underflow Tailings				
TESTED BY:	FG	CHECKED BY:	BY	SHEET	1 of 2

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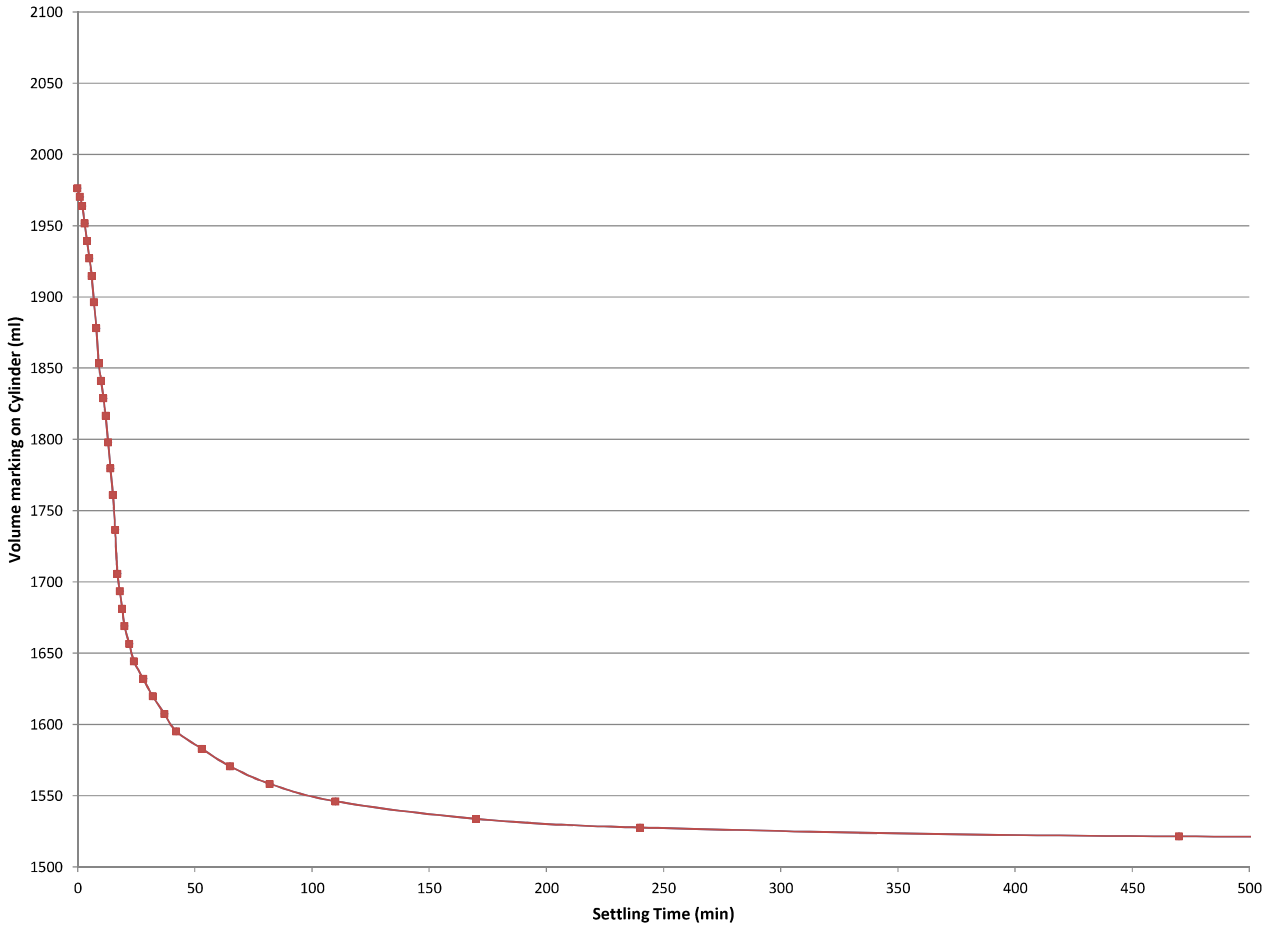
Settlement Test for Underflow Tailings at 60% Solids Content



Klohn Crippen Berger	PROJECT NO:	M09638A03		DATE	3-Dec-12
	PROJECT:	Minto Mine		LOCATION:	Yukon
	DETAIL:	60% Solids Content			
	DESCRIPTION:	Underflow Tailings			
	TESTED BY:	FG	CHECKED BY BY	SHEET	2 of 2

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APPENDIX IV

Specific Gravity

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SPECIFIC GRAVITY OF SOIL SOLIDS
(ASTM-D854)

Sample No.		Underflow Tailings					
Flask No.		KL-2	KL-3	KLA-4			
Volume of Flask @ 20° C	ml	500	500	500			
Method of Air removal		Boiling	Boiling	Boiling			
De-airing Period	hr	2	2	2			
Test temperature	° C	22.8	22.8	22.8			
Mass of Flask+Water (M_a)	g	675.48	676.08	678.54			
Mass of Flask+Water+Soil (M_b)	g	712.53	712.98	715.36			
Mass of Dish/Flask+Soil		235.75	235.77	238.34			
Mass of Dish/Flask		177.12	177.39	180.06			
Mass of Dry Soil (M_o)	g	58.63	58.38	58.28			
Correction factor (K) @ Test Temperature		0.9994	0.9994	0.9994			
Specific Gravity of Solids @ 20° C		2.715	2.716	2.714			
Average Specific Gravity of Solids @ 20° C		**2.72**					

Sample No.							
Flask No.							
Volume of Flask @ 20° C	ml						
Method of Air removal							
De-airing Period	hr						
Test temperature	° C						
Mass of Flask+Water (M_a)	g						
Mass of Flask+Water+Soil (M_b)	g						
Mass of Dish/Flask+Soil							
Mass of Dish/Flask							
Mass of Dry Soil (M_o)	g						
Correction factor (K) @ Test Temperature							
Specific Gravity of Solids @ 20° C							
Average Specific Gravity of Solids @ 20° C							

Specific Gravity of Solids @ 20° C = $(K \times M_o)/(M_o + M_a - M_b)$

Klohn Crippen Berger

PROJECT NO.:	M09638A03
PROJECT:	Minto Mine
LOCATION:	Yukon
DATE:	29-Nov-12
TESTED BY:	FG CHECKED BY BY

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Appendix E: Main Dam Probable Maximum Flood Estimation

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Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Victor Muñoz	**Project No:**	1CM002.018
Cc:	Maritz Rykaart, Dylan MacGregor, John Duncan, Murray McGregor	**Date:**	July 14, 2014
Subject:	Main Dam Probable Maximum Flood (PMF) Estimation		

1 Introduction

This memo summarizes the estimation of the probable maximum flood (PMF) for the Minto Main Dam.

2 Approach

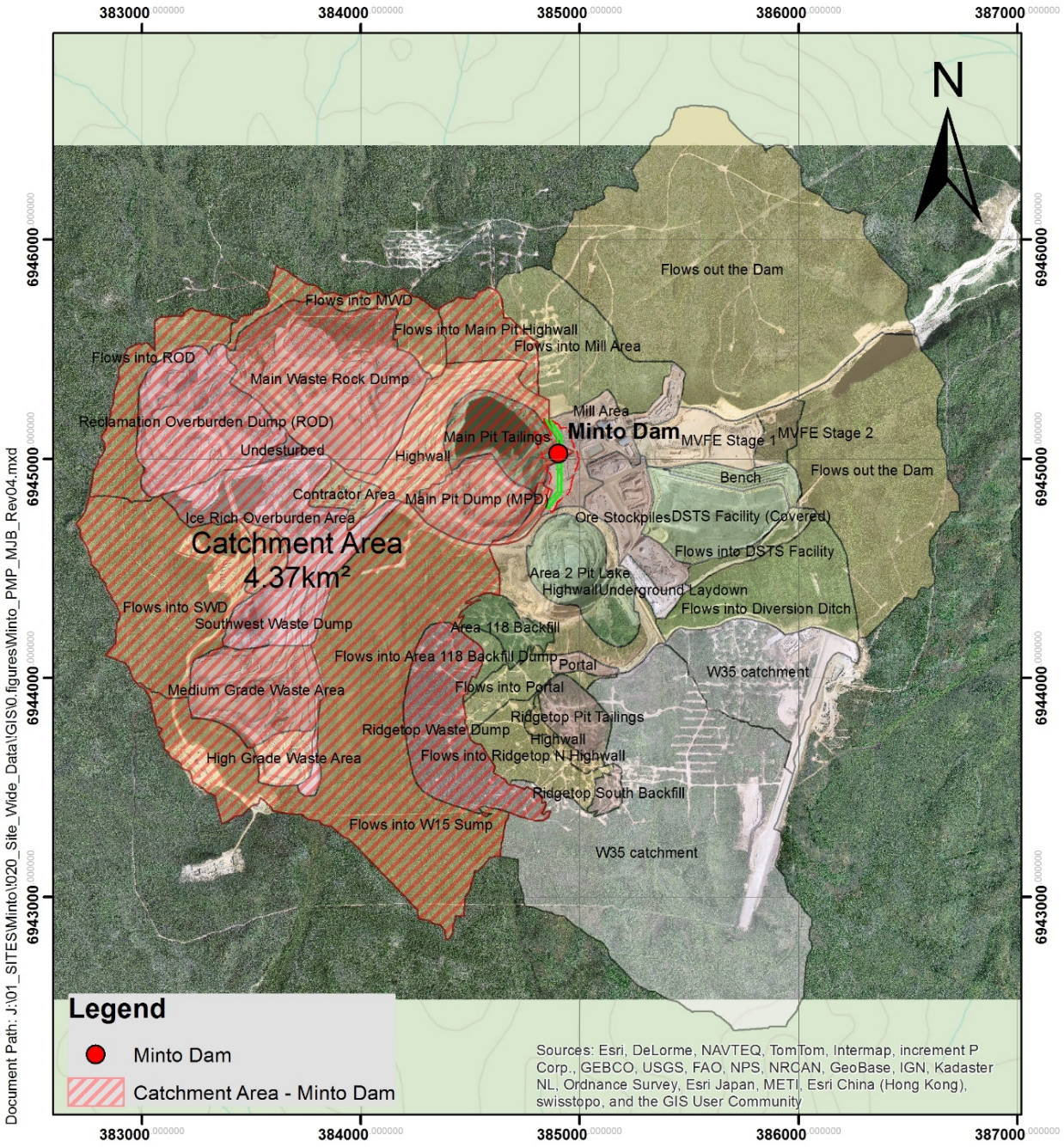
2.1 Supporting Information

Supporting information for this memo includes topographical data and runoff gauging records. Topographical information was obtained from Geogratis and LiDAR topography flown in 2013. Geogratis is a public geospatial data service provided by Natural Resources Canada (NRCan 2014). Its digital elevation model (DEM) has a point density of 12 x 24 m in the easting and northing, respectively. Capstone provided LiDAR topographic data, in the form of a high precision Digital Elevation Model (DEM) of the mine site, with a precision of 0.2 x 0.2 m in the easting and northing.

Runoff information from regional gauging stations was obtained from Environment Canada (EC 2012). Data included maximum daily and instantaneous peak flows from 34 stations in the Yukon.

2.2 Topography and Watershed Definition

The catchment area for the Main Dam was defined using supporting topographical information from Capstone, and was prepared using Global Mapper™ and ArcGIS™. This software performs digital elevation mapping (DEM) manipulations, including catchment delineation and contour line definition. The catchment areas were corrected based on existing mine infrastructure and are presented in Figure 1.



Coordinate System: NAD 1983 UTM Zone 8N
Projection: Transverse Mercator
Datum: North American 1983

0 0.3 0.6 1.2 km

Figure 1: Main Dam Catchment

General hydrologic parameters for the watershed were estimated, including time of concentration, average slope, and maximum, minimum, and average elevation. Time of concentration was estimated using multiple expressions, including Chow, California Highways, Watt & Chow, Lindley, Papadakis & Kazan and TxDOT expressions, (Loukas 1996) and (Li 2008). These equations are based on the geomorphological parameters of the watershed. The evaluations for these expressions are presented in Table 1. The overall recommended time of concentration for the catchment is approximately 25 minutes, which corresponds to the average of the time of concentrations from the cited expressions.

Table 1: Time of Concentration for the Main Dam area

Max. Elevation [m]	Min. Elevation [m]	Elevation Difference [m]	Path Length [m]	Average Slope		Manning's Roughness	Runoff Coefficient	Time of Concentration [min]						
				[%]	[m/km]			Chow	California Highways	Watt & Chow	Lindley	Papadakis & Kazan	TxDOT	Suggested
1016	791	225	2100	0.14	137.3	0.025	0.5	29	17	30	26	11	37	**25**

3 Probable Maximum Flood Estimation

3.1 Methodology

The Creager methodology (Neill, 1986) was used to estimate the PMF for the Main Dam. Creager diagrams describe empirical relationships between the PMF and the drainage area, based on the Creager coefficient, “C”, which varies based on regional information. Figure 2 presents the Creager curves, for “C” equivalents of 30, 60, and 100 (Neill 1986) based on maximum flood events in United States and Canada.

By definition, the PMF is the resulting flood presented after a 24 hour probable maximum precipitation (PMP) storm. The “C” parameter was revised based on regional and national information, and was adjusted for the duration of the storm event.

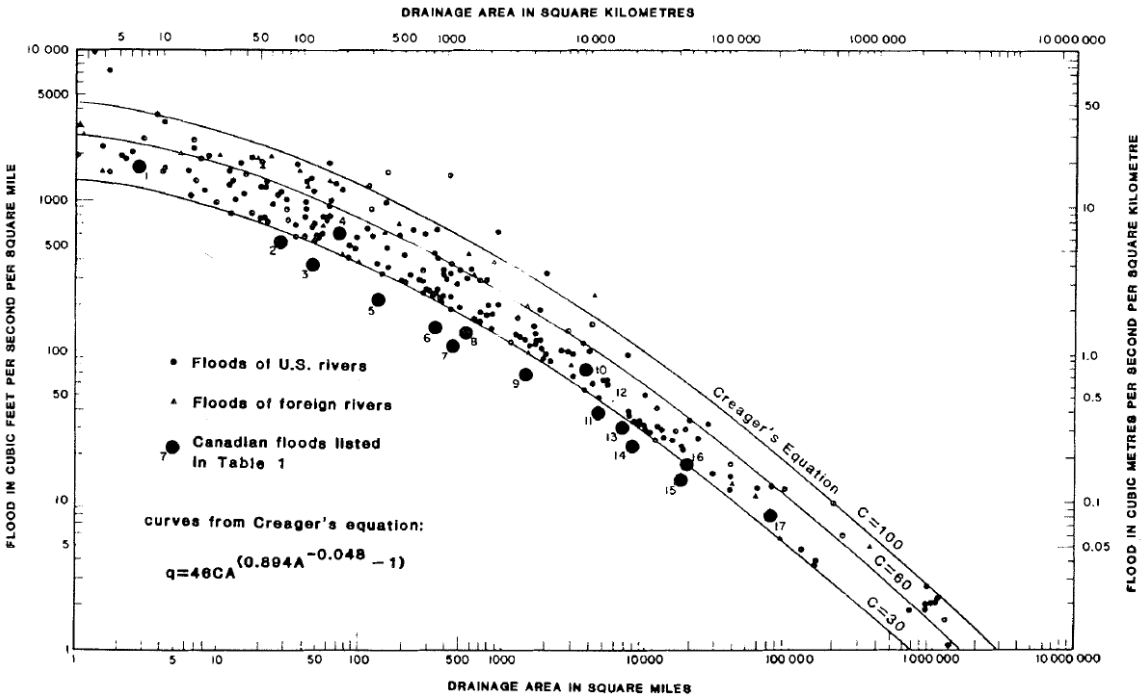


Figure 2: Creager Diagram (Neill, 1986)

3.2 Compilation of Yukon Maximum Gauging Records with Creager Diagram

The Creager diagram presents a PMF function based on catchment areas. The “C” parameter has to be defined based on regional characteristics. Regional gauging information from the Yukon Territory was compiled in order to establish regional characteristics for the site.

Thirty-four gauging stations were selected in the Yukon Territory, and the maximum instantaneous flood was estimated. Two records were compiled for each station: the maximum daily flow and maximum annual instantaneous flow. The maximum daily flows were then transformed into maximum instantaneous flows using the Sangal expression (Sangal 2010). Both sets of maximum flow records were plotted against their respective areas within the Creager plots.

Table 2 presents the monthly maximum flow, the maximum daily flow, the estimated maximum instantaneous flow using Sangal’s expression, as well as the maximum instantaneous flow from the Environment Canada records. Both maximum flows and station catchment areas are displayed within the Creager curve plot in Figure 3, with a maximum “C” coefficient of 16.

Table 2: Maximum historical flow for stations in Yukon Territory

Station ID	Average daily maximum flow [m³/s]												Max daily flow [m³/s]	Estimated Max peak flow from Daily Flows Sangal(2010) [m³/s]	Max. Instantaneous Peak Flow [m³/s]
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec			
09AA011	0.24	0.24	0.34	1.42	4.08	1.53	1.37	0.60	0.73	0.66	0.34	0.25	4.08	5	3
09AA012	3.12	2.31	2.18	10.70	49.60	91.50	78.90	54.50	41.60	16.00	7.36	5.20	91.5	100	103
09AA014	4.92	3.94	7.05	13.40	84.80	139.00	147.00	180.00	215.00	98.00	30.60	10.30	215	298	228
09AA015	1.93	1.70	1.78	1.41	31.30	55.50	56.50	40.80	42.10	23.50	7.15	3.06	56.5	58	57
09AB001	216.00	197.00	209.00	229.00	385.00	544.00	637.00	646.00	635.00	576.00	524.00	286.00	646	654	643
09AB008	7.00	5.70	5.65	29.20	101.00	104.00	50.10	33.40	33.30	26.90	13.00	8.90	104	112	113
09AB009	246.00	224.00	220.00	224.00	331.00	796.00	813.00	824.00	818.00	738.00	568.00	368.00	824	827	827
09AC001	25.10	19.90	17.70	35.00	117.00	297.00	343.00	340.00	487.00	176.00	75.30	47.60	487	535	329
09AC004	19.70	12.90	12.00	11.70	57.00	279.00	264.00	228.00	170.00	117.00	59.70	33.60	279	285	280
09AC007	1.09	0.82	0.70	4.76	14.80	37.30	25.00	9.40	17.10	4.57	7.28	1.72	37.3	48	47
09AD001	48.80	24.80	25.10	111.00	545.00	713.00	590.00	185.00	213.00	229.00	103.00	80.50	713	731	720
09AD002	1.97	1.32	1.25	6.62	53.20	58.60	40.00	10.00	13.90	16.90	3.88	2.62	58.6	72	66
09AE001	192.00	151.00	138.00	137.00	1200.00	1840.00	1790.00	867.00	736.00	617.00	492.00	274.00	1840	1850	1730
09AE004	8.64	7.80	9.70	7.78	62.60	120.00	80.00	44.20	41.30	32.60	23.10	12.70	120	122	120
09AE006	6.67	5.28	4.10	21.00	144.00	143.00	86.50	55.10	62.90	41.80	19.00	11.50	144	149	146
09AF001	221.00	173.00	130.00	137.00	1100.00	1850.00	1790.00	872.00	782.00	663.00	436.00	255.00	1850	1860	1860
09AG001	42.30	30.30	27.00	65.00	462.00	671.00	397.00	262.00	220.00	141.00	93.00	59.00	671	766	685
09AG003	2.09	1.34	1.04	5.60	76.90	58.70	45.00	25.60	21.00	10.40	5.10	3.40	76.9	103	86
09AH001	517.00	456.00	435.00	960.00	1740.00	3600.00	3260.00	2070.00	1670.00	1620.00	1160.00	651.00	3600	3760	2890
09AH003	1.93	0.80	0.62	142.00	214.00	134.00	243.00	249.00	104.00	27.10	5.47	2.98	249	337	371
09AH004	9.40	7.60	7.10	117.00	199.00	185.00	103.00	120.00	101.00	80.50	24.70	13.10	199	221	244
09AH005	3.77	2.36	2.46	2.85	20.00	27.80	15.40	25.40	22.50	17.60	10.90	5.32	27.8	31	31
09BA001	18.60	12.90	11.10	231.00	716.00	742.00	450.00	263.00	225.00	232.00	54.60	32.90	742	769	762
09BA002	21.20	11.70	9.75	38.00	611.00	596.00	289.00	271.00	214.00	232.00	71.90	29.00	611	675	621
09BB001	4.68	2.67	2.39	12.40	163.00	192.00	141.00	84.50	105.00	68.00	13.50	7.10	192	217	208
09BB002	38.30	28.00	23.80	286.00	1110.00	1100.00	555.00	409.00	489.00	348.00	94.00	54.50	1110	1165	1130
09BC001	143.00	103.00	84.40	566.00	4300.00	4130.00	2170.00	1430.00	1350.00	1370.00	490.00	244.00	4300	4820	4160
09BC002	41.30	30.60	24.90	70.80	1550.00	2010.00	1070.00	507.00	498.00	354.00	108.00	68.50	2010	2180	2010
09BC003	0.47	0.23	0.22	0.39	32.60	30.30	15.50	9.94	7.93	3.26	1.03	0.79	32.6	34	-
09BC004	57.50	45.10	37.60	175.00	1520.00	1800.00	858.00	828.00	716.00	802.00	177.00	97.10	1800	3600	1550
09BC005	7.50	6.28	5.45	43.30	179.00	185.00	152.00	116.00	107.00	58.50	21.10	14.00	185	188	189
09CA002	32.80	27.40	24.60	27.50	66.50	209.00	340.00	413.00	320.00	209.00	87.10	48.50	413	429	427
09CA003	35.50	27.30	38.40	86.00	261.00	682.00	1430.00	976.00	587.00	300.00	125.00	56.00	1430	1749	1700
09CA004	2.18	1.90	1.66	10.10	36.70	122.00	308.00	69.50	60.70	12.50	4.31	3.10	308	418	573

Source: Z:\01_SITES\Minto\!020_Site_Wide_Data\PMP_+_PMF\Peak Flows\Peak Flows Yukon_VM_Rev1.xlsx

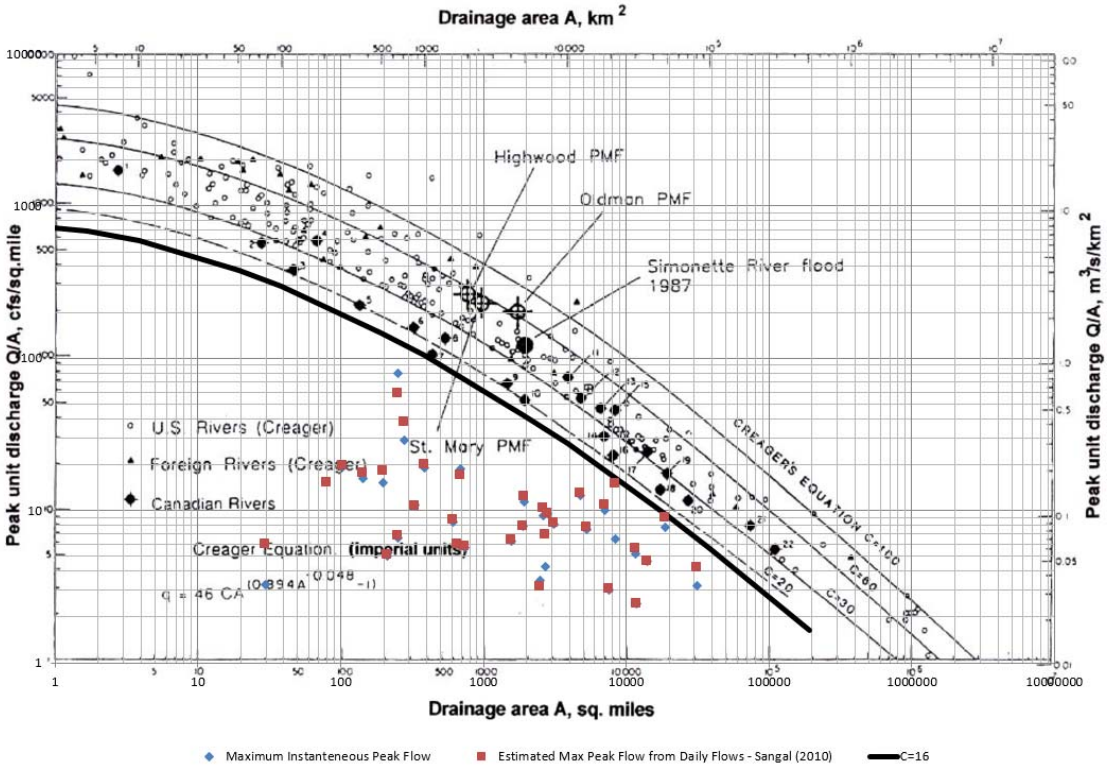


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Figure 3: Creager Curve with Maximum Flows within the Yukon Territory

3.3 Normalizing for Regional and National Data

The Creager curves were compared to national PMF estimations. Neill (1986) suggests that for Canadian floods, “C” values are expected to be within the range of 20 to 45. Abrahamson (2010) proposes using different methodologies to estimate the PMF in British Columbia and suggests “C” values between 20 and 70 for catchments with areas in the range of 20 – 200,000 km².

3.4 Correction for PMF Duration

The time of concentration for the Main Dam catchment was determined to be 25 minutes. Since the PMF from the Creager curves is based on a 24 hour duration event, it must be corrected for a duration closer to the time of concentration.

Abrahamson provided several regression expressions to convert the 24 hour duration event into various smaller durations for British Columbia watersheds (Abrahamson, 2010). The PMF for the Main Dam was adjusted based on the 1 hour conversion expression.

3.5 Resultant PMF for the Main Dam area

The PMF was estimated using the Creager curve, with a Creager coefficient of 16 and 20 from regional information in the Yukon Territory. A “C” value of 20 is the minimum recommended for the mine site based on suggestions from Neill (1986) and Abrahamson (2010). Based on the time of concentration of 25 minutes for the catchment, the PMF was then corrected for a one-hour duration. The result is a PMF for the Main Dam of 55 m³/s (or 12.5 m³/s/km²). This information is summarized in Table 3.

Table 3: Probable Maximum Flood for the Main Dam

Parameter	Value	Units	Comments
PMF (24 h)	32.9	[m³/s]	Creager (C=16)
PMF (24 h)	41.1	[m³/s]	Creager (C=20)
PMF (1 h)	54.6	[m³/s]	Relationship between PMF 24 h to PMF 1 h. (Abrahamson, 2010)

Source: Z:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Water Resources\PMF\PMF_Minto Site-West_VM_Rev3.xlsm

After the adjustment for the shorter storm duration, the PMF for the Main Dam has a Creager coefficient “C” of 26, which is within the expected range.

4 Benchmarking

4.1 Probable Maximum Precipitation (PMP)

Miller (1963) prepared the probable maximum precipitation (PMP) for Alaska for a 24-hour duration event. In this report, a PMP of 254.0 mm (10 inches) is suggested for the Minto area, which is based on a station under the U.S. authority located 200 km west of the site. For the Wareham Dam at Mayo, which is located 130 km from the site, Hogg (2002) prepared a probable maximum precipitation (PMP) for a 24-hour duration storm. This estimation suggests a PMP of 116.1 mm. Another example, the Faro mine site, which is located 200 km to the east of the site, Taylor et al. (2006) presents a PMP of 184.1 mm for a 24-hour duration event.

The rainfall from the 24-hour duration PMP event is multiplied by the catchment area to obtain the total volume of storm water. This volume of water is similar to the total amount of runoff from a 24-hour duration triangular distribution event. Under this assumption, no losses have been included and the total amount of precipitation is completely translated into runoff. Based on this simplified approach, the PMP values previously described would result in peak flows of 25.7, 11.7 and 18.6 m³/s from Miller (1963), Hogg (2002) and Taylor (2006), respectively. These numbers are not PMF estimations; however, they demonstrate the likely order of magnitude for a PMF at the Main Dam.

4.2 Probable Maximum Flood (PMF)

Water Management Consultants (2006) presented a detailed analysis about the probable maximum flood in Rose Creek for the Faro Mine site. This analysis included the estimation of the PMP for three sub-catchments. The results suggested peak flows of 384, 674 and 692 m^3/s for areas of 122.5, 83.5 and 11.3 km^2, respectively. These numbers produce unit area peak flows of 3.14, 8.07 and 61.2 $m^3/s/km^2$, and are comparable with the 12.5 $m^3/s/km^2$ (55 m^3/s) PMF value suggested for the Main Dam.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

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Appendix F: Main Dam Spillway Hydralic Design

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Memo

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Cc:	Maritz Rykaart, Dylan MacGregor, John Duncan, Murray McGregor	**Date:**	July 15, 2014
Subject:	Main Dam Spillway Hydraulic Design		

1 Design Assumptions

This memo presents the preliminary hydraulic design for the Main Dam spillway, including the size and form of the erosion protection. The primary design assumptions include:

- Design flow: 54.6 m^3/s, which is associated with a one-hour probable maximum flood (PMF) event as documented in SRK (2014).
- Parameter estimation: Manning's roughness coefficient was calculated using an iterative process around the chosen riprap size, using five empirical expressions (Strickler (1923), Anderson *et al*. (1970), Abt *et al*. (1988), Limerinos (1970), and Jarrett (1984) all cited in Abt *et al*. (1988)).
- Riprap: The riprap size (D_{50}) was estimated using empirical expressions by Isbach (USACE 1991), Robinson *et al*. (1998) and Khan and Ahmad (2011). The minimum riprap size of 200 mm was based on BCMOT (2006).
- Geometry: The side slopes (transverse) spillway slopes were set at 3H:1V, based on what was deemed geotechnically stable. The dam crest is assumed to be at an elevation of 812 masl, with a 3 m freeboard which means the spillway invert level is set at elevation 809 masl. The spillway will consist of three zones; Upper reach with a longitudinal slope of 6%; middle reach with a longitudinal slope which transitions from 100% (1H:1V) to 50% (2H:1V); and a bottom reach with 0% slope which will act as a stilling basin. A schematic of this design is shown in Figure 1.
- Flow conditions: The maximum spillway flow depth was set at 1 m and the spillway was designed for steady state conditions.
- Foundation conditions: The spillway will be founded in erodible permafrost overburden, which transitions to weathered bedrock at its outlet.

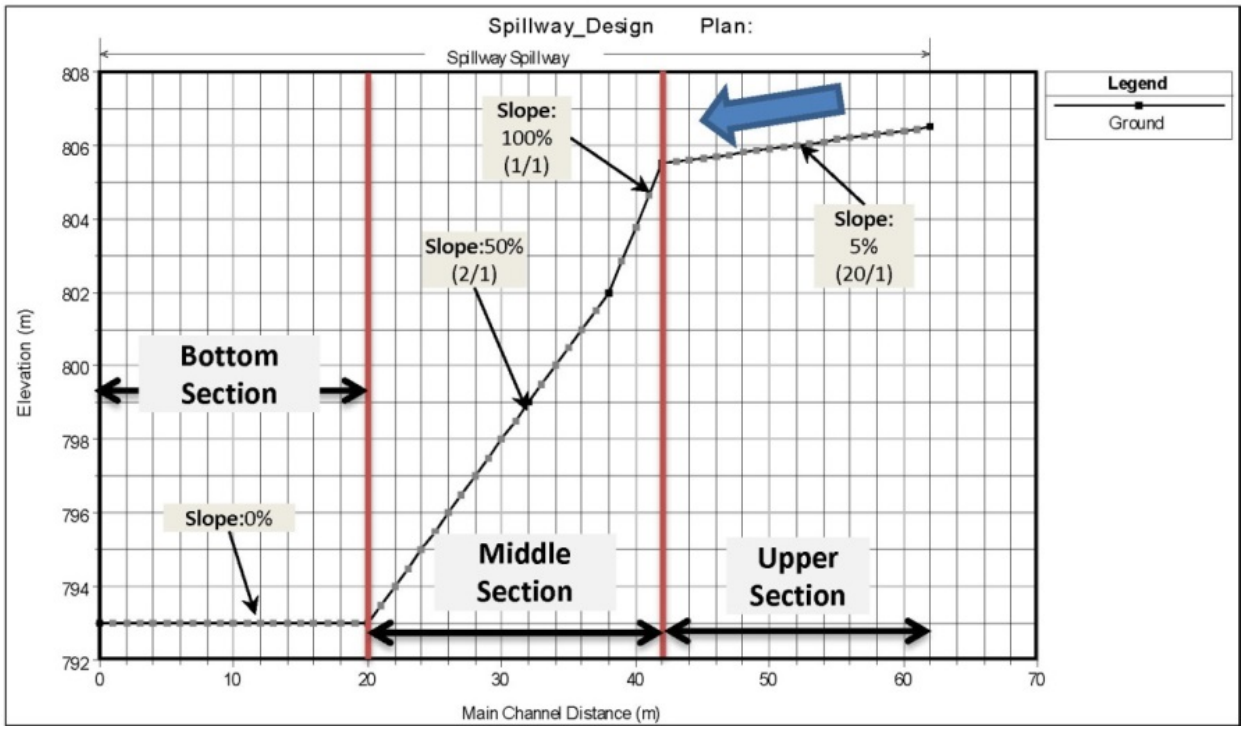

Figure 1: Spillway Design Profile (Produced using HEC-RAS Software)

2 Hydraulic Analysis

2.1 Upper Reach Design

A sensitivity analysis was conducted on the spillway's Upper Section assuming a 25 m width and a 6% longitudinal slope. The results presented in Table 1 demonstrates that in all cases the water flow depth would be less than 1 m, and the riprap (D_{50}) must be at least 0.3 m in diameter. The flow in this reach is supercritical (Froude number close to 1.7). Based on hydraulic analysis, SRK recommends a minimum D_{50} riprap size of 300 mm, and minimum riprap thickness of 2 x D_{50} for this reach.

2.2 Middle Reach Design

The U.S. Army Core of Engineers' HEC-RAS software was used to conduct a sensitivity analysis on the compound middle reach of the spillway. The spillway width was assumed to be 35 m wide and the slope transitions from 100% (1H:1V) at the top to 50% (2H:1V) lower down. The results of the analysis are presented in Table 2. The model estimates a water depth of approximately 0.3 m with water velocities from 5.2 to 6.6 m/s with corresponding riprap D_{50} between 0.6 and 0.9 m.

Due to the steepness of the longitudinal slopes, the flow regime would be supercritical, with an associated Froude number as high as 4.0. The recommended minimum D_{50} riprap size for this reach is 1,000 mm, placed at a thickness of 2 x D_{50}.

Table 1: Normal Depth Calculations Based on Longitudinal Slope of 6%

		Width : 25 [m]			
		D50:0.2[m]	D50:0.3[m]	D50:0.4[m]	D50:0.5[m]
Channel Input					
Max Flow	[m³/s]	54.6	54.6	54.6	54.6
Width	[m]	25	25	25	25
Transversal Slope	[h/v]	3	3	3	3
Longitudinal Slope	[m/m]	0.06	0.06	0.06	0.06
Normal Depth					
Depth	[m]	0.52	0.54	0.55	0.56
Area of Flow	[m²]	13.76	14.31	14.73	15.06
Wetted Perimeter	[m]	28.28	28.40	28.49	28.57
Hydraulic Radius	[m]	0.49	0.50	0.52	0.53
Average Velocity	[m/s]	3.97	3.82	3.71	3.63
Critical Velocity	[m/s]	2.19	2.23	2.26	2.28
Top Width (T)	[m]	28.11	28.23	28.31	28.38
Froude Number	[-]	1.81	1.71	1.64	1.59
Manning's Roughness Estimation					
Strickler (1923)	[-]	0.037	0.039	0.041	0.042
Anderson et al. (1970)	[-]	0.037	0.039	0.041	0.043
Jarrett (1984)	[-]	0.124	0.124	0.123	0.123
Limerinos (1970)	[-]	0.058	0.068	0.079	0.090
Abt et al. (1988)	[-]	0.040	0.043	0.045	0.047
Suggested Manning's Roughness	[-]	0.038	0.041	0.043	0.044
Riprap Estimation					
Isbach Methodology | Source: Riprap U.S Army Corps of Engineers (EM-1110-2-1601,1994)					
D50	m	**0.34**	**0.31**	**0.30**	**0.28**
Design of Rock Chutes (Robinson et al. 1998)					
D50	m	**0.25**	**0.25**	**0.25**	**0.25**
Stabilization of Angular -shaped riprap under overtopping flows (Khan and Ahmad, 2011)					
D_{50}	m	**0.15**	**0.19**	**0.23**	**0.26**
Minimum D50	m	**0.20**	**0.20**	**0.20**	**0.20**
Riprap Protection Suggested					
D_{15}	m	0.08	0.10	0.15	0.18
D_{30}	m	0.15	0.20	0.30	0.35
D_{50}	m	**0.20**	**0.30**	**0.40**	**0.50**
D_{84}	m	0.25	0.35	0.45	0.55
D_{100}	m	0.30	0.45	0.60	0.75
Minimum Thickness of Riprap Layer	m	**0.40**	**0.60**	**0.80**	**1.00**

Table 2: Normal Depth Calculations Based on Longitudinal Slope of 1%

		Width : 35 [m]
		D50:1.4[m]
Channel Input		
Max Flow	[m³/s]	54.6
Width	[m]	35
Transversal Slope	[h/v]	3
Longitudinal Slope	[m/m]	1.00
Normal Depth		
Depth	[m]	0.28
Area of Flow	[m²]	8.25
Wetted Perimeter	[m]	29.00
Hydraulic Radius	[m]	0.28
Average Velocity	[m/s]	6.61
Top Width (T)	[m]	3.96
Froude Number	[-]	0.28
Manning's Roughness Estimation		
Strickler (1923)	[-]	0.047
Anderson et al. (1970)	[-]	0.048
Jarrett (1984)	[-]	n/a
Limerinos (1970)	[-]	n/a
Abt et al (1988)	[-]	0.082
Suggested Manning's Roughness	[-]	0.059
Riprap Estimation		
Isbach Methodology | Source: Riprap U.S Army Corps of Engineers (EM-1110-2-1601,1994)		
D50	m	**0.94**
Design of Rock Chutes (Robinson et al. 1998)		
D50	m	**0.63**
Stabilization of Angular -shaped riprap under overtopping flows (Khan and Ahmad, 2011)		
D_{50}	m	**0.67**
Minimum D_{50}	m	**0.20**
Riprap Protection Suggested		
D_{15}	m	0.35
D_{30}	m	0.70
D_{50}	m	**1.00**
D_{84}	m	1.05
D_{100}	m	1.50
Minimum Thickness of Riprap Layer	m	**2.00**

2.3 Bottom Reach Design

The Bottom Reach design is characterized by a 35 m width and a 0% slope, and receives highly turbulent flows from the Middle Reach. Due to the initial high turbulence and energy, riprap sizing from middle section should be extended for the first 5 m of the stilling basin.

The stilling basin section is expected to be located partially or completely on bedrock. For any portion that is not directly on bedrock, riprap protection similar to the Upper Reach is recommended (i.e., minimum D_{50} of 300 mm with a minimum total thickness of 2 x D_{50}).

Figure 2 presents a 3D view of the spillway considering every reach from the HEC-RAS software.

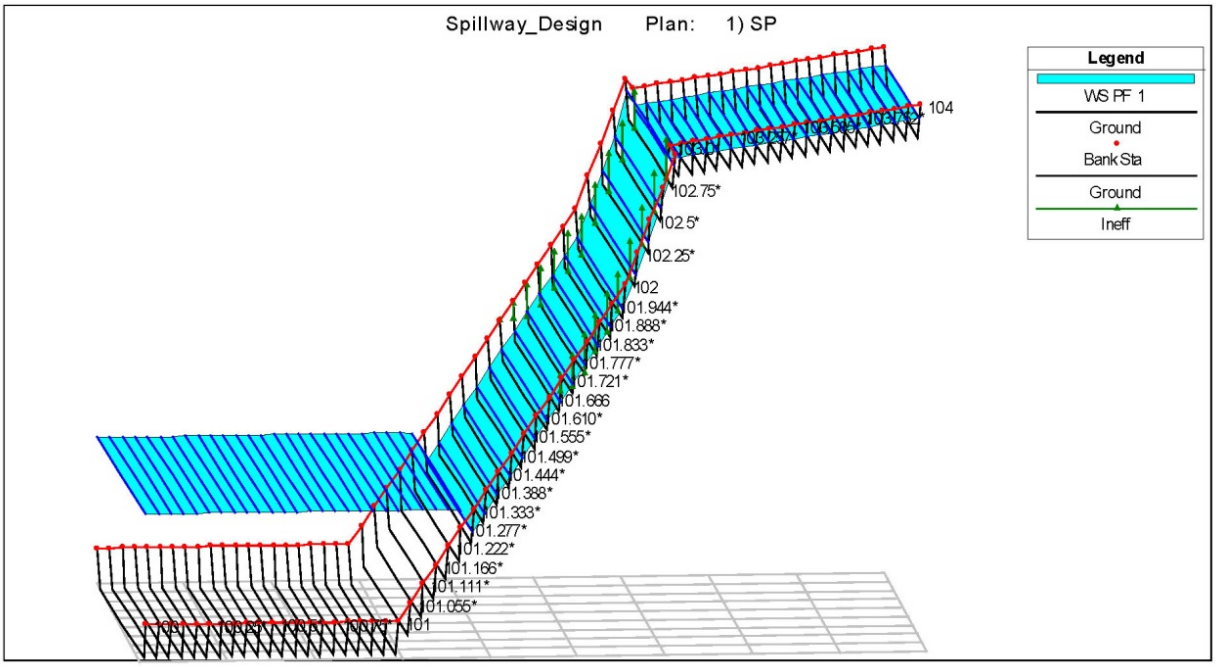

Figure 2: 3D Spillway View (Produced using HEC-RAS Software)

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Appendix G: Main Dam Freeboard Design

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Cc:	Maritz Rykaart, Dylan MacGregor, John Duncan, Murray McGregor	**Date:**	July 18, 2014
Subject:	Minto Main Dam Wave Run-up Freeboard and Riprap Design		

1 Introduction

This memo documents the wave run-up freeboard and riprap erosion protection design for the Main Dam. The wind setup, wave run-up and upstream riprap design are based on a fixed full supply level and a wind speed return period of 100 years. No consideration for climate change is included.

Topographical information for the project site consists of LiDAR, provided by Capstone. The preliminary Main Dam alignment and cross section was used to estimate fetch length.

Meteorological information was obtained from Environment Canada (EC 2014), including hourly wind data for six stations; Dawson, Dawson A, Carmacks, Mayo A, Aishihik A and Whitehorse. Figure 1 presents the locations of the meteorological stations and the mine site. This data was complimented with historical wind data from the mine site.

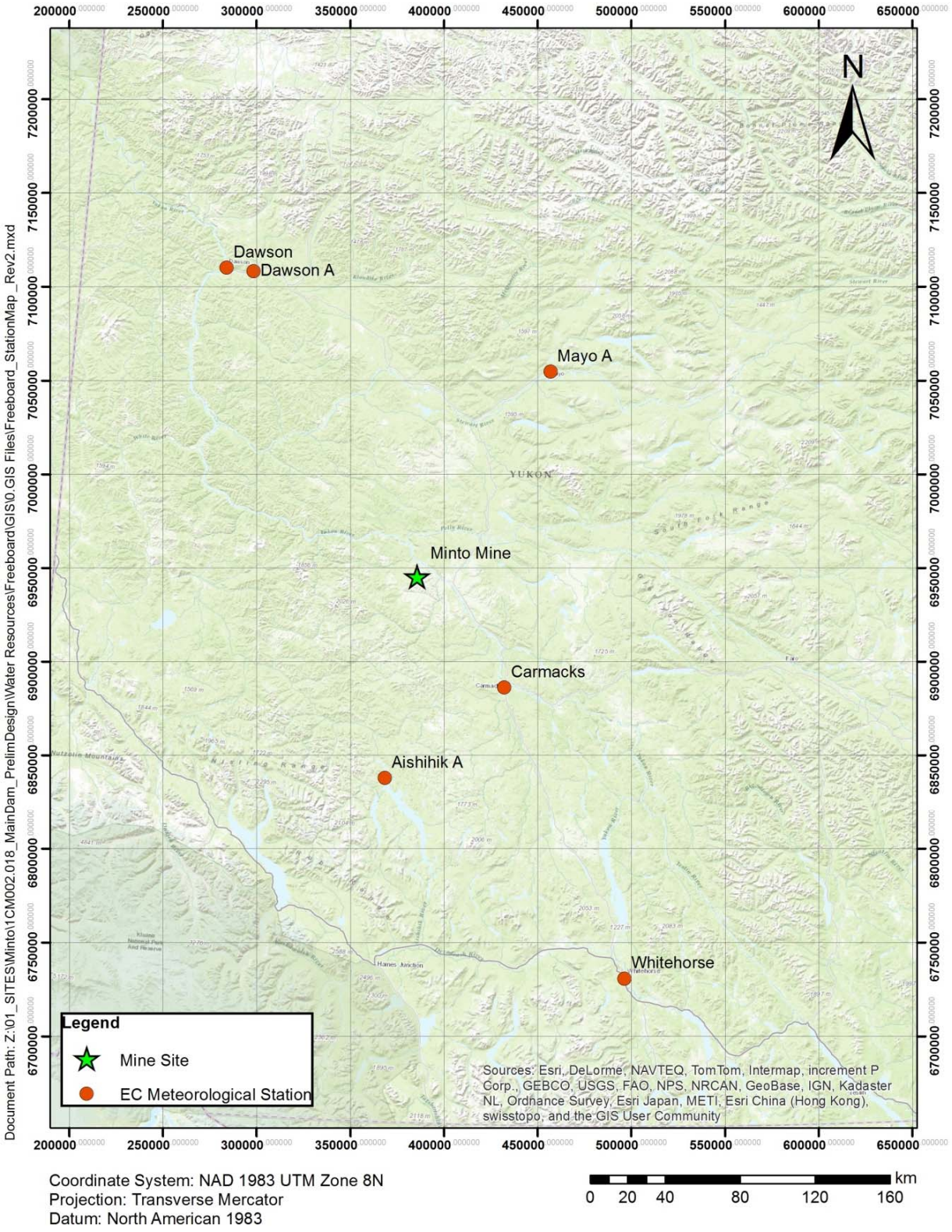


Figure 1: Meteorological stations in the vicinity of the project site

2 Wave Run-up Freeboard Design

Wave run-up freeboard for the Main Dam is estimated by taking the sum of the wind setup and the wave run-up. The following subsections describe the estimation of each parameter.

2.1 Wind Setup

The wind setup is defined as the vertical water height above the static water level which may result from wind stress over the water surface. The US Army Corps of Engineers (USACE 1989) estimates the wind setup relative to the supply water level (SWL) using the following expression:

$$S = \frac{U^2 F}{1400 d} \quad \text{(Eq. 1)}$$

Where:

S is the wind setup relative to the SWL (ft);
U is the wind speed (mph);
F is the effective fetch length (miles); and,
d is the average water depth over the fetch (ft).

2.1.1 Wind Speed

The wind speed was estimated from a regional analysis of meteorological stations surrounding the project site. For this analysis, only stations with historical records longer than 20 years were considered. Using the records from a total of five nearby stations, an hourly wind speed frequency analysis was prepared for return periods between 2 and 100 years. The resulting information is presented in Table 1

Table 1: Maximum hourly velocity at meteorological stations close to project site

Station Name	Station ID	Lat. [degrees]	Long. [degrees]	Elev. [m]	# of Yrs of Data	Max Wind Speed (km/hr)						
						Return Period (yrs)						
						100	50	20	10	5	3	2
Dawson	2100400	64.05	-139.43	320	60	54	52	49	46	43	40	38
Dawson A	2100402	64.04	-139.13	370								
Carmacks	2100300	62.10	-136.30	525	14							
Mayo A	2100701	63.62	-135.87	504	59	56	53	49	46	43	40	38
Aishihik A	2100100	61.65	-137.48	966	*13*							
Whitehorse	2101303	60.71	-135.07	706	59	**77**	74	70	67	63	59	56
Site Data	N/A	62.62	-137.23	795	6	54	50	44	40	35	31	28

Table 1 shows Dawson and Dawson A as one station, due to the proximity of both stations, and their complimentary historical records. Carmacks and Aishihik do not present wind records but are presented in Table 1 for regional completeness.

The design wind recurrence period for wave run-up calculations for the Main Dam was selected as the 100 year maximum hourly wind speed. The maximum 100-year wind speed from the five stations was found in the frequency analysis of the Whitehorse station, corresponding to a wind speed of 77 km/hr. The corresponding design value is 77 km/hr over land, based on data from the Whitehorse station.

2.1.2 Fetch Length

The US Bureau of Reclamation (USBoR 1981) suggests estimating the effective fetch length using Equation 3.2.

$$Fe = \frac{\sum x_i cos(\alpha_i):}{\sum cos(\alpha_i):} \quad \text{(Eq. 2)}$$

Where:

α_i is the angle between the central radial and radial$_i$; and
x_i is the length of the projection of radial i on the central radial$_i$.

This concept is presented in Figure 2, and the results are expressed in Table 2. Subsequently the maximum fetch length is equal to 519 m and the effective fetch length, is equal to 417 m.
The maximum fetch length value was used in this analysis instead of the effective fetch length, in order to obtain a more conservative wind setup estimate.

Table 2: Estimation of maximum and effective fetch

Angle [deg]	Cos (α_i)	Distance [m]	x_i Cos (α_i)
42	0.74	227.0	168.7
36	0.81	271.1	219.3
30	0.87	342.8	296.9
24	0.91	441.4	403.2
18	0.95	471.6	448.5
12	0.98	504.4	493.4
6	0.99	487.0	484.3
0	1.00	**519.0**	519.0
-6	0.99	480.5	477.9
-12	0.98	496.8	485.9
-18	0.95	495.5	471.2
-24	0.91	431.9	394.6
-30	0.87	367.5	318.3
-36	0.81	309.4	250.3
-42	0.74	269.6	200.4
$\sum cos(\alpha_i):$	13.5	$\sum x_i cos(\alpha_i):$	5631.9

Maximum Fetch	519 [m]
Effective Fetch	417 [m]

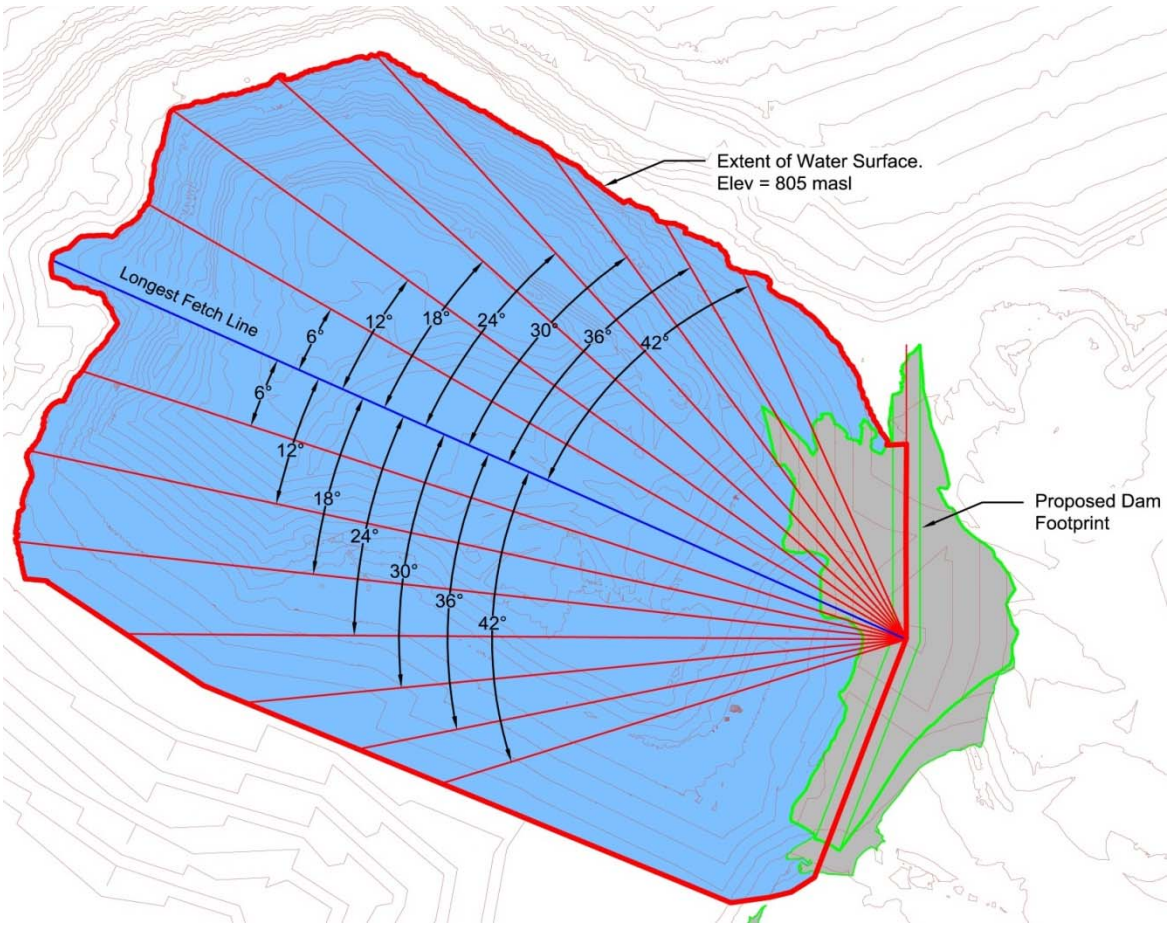


Figure 2: Fetch measurement considered for the effective fetch calculation

These fetch length estimations are based on a water level of 805 masl. A sensitivity analysis was performed to assess the impact of variations in the water level on the fetch length calculations. A change in the water level that is less than 10 m up or down will not significantly affect the validity of this evaluation.

USBoR (1981) and USACE (1997) suggest corrections for meteorological wind speed over water. These correction factors are based on the considered fetch length, and are presented in Table 3. Based on the maximum fetch length of 519 m, which is approximately 0.32 miles, the correction factor used in the analysis is 1.04. This correction factor was applied to the maximum wind speed calculated in Section 2.1.1

Table 3: Wind over water correction factors

Fetch km (miles)	Wind Ratio Over Water/Over Land
0.8 (0.5)	1.08
1.6 (1)	1.13
3.2 (2)	1.21
4.8 (3)	1.26
6.4 (4)	1.28
8.0 (5)	1.30

Source: USACE (1997)

2.1.3 Average Water Depth over Fetch

The average water depth over fetch was estimated for the maximum fetch length. Capstone provided LiDAR topographic data, in the form of a Digital Elevation Model (DEM) of the mine site, with a precision of 0.2 x 0.2 m in the easting and northing. Using the fetch length and the site DEM, the average water depth was found to be 43 m, as presented in Figure 3.

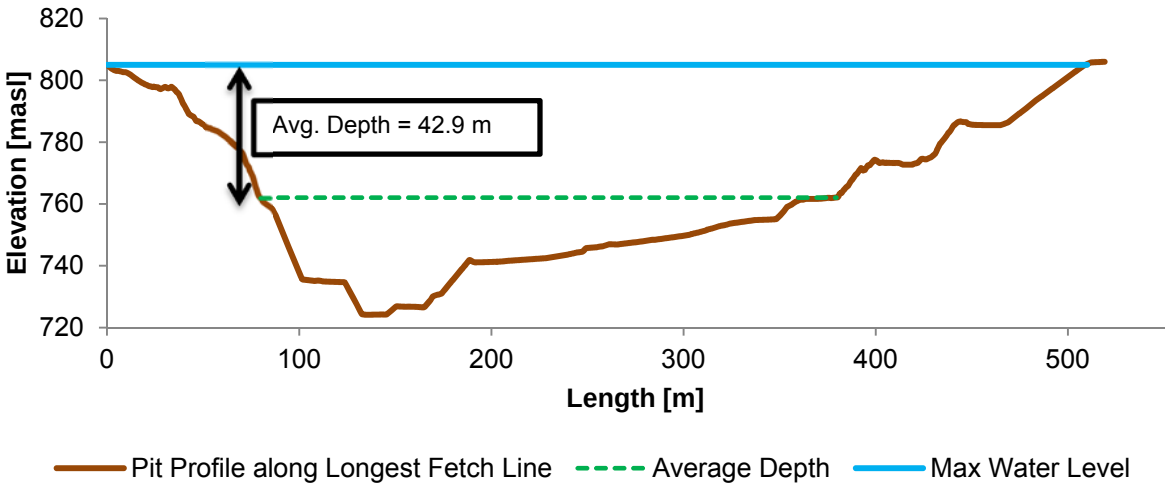


Figure 3: Average water depth along the maximum fetch

2.1.4 Resultant Wind Setup

The calculation of the average water depth assumes the tailings dam is completely full of water. A sensitivity analysis was performed by varying the water depth by 20 m above or below the average water depth. The resultant change in the wind setup height is from 0.001 m to 0.002 m. This height change was assumed to be diminishable for the evaluation of the wind setup, considering the magnitude of the wind run-up.

The resultant wind setup for 43 m of average water depth at the Main Dam is 0.001 m.

2.2 Wave Run-up

Wave run-up is defined as the maximum vertical extent of a wave uprush on a beach or structure. The estimation of the wave run-up requires the estimation of the wave height. Two methodologies were evaluated to estimate the wave height, and the maximum was applied in the wave run-up estimation.

2.2.1 Wave Height Estimation

The wave height was estimated using two sources; the USBoR, and the South African National Committee on Large Dams (SANCOLD).

The USBoR Design Standards No. 13 for Embankment Dams (USBoR 2012) presents Equation 3.3 for the significant wave height

$$.H_S = 0.0245\sqrt{F}W_v\sqrt{(1.1 + 0.0156W_v)} \quad \text{(Eq. 3)}$$

Where:

H_s is the significant wave height (ft);
F is the fetch length (mi); and
W_v is the wind velocity (ft/s).

SANCOLD (1990) further suggests the estimation of significant wave height using a relationship between effective fetch length and over-water wind speed, as shown in Figure 4.

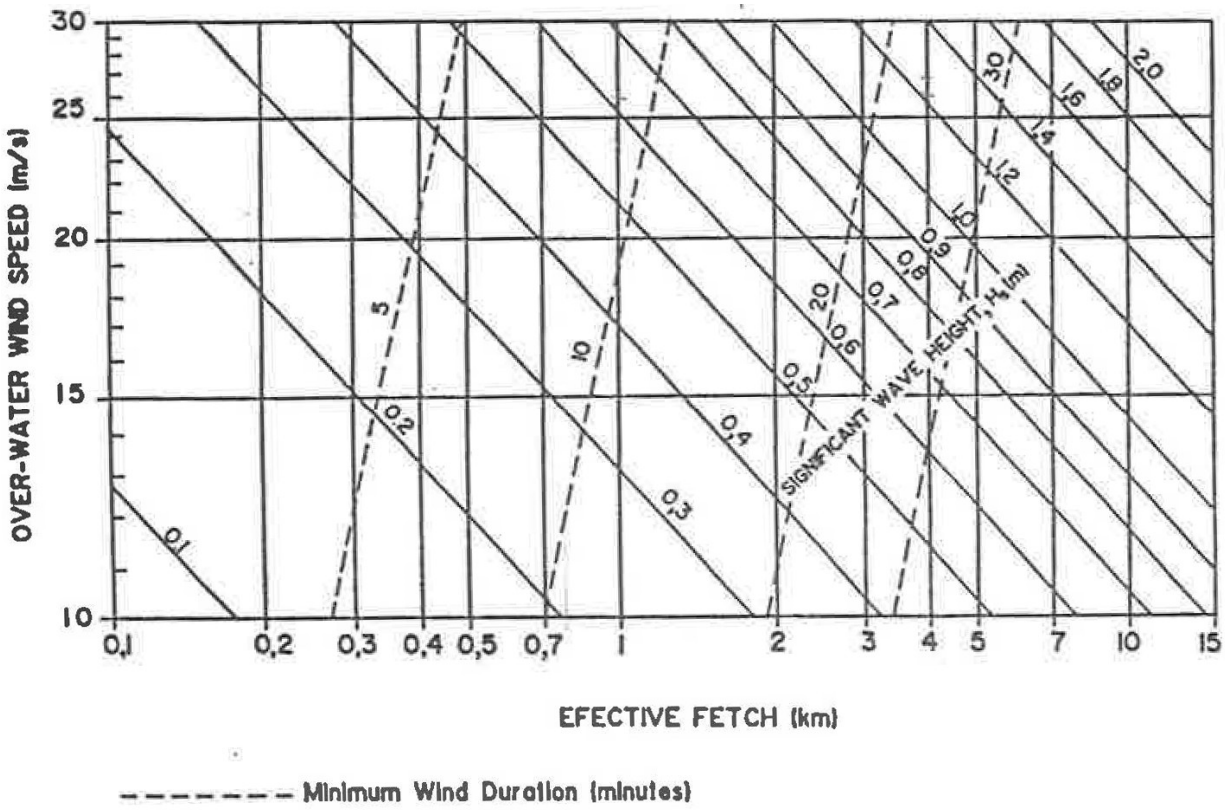


Figure 4: Significant wave height from SANCOLD (1990)

Using a Rayleigh distribution, wave height measurements can be estimated from the significant wave height calculated in Equation 3 or Figure 4. Table 4 from USBoR (2012) shows common wave height relationships. From this table, the average wave height from the highest 1% of waves in a given spectrum is approximately 1.67 H_s.

Table 4: Common wave height relationships

Percent of total number of waves in series averaged to compute specific wave height	Ratio of specific wave height, H, to average wave height, H_{ave}	Ratio of specific wave height, H, to significant wave height H_s	Percent of waves exceeding specific wave height
(H)	(H/H_{ave})	(H/H_s)	(H)
1.00	2.66	1.67	0.4
5.00	2.24	1.40	2.0
10.00	2.03	1.27	4.0
20.00	1.80	1.12	8.0
25.00	1.71	1.07	10.0
30.00	1.64	1.02	12.0
33.33	1.60	1.00	13.0
40.00	1.52	0.95	16.0
50.00	1.42	0.89	20.0
75.00	1.20	0.75	32.0
100.00	1.00	0.62	46.0

Source: USBoR (2012)

The design wave height was selected as the maximum of the significant wave heights obtained from the USBoR method and the SANCOLD method. The maximum wave height is then multiplied by 1.67 to obtain the highest 1% of waves, as presented in Equation 4.

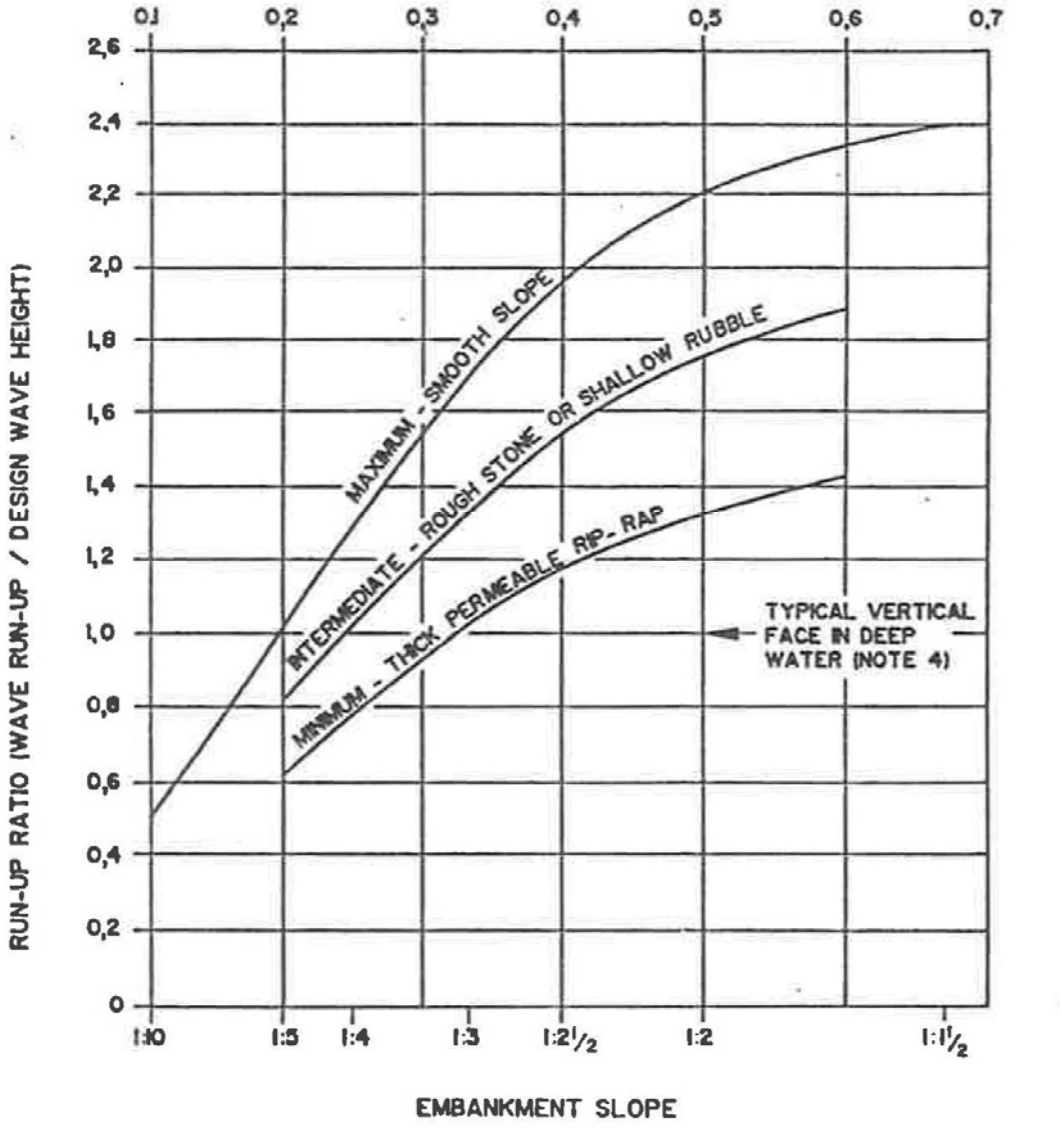
$$Design\ Wave\ height\ [m] = 1.67x \max(H_S) \qquad \text{(Eq. 4)}$$

The calculated maximum wave height was 0.63 m, and was calculated using the SANCOLD (1990) approach.

2.2.2 Wave Run-up Estimation

The wave run-up was calculated as the maximum from three methodologies, described in SANCOLD (1990), USBoR (1981) and USBoR (2012).

SANCOLD (1990) presents a relationship between the wave height, the embankment slope and the embankment protection. The wind run-up ratio is obtained from Figure 5.



NOTES:

1. MAXIMUM LINE FROM SAVILLE ET AL (1962) FOR TYPICAL WAVE STEEPNESS (SIGNIFICANT WAVE HEIGHT/LENGTH) = 0,05.
2. INTERMEDIATE LINE IS 0,8 x MAXIMUM.
3. MINIMUM LINE IS 0,6 x MAXIMUM.
4. FOR FACES OFF-VERTICAL THE RUN-UP RATIO RISES ABOVE UNITY AND CAN APPROACH 2 IN SOME CIRCUMSTANCES WHERE THE DEEP WATER CONDITION IS NOT FULFILLED.

Figure 5: Wave run-up estimation from SANCOLD (1990)

USBoR (1981) uses a relationship originally prepared by the USACE (1980), which correlates the rate of wave run-up divided by wave height, as well as the embankment slope and wave period.

USBoR (2012) estimates wave run-up based on several parameters, including wave height, wave length, wave period, incident fetch angle, embankment angle and roughness of the embankment protection.

The maximum wave run-up from the three methodologies was calculated using SANCOLD (1990), which corresponds to a wave run-up of 0.98 m over the water supply level.

2.3 Wave run-up Freeboard Estimate

The wave run-up (i.e. hydraulic) freeboard is the sum of the wind setup (0.001 m) and wave run-up (0.98 m). In the case of the Main Dam, this freeboard was determined to be approximately 1 m.

3 Riprap Design

The riprap for the upstream face of the dam is calculated with the Hudson's expression cited in USACE (1986), presented in Equation 5.

$$W_r = \frac{\rho_S g H^3}{K_d\left(\frac{\rho_S}{\rho_W}-1\right)^3 cot(\alpha)} \qquad \text{(Eq.5)}$$

Where:

W_r is the weight of the armor unit in pounds;
ρ_s is the unit weight of the armor unit in pounds per cubic foot;
ρ_w is the unit weight of the fluid, where the protection is constructed;
H is the design wave height in feet;
α is the angle of the structure measured from the horizontal in degrees; and
K_d is a stability coefficient which is dimensionless.

Table 5: Evaluation from USACE (1986)

Table 4-2. Suggested K_D Values for Use in Determining Armor Unit Weight. (No-Damage Criteria and Minor Overtopping)

Armor Units	n[a]	Placement	Structure Trunk K_D[b] Breaking Wave	Structure Trunk K_D[b] Nonbreaking Wave	Structure Head K_D Breaking Wave	Structure Head K_D Nonbreaking Wave	Slope Cot α
Quarrystone							
Smooth rounded	2	Random	1.2	2.4	1.2	1.9	1.5 to 3.0
Smooth rounded	>3	Random	1.6	3.2	1.4	2.3	(c)
Rough angular	1	Random [d]	(d)	2.9	(d)	2.3	(c)
Rough angular	2	Random	2.0	4.0	1.9 1.6 1.3	3.2 2.8 2.3	1.5 2.0 3.0
Rough angular	>3	Random	2.2	4.5	2.1	4.2	(c)
Rough angular	2	Special[e]	5.8	7.0	5.3	6.4	(c)
Parallelepiped[f]	2	Special[e]	7.0-20.0	8.5-24.0	--	--	(c)
Tetrapod and Quadripod	2	Random	7.0	8.0	5.0 4.5 3.5	6.0 5.5 4.0	1.5 2.0 3.0
Tribar	2	Random	9.0	10.0	8.3 7.8 6.0	9.0 8.5 6.5	1.5 2.0 3.0
Dolos	2	Random	15.0[g]	31.0[g]	8.0 7.0	16.0 14.0	2.0[h] 3.0
Modified Cube	2	Random	6.5	7.5	--	5.0	(c)
Hexapod	2	Random	8.0	9.5	5.0	7.0	(c)
Toskanes	2	Random	11.0	22.0	--	--	(c)
Tribar	1	Uniform	12.0	15.0	7.5	9.5	(c)
Quarrystone (K_{RR}) Graded angular	-	Random	2.2	2.5	--	--	--

[a] n is the number of units comprising the thickness of the armor layer.

[b] Applicable to slopes ranging from 1 on 1.5 to 1 on 5.

[c] Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes ranging from 1 on 1.5 to 1 on 3. Some armor units tested on a structure head indicate a K_D slope dependence.

[d] The use of a single layer of quarrystone armor units subject to breaking waves is not recommended, and only under special conditions for nonbreaking waves. When it is used, the stone should be carefully placed.

[e] Special placement with long axis of stone placed perpendicular to structure face.

[f] Long slablike stone with the long dimension about three times its shortest dimension.

[g] Refers to no-damage criteria (<5 percent displacement, rocking, etc.); if no rocking (<2 percent) is desired, reduce K_D 50 percent.

[h] Stability of dolosse on slopes steeper than 1 on 2 should be substantiated by site-specific model tests.

NOTE: Breaking wave stability coefficients for stone and dolos were developed using a 1V:10H foreslope.

The following assumptions were made for the estimation of riprap along the upstream dam face:

- Riprap protection density (ρ_s) of 2,650 kg/m³ (165 lb/ft³);
- Water density of 1,000 kg/m³ (62 lb/ft³);
- Gravity constant of 32.17 ft/s²;
- Upstream dam slope of 2.5:1 (H:V); and
- K_d is 1.45 for rough angular rock, structure head and breaking wave, double riprap layer. This expression follows Table 5 from USACE (1986).

From Equation 5, the suggested weight is 42 kg (92 lbs), based on the assumptions stated. This corresponds to a riprap D_{50} diameter of 0.3 m. The riprap needs to be within a weight range of ± 25% of the suggested D_{50} weight.

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4 References

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Appendix H: Minto Main Dam –Deformation Analysis

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Memo

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From:	John Kurylo	**Project No:**	1CM002.018
Cc:	Peter Mikes, Maritz Rykaart, Cam Scott	**Date:**	July 18, 2014
Subject:	Minto Main Dam Deformation Analysis		

1 Introduction

Settlement of the Main Dam foundation is expected as a result of consolidation, as well as from thaw settlement (SRK 2014a). The largest magnitudes of deformation are expected beneath the southern portion of the dam where ice-rich fine grained overburden is present.

This memo presents the analysis completed to evaluate the potential deformation, caused by consolidation and thaw settlement, and to assess their effects on the Main Dam.

2 Deformation Analysis

2.1 Methodology

The deformation analysis was completed using the two-dimensional (2D) finite element (FE) software program PLAXIS 2D Anniversary Edition (AE) (2014a). Additional details on the PLAXIS software are available in the Reference Manual (Plaxis 2014b).

Deformations from thaw settlement (due to melting of excess ice in the foundation soils) were computed based on the results of the thermal analysis presented in SRK (2014a). Zero degree isotherms from the thermal models were imported into the FE models and used to define the frozen and unfrozen zones. Unfrozen soils were assumed to be fully saturated in order to allow for computation of the foundation consolidation.

These analyses included water pressure induced by compression and consolidation of all thawed overburden materials, solved simultaneously with the soil and rock mass stress field. These results were also coupled with the computation of a strength reduction factor (similar to a FOS) via the strength reduction technique; completed to get a better understanding of expected failure mechanisms and areas of larger displacements.

2.2 Model Set-Up

2.2.1 Model Geometry

The location of the proposed Main Dam and the 2D cross-section geometry used in the deformation analysis are shown in Figure 1.

The section location was selected along the dam section that intersects borehole 13-MPD-06. This section is expected to represent the least favourable foundation conditions as it has significant fine grained materials close to the surface and contains the most excess ice found during the 2013 geotechnical drilling investigation (SRK 2014b). This section was also analyzed in the Main Dam thermal assessment (SRK 2014a) to estimate the amount of thaw settlement that could be expected with time. The overburden at this location is 40 m thick and generally consists of frozen fine grained silts and clays with a 4 m sand and gravel layer located approximately 7 m below the original surface.

The ground surfaces are based on topographic site surveys provided on January 2014. The bedrock contact was generated from the available borehole database and the lithology is based on SRK interpretations of the borehole logs.

The proposed dam has a crest width of 14.5 m with a 2.5H:1V upstream slope and a 3H:1V downstream slope. The dam is a rockfill structure with an internal geomembrane liner system and a fine-grained core that is keyed into fine grained overburden and/or bedrock. The key trench has a variable depth with 2H:1V side-slopes.

All base case deformation models were evaluated only for an 807 m crest height dam with the maximum water surface reaching up to elevation 804 m. Sensitivity analysis was completed to evaluate the effects of raising the dam to an elevation of 812 m. For simplicity, the internal transition zones, filter materials, and the geomembrane liner system were not modelled as separate domains within the dam.

2.2.2 Constitutive Soil Models

The constitutive soil models (Plaxis 2014c) used in this FE analysis are summarized in Table 1.

Table 1: Constitutive Soil Models

Model	Description	Units
Linear Elastic (LE)	A perfectly elastic constitutive model that does not account for plastic deformations; therefore; soil failure is not accounted for.	• Bedrock • Frozen soils
Mohr-Coulomb (Mohr-C)	A bi-linear soil model that accounts for soil failure according to the Mohr-Coulomb failure criterion. This model does not account for plastic straining before soil failure.	• Weathered bedrock • Waste Rock
Hardening Soil Model with Small Strain Stiffness (HS-small)	This advanced nonlinear soil model allows for inputs that help to design stress dependency of the soil stiffness and accounts for the increased stiffness of soils at small strains. At low strain levels most soils exhibit a higher stiffness than at engineering strain levels, and this stiffness varies non-linearly with strain (Plaxis 2014b). This model gives more reliable displacements, specifically for working load conditions. The isotropic hardening model properly accounts for: • Increases in stiffness with confining pressure • Small-strain elasticity • Pre-failure hardening plasticity with hyperbolic stress-strain response Compression plasticity allowing for the simulation of consolidation and one-dimensional compression.	• Sand and Gravels • Clays and Silts • Tailings

2.3 Modelled Scenarios

The FE models were developed for static analysis for expected and conservative case scenarios. The expected scenario consists of a fully developed tailings beach on the front face of the dam as planned. The conservative/worst case scenario assumes that the Main Pit floods with water during the first freshet. Models were verified with manual calculations and additional model runs were completed assuming all foundation materials were fully thawed. Examples of 1D consolidation hand calculations are presented in SRK (2014c). Figure 2 overviews the general model set-ups used in Plaxis and Table 2 summarizes the Main Dam scenarios analyzed.

Table 2: Main Dam Scenarios Analysed as Part of the Deformation Analyses

Scenario		Description
1	Pit Lake	807 m crest dam with the Main Pit filled with water up to the spillway elevation of 804 m.
2	Tailings Beach	807 m crest dam with a tailings beach developed up to 805 m.
3	Pit Lake – Dam with No Core	Sensitivity Analysis/Calibration – model run without core to initially calibrate section and to better investigate influences of having silt core in the preliminary design.
4	Pit Lake – No Core & Fully Thawed	Sensitivity Analysis / Calibration – 807 m crest dam with no core and the foundation assumed to be fully thawed.
5	Pit Lake – Fully Thawed	Sensitivity Analysis – 807 m crest dam with Main Pit filled with water and the foundation assumed to be fully thawed.
6	Pit Lake – Raised Dam with No Core	Sensitivity Analysis – 812 m crest dam with Main Pit filled with water. Model to confirm expectations about effects of raising dam from 807 m.

2.3.1 Mesh Elements

Figure 3 shows the mesh and basic triangular elements used for the deformation models. The size of the mesh elements was adjusted to capture the relevant information of the model behavior under static loads. For the final model runs, a mesh with an average element size of around 0.5 m was used (relative element size of approximately 0.1 m).

Overall up to around 300,000 nodes and 40,000 fifteen-Node triangular elements were typically used for the final model runs.

2.3.2 Boundary Conditions

Figure 4 shows the boundary conditions used for the deformation models. Two sets of boundary conditions were used in the FE deformation model. Specifically, the sides of the model were fixed horizontally, or with normal in x-direction and free in y direction (i.e. model allowed to move vertically), and the bottom of the model was specified to be fully fixed (horizontal and vertical both fixed).

2.4 Material Properties

The material properties are presented in Tables 3 to 11 and were based on a review of properties presented in SRK (2014a, b, c, d, e and f). These values formed the base cases for the conventional geotechnical analyses. Where required, constitutive soil parameters for the FE analyses were calculated and/or adopted based on engineering judgement (such as for the HS-small soil model units). Additional details on some of the initial assumptions made for some of the advance HS-small material properties is presented in Attachment 1.

Table 3: Material Parameters for Waste Rock (Dam Fill) and Weathered Bedrock

Property/Parameter	Symbol	Unit	Value	Notes/Reference
Constitutive model	-	-	Mohr-C	Mohr-Coulomb [1]
Unit weight - unsaturated	γ_{unsat}	kN/m^3	20.0	General
Unit weight - saturated	γ_{sat}	kN/m^3	20.0	General – based on SRK 2014d and back calculated based on void ratio
Initial void ratio	*e*	-	0.4	General
Cohesion	c’	kPa	0.0	Strength – from SRK 2014d
Friction angle	Φ’	º	37.0	Strength– from SRK 2014d
Dilatancy angle	ψ	º	0.0	Strength – based on engineering judgement
Young’s modulus – effective	E’	MPa	50.0	Stiffness
Poisson’s ratio - effective	*v’*	-	0.25	Stiffness – based on engineering judgement
Initial stress ratio	K_0	-	0.4 - 0.5	State parameter – see Section 2.5.1 for more details
Horizontal & vertical permeability	k_x & k_y	m/s	$1.0x10^{-3}$	Same as hydraulic conductivity as used in SRK 2013e

Notes:
(1) This material model was seen as appropriate for these materials as they their stiffness is not expected to be significantly dependent on pressure.

Table 4: Material Parameters for Compacted Core (Gravelly Silt)

Property / Parameter	Symbol	Unit	Value	Notes / Reference
Constitutive model	-	-	HS-Small	Hardening soil model with small-strain stiffness
Unit weight - unsaturated	γ_{unsat}	kN/m^3	18.5	General
Unit weight - saturated	γ_{sat}	kN/m^3	20.0	General - back calculated based on void ratio
Moisture content	Ω	%	15.5	General – based on proctor test (sample 08-ROD-0B01)
Void ratio	E	-	0.4	General Properties – assumed to be a stiff / well compacted material
Initial shear modulus	G_0^{ref}	MPa	79.2	Stiffness – based calcuations from other input parameters listed in table and adjusted based on engineering judgement
Unload stiffness	E_{ur}^{ref}	MPa	58.0	Stiffness – see above comment
Secant stiffness	E_{50}^{ref}	MPa	8.1	Stiffness – see above comment
Oedometric stiffnesss	E_{oed}^{ref}	MPa	6.4	Stiffness see above comment
Cohesion	c’	kPa	0.0	Strength – assumed to be normally consolidated
Friction angle	Φ’	º	27.5	Strength – based on diret shear (sample 08-ROD-0B01) and SRK 2014f
Dilatancy angle	Ψ	º	3.0	Strength – based on engineering judgement
Poisson ratio for unloading/reloading	ν'_{ur}	-	0.20	Advance stiffness parameter
Overconsolidation ratio	OCR	-	1	Consolidation parameter
Pre-overburden pressure	POP	kPa	50	Similar to preconsolidation pressure
Initial stress ratio	K_0	-	0.5 – 0.6	State parameter – see Section 2.5.1 for more details
Horizontal & vertical permeability	k_x & k_y	m/s	2.0×10^{-7}	Same as hydraulic conductivity as used in SRK 2013e

Table 5: Material Parameters for Tailings

Property / Parameter	Symbol	Unit	Value	Notes / Reference
Constitutive model	-	-	HS-Small	Hardening soil model with small-strain stiffness
Unit weight - unsaturated	γ_{unsat}	kN/m^3	13	General
Unit weight - saturated	γ_{sat}	kN/m^3	15	General – back calcuated based on specific gravity results and void ratio
Void ratio	E	-	0.8	General – based on tailings lab testing results completed by Klohn Crippen Berger (KCB) on samples from underflow tailings in December 2012
Compression index	C_c	-	0.15	Stiffness – see comment above. Based on KCB 2012 consolidation tests
Reloading or wwelling index	C_s	-	0.015	Stiffness
Initial shear modulus	G_0^{ref}	MPa	117.7	Stiffness – adjusted based on engineering judgement, ie. calcuations use slightly higher void ratio
Cohesion	c'	kPa	0	Strength – from SRK 2014d
Friction angle	Φ'	º	8.0	Strength – from SRK 2014d
Dilatancy angle	Ψ	º	3.0	Strength – basd on engineering judgement
Poisson ratio for unloading/reloading	ν'_{ur}	-	0.30	Advance stiffness parameter
Overconsolidation ratio	OCR	-	1	Consolidation parameter
Pre-overburden pressure	POP	kPa	50	Similar to preconsolidation pressure
Initial stress ratio	K_0	-	0.7 – 0.8	State parameter – see Section 2.5.1 for more details
Horizontal & vertical permeability	k_x & k_y	m/s	1.0×10^{-6}	Hydraulic properties based on KCB 2012 tests.

Table 6: Material Parameters for Silt

Property / Parameter	Symbol	Unit	Value	Notes / Reference
Constitutive model	-	-	HS-Small	Hardening soil model with small-strain stiffness
Unit weight - unsaturated	γ_{unsat}	kN/m^3	16.5	General
Unit weight - saturated	γ_{sat}	kN/m^3	18.0	General – from SRK 2014d
Moisture content	Ω	%	15.0 - 30.0	General properties - volumetric water content used in thermal anaysis
Void ratio	E	-	0.7	General – typical value for silt
Compression index	C_c	-	0.15	Stiffness – roughly based on availble concolidation tests (SRK 2013a) and adjusted based on void ratio and engineering judgement
Reloading or wwelling index	C_s	-	0.015	Stiffness – 10% of compression index
Initial shear modulus	G_0^{ref}	MPa	136.4	Stiffness – based calcuations from other input parameters listed in table and adjusted based on engineering judgement
Cohesion	c’	kPa	0	Strength – from SRK 2014d
Friction angle	Φ’	º	26.0	Strength – from SRK 2014d
Dilatancy angle	Ψ	º	3.0	Strength – based on egineering judgement
Poisson ratio for unloading/reloading	ν'_{ur}	-	0.25	Advance stiffness parameter
Overconsolidation ratio	OCR	-	1	Consolidation parameter
Pre-overburden pressure	POP	kPa	50	Similar to preconsolidation pressure
Initial stress ratio	K_0	-	0.5 – 0.6	State parameter – see Section 2.5.1 for more details
Horizontal & vertical permeability	k_x & k_y	m/s	7.0×10^{-6}	Based on gradation

Table 7: Material Parameter for Sand and Gravels

Property / Parameter	Symbol	Unit	Value	Notes / Reference
Constitutive model	-	-	HS-Small	Hardening Soil model with small-strain stiffness
Unit weight - Unsaturated	γ_{unsat}	kN/m^3	18.0	General
Unit weight - Saturated	γ_{sat}	kN/m^3	19.0	General – from SRK 2014d
Moisture content	Ω	%	15.0	General - volumetric water content used in SRK 2014a
Void ratio	E	-	0.6	General – back calcuated off lab indicator data
Cohesion	c’	kPa	0.0	Strength – from SRK 2014d
Friction angle	Φ,	º	38.0	Strength – from SRK 2014d
Dilatancy angle	Ψ	º	2.0	Strength – based on engineering judgement
Initial shear modulus	G_0^{ref}	MPa	158.0	Stiffness – based calcuations from other input parameters listed in table and adjusted based on engineering judgement
Unload stiffness	E_{ur}^{ref}	MPa	33.1	Stiffness – see above comment
Secant stiffness	E_{50}^{ref}	MPa	4.6	Stiffness – see above comment
Oedometric stiffnesss	E_{oed}^{ref}	MPa	3.7	Stiffness see above comment
Poisson ratio for unloading/reloading	ν'_{ur}	-	0.20	Advance stiffness parameter
Initial stress ratio	K_0	- -	0.4 - 0.5	State parameter – see Section 2.5.1 for more details
Overconsolidation rtio	OCR	-	1	Consolidation parameter
Pre-overburden pressure	POP	kPa	0.1	Similar to preconsolidation pressure
Horizontal & vertical permeability	k_x & k_y	m/s	$1.0x10^{-5}$	Same as hydraulic conductivity as used in SRK 2013e for fine fiter

Table 8: Material Parameter for Clay and Silt

Property / Parameter	Symbol	Unit	Value	Notes / Reference
Constitutive model	-	-	HS-Small	Hardening soil model with small-strain stiffness
Unit weight - unsaturated	γ_{unsat}	kN/m^3	16.0	General
Unit weight - saturated	γ_{sat}	kN/m^3	18.0	General – from SRK 2014d
Moisture content	ω	%	33.0	General - volumetric water content used in SRK 2014c
Void ratio	e	-	0.8	General – based on SRK 2014b (geotech invest) lab testing
Plasticity index	PI	%	22	General – based on SRK 2014b (geotech invest) lab testing
Compression index	C_c	-	0.2	Stiffness – roughly based on availble concolidation tests (SRK 2013a) and adjusted based on void ratio and engineering judgement
Reloading or swelling index	C_s	-	0.02	Stiffness
Initial shear modulus	G_0^{ref}	MPa	117.7	Stiffness – adjusted based on engineering judgement, ie. calcuations use slightly higher void ratio.
Cohesion	c'	kPa	0	Strength – from SRK 2014d
Friction angle	Φ'	º	19.0	Strength – from SRK 2014d
Dilatancy angle	ψ	º	5.0	Strength – basd on engineering judgement
Poisson ratio for unloading/reloading	ν'_{ur}	-	0.30	Advance stiffness parameter
Overconsolidation ratio	OCR	-	1	Consolidation parameter
Pre-overburden pressure	POP	kPa	50	Similar to preconsolidation pressure
Initial stress ratio	K_0	-	0.5 – 0.6	State parameter – see Section 2.5.1 for more details
Horizontal & vertical permeability	k_x & k_y	m/s	2.0×10^{-7}	Same as hydraulic conductivity as used in SRK 2013e

Table 9: Material Parameters for Frozen Soil

Property / Parameter	Symbol	Unit	Value	Notes / Reference
Constitutive model	**-**	**-**	Lin. elastic	Linerar elastic[1]
Unit weight - unsaturated	γ_{unsat}	kN/m^3	18.0	General
Unit weight - saturated	γ_{sat}	kN/m^3	18.0	General – based on engineering judgement
Void ratio	E	-	0.4	General
Young's modulus - effective	E	MPa	1000	Stiffness – based on enginering judgement and the Young's Modulus of Ice (Nimmo 2004)
Initial stress ratio	K_0	-	0.3	State parameter – see Section 2.5.1 for more details
Poisson's ratio - effective	*v'*	-	0.20	Stiffness
Horizontal & vertical permeability	k_x & k_y	m/s	1.0×10^{-8}	Assumed [2]

Note:

1. For modelling massive structural elements and bedrock layers, linear elastic is typically appropriate and have been used in these models.
2. *This parameter does not have a large effect on the deformation analysis results.*

Table 10: Material Parameters for Bedrock

Property / Parameter	Symbol	Unit	Value	Reference
Constitutive model	**-**	**-**	Lin. elastic	Linerar elastic[1]
Unit weight - unsaturated	γ_{unsat}	kN/m^3	24.0	General
Unit weight - saturated	γ_{sat}	kN/m^3	24.0	General – from SRK 2014d
Void ratio	E	-	0.3	General – based on engineering judgement
Young's modulus - effective	E	MPa	15000	Stiffness
Poisson's ratio - effective	*v'*	-	0.15	Stiffness
Initial stress ratio	K_0	-	0.3	State parameter – see Section 2.5.1 for more details
Horizontal & vertical permeability	k_x & k_y	m/s	8.0×10^{-8}	Same as hydraulic conductivity as used in SRK 2013d [2]

Note:

1. For modelling massive structural elements and bedrock layers, linear elastic is typically appropriate and have been used in these models.
2. *This parameter does not have a large effect on the current deformation analysis.*

Table 11: Default Strain, Stiffness and Strength Parameters for HS-Small Constitutive Models

Property / Parameter	For hardening soil models with small-strain stiffness		
Reference shear strain	$\gamma_{0.7}$	-	1×10^{-4}
Stress exponent	m	-	Typically 1 (0.5 used for sands)
Reference stress	P^{ref}	kPa	100.0
Failure ratio	R_f	-	0.90

2.5 Model Stages

2.5.1 Initial Conditions

The initial conditions, or the initial stage, represents the site prior to the Main Dam construction.

An initial stress ratio (K_0) is used to compute the initial stress state of the soil mass. K_0 can be thought of as being somewhat similar to the use of an earth pressure coefficient for the soil elements. An increase in K_0 has no influence on the friction angle (material parameter of the model) and increases the preconsolidation pressure. Therefore, an increase of K_0 will have negligible impact on the computed FOS and only have a small impact on settlements. In the absence of measured values, an initial stress ratio K_0 was assumed based on the input values outlined in Section 2.4, to define the initial stresses due to the soil self-weight and the hydrostatic water pressures prior to the dam construction. The adopted K_0 values are outlined in Tables 2 to 10 in Section 2.4. Figure 5 presents the initial conditions used in the deformation models.

2.5.2 Construction Stages

The subsequent construction stages are illustrated on Figure 6 and can be summarized as follows:

- The second stage is following the dam construction and plastic deformation and/or one day consolidation is calculated.
- The third stage added the tailings and/or water and calculates consolidation one year after the dam construction.
  - Consolidation in this stage is a time-dependent analysis of deformation and excess pore pressure which require the input of soil permeability.
- The fourth stage is a shear strength reduction analysis which allows for the calculation of safety factors (effectively FOS values).
  - Calculations of the global safety factors are completed by reducing the material shear strength values and then recomputing the model results to see if a solution is generated. This is done as an iterative process, further reducing the shear strength values at each subsequent step. This is completed until the reduced material shear strength values generate computed deformations that are unacceptable or until the finite element

solutions do not converge (for example if the equations are now dividing by a very small number that is effectively zero then they will not converge).

3 Results

3.1 Pit Lake (Worst Case Scenario)

3.1.1 Displacements

Displacements, due primarily to consolidation (vertical displacement of the top surface of the dam), were found to be less than 0.80 m for the 20-year operational design life (excludes thaw settlements). The largest displacement was observed near the upstream crest. Figure 7 shows scaled deformed meshes and contoured plots of total displacements for the Pit Lake runs.

Cases run with an entirely thawed foundation resulted in, as expected, greater ultimate consolidation settlements. This scenario is extremely unlikely to occur as even if the dam was a water retaining structure, it would be decommissioned prior to the foundation completely thawing. Typically an increase in total settlement, after thaw settlement, of about 1.3 m (i.e. total settlement of 2.1 m) was noted in a fully thawed foundation. Figure 10 shows results from the fully thawed Pit Lake runs.

3.1.2 Differential Settlement

Horizontal sections cut through the clay core show that some differential settlement is expected. Modelled differential settlement estimates of the core were noted to be around 0.35 m, over the six meter core width. Figure 8 shows sections through the FE deformation model that intersect the core. These differential settlements lead to some shear strains and distortions of the core. As a result, it is observed that some plastic points of the modelled core yield (exceed allowable stresses); however, a full failure plane through the core is not apparent in the deformation models.

Although the displacements were noted to be higher in the fully thawed foundation case, the differential settlement of the core were noted to be slightly lower (approximately 0.2 m across the six meter core) when compared to the partially frozen foundation case. Figure 11 shows sections through the Pit Lake runs completed with a fully thawed foundation.

3.1.3 Shear Strain

As shown in Figure 9 the shear strain changes between one day of consolidation and one year of consolidation. The strains in the deformation model start by being largest, in compression, in the foundation below the upstream toe (in the one day consolidation model) and then become more uniform in the foundation and throughout the core as the consolidation time increases and the Pit Lake water is added. As the water table is conservatively added in one step in the model and the hydraulic conductivity of the foundation materials is low, a short-term load is shown in the FE models as the water has not yet infiltrated into the foundation materials.

3.1.4 Shear Strength Reduction

The strength reduction factors/FOS for the Pit Lake scenario were typically above 1.3 under static conditions. The predominant failure mechanism is an upstream failure surface along/near the interface with the core. The fully thawed foundation scenario resulted in a FOS of 1.2 with a failure mechanism towards the downstream side of the dam through the finer silt and clay foundation materials.

Further details on the shear strength model runs for this scenario can be found in Figure 12.

3.2 Tailing Beach Scenario

For the typical tailings beach scenario, the tailings are expected to freeze to the surface upstream of the dam in the initial years (SRK 2014a). As surface temperatures warm due to climate change, the frozen tailings then act to slow the rate of thaw on the upstream slope of the dam. As a result, the active layer (relative to the base of the dam) at the upstream toe and centerline of the dam is shallower in year 20 compared to at the time when the dam is constructed. At the downstream toe of the dam, the active layer was predicted to increase by around 0.4 m after 20 years.

3.2.1 Displacements

As most of the foundation is expected to be remain frozen during operations, minor to no thaw settlements is predicted in the first 20 years and little additional settlement due to consolidation of the unfrozen material is expected. Overall, settlements due to consolidation were found to be less than 0.1 m (after thaw settlements) for the 20 year dam operation life; with the largest displacement occurring near the downstream toe of the dam. Under the fully thawed scenario, results similar to what was found in the Pit Lake runs (displacements around two meters) were noted; however, full melting of the permafrost foundation is expected to take multiple decades to centuries.

3.2.2 Differential Settlement

The differential settlements across the core were noted to be minimal; less than 0.05 m over the 6 m core width. Figures 13 and 14 show scaled deformed meshes and contoured plots of total displacements for the tailings beach model runs.

3.2.3 Shear Strength Reduction

The FOS for the typical tailings beach scenario was found to be around 1.5. The shear strength reduction computations showed that the predominant failure mechanism appears to be a failure surface in the downstream foundation that progressed down into the foundation materials.

Further details on the shear strength model runs for this scenario can be found in Figure 15.

3.3 Rates of Displacements

Thawing of the foundation is expected to occur very gradually and as a result the consolidation time is expected to be insignificant compared to the time to thaw. If thawing occurred instantaneously in the upper silt unit, the consolidation time would be on the order of 6 months (or less) for a 5 m or thinner layer beneath the dam. If the entire foundation instantaneously thawed, the consolidation time was estimated to be in the order of one year over the approximately 40 m thick overburden section. If the hydraulic conductivity of the fine foundation materials is found to be lower than 2.0×10^{-7} m/s then the consolidation times would increase.

3.4 812 m Crest Elevation Dam

If the Main Dam crest height is raised to 812 m, an increase in load is expected in some areas of the dam and on the dam foundation; however, the results are expected to be similar to the deformation results for an 807 m dam. For the 807 m crest dam with a developed tailings beach, slightly larger settlements are expected on the downstream. As most of the foundation is expected to be frozen (SRK 2014a), the increase in foundation consolidation settlement for the 812 m dam is expected to be minor (increase less than approximately 0.1 m). Figure 16 presents some additional notes to compare the 807 m and 812 m crest elevation dams.

4 Final Remarks

Based on this analysis, the Main Dam design has FOS and deformations after thaw settlements that are predicted to be within acceptable operational levels. As part of the detailed design stage it is suggested that additional deformation analyses be performed to further optimize the core geometry and/or material type to assist with further increasing the FOS and to mitigate against the potential of the core cracking.

This analysis was completed on a section of the dam founded on overburden with a thickness of 40 m. If a thicker section of overburden is apparent under the dam it would be expected that the deformation would not be significantly different than what was presented in this analysis; and may be less if a smaller height of dam is noted. Deformations on portions of the dam founded on thicker overburden are not expected to change the results presented. Deformations on the portions of the dam founded on thinner overburden and/or on bedrock, are expected to be less than what was derived from this analysis.

5 References

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Figures

PLAN LEGEND

- Phase IV/V/VI Pit Shell
- Phase V/VI Waste Dump
- Approximate Tailings Area
- Approximate limits of South Wall Failure
- 2013 Main Dam Drillholes

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Main Dam Deformation Analysis

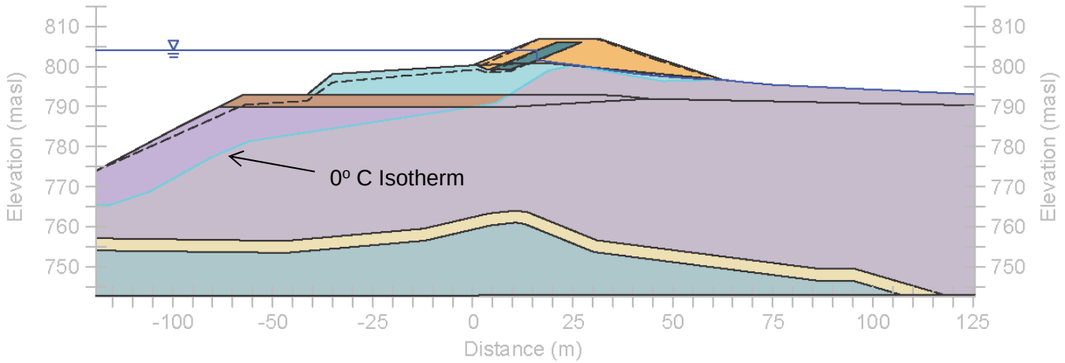
Site Plan & Section Location

Job No: 1CM002.018
Filename: Dam_Deformation1CM002.018.ppt

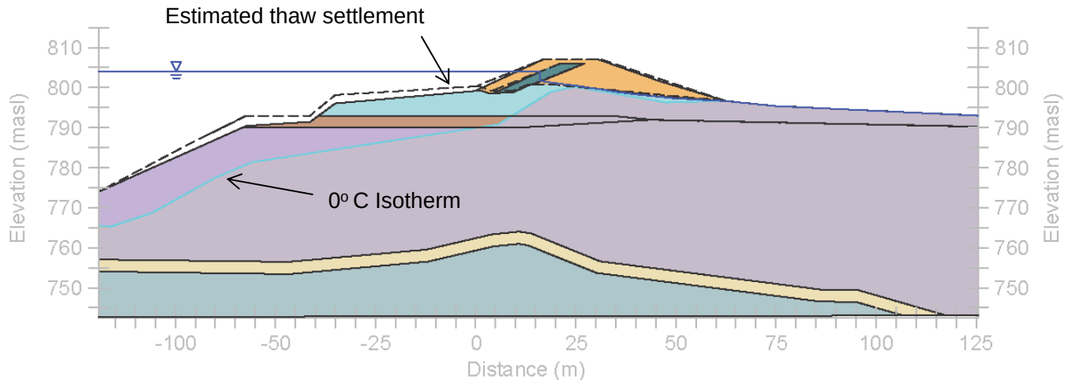
Minto Mine

Date:	Approved:	Figure:
May 2014	JBK	1

Pit Lake Scenario

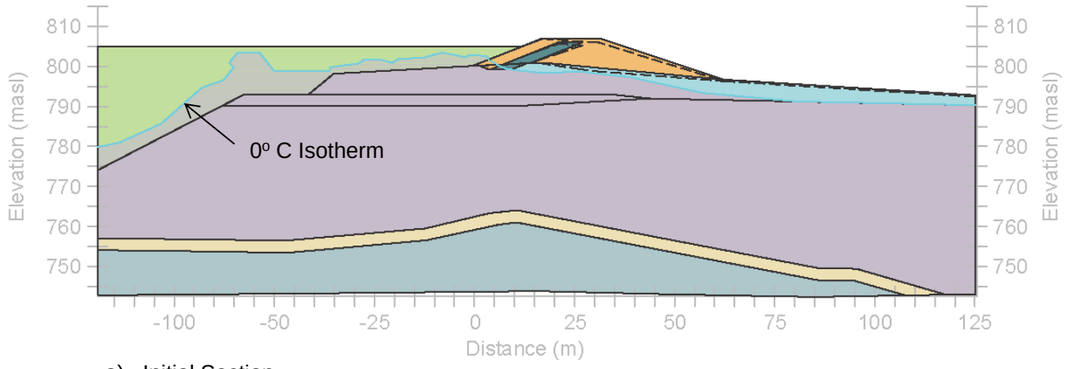


a) Initial Section



b) Section adjusted for thaw settlement

Tailings Beach Scenario



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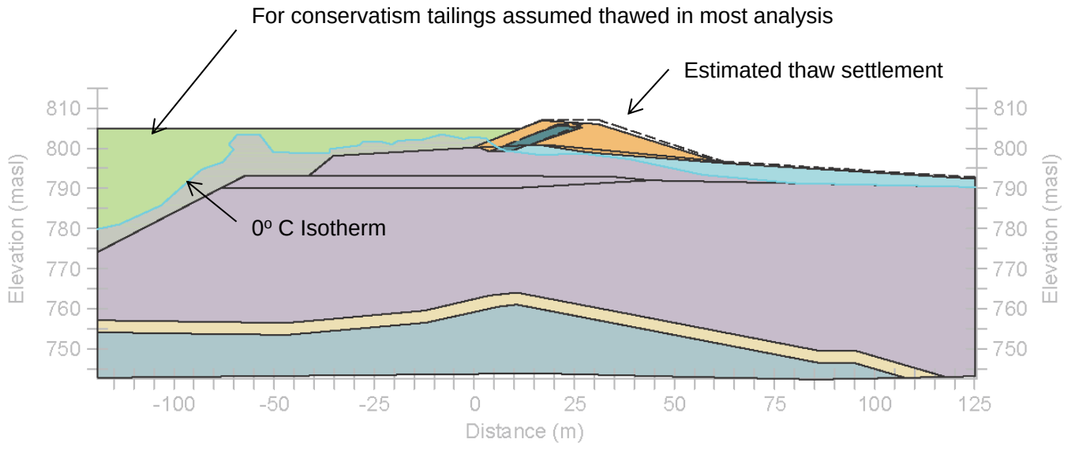

b) Section adjusted for thaw settlement

Legend

- Waste Rock
- Silt (unfrozen)
- Frozen Soil
- Gravelly Silt Core
- Sand & Gravel (unfrozen)
- Weathered Bedrock
- Tailings
- Clay (unfrozen)
- Bedrock

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Job No: 1CM002.018
Filename: Fig_DamThermal_11x17land.pptx

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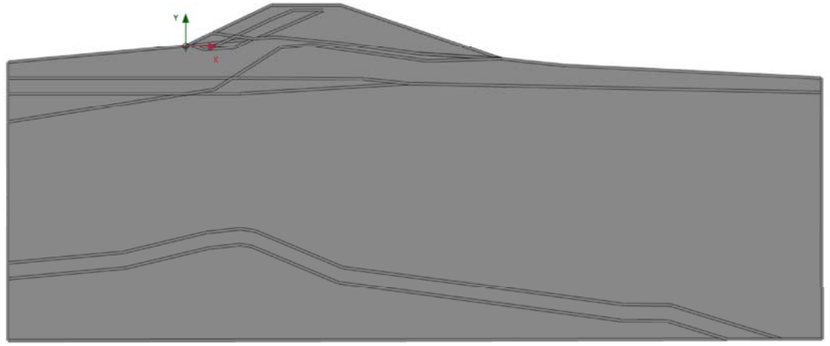
Minto Mine

Main Dam Deformation Analysis

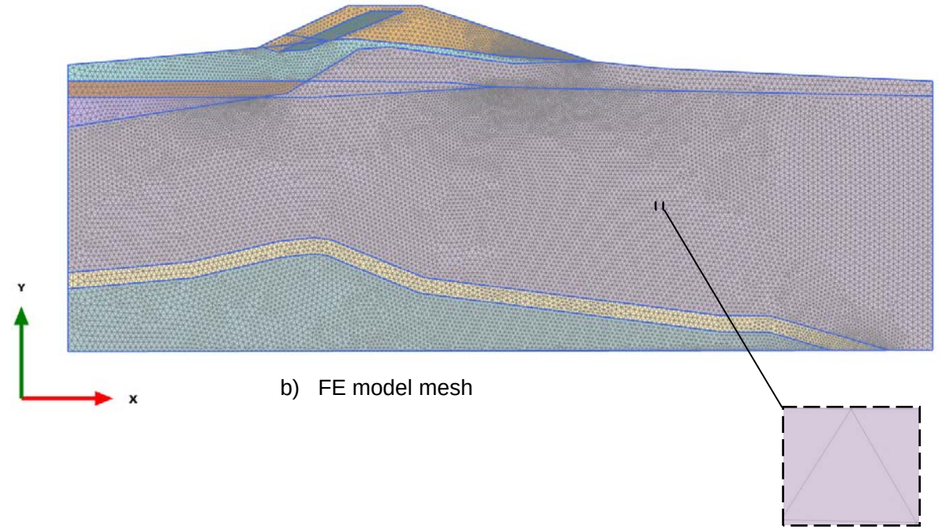
General Model Set-Up

Date:	Approved:	Figure:
May 2014	JBK	2

Pit Lake Scenario

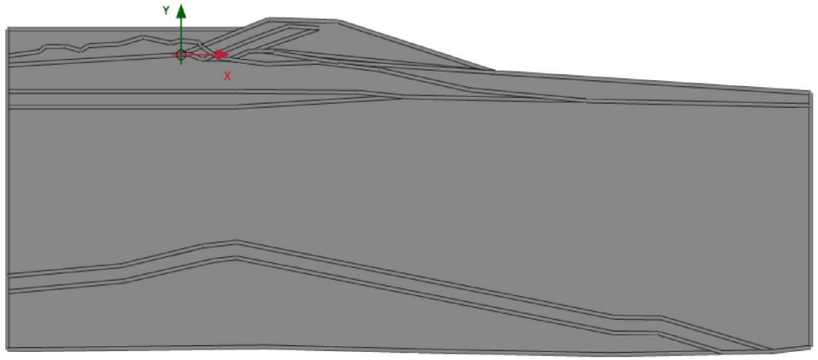


a) Model set-up for meshing / discretization



b) FE model mesh

Tailings Beach Scenario



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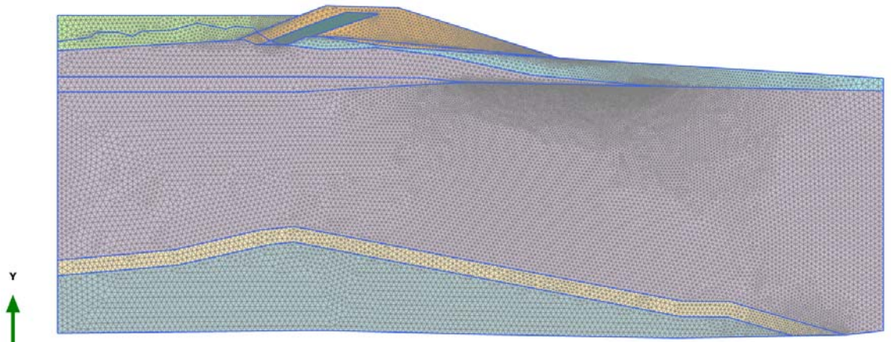
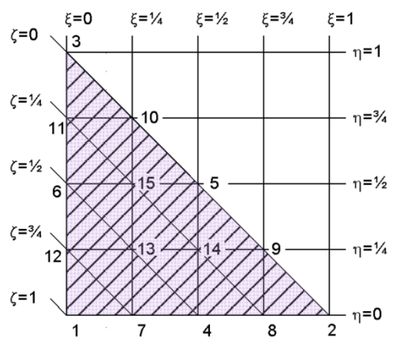


b) FE model mesh

15-Node Triangular Element Overview



Local numbering and position of notes of a 15-node triangular element (Plaxis 2014b) shown in diagram to the left.

For triangular elements in Plaxis there are two local coordinates (ξ and η). These are used to describe the shape functions for the elements.

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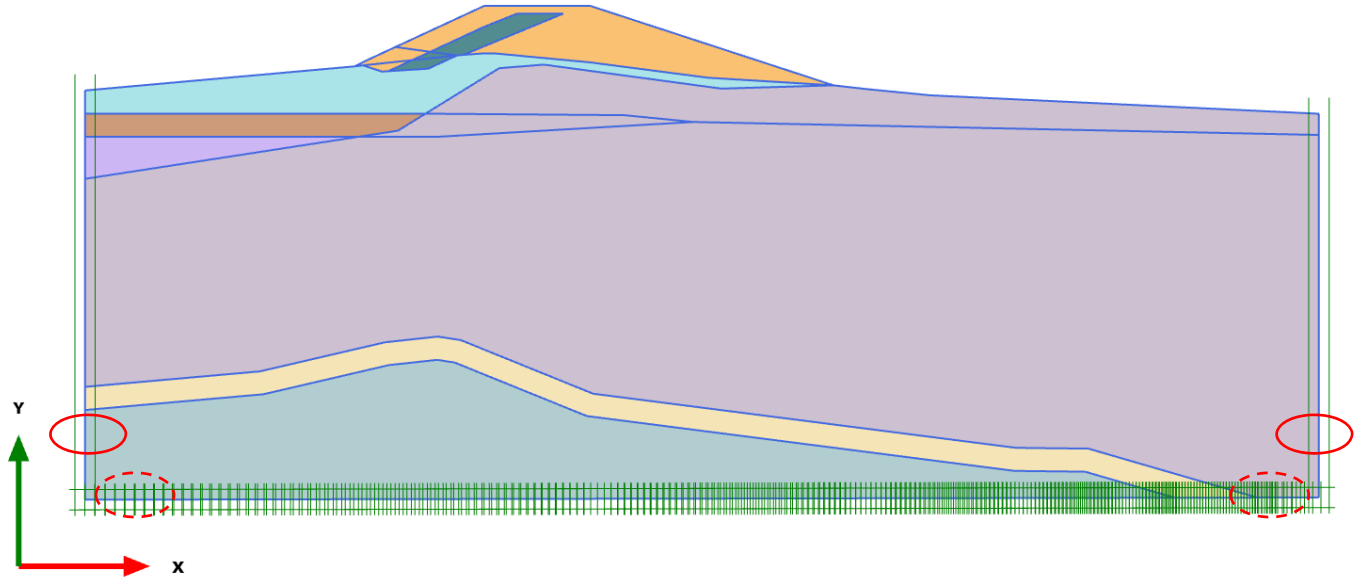
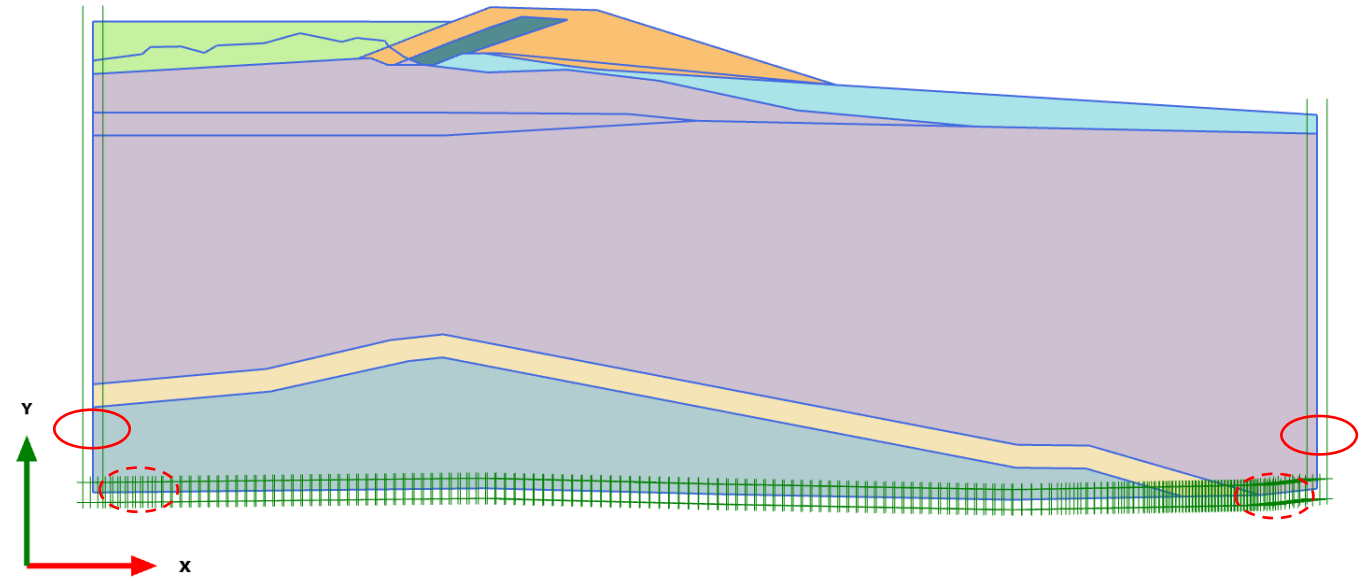
		FE Model Mesh
Job No: 1CM002.018 Filename: Fig_DamThermal_11x17land.pptx	Minto Mine	Date: May 2014 / Approved: JBK / Figure: **3**

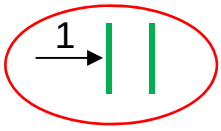
Pit Lake Scenario



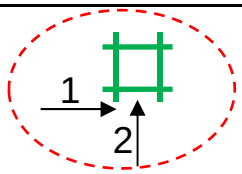
Tailings Beach Scenario



Tailings Beach Scenario



fixed horizontally, or with normal in x-direction and free in y direction



fully fixed, horizontal and vertical both fixed

Sources:
a) x

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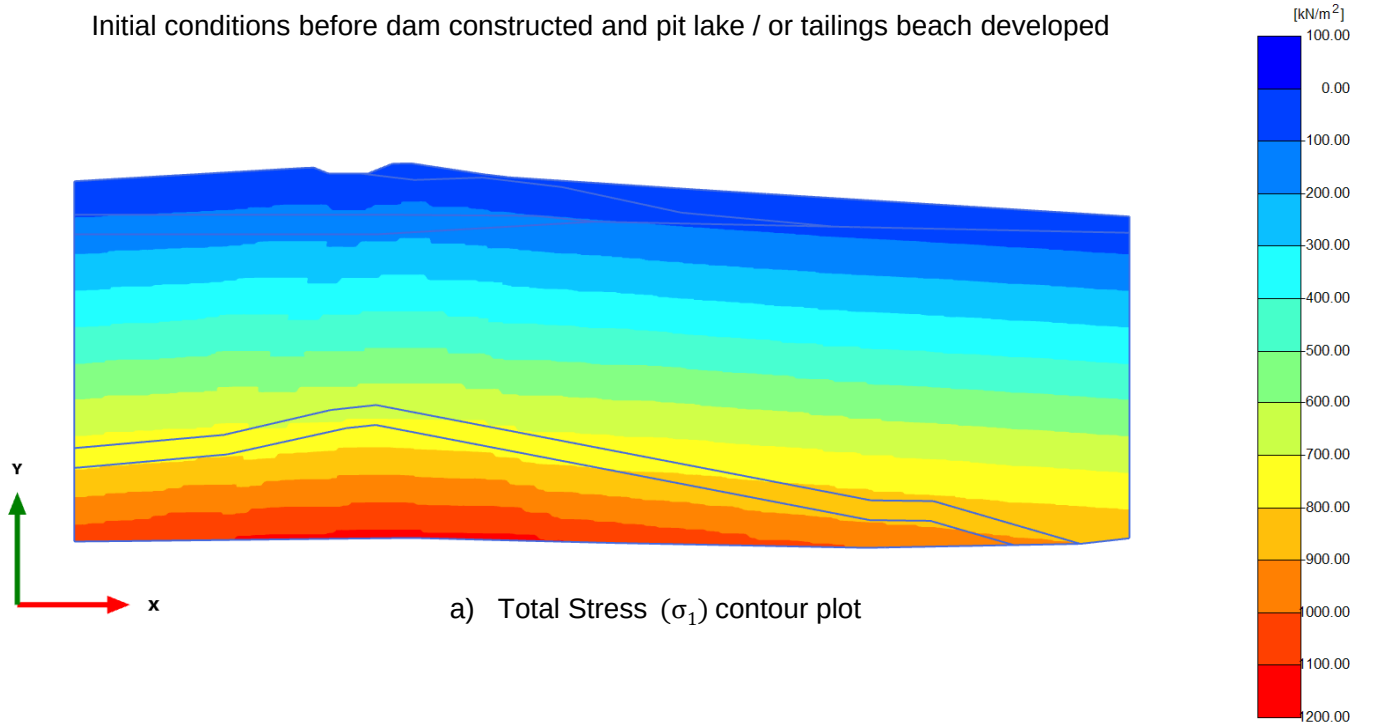
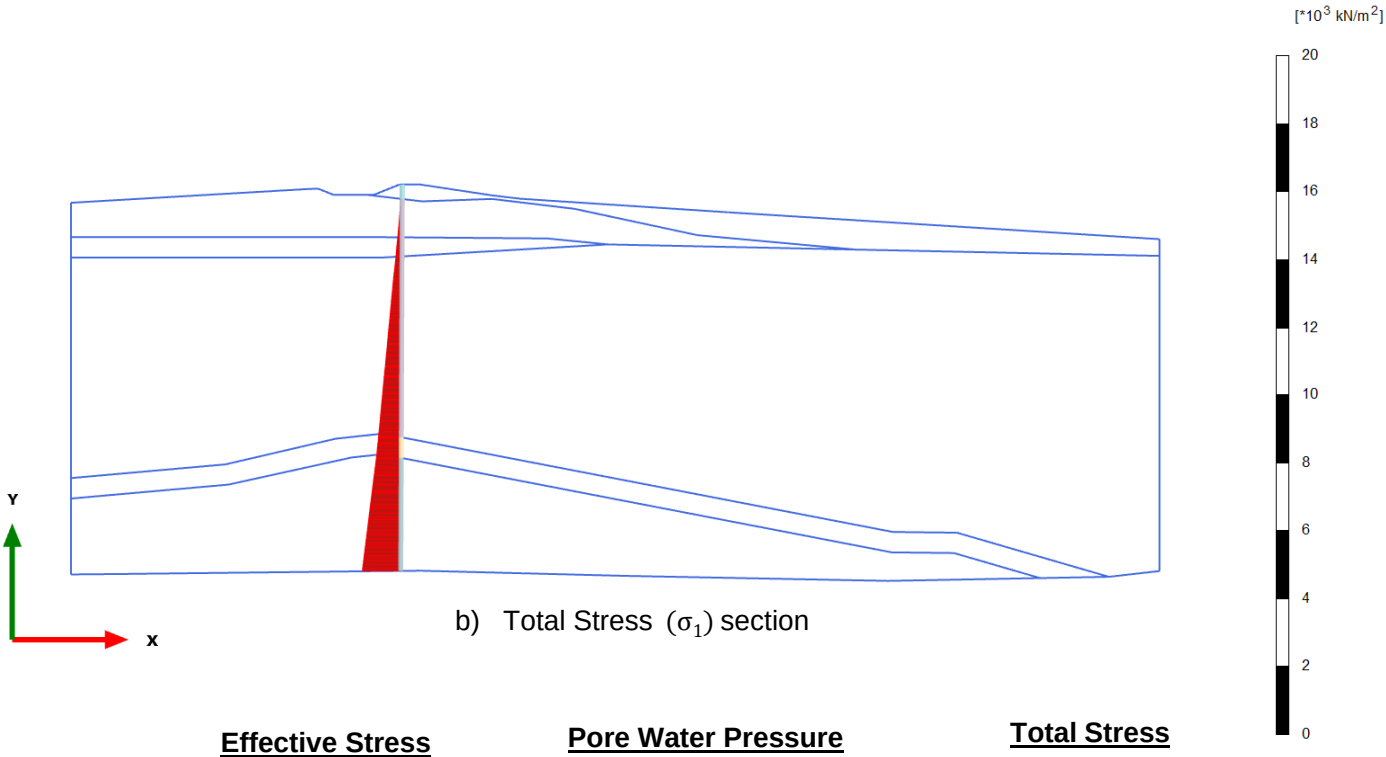
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014 | Approved: JBK | Figure: **4**

Initial conditions before dam constructed and pit lake / or tailings beach developed



a) Total Stress (σ_1) contour plot



b) Total Stress (σ_1) section

Effective Stress

$$\sigma'_v = \gamma' * Z$$
$$\sigma'_h = K_0 * \sigma'_v$$

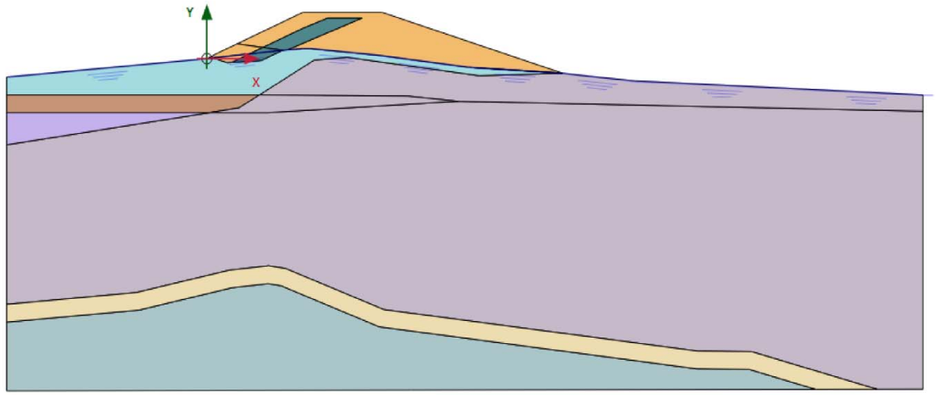
Pore Water Pressure

$$\sigma_w = \gamma w * z$$

Total Stress

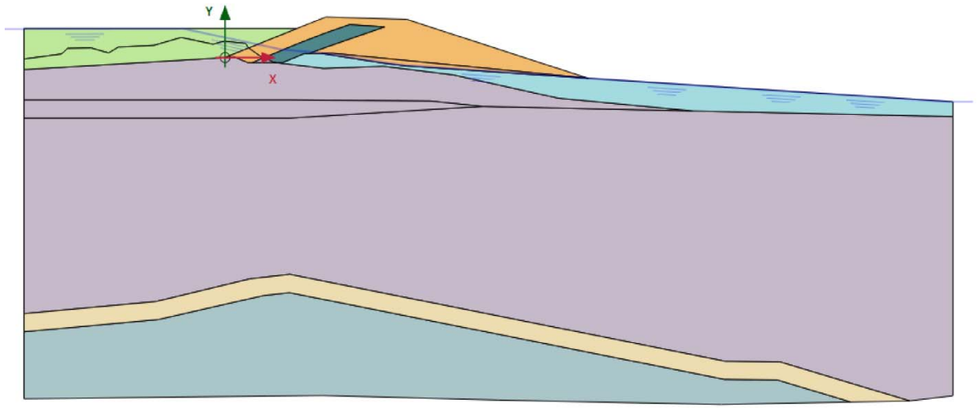
$$\sigma_v = \sigma'_v + \sigma w$$

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Initial Conditions**		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: 5



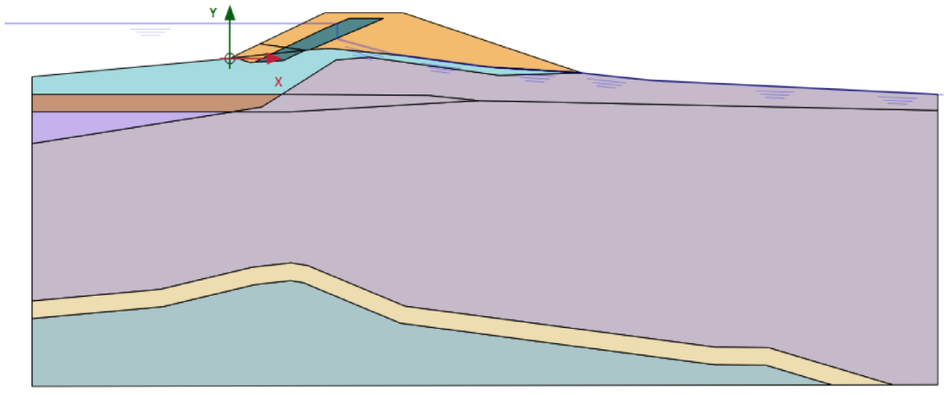
2. Second Stage – Dam construction

Plastic deformation and / or one day consolidation calculated



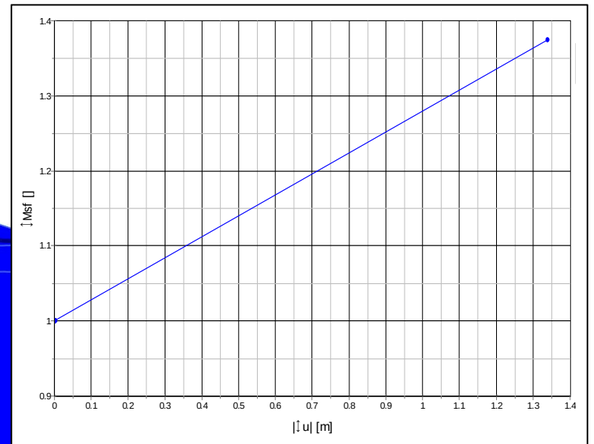
3. Third Stage – Tailings added

Consolidation after one year calculated



3. Alternate Third Stage – Water added

Consolidation after one year calculated



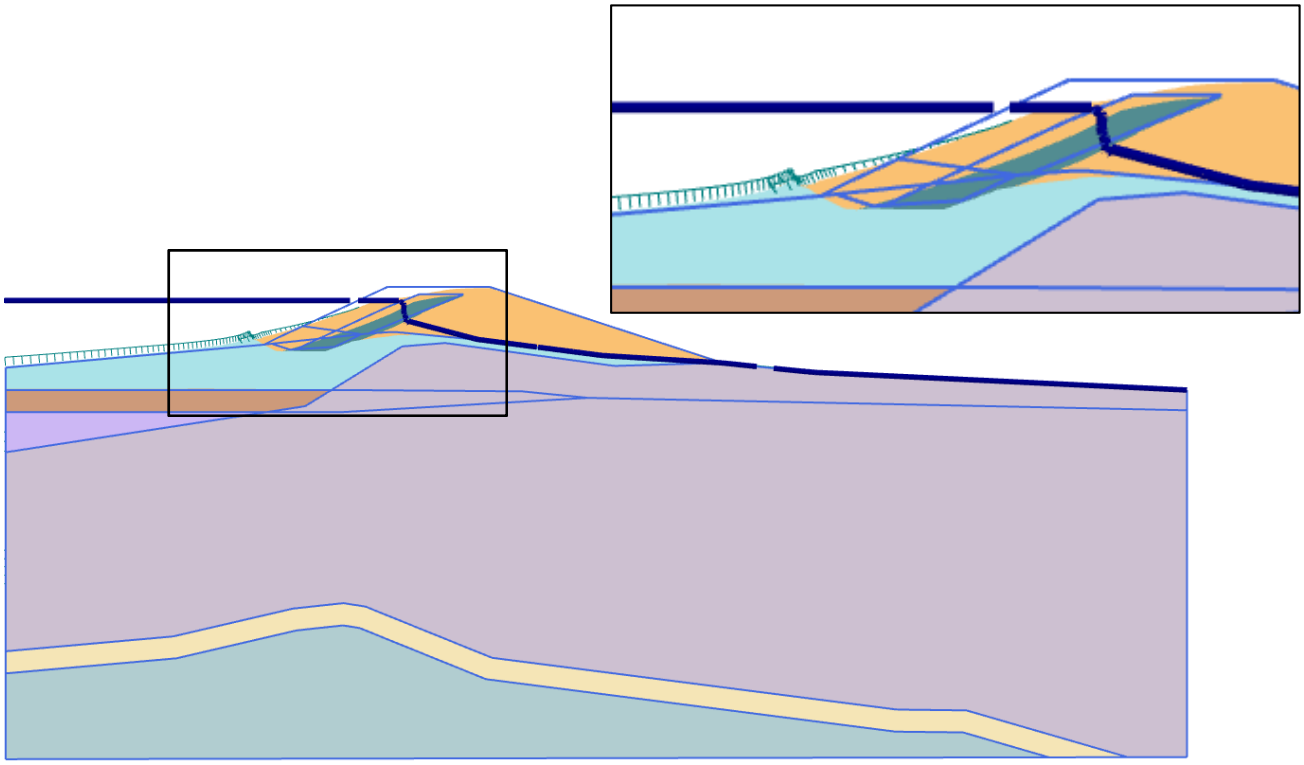
Decreasing materials shear strengths and recomputing deformations until failure

4. Fourth Stage – Shear strength reduction analysis

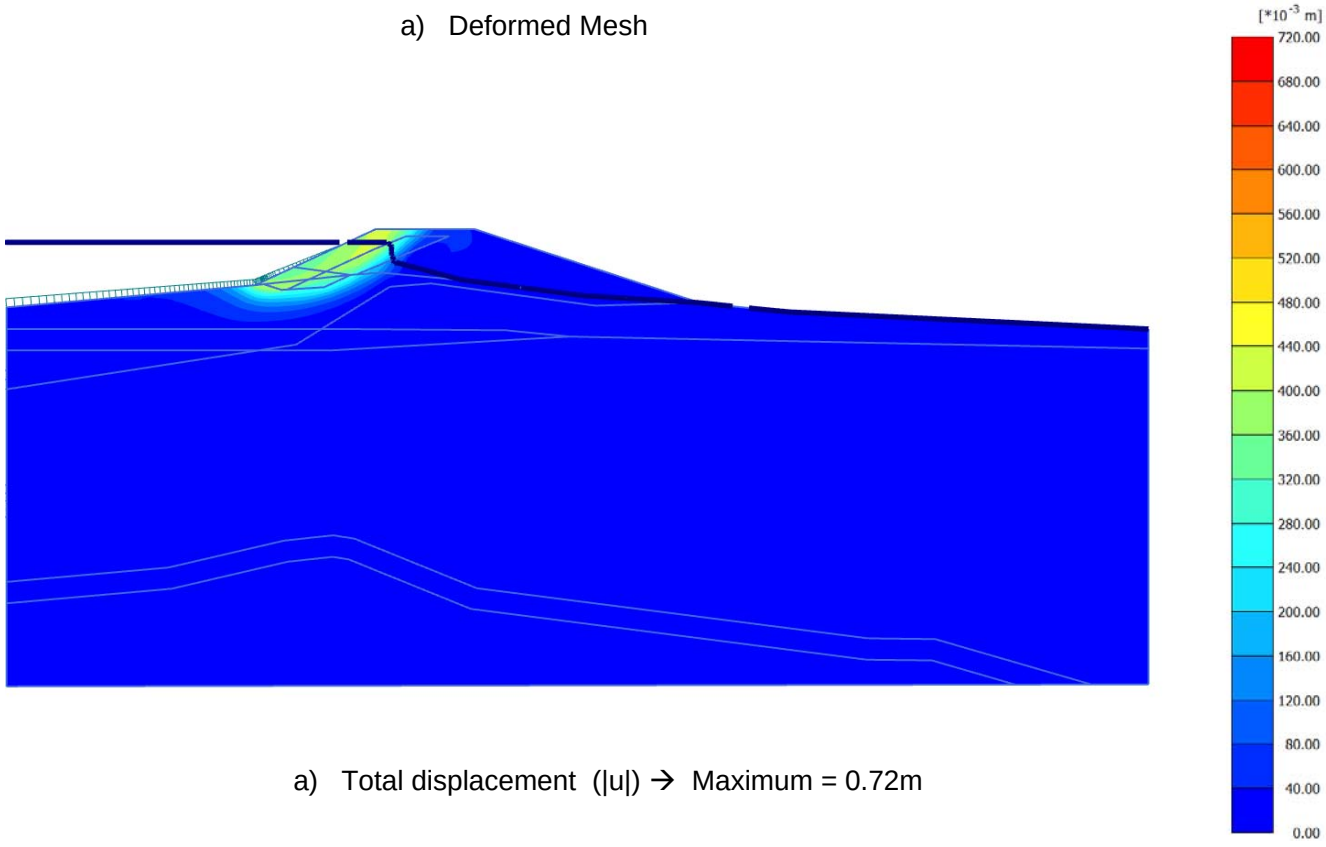
Safety factor (or FOS) computed

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis		
		Construction Stages		
Job No: 1CM002.018 Filename: Fig_DamThermal_11x17land.pptx	Minto Mine	Date: May 2014	Approved: JBK	Figure: **6**

807 m crest dam with the Main Pit filled with water up to the spillway elevation of 804 m.



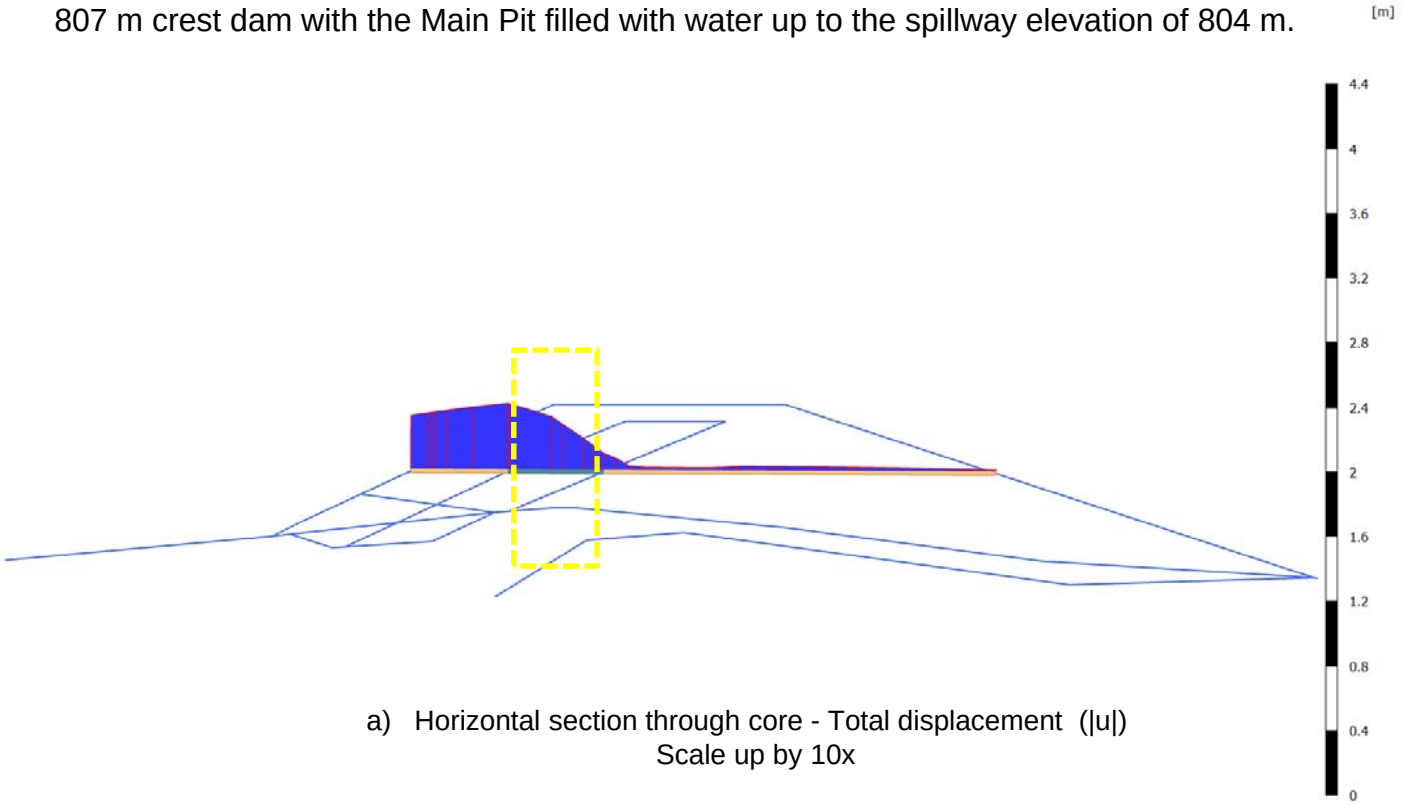
a) Deformed Mesh



a) Total displacement (|u|) → Maximum = 0.72m

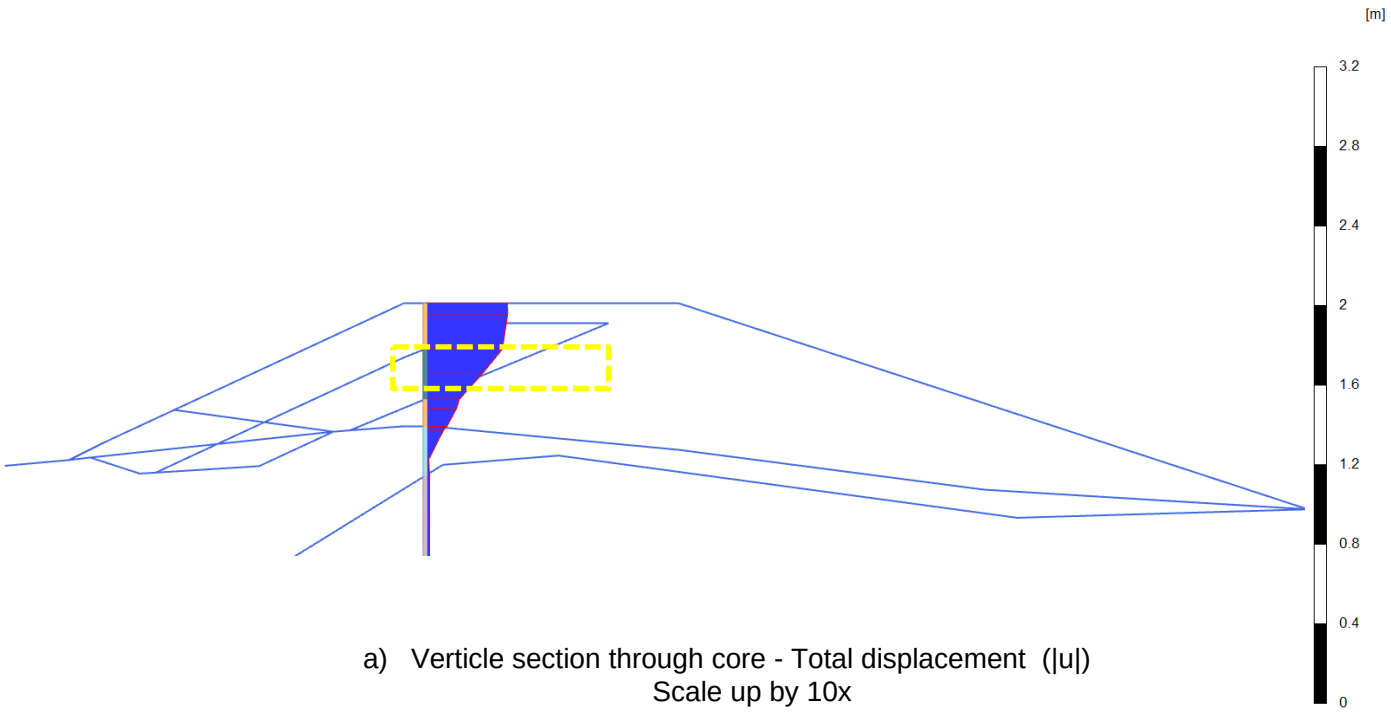
srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis		
		Pit Lake Scenario (20yr Thaw) Displacements		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: 7

807 m crest dam with the Main Pit filled with water up to the spillway elevation of 804 m.



a) Horizontal section through core - Total displacement (|u|)
Scale up by 10x

Differential Settlement across 6m core ~0.35m

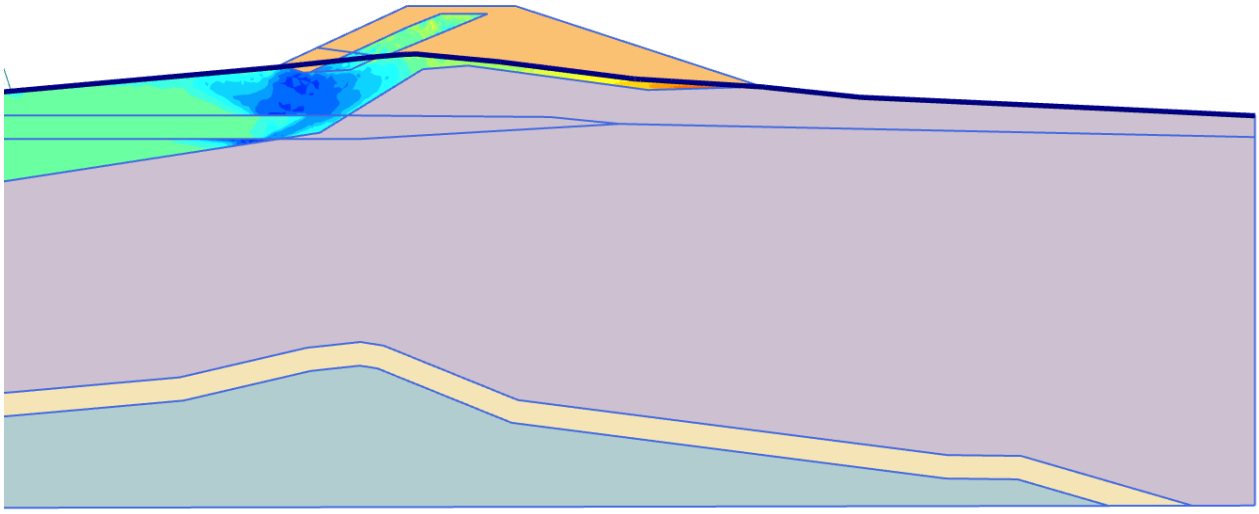


a) Verticle section through core - Total displacement (|u|)
Scale up by 10x

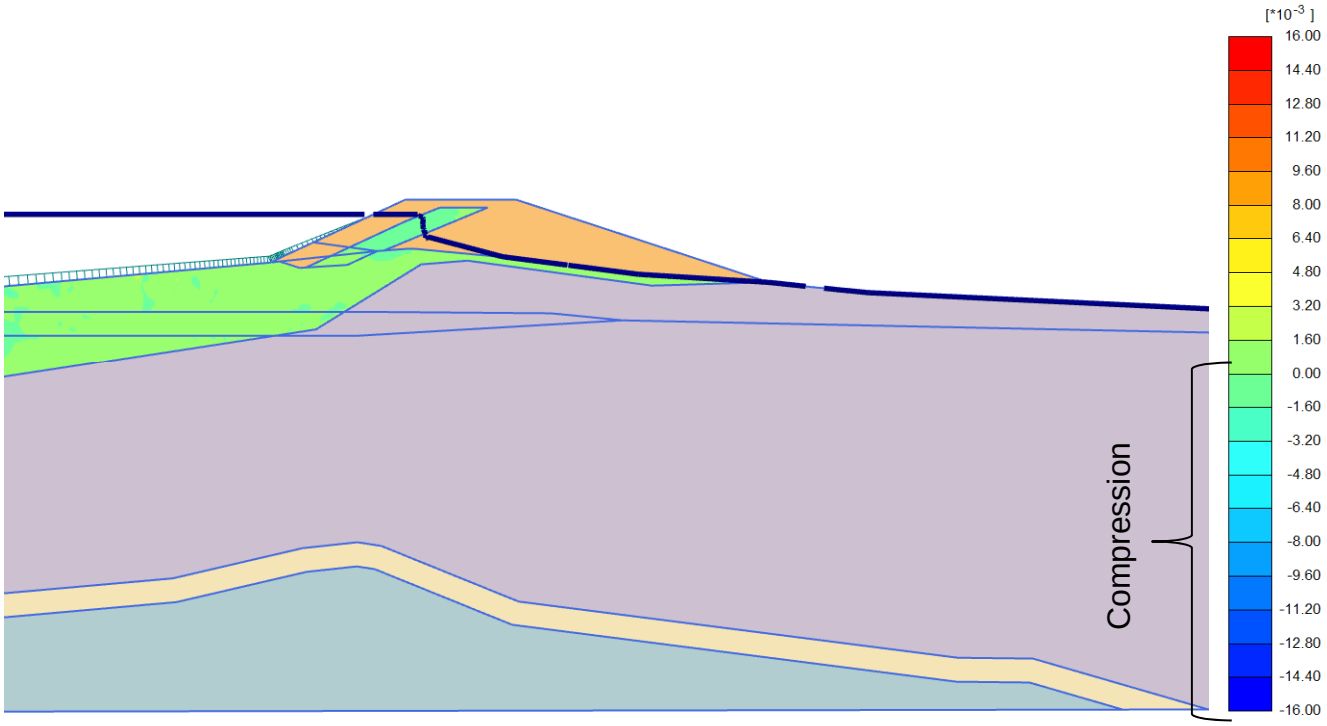
Max displacement point ~0.4m

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Pit Lake Scenario (20yr Thaw) Sections Through Core**
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014 / Approved: JBK / Figure: **8**

807 m crest dam with the Main Pit filled with water up to the spillway elevation of 804 m.



Strain History – after 1 day consolidation

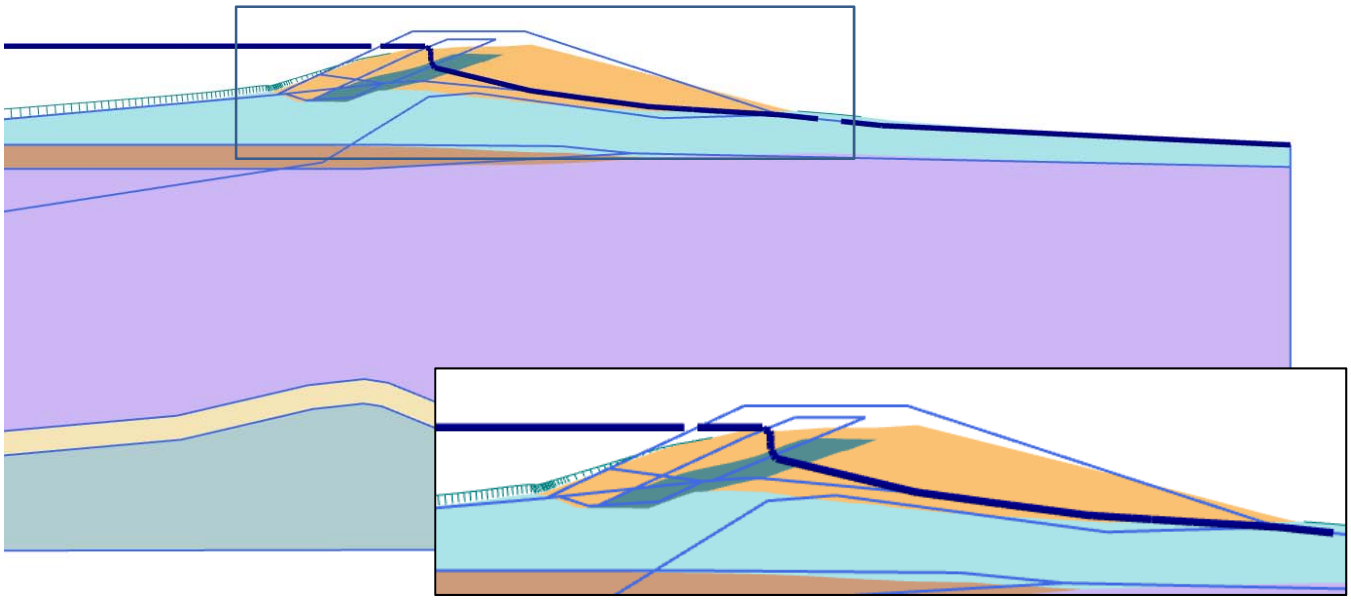


Strain History – after 1 year consolidation

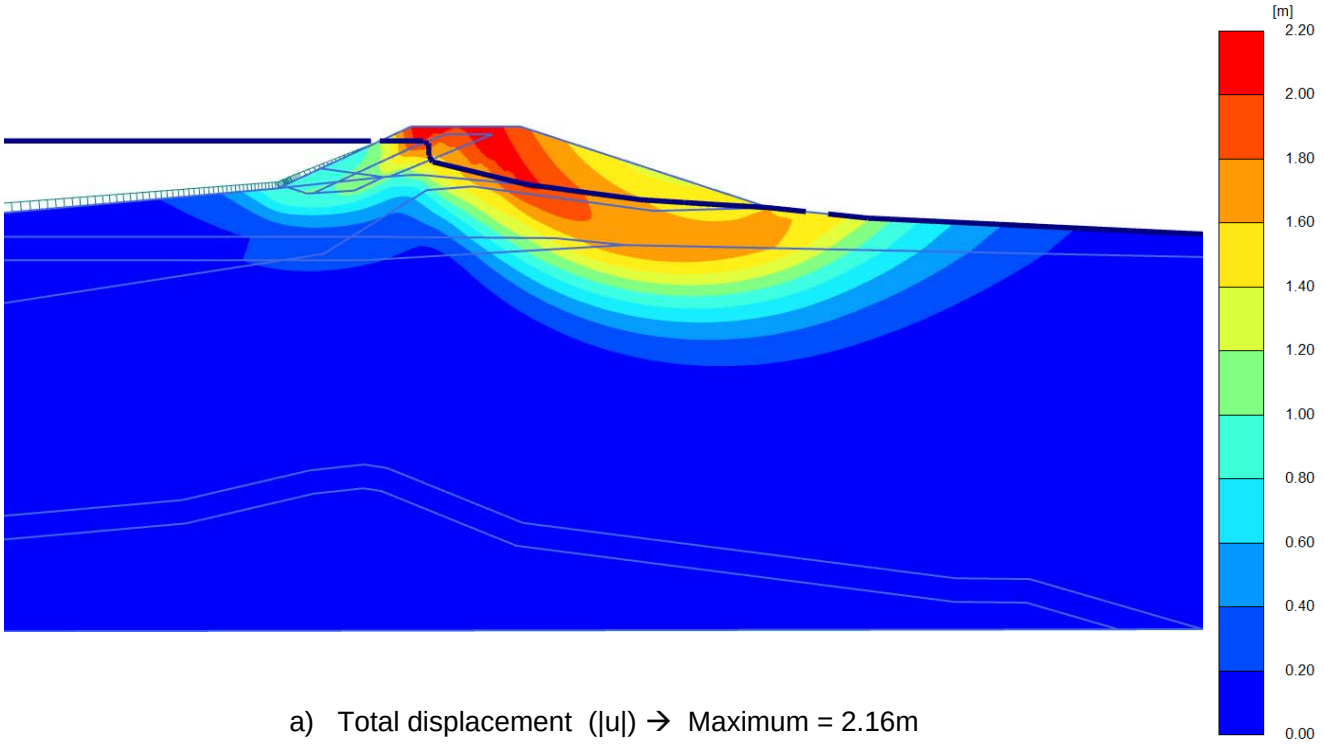
srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Pit Lake Scenario (20yr Thaw)** **Shear Strains**		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: **9**

Sensitivity Analysis

807 m crest dam with Main Pit filled with water and the foundation assumed to be fully thawed



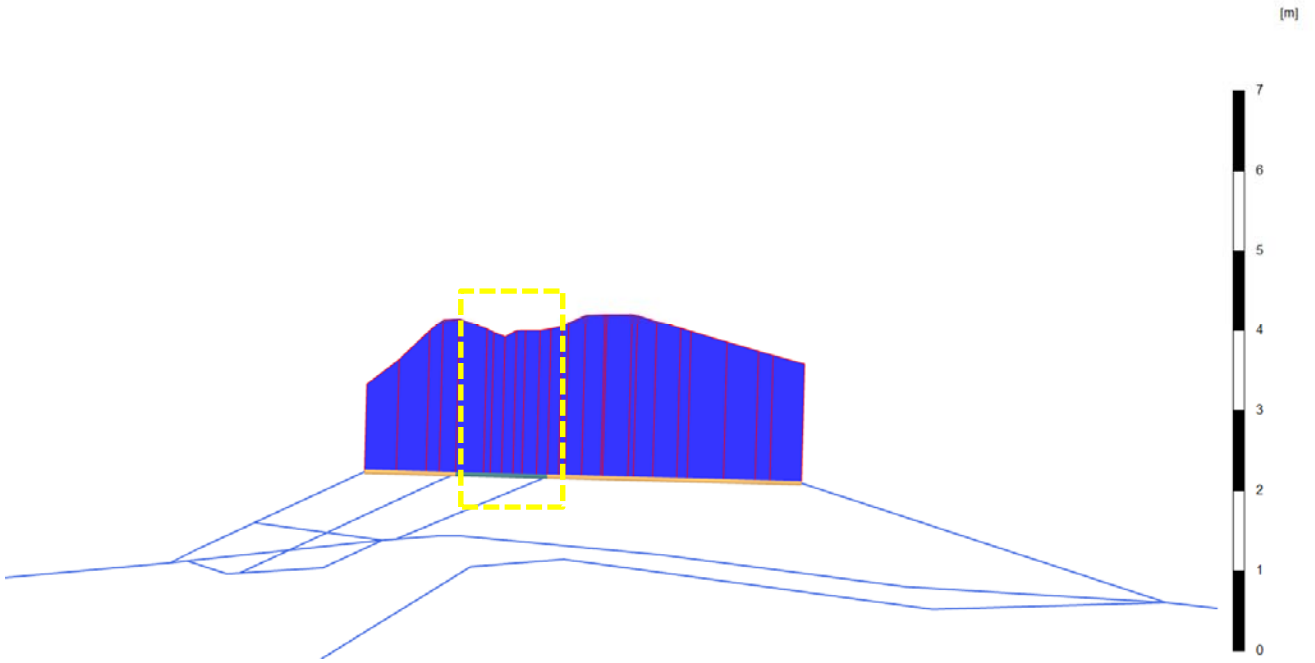
a) Deformed Mesh (true scale)



a) Total displacement (|u|) → Maximum = 2.16m

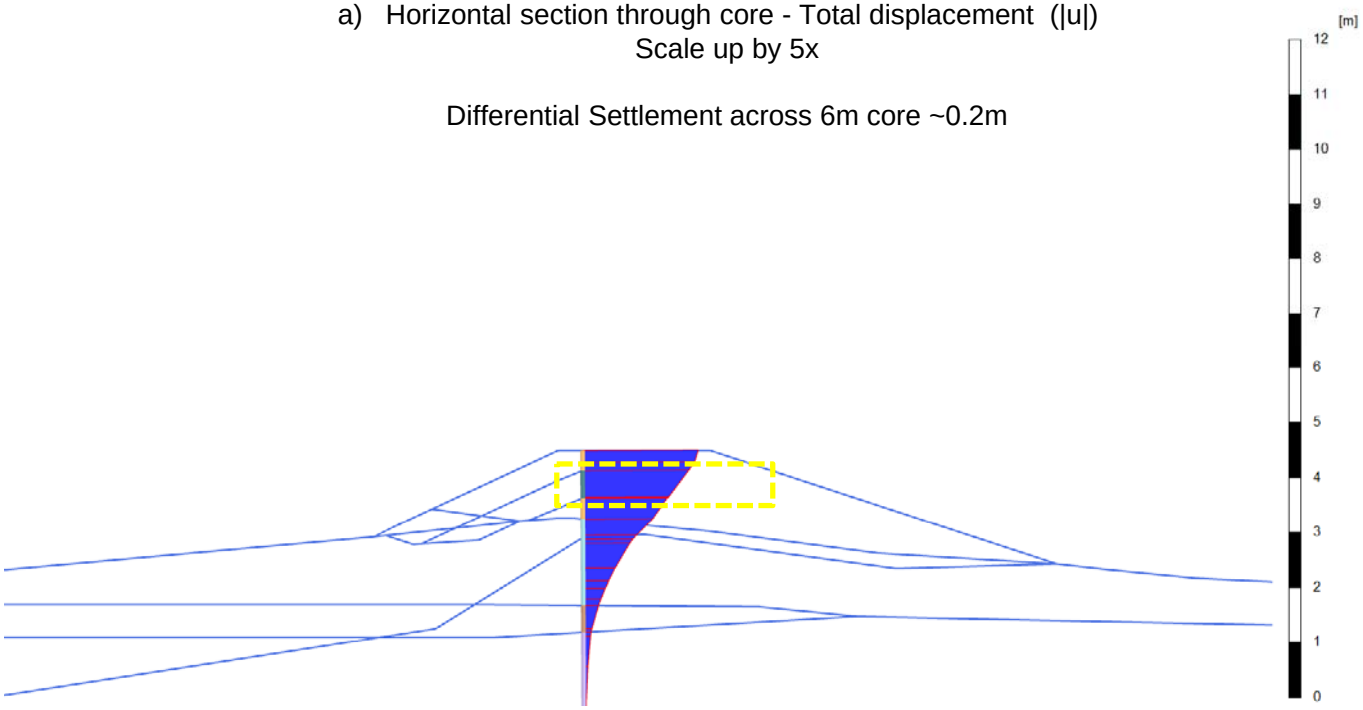
srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Pit Lake Scenario (Full Thaw) Displacements**		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: **10**

807 m crest dam with Main Pit filled with water and the foundation assumed to be fully thawed



a) Horizontal section through core - Total displacement (|u|)
Scale up by 5x

Differential Settlement across 6m core ~0.2m

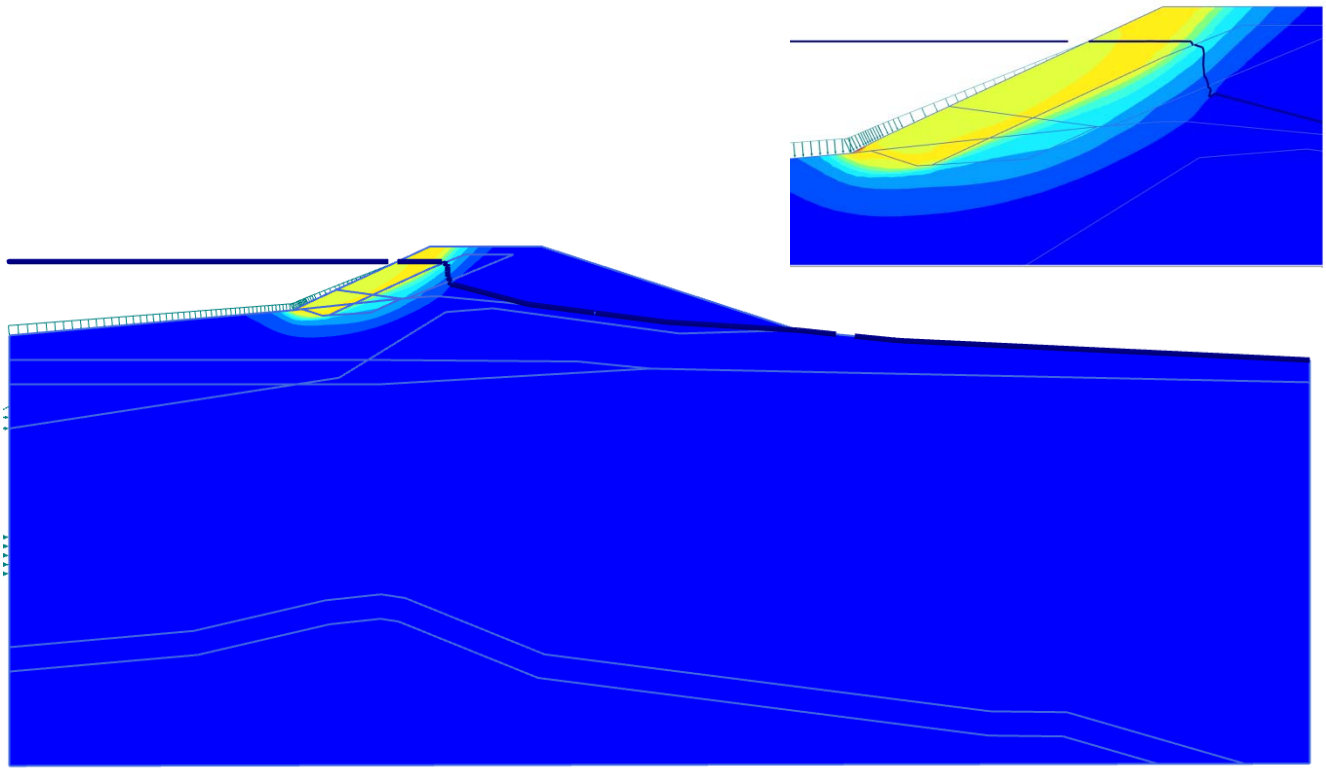


a) Vertical section through core - Total displacement (|u|)
Scale up by 5x

Max displacement point ~2.1m

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Pit Lake Scenario (Full Thaw) Sections Through Core**		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: **11**

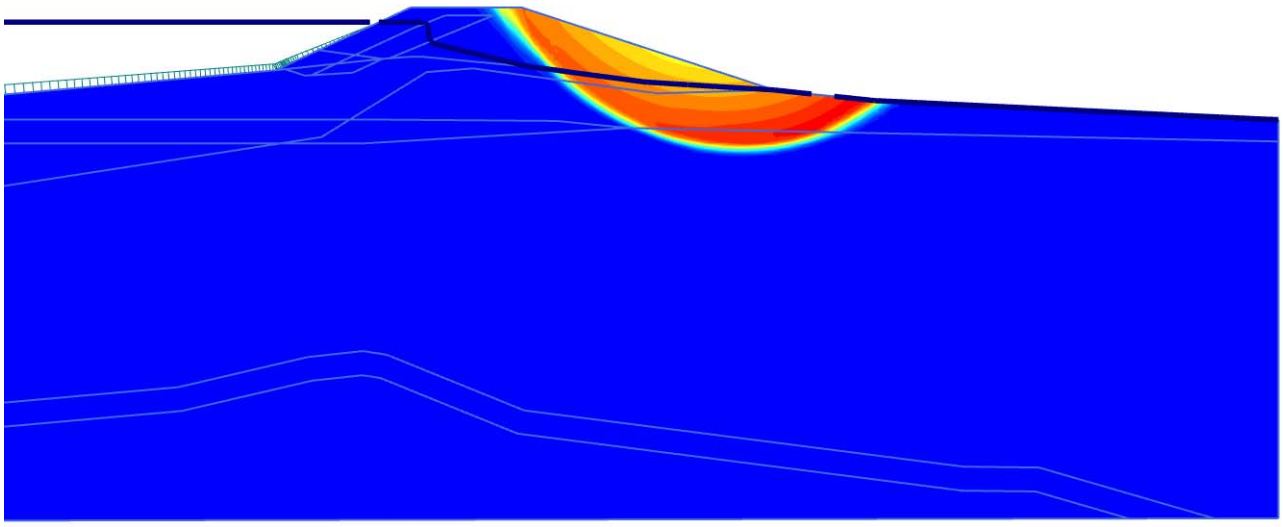
Conservative Case – 20 yr Thaw



Shear strength reduction / FOS = 1.3

Total displacement shown on plots to highlight failure mechanism.

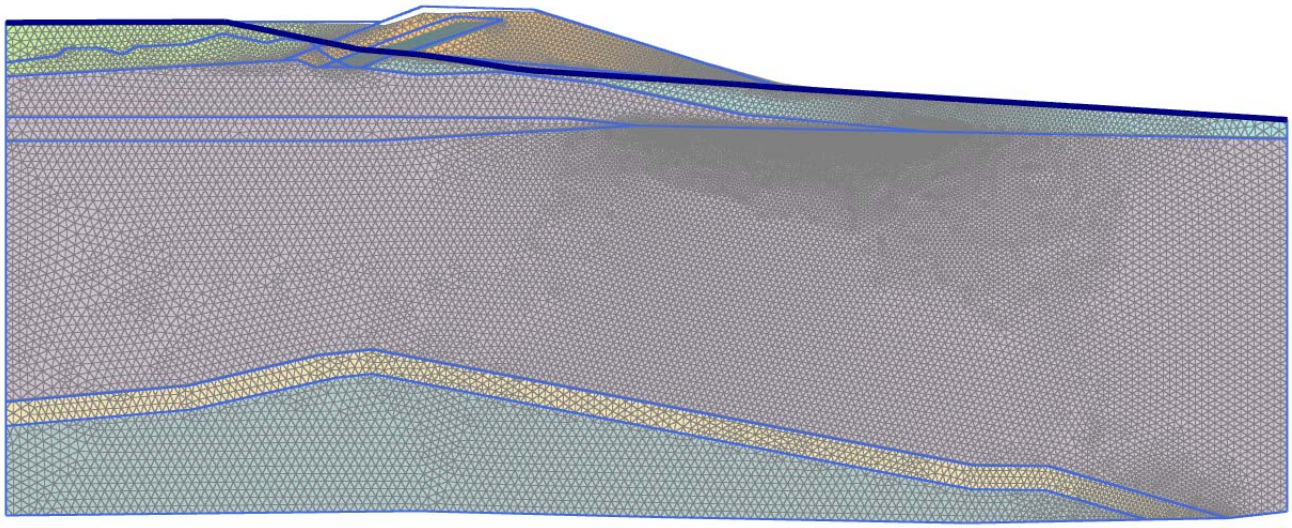
Sensitivity Analysis / Unexpected– Full Foundation Thaw



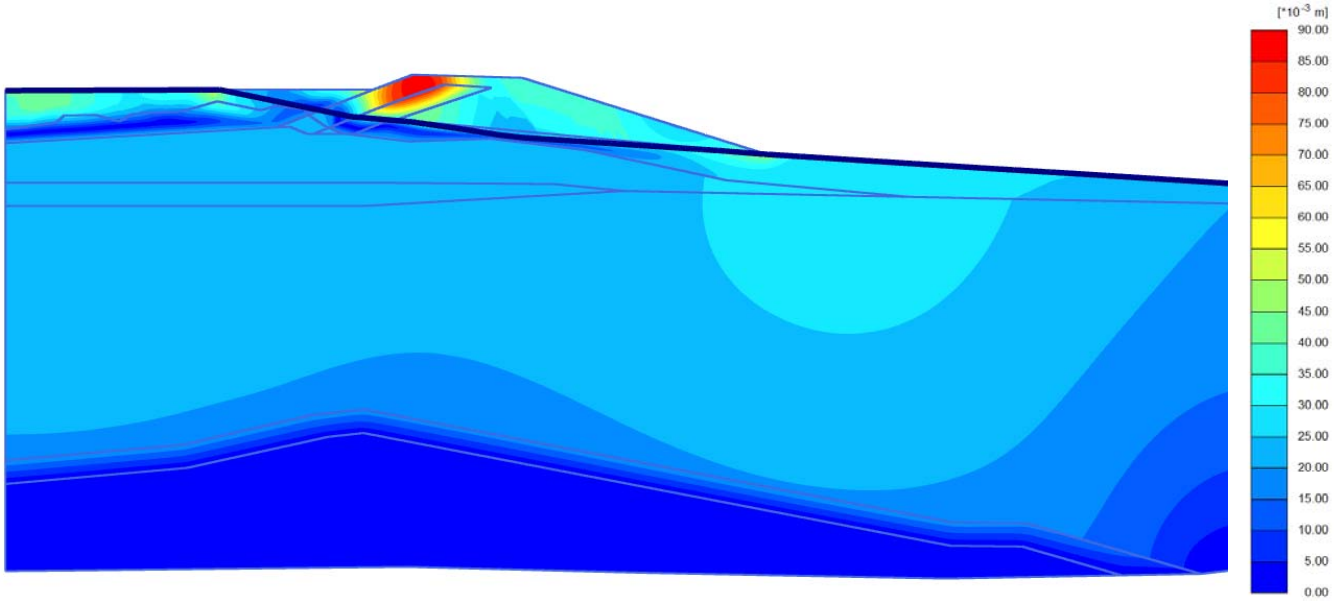
Shear strength reduction / FOS = 1.2

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Pit Lake Scenarios** **Shear Strength Reduction**		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: **12**

807 m crest dam with a tailings beach developed up to 805m.



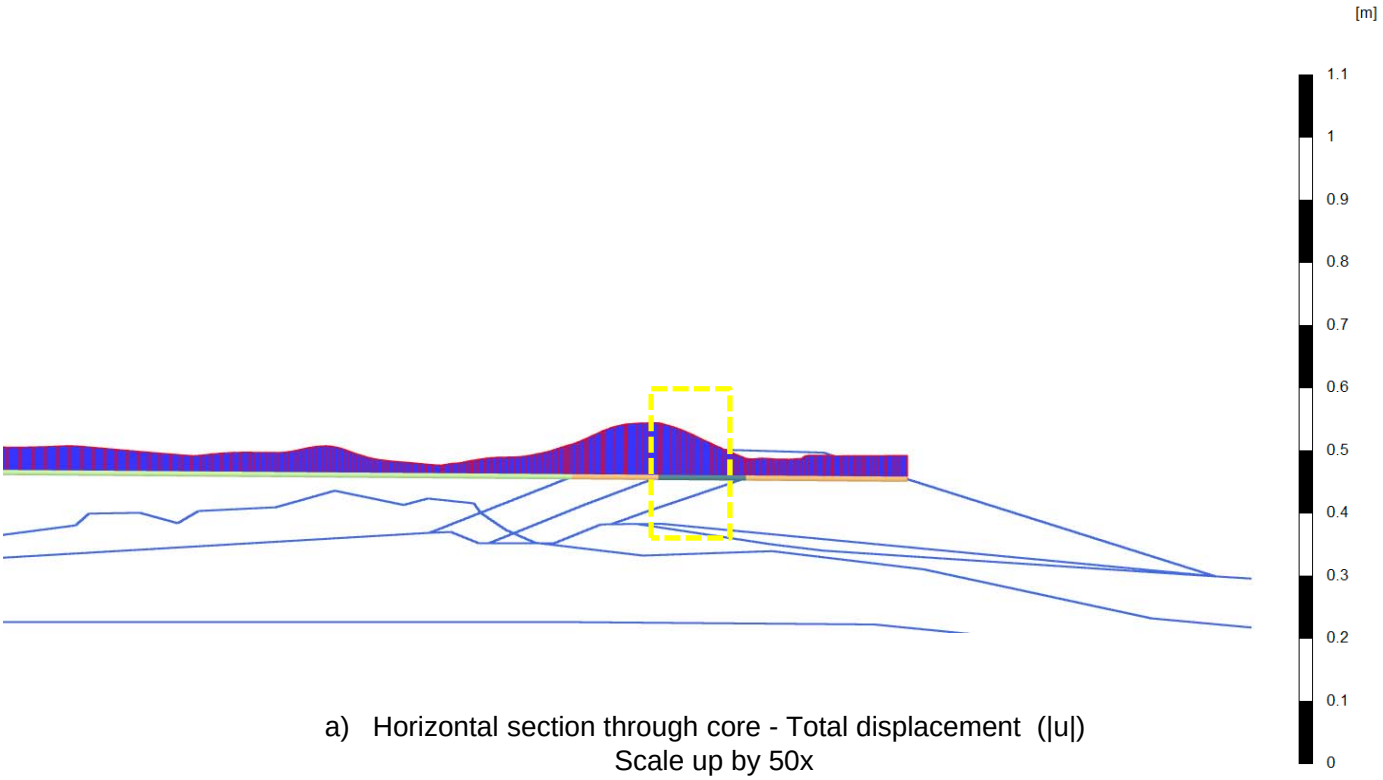
a) Deformed Mesh



a) Total displacement (|u|) → Maximum = 0.09m

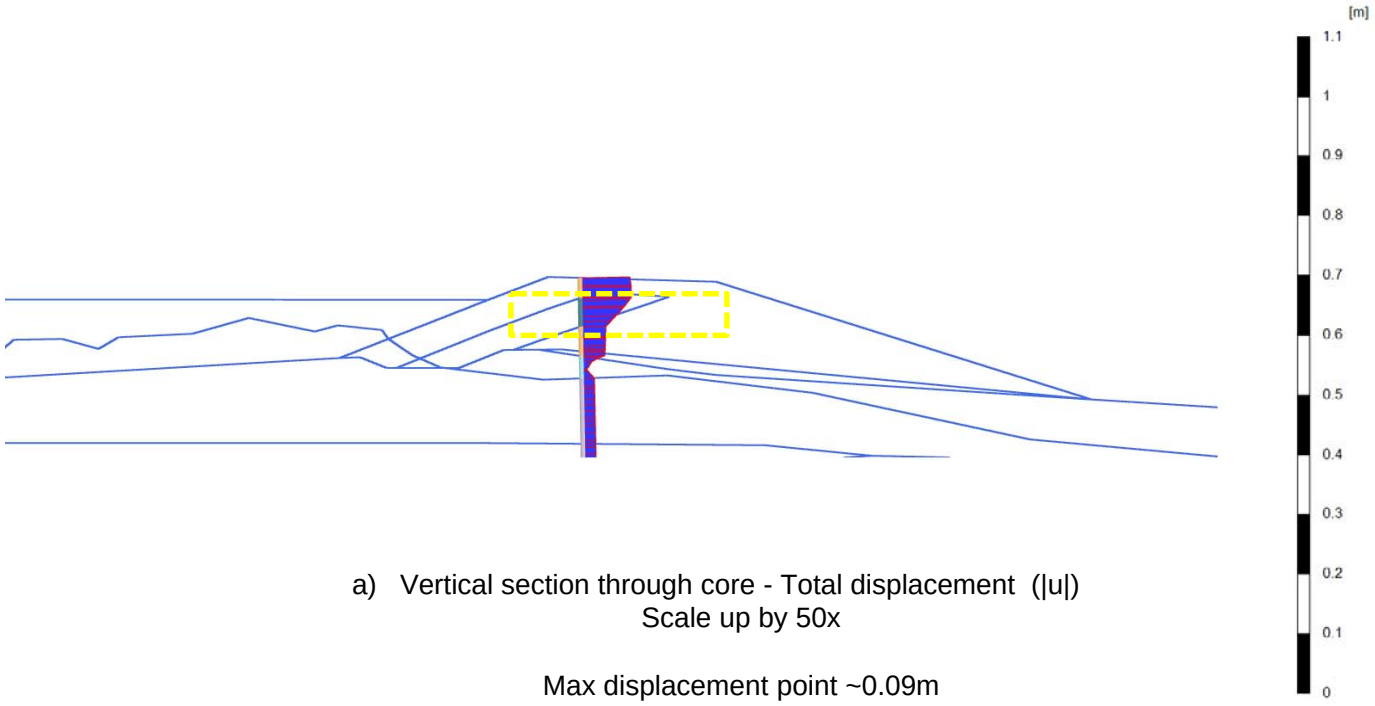
srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Tailings Beach Scenario Displacements**		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: **13**

807 m crest dam with a tailings beach developed up to 805m.



a) Horizontal section through core - Total displacement (|u|)
Scale up by 50x

Differential Settlement across 6m core ~0.05m

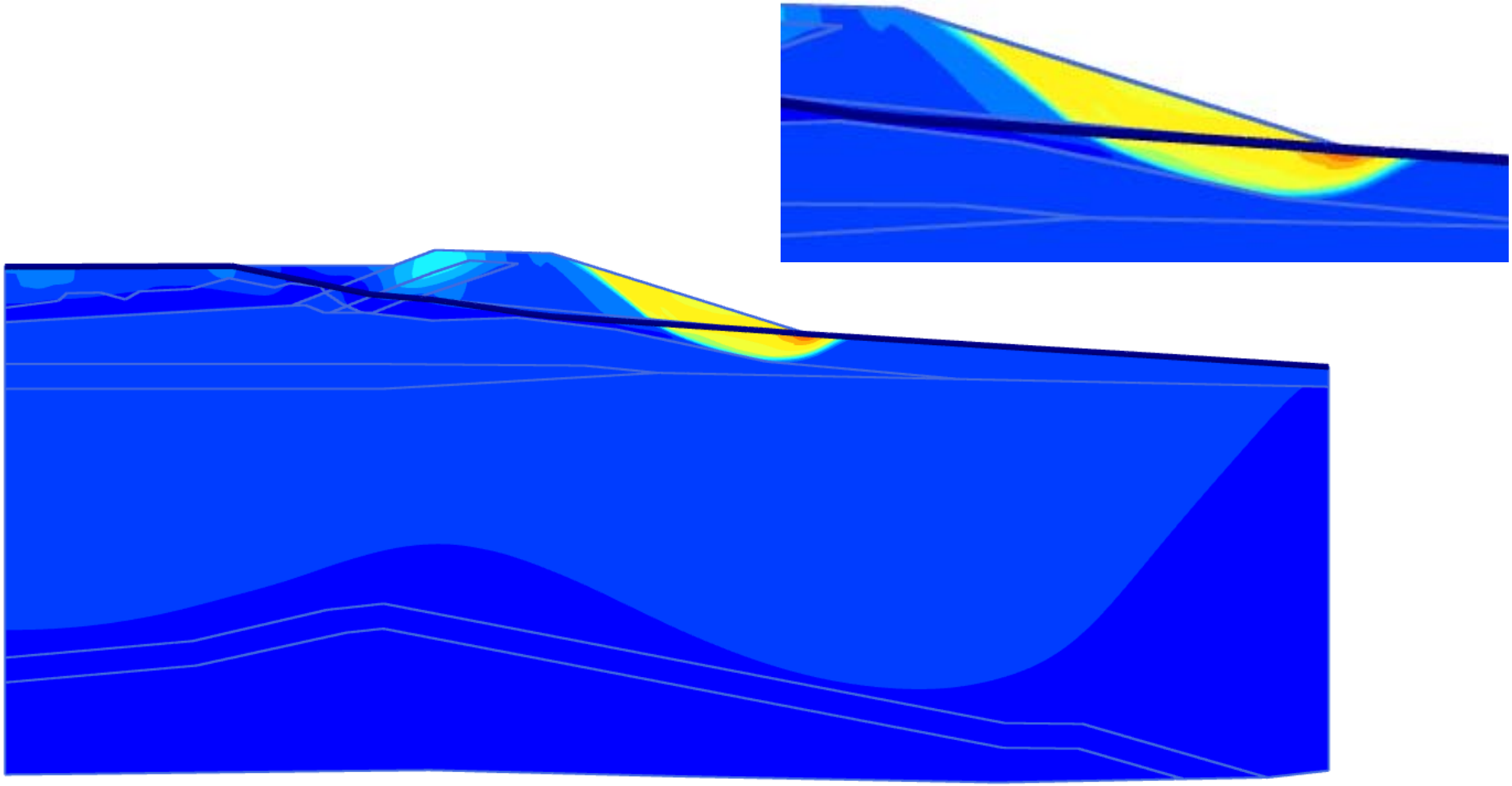


a) Vertical section through core - Total displacement (|u|)
Scale up by 50x

Max displacement point ~0.09m

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis **Tailings Beach Scenario Sections Through Core**		
Job No: 1CM002.018 Filename: Fig_DamThermal_LetterPortrait.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: **14**

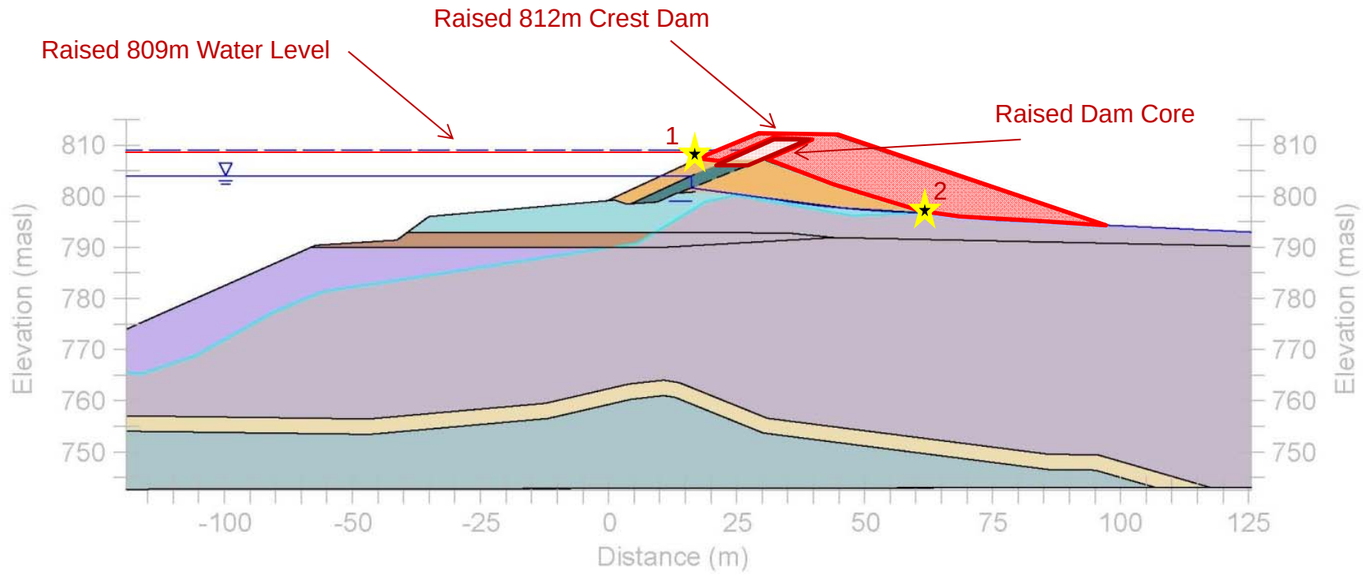
807 m crest dam with a tailings beach developed up to 805m.



Shear strength reduction / FOS = 1.5

Total displacement shown on plots to highlight failure mechanism.

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Deformation Analysis		
		Tailings Beach Scenario Shear Strength Reduction		
Job No: 1CM002.018 Filename: Dam_Deformation1CM002.018.ppt	Minto Mine	Date: May 2014	Approved: JBK	Figure: **15**



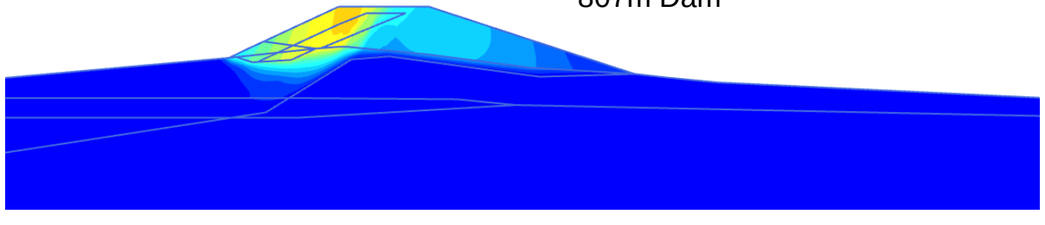
Notes

1 ★ The displacements at this point for the 812m case are expected to be very similar as to the 807m dam crest for the Pit Lake scenario (as the load on this point it not substantially increasing). The largest displacements are still expected on the upstream somewhere between this point and the crest of the 812m dam.

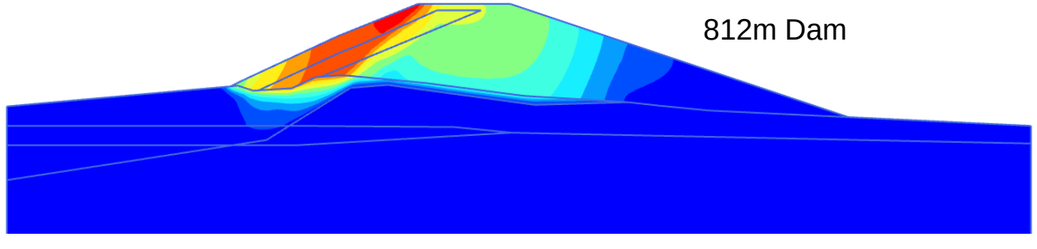
2 ★ For the Tailings Beach scenario slightly large settlements are expected at the downstream. As most of the foundation is frozen, the increase in consolidation settlement for the 812m dam is expected to be minor for the Tailings Beach scenario.

Model Check (both models run without core; i.e. only waste rock for dam fill in model)

807m Dam



812m Dam



For the Pit Lake scenario the largest increase in displacement are expected around the core, with minimal changes in the foundation settlements. Overall changes only on the order of centimeters are expected.

Legend

- Waste Rock
- Silt (unfrozen)
- Frozen Soil
- Gravelly Silt Core
- Sand & Gravel (unfrozen)
- Weathered Bedrock
- Clay (unfrozen)
- Bedrock

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Job No: 1CM002.018
Filename: Fig_DamThermal_11x17land.pptx

capstone MINTO MINE
OPERATED BY MINTO EXPLORATIONS LTD.

Minto Mine

Main Dam Deformation Analysis

Comparison of 807 and 821m Dams - Deformation Section

Date:	Approved:	Figure:
May 2014	JBK	16

Attachment 1

Advanced Finite Element Constitutive Parameters

This document provides some background and the general initial assumptions that were utilized to generate the initial advanced material properties, primarily for the Hardening Soil Model with Small-Strain Stiffness (HS-Small) small material models. These values were then adjusted based on: guidance outlined in the Plaxis 2D reference manuals (2014a, b, c, d), element tests performed in the Plaxis program, and engineering judgement. The values final advanced constitutive parameters utilized are outlined in Tables 2 to 10 in the deformation memo text.

1 Stiffness

1.1 Small strain shear stiffness

Small strain shear modulus is estimated using (EPRI 1990)

$$G = G_0^{ref} \cdot \left(\frac{p}{p_{ref}} \right)^{n_e}$$

where n_e = 0.5 is adopted for all soils (EPRI 1990); and

$$G_0^{ref} = 840 \cdot \frac{\left(2.17 - e\right)^2}{1+e} p_{ref}$$

for clean sands; and

$$G_{ref} = 450 \cdot \frac{\left(2.97 - e\right)^2}{1+e} p_{ref}$$

for sandy silts and clays. The expression allows for the direct determination of small-strain stiffness parameters for HS-Small constitutive models. Note that in the equations above *e* is the void ratio.

1.2 Elastic unload-reload stiffness

The unload/reload mean stiffness E_{ur}^{ref} is computed using expressions of theory of elasticity. For a shear strain $\gamma=10^{-3}$, for $\gamma_{0.7}=10^{-4}$ and assuming a Poisson's ratio v = 0.2 the folllowing was used (Shantos & Correia 2001):

$$G_{ur}^{ref} = \frac{G_0^{ref}}{1 + 0.385 \frac{10^{-3}}{10^{-4}}} \wedge E_{ur}^{ref} = G_{ur}^{ref} \cdot 2(1+\nu) \rightarrow E_{ur}^{ref} = 0.41 \cdot (1+\nu) \cdot G_0^{ref}$$

For a Poisson's ratio v=0.2, the unloading/reloading stiffness is simplified as being approximately computed as

$$E_{ur}^{ref} = 0.5 \cdot G_0^{ref}$$

1.3 Secant stiffness

For the HS-Small constitutive model, the secant elastoplastic stiffness at 50% failure load and 100kPa confining pressure was estimated as a first pass, as

$$E_{50}^{ref} = 0.33 \cdot E_{ur}^{ref}$$

And for more granular materials as

$$E_{50}^{ref} = 0.50 \cdot E_{ur}^{ref}$$

These values were then adjusted based on the other input parameters and on the base case guidelines outlined in Plaxis (2014b, c).

1.4 Oedometric stiffness

For granular materials, where oedometric compression is not a primary source of plastic deformations, the oedometric stiffness can be estimated (based on theory presented in Plaxis 2014c) as:

$$E_{oed}^{ref} = \frac{\sqrt{K_0^{nc}}}{3} E_{ur}^{ref}$$

where K_0^{nc} is the vertical to horizontal stress ratio for normally consolidated material

$$K_0^{nc} = 1 - \sin\left(\phi_{cv}\right)$$

Therefore,

$$E_{oed}^{ref} = 0.25 \cdot E_{ur}^{ref}$$

For fine soils, the oedometric stiffness is calculated using

$$E_{oed}^{ref} = \frac{2.3\left(1+e\right)}{C_c} p_{ref}$$

where C_c is the compression index from 1-D compression tests.

2 References

[EPRI] Electric Power Research Institute, 1990. Manual on Estimating Soil Properties for Foundation Design. Product ID: EL-6800.

Plaxis bv. 2014a. Plaxis 2D Anniversary Edition, Finite Element Software Package. Delft, Netherlands.

Plaxis bv. 2014b. Plaxis 2D Anniversary Edition, Reference Manual. Delft, Netherlands.

Plaxis bv. 2014c. Plaxis 2D Anniversary Edition, Material Models Manual. Delft, Netherlands.

Plaxis bv. 2014d. Plaxis Scientific Manual. Delft, Netherlands.

Santos, J.A., Correia, A.G. 2001. Reference threshold shear strain of soil - its application to obtain a unique strain-dependent shear modulus curve for soil. In proceedings for the 15th International Conference on Soil Mechanics and Geotechnical Engineering. Istanbul, Turkey, volume 1, 267-270.

Appendix I: Minto Main Dam Preliminary Design – Filter Design

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www.srk.com

Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Emma Helmers	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 15, 2014
Subject:	Main Dam Preliminary Design: Filter Design and Core Suitability		

1 Introduction

The Minto Main Dam design is a fine grained core with an upstream liner. A schematic of a typical dam section is provided in Figure 1. Two filters are required between the low permeability core and the outer waste rock shell on the downstream side of the dam. The first filter located immediately downstream of the core will be referred to as the fine filter (or Transition 1), and the second filter will be referred to as the coarse filter (or Transition 2).This memo summarizes these filter designs, as well as considers whether locally available fine-grained material would be suitable for construction of the dam core.

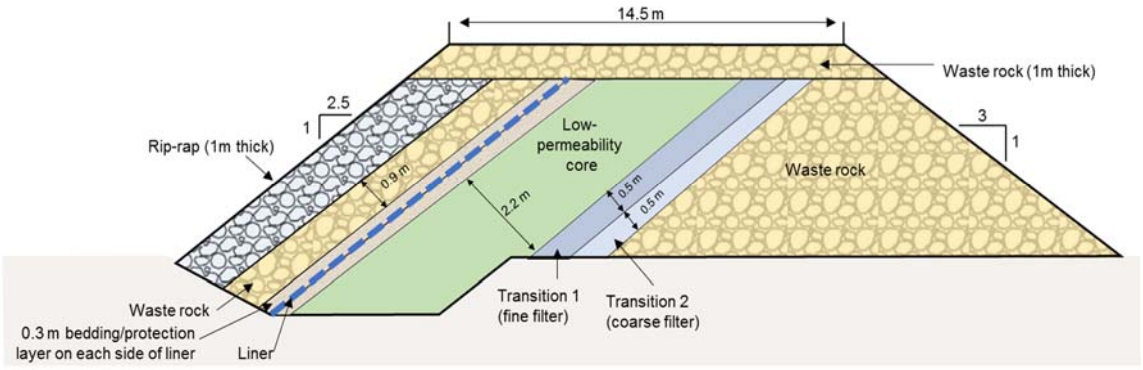

Figure 1: Main Dam Typical Section

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2 Material Suitability and Availability

Locally available overburden materials were evaluated, using primarily particle size distribution curves to assess their suitability for use as both core and filter materials.

2.1 Core Material

A test pit investigation program was completed by Minto in 2012 to characterize overburden materials across the site and to evaluate the acceptability of the materials for use as closure cover materials (SRK 2013a). Test pits were located in the vicinity of Area 2, Main Waste Dump (MWD), Reclamation Overburden Dump (ROD), Ice Rich Overburden Dump (IROD) and Minto North. Samples collected during the program were analyzed in a geotechnical laboratory testing particle size distribution (PSD), Atterberg limits, specific gravity, and Standard Proctor tests. Triaxial hydraulic conductivity and soil water characteristic curve testing were performed on select samples.

Three criteria were used to analyze the suitability of these materials for construction of the dam core: (1) the core material must be fine-grained, (2) have a low hydraulic conductivity (approximately $1x10^{-7}$ m/s), and (3) the material must be accessible.

Although the MWD samples appeared to be, on average, the finest material, SRK understands that this material is inaccessible at the current time. Samples collected from the ROD also contain significant fines and may be sufficient for use as dam core material. Its hydraulic conductivity was estimated based on its grain size distribution to be approximately $2x10^{-7}$ m/s using the United States Department of Agriculture program Rosetta (Skaggs 1999). As this estimate likely contains significant errors, further characterization of the ROD material is recommended to improve the material property estimates for filter design and seepage estimation.

Figure 2 presents the particle size distribution curves of the ROD samples as well as the overall particle size distribution envelope for this material.

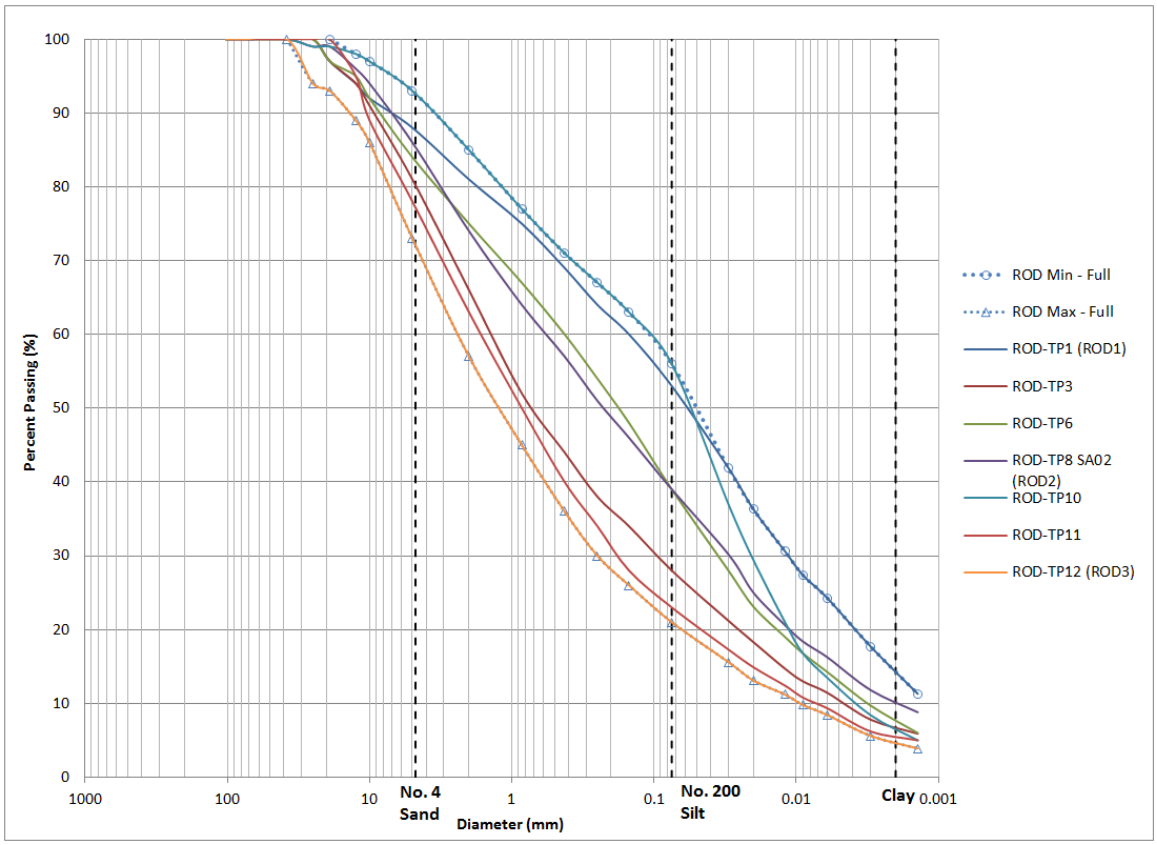

Figure 2: Reclamation Overburden Dump (ROD) Particle Size Distribution Envelope

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2.2 Filter Material

Soil filters are required between the core and run-of-mine rock fill zones to support the fine-grained core material and to prevent it from being eroded by seepage, as well as to allow water to drain and prevent build-up of pore water pressures.

The available data from the SRK (2013a) field program suggest that those tested sources are quite variable, and predominantly have high fines content. Other geotechnical field investigations, such as the Ridgetop field investigation (SRK 2013b), have shown that there are areas where coarser grained soils can be found. Geomorphologically it is expected that some of the higher elevation areas may contain higher quantities of colluvium, or weathered bedrock, which would have a higher sand content which in turn would be more conducive towards constructing filters.

Since the filters will require strict gradation envelopes, an additional more focused on-site borrow characterization program will be required to better define availability and suitability of locally available materials as filter media, or alternately off-site sources must be identified.

3 Filter Design Criteria

3.1 Approach

Filter design criteria are generally empirical and based on experimental work. For this assessment, the design criteria were selected from Best Practices published by the US Federal Emergency Management Agency (FEMA 2011). This document provides a step-by-step procedure based on extensive experience of the US Bureau of Reclamation (USBR), the US Army Corps of Engineers (USACE) and the US Natural Resource Conservation Service (NRCS) (FEMA 2011).

Prior to applying the filter design criteria, the core material grain size distribution curves were computationally re-graded to correct for broadly graded soils as recommended in FEMA (2011). The re-grading procedure adjusted the grain-size distribution curves by excluding the soil particles that are larger than the D_{50} of the material. This correction is intended to ensure particle retention of the finer fraction of the core material.

The four filter design criteria used in this assessment are:

- Filtering criteria: to prevent continued migration of fines from the low permeability core;
- Permeability criteria: to prevent excessive build-up of hydraulic head within the dam;
- Gradation criteria: to prevent internal erosion of the filter material; and
- Maximum particle size criteria: to aid in constructability.

The following sections describe these filter criteria in greater detail.

3.1.1 Filtering Criteria

The filtering criteria are intended to prevent migration of the core material in the downstream direction. The maximum D_{15} (the particle diameter at which 15% of the material is finer) of the filter material is based on the D_{85} (the particle diameter at which 85% of the material is finer) of the core material. In this memo, the subscript 'f' will denote particle sizes of the filter material (ex. D_{15f}) and the subscript 'b' will denote the particle size of the base material, or the material to be retained (ex. D_{85b}). The equation used to calculate D_{15f} depends on the amount of material passing the Number 200 sieve (0.075 mm, or the maximum particle diameter of silt) in the base material. **Error! Reference source not found.** presents the base soil categories.

Table 1: Filtering Criteria Base Soil Categories

Base Soil Category	% Finer than No. 200 Sieve	Base Soil Description
1	>85	Fine silt and clays
2	40-85	Sands, silts, clays, and silty sands
3	15-39	Silty and clayey sands and gravels
4	<15	Sands and gravels

http://www.fema.gov/media-library-data/20130726-1813-25045-0568/fema_filter_manual_links.pdf

Table 2 presents the equations (and conditions) used to calculate D15f based on the base soil category.

Table 2: Filtering Criteria Maximum D_{15f} Based on Base Soil Category

Base Soil Category	Filtering Criteria – Maximum D_{15f}
1	The maximum D_{15f} should be ≤ 9 x D_{85b}, but not less than 0.2 mm, unless oils are dispersive. Dispersive soils require a maximum D_{15f} that is ≤ 6.5 x D_{85b} size, but not less than 0.2 mm
2	The maximum D_{15f} should be ≤ 0.7 mm unless soil is dispersive, in which case the maximum d_{15f} should be < 0.5 mm
3	For non-dispersive soils, the maximum D_{15f} should be $\leq \left(\frac{40-A}{25}\right) * (4 * D_{85b} - 0.7mm) + 0.7mm$, Where A is the % passing the No. 200 sieve. When 4 x D_{85b} is less than 0.7 mm, use 0.7 mm. For dispersive soils, use 0.5 mm as opposed to 0.7 mm
4	The maximum d_{15f} should be ≤ 4 x D_{85b}

http://www.fema.gov/media-library-data/20130726-1813-25045-0568/fema_filter_manual_links.pdf

3.1.2 Permeability Criteria

Permeability criteria are intended to ensure that the filter is more permeable than the soil being retained to prevent a build-up of pore water pressure that could lead to internal erosion and instability. The following criterion was used to calculate the minimum allowable D_{15f} values for the fine and coarse filters:

Minimum D_{15f} ≥ 3 x maximum D_{15b} (FEMA 2011)

3.1.3 Gradation Criteria

The filter material must be suitably well graded to prevent particle migration of fines leading to loss of material or clogging of the filter. The following gradation criterion was used in the design of the filter:

2 ≤ C_u ≤ 6 (FEMA 2011)

where C_u is the coefficient of uniformity and is calculated using the formula: $C_u = \frac{D_{60}}{D_{10}}$

3.1.4 Maximum Particle Size

FEMA (2011) recommends a maximum filter particle size (D_{100f}) of 51 mm and that the filter grain size curve should be parallel to the base material curve wherever possible The 51 mm particle size is recommended for constructability purposes as larger diameter particles can limit the effectiveness of compaction. In this assessment, the maximum grain size of the filter material was generally limited by the recommendation for the filter soil envelope curve to be parallel to the base material curve.

4 Results

4.1 Core Material

Figure 3 presents estimated particle size distribution envelopes of the core material based on laboratory test data from the ROD (SRK 2013a). The series labelled "ROD Min – Full' and "ROD Max – Full" represent the minimum and maximum distribution curves of the ROD samples and are the same as those presented in Figure 1. The series labelled "ROD Min – Fine" and "ROD Max- Fine" are the adjusted minimum and maximum particle size distribution envelopes following removal of all particles with a diameter larger than the D_{50} of the full particle size distribution curve. These adjusted curves were subsequently used to determine the filter requirements. Table 3 describes the maximum and minimum diameters that were used in the design of the fine filter.

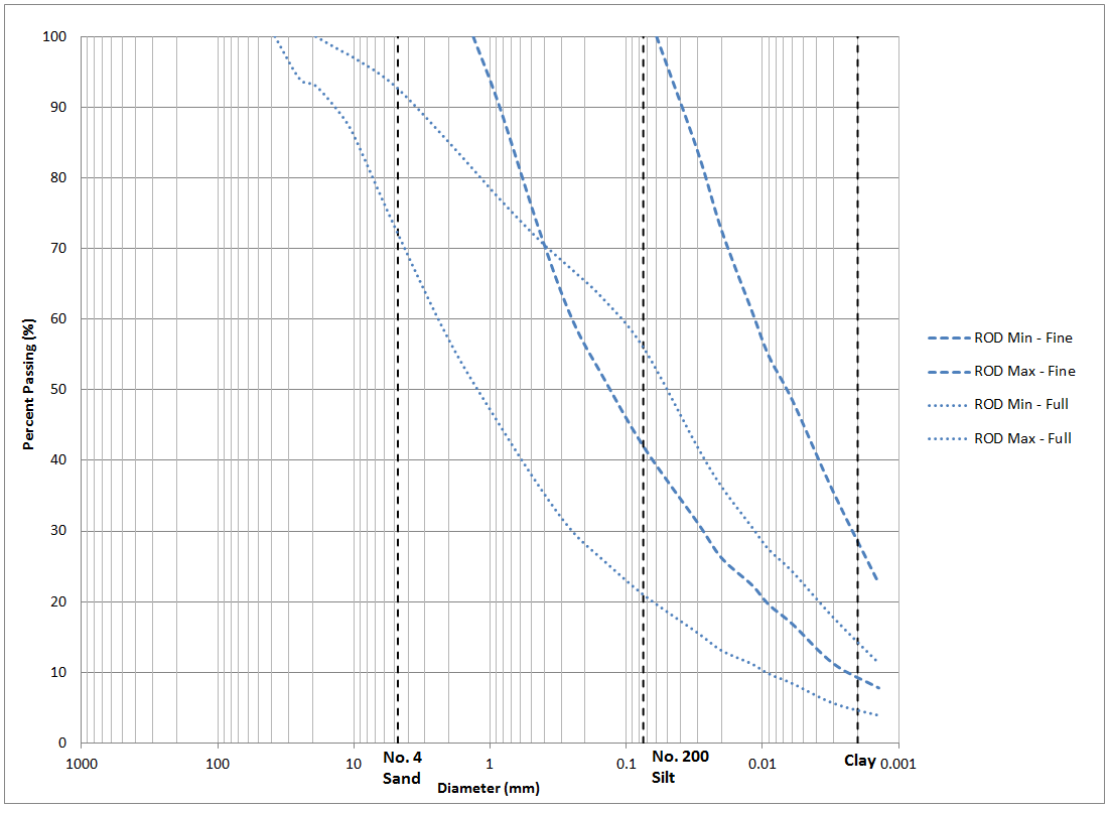

Figure 3: ROD Re-graded Particle Size Distribution Envelope

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Table 3: Core Material Particle Sizes Used in Design

Diameter (% passing)	Minimum Diameter (mm)	Maximum Diameter (mm)
D_{15}	0.001	0.043
D_{85}	0.032	0.730
D_{100}	0.060	1.300

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4.2 Fine Filter

Figure 4 presents the particle size distribution envelope for the fine filter, and Table 4 lists the maximum and minimum envelope diameters. The resulting fine filter will consist mostly of sand and silt.

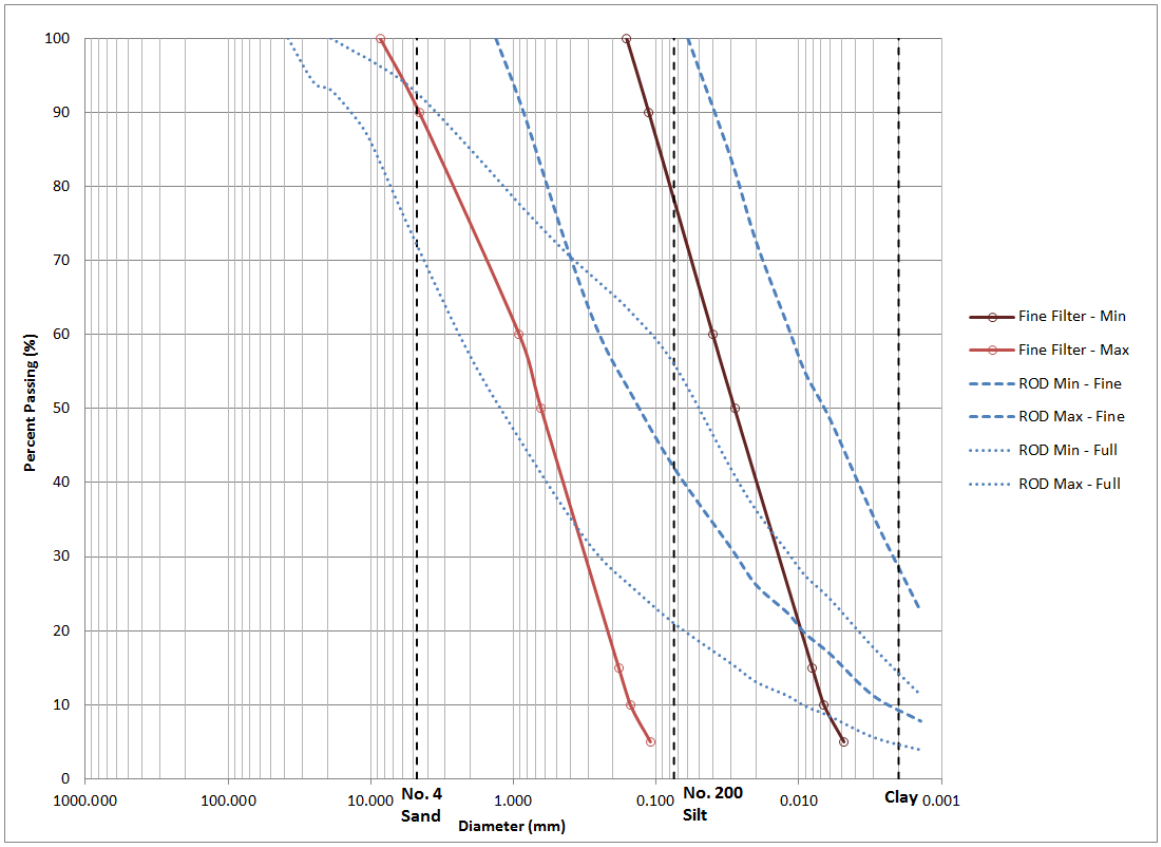

Figure 4: Fine Filter Particle Size Distribution Requirements

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Table 4: Fine Filter Particle Size Requirements

Diameter (% passing)	Minimum Diameter (mm)	Maximum Diameter (mm)
D_{15}	0.008	0.18
D_{85}	0.100	4.00
D_{100}	0.160	8.40

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4.3 Coarse Filter

Figure 5 presents the particle size distribution envelope for the coarse filter. Table 5 lists the maximum and minimum envelope diameters. The resulting coarse filter will be comprised mostly of sand, with some gravel and some silt.

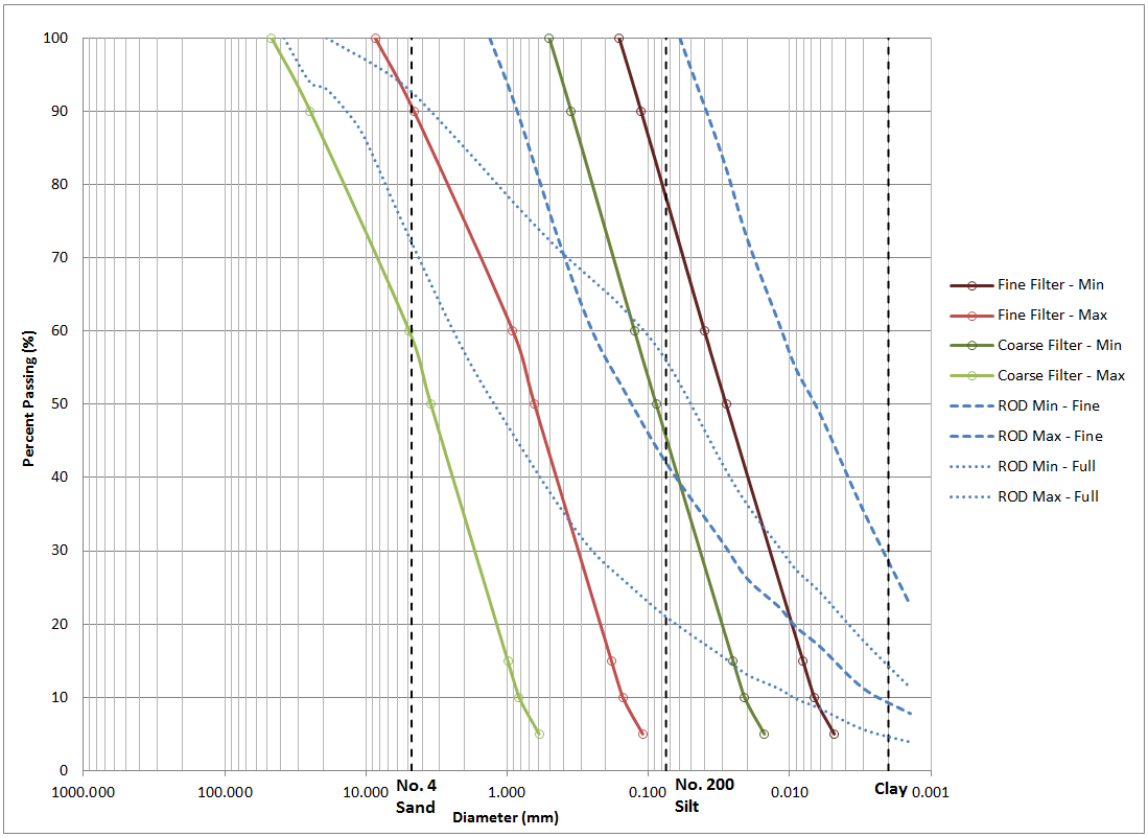

Figure 5: Coarse Filter Design

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Table 5: Particle Sizes Used in Design, Coarse Filter Envelope

Diameter (% passing)	Minimum Diameter (mm)	Maximum Diameter (mm)
D_{15}	0.025	0.99
D_{85}	0.300	17.00
D_{100}	0.500	47.00

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Appendix J: Review of Geotechnical Strength Properties at Minto Mine

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Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Peter Mikes	**Project No:**	1CM002.018.110
Cc:	Cam Scott, Maritz Rykaart	**Date:**	July 17, 2014
Subject:	Review of geotechnical strength properties at Minto Mine		

1 Introduction

This memo presents a review of all the geotechnical soil strength testing completed on Minto Mine projects and presents recommended properties for use in subsequent stability analyses.

This review was undertaken due to confusion regarding different naming conventions to describe the same material, as well as different material properties used to represent the same material depending on the purpose of the project. For example, a previous waste rock design report may have adopted a conservative friction angle to calculate a conservative factor of safety, and this value was later adopted as a typical friction angle for the material.

This memo is intended to provide a compilation of previous direct shear and triaxial test results and derive 'typical' values for various materials based on the results as well as engineering judgement. This memo is not intended to provide definitive geotechnical design parameters, but is to be used as a general guideline. Where more rigorous geotechnical design parameters are required, site specific measurements must be made, or alternately appropriate sensitivity analysis and design mitigations must be included to compensate for any uncertainty associated with geotechnical parameters.

2 Soil Strength Parameters

2.1 Review of Available Strength Test Data

A review of available information found a total of nineteen direct shear test results and five triaxial test results of overburden soils. Of these tests, the laboratory raw data was not available for seven direct shear and one triaxial test with only the strength parameters stated in a report. As a result, these tests could not be substantiated by SRK. A compilation of the strength test results is provided in Attachment 1. The attachment includes four tables, whose contents are described in Table 1.

Table 1: Attachment 1 Strength Test Result Compilation

Table	Title	Description
A1-1	Soil Property Data	Provides the sample location, description and other known material property for each strength test sample, as well as the document source of the test data. Results provided without the laboratory test sheets are noted in the comments column.
A1-2	Direct Shear Test Results	Provides the normal, peak and residual stress for each test and the apparent friction angle and cohesions. For each sample, three sets of strength parameters are listed: 1. The first set is the raw data best-fit linear trend-line corresponding to the cohesion and friction angles provided by the laboratory. 2. The second set adjusts the friction angle by forcing the cohesion to zero. 3. The third set is the recommended adjusted strength parameters for consideration in stability analyses. In all cases, the cohesion has been set to zero, and the recommended friction angle is between the set 1 and 2 friction values. Figures 1 to 3 provide graphical results of the direct shear tests sorted by soil type. The plotted trend-lines are the 'best-fit' linear lines corresponding the reported lab results (Set 1).
A1-3	Triaxial Test Results	Provides the consolidated undrained test results for each test (3 points). The peak friction angle was calculated as the average of the three test points. The residual friction angle was obtained from the p-q graph on the test results and adjusted to residual values through the formula: $\phi = \sin\text{-}1(\tan \alpha)$ where ϕ is the residual friction angle, and α is the critical friction angle.
A1-4	Soil Properties by Soil Type	Provides the recommended peak and residual properties sorted by the following soil types: 1. Coarse grained soils (sands and gravels) 2. Silts 3. Clays 4. South Wall Failure Slide Debris (clays) Reconstituted samples are noted where applicable. Generally, the reconstituted samples were compacted to a specific standard proctor and as a result, the friction angles are lower than 'undisturbed' samples of similar soil types.

Figures 1 to 3 provide plotted direct shear test results for the coarse grained soils, silts, and clays, respectively. The figures include a summary of the strength parameters obtained directly from the laboratory test sheets that represent the best-fit linear trend line (no engineering judgement). In two instances, plotting of the trend line resulted in negative cohesion values (TP97-02 and SWF-1C-003).

No undrained shear test data was found on any overburden materials on site. Previous reports have used undrained shear strengths of 50 kPa (EBA (2011a) and 60 kPa (EBA 2011b) to represent the ice rich permafrost clay. No basis to the 50 kPa value could be found. The 60 kPa value is stated by EBA to be based on back analysis results and research at the Norwegian

Geotechnical Institute on Svea clay from Svalbard off the northeast tip of Greenland (Berggren, 1983).

2.2 Suggested Soil Strength Parameters

Based on the strength test data review, suggested strength parameters were selected for each major material type. Table 2 provides a summary of the suggested properties for use in stability analyses.

No laboratory test data was found of waste rock and tailings materials. The basis of their suggested strength properties are provided in the Table 2 notes below. Properties of the 'shear zone' used in the SRK (2013) Dry Stack Tailings Storage Facility (DSTSF) and at the South Wall Failure area have been excluded from this table but are discussed in Section 3.

Table 2: Summary of Material Strength Properties for Stability Analysis

		Peak Strength		Residual Strength		
Material	**Bulk Density (t/m^3)**	**Cohesion (kPa)**	**Friction Angle (°)**	**Cohesion (kPa)**	**Friction Angle (°)**	**Comments**
Clean coarse grained soils	1.9	0	40	0	38	See note 1
Silty sands, residuum, weathered bedrock	1.9	0	35	0	32	See note 1
Silts	1.8	0	30	0	26	See note 1
Clays, including high plasticity, ice rich clays	1.8	0	23	0	19	See note 1
South Wall Failure Slide Debris	1.8	-	-	0	10	See note 1
Waste Rock	2.1	0	37	0	37	See note 2
DSTSF compacted tailings	1.9	0	35	0	32	See note 3
Conventional tailings	1.3	0	8	0	8	See note 4

Source: Minto Material Properties.xlsx

Note(s):

(1) Properties based on test results provided in Table 2.

(2) Waste rock density based on a specific gravity of 2.7 and a 1.3 swell factor. The swell factor is based on the Waste Rock and Overburden Management Plan (Minto 2013a). The waste rock friction angle is based on the angle of repose measured at site and is considered to be a lower bound estimate.

(3) Dry stack tailings density is based on the typical density measured in the EBA (2010) memo "Review of compaction and moisture content at the Dry Stack Tailings Storage Facility". The peak friction angle is based on a direct shear test noted in Table 6 of the OMS manual (EBA 2011c), however the test data is unavailable.

(4) The conventional tailings density was obtained from the Phase V/VI Tailings Management Plan (Minto 2013b). The friction angle is based on the gradation distribution and literature values from Shamsai *et al*. (2007).

2.3 Deep-Seated Shear Zone Material Properties

Deep-seated foundation movements have been observed at the DSTSF and at the south wall of the Main Pit. The movements have occurred at depth; generally 5 m to 10 m above the bedrock contact and are associated with ice-rich high plastic clays. The shear zones are in areas of warm permafrost with temperatures between -0.6 and -0.1 °C.

Various back analyses of the movements at the DSTSF and south wall of the Main Pit have been completed to estimate the shear zone properties. The results of the calculations are described in Table 3. The results show a large degree of variance in the calculated properties and as a result, no single set of parameters is recommended and a sensitivity analysis should be completed that use a range of parameters.

Table 3: Summary of Back-Calculated Shear Zone Properties

Reference	Area	Results
SRK (2009)	South Wall Main Pit	SRK (2009) back-calculated the failure of the south wall of the Main Pit and found that shear strengths of approximately 10 degrees with zero cohesion would have been required for displacements to occur without the influence of an external force such as pore water pressure. The analysis was completed using a limit equilibrium model.
Norwest (2014a)	South Wall Main Pit	Norwest (2014a) back-calculated the failure of the south wall of the Main Pit and estimated the undrained shear strength of the shear zone to be 73 kPa at limit equilibrium conditions. Under drained conditions, with friction angles ranging between 10 to 14 degrees, pore pressure ratios (ratio of pore to overburden pressure) of 0.5 to 0.7 was required to result in FOS of 1. The analyses were completed using limit equilibrium methods.
Norwest (2014b)	DSTSF	Norwest (2014b) back-calculated the DSTSF failures resulting in residual friction angles ranging between 10 to 14 degrees with pore pressures equivalent to pore pressure ratio values of 0.6 to 0.7. Undrained shear strengths were estimated to range between 60 and 80 kPa.
SRK (2014)	DSTSF	SRK (2014) back-calculated the DSTSF mobilized shear strengths of the ice-rich clay to range between 10 and 16 kPa. This analysis was completed using a wedge analysis. Appendix A of SRK (2014) uses a limit equilibrium approach to the same sections as used in the wedge analysis and resulted in undrained shear strengths ranging from 35 to 60 kPa [1]

Note(s):

(1) These strengths were calculated at Sections A and B of the report. Section C is not included in this report as movement rates are lower at this section compared to A and B and analysis of its movement is suspected to require a 3D analysis.

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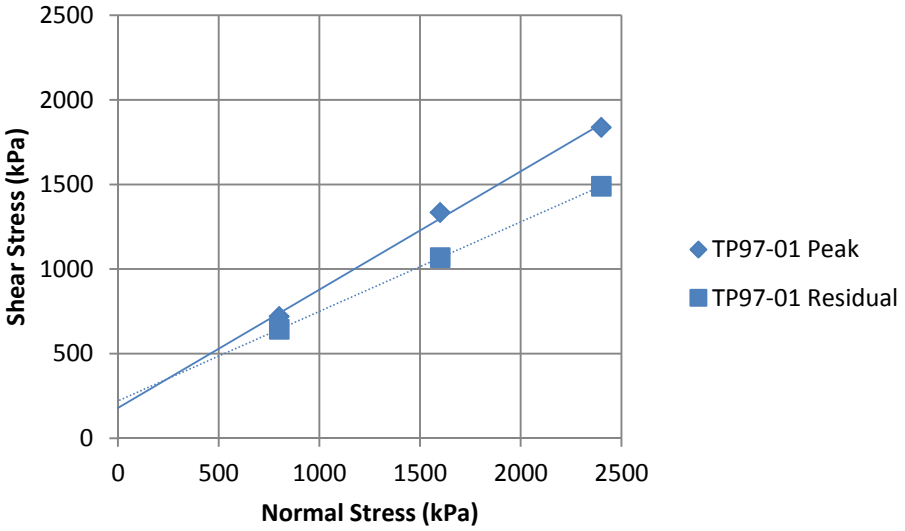
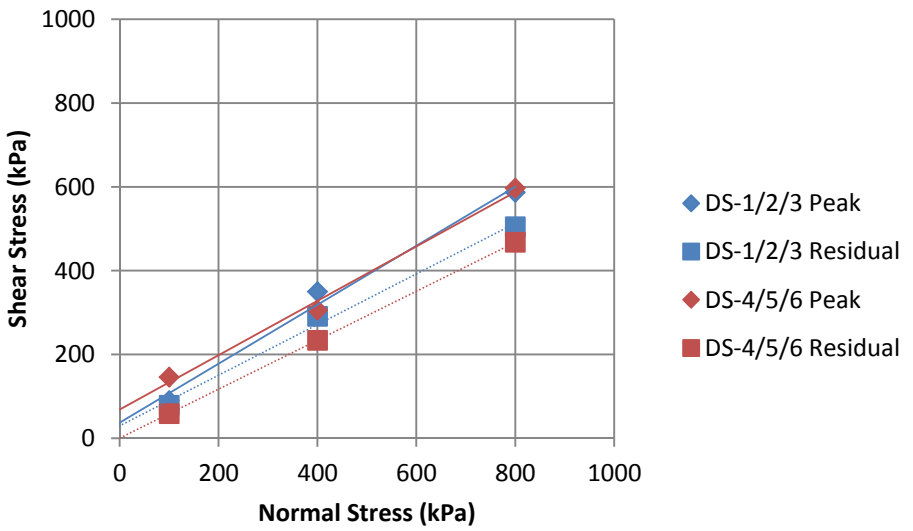
Figures



a) EBA (1998) Results



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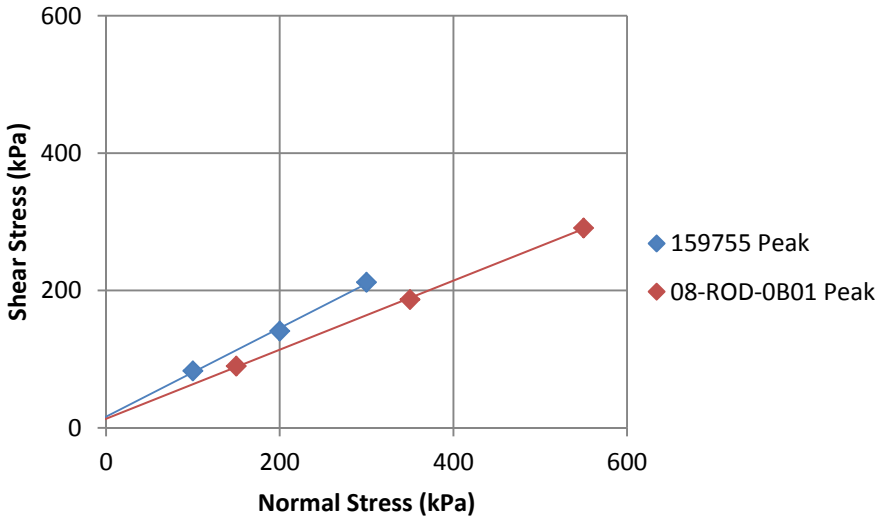


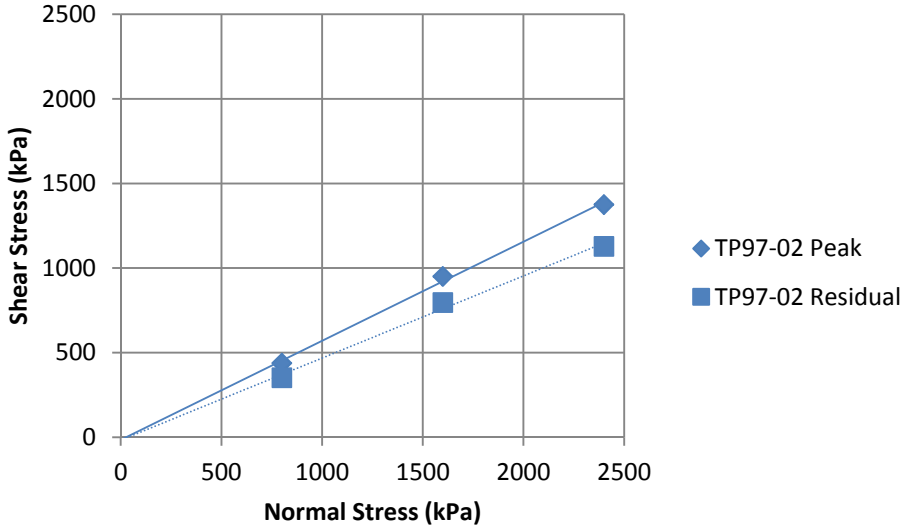
c) SRK (2013) and EBA (2008) Results

			Peak		Residual	
Sample ID	USCS	Description	Apparent Cohesion (kPa)	Friction Angle (°)	Apparent Cohesion (kPa)	Friction Angle (°)
TP97-01	SM	Silty sand colluvium	180.0	35.0	220.0	28.0
DS-1/2/3	SM	Silty sand with gravel	37.4	35.1	31.2	29.7
DS-4/5/6	SM	Silty sand with clay and gravel	68.5	33.0	0.5	30.3
159755	SM	Silty sand	16.0	32.8	-	-
08-ROD-0B01	GM	Silty gravel with clay and sand	13.5	26.7	-	-

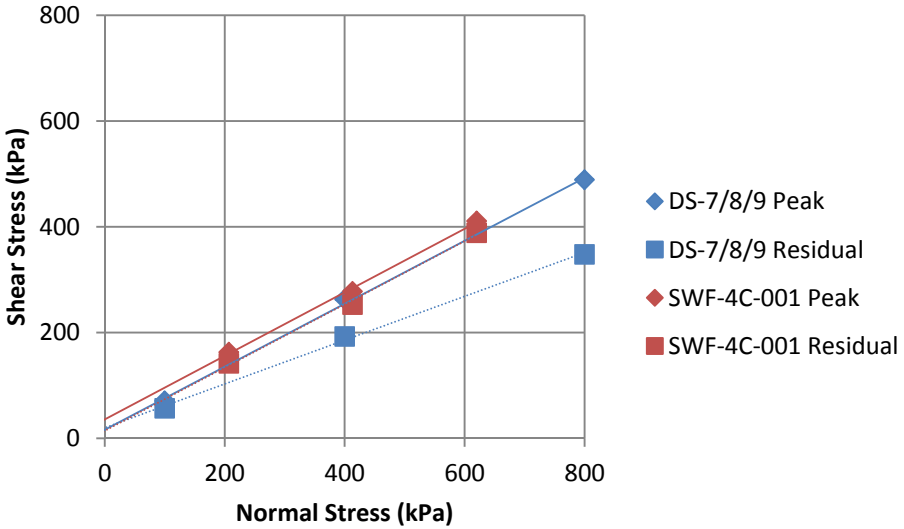
Notes:

1. Strength parameters listed obtained from graph linear trendlines of the raw laboratory data .
2. Sample 159755 was remolded to a initial bulk density of 1.6 t/m^3 and is likely less dense than actual conditions.
3. Sample 08-ROD-0B01 was compacted to 88% SP prior to testing.

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Strength Test Parameter Review
Job No: 1CM002.018.110 Filename: MintoMaterialProperties_Figures.pptx	Minto Mine	**Direct Shear Test Results - Coarse Grained Soils**
		Date: May 2014 / Approved: PM / Figure: 1



a) EBA (1998) Results – Main Waste Dump Area



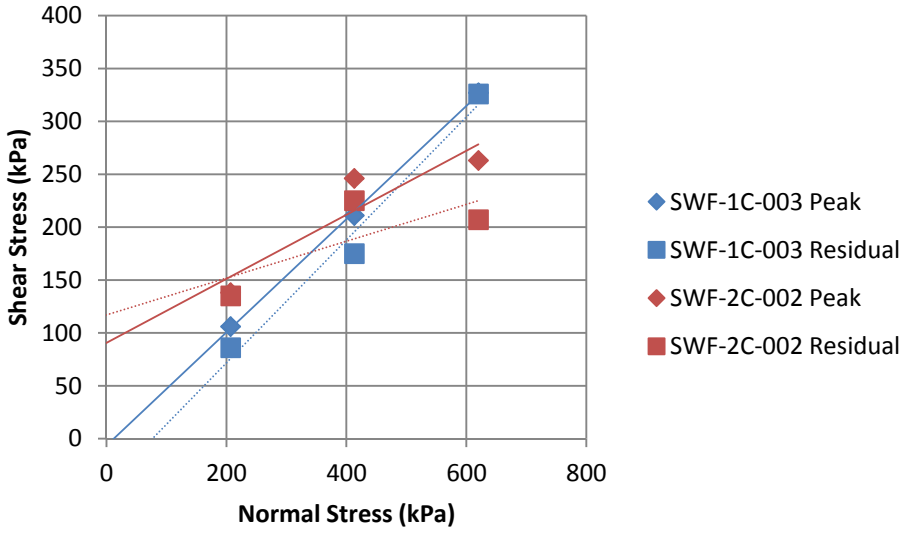
b) EBA/SRK (2009) Results – South Wall Failure Area

			Peak		Residual	
Sample ID	USCS	Description	Apparent Cohesion (kPa)	Friction Angle (°)	Apparent Cohesion (kPa)	Friction Angle (°)
TP97-02	ML	Clayey silt colluvium	-16.0	30.0	-17.0	26.0
DS-7/8/9	ML	Clayey silt with sand	16.7	30.7	19.9	22.5
SWF-4C-001	ML	Silt, low plasticity	36.2	30.9	14.1	30.9

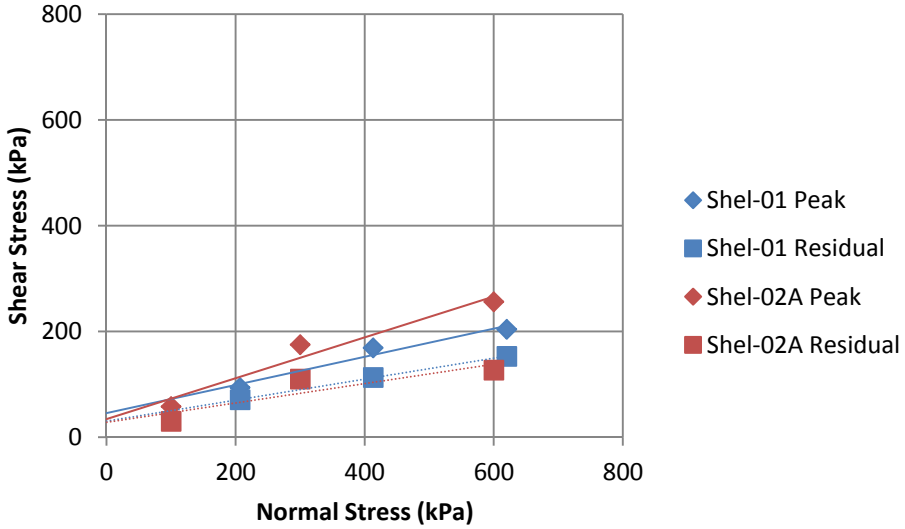
Notes:

1. Strength parameters listed obtained from graph linear trendlines of the raw laboratory data.

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Strength Test Parameter Review
Job No: 1CM002.018.110 Filename: MintoMaterialProperties_Figures.pptx	Minto Mine	**Direct Shear Test Results - Silts**
		Date: May 2014 / Approved: PM / Figure: **2**



a) SRK and EBA (2009) Results – South Wall Failure Area



b) SRK and EBA (2009) Results – South Wall Failure Slide debris

			Peak		Residual	
Sample ID	USCS	Description	Apparent Cohesion (kPa)	Friction Angle (°)	Apparent Cohesion (kPa)	Friction Angle (°)
SWF-1C-003	CI	Clay, medium plasticity	-6.5	28.1	-44.6	30.2
SWF-2C-002	CL	Clay, low plasticity	90.0	16.9	117.0	9.9
Shel-01	CH	Clay, high plasticity	45.4	14.9	31.1	11.1
SHEL-02A	CL	Silty clay with low plasticity	34.3	21.1	27.9	10.4

Notes:

1. Strength parameters listed obtained from graph linear trendlines of the raw laboratory data.

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Strength Test Parameter Review
		Direct Shear Test Results - Clays
Job No: 1CM002.018.110 Filename: MintoMaterialProperties_Figures.pptx	Minto Mine	Date: May 2014 / Approved: PM / Figure: 3

Attachment 1: Compilation of Strength Test Results

Table A1.1: Soil Property Data

				Depth		Descriptions					Atterberg Limits			Moisture	Before testing			Grain Size Distribution				
Sample ID	Source	Area	Borehole ID	Top (m)	Bottom (m)	UCSC	Soil Description	Field Description	Ice Description	Logged by	Plastic Limit	Liquid Limit	Plasticity Index	Content (%)	Bulk Density (Mg/m^3) [1]	Dry Density (Mg/m^3) [1]	Test Type	% Gravel	% Sand	% Silt	% Clay	Comments
Clay-1	Golder (1974)	Main Pit	-	-	-	SM	Silty Sand with gravel	Sandy silt with clay	n/a	Golders	16	27	11	17.2	-	-	DS	18	36	33	13	Samples collected from the 'suspected solifluction areas) No lab test data available.
Clay-2	Golder (1974)	Main Pit	-	-	-	SM	Silty Sand with gravel	Sandy silt with clay	n/a	Golders	30	41.9	11.9	31.4	-	-	DS	28	38	20	14	Samples collected from the 'suspected solifluction areas) No lab test data available.
RS-1	Golder (1974)	Main Pit	-	-	-	S	Coarse sand	Residual Soil, coarse sand	n/a	Golders				4.2	-	-	DS		100			No lab test data available.
RS-2	Golder (1974)	Main Pit	-	-	-	S	Coarse sand	Residual Soil, coarse sand	n/a	Golders				4.4	-	-	DS		100			No lab test data available.
RS-3	Golder (1974)	Main Pit	-	-	-	S	Coarse sand	Residual Soil, coarse sand	n/a	Golders				3.2	-	-	DS		100			No lab test data available.
RS-4	Golder (1974)	Main Pit	-	-	-	S	Coarse sand	Residual Soil, coarse sand	n/a	Golders				3.2	-	-	DS		100			No lab test data available.
Clay	EBA (Dec 1995)	Access road at the E end	-	-	-	CI	Clay, medium plasticity	n/a	n/a	EBA	19	53	34	19.3	1.9	1.6	Triaxial	0	20	22	58	From Geotech Design Tailings-Water Dam Report, no lab test data available
2187	EBA (Dec 1995)	Access road at the east end of lease boundary	-	-	-	SM	Silty Sand with gravel	Residuum	n/a	EBA				10	2	1.8	DS	14	72.4	10.2	3.4	No lab test data available.
TP97-01	EBA (1998)	Main Dump	TP97-01	0.3	0.6	SM	Sand, Silty, trace of clay and gravel	Silty sand colluvium	n/a	EBA				10.3	2.3	2.1	DS	20.7	40.4	31.7	20.7	
TP97-02	EBA (1998)	Main Dump	TP97-02	1	1.3	ML	Clay, silty, sandy, low plastic	Silty clay colluvium	n/a	EBA				20	2.1	1.9	DS	17.7	27.7	32.9	21.7	
DS-7	See note 2	Main Pit	SWF09-3R	36	36.3	ML	Clayey silt with sand	n/a	n/a	EBA	17	34	17	31.4	2	1.5	DS	8	26	36	30	
DS-1	See note 2	Main Pit	SWF09-4R	35.4	35.7	SM	Silty Sand with gravel	n/a	n/a	EBA	20	25	5	16	2.2	1.9	DS	12	38	43	7	
DS-4	See note 2	Main Pit	SWF09-4R	39.4	39.8	SM	Silty Sand with clay and gravel, medium plasticity	n/a	n/a	EBA	20	37	17	19	2.2	1.8	DS	15	39	32	14	
SHEL-02A/B	See note 2	Main Pit	-	-	-	CL	Silty clay with low plasticity	Slide debris	n/a	EBA	21	46	25	35.4	1.8	1.4	DS	11	14	27	48	Shelby tube sample pushed into the Main Pit overburden
SHEL-01	See note 2	Main Pit	-	-	-	CH	Clay, high plasticity	Slide debris	n/a	SRK	19	73	54	42	1.9	1.3	DS	0	2	98		Shelby tube sample pushed into the Main Pit overburden
SWF-1C-003	See note 2	Main Pit	SWF09-1C	27.7	28	CI	Clay, medium plasticity	Medium firm clay	n/a	SRK	15	50	35	22	2.1	1.8	DS			74		
SWF-2C-002	See note 2	Main Pit	SWF09-2C	53.5	53.8	CL	Clay, low plasticity	Stiff clay; 40% sand, 10% clasts; poorly sorted gravelly clay.	n/a	SRK	14	39	25	17.4	2.1	1.8	DS			69		
SWF-4C-001	See note 2	Main Pit	SWF09-4C	36.3	36.6	ML	Silt, low plasticity	Firm dark grey clay	n/a	SRK	22	30	8	27	2	1.6	DS			53		
08-ROD-OB01	See note 3	Overburden dump	E. section of 796 bench	0	0	GM	Silty gravel with clay and sand	Silt, gravelly, some clay, sand - yellowish brown	n/a	EBA	-	-	-	16.6	1.9	1.6	DS	38	15	28	19	
96477 (CU-1)	SRK (2013a)	DSTSF	DSI-16	30.7	31	ML	Silt - sandy, trace gravel	Silt, few sand, trace gravel, low plasticity	Nbn	SRK	22	24	2	27.7	1.9	1.5	Triaxial	1	27	72	0	Grain size and moisture content from sample 2m lower (96478); Peak friction angle is the average of three triaxial tests
159753 (CU-1)	SRK (2013b)	Main dam	13-MPD-05	17.9	18.1	CL	Clay, low plasticity	Clay and silt, little sand, greyish brown, stiff, low moisture	Vr, 2% excess ice	SRK	20	46	26	20	2	1.6	Triaxial	2	9	50	39	Peak angle based on average of 3 triaxial tests
159756 (CU-2)	SRK (2013b)	Main Dam	13-MPD-05	24.7	24.8	SM	Silty sand	Sand, few gravel, medium dense, well graded	Unfrozen	SRK	-	-	-	12.5	2.2	2	Triaxial	7	60	26	7	
159755	SRK (2013b)	Main Dam	13-MPD-05	21.3	21.8	SM	Silty sand	Sand, little clay, few gravel	Unfrozen	SRK	-	-	-	14.8	1.6	1.4	DS	14	54	22	10	Sample was remoled to a void ratio of 0.81 (loose)
160022 (CU-3)	SRK (2013b)	Main Dam	13-MPD-06	30.8	31.1	CL	Clay, silty, some sand, gravel, low plasticity	Clay, little sand, little gravel	Vr, well bonded with no excess ice, few lenses 1-2 cm thick.	SRK	16	40	24	18.5	2	1.7	Triaxial	12	29	30	29	

Notes:

1. Bulk and dry density samples are prior to testing
2. Samples were collected and tested in 1999 as part of the geotechnical investigation of the Main Pit south wall failure conducted by EBA and SRK. No report was produced of this invesigation.
3. Sample was collected and tested by EBA in 2008. No report was produced.

Table A1-2: Direct Shear Test Results

			Test Data				Peak		Residual		
Sample	USCS	Description	Normal Stress (kPa)	Peak Shear Stress (kPa)	Residual Shear Stress (kPa)		Apparent Cohesion (kPa)	Friction Angle (°)	Apparent Cohesion (kPa)	Friction Angle (°)	Comments
TP97-01	SM	Silty sand colluvium	800	720	644	Raw data linear trendline values	180	35	220	28	
			1600	1335	1068	Adjusted trendline w/ no cohesion	-	37	-	32	Excludes the 1600kPa and 2400kPa normal stress tests
			2400	1838	1490	Recommend values for assessment	-	37	-	32	Angles expected to be higher under lower normal loads
TP97-02	ML	Clayey silt colluvium	800	438	352	Raw data linear trendline values	-16	30	-17	26	
			1600	951	797	Adjusted values w/ no cohesion	0	30	0	25	Excludes the 800kPa normal stress test
			2400	1375	1129	Recommend values for assessment	0	30	0	26	
SHEL-02A	CL	Silty clay with low plasticity	100	58	30	Raw data linear trendline values	34.3	21.1	27.9	10.4	Slide debris
			300	175	110	Adjusted values w/ no cohesion	0	25	0	14	
			600	256	127	Recommend values for assessment	0	21	0	10	
DS-7/8/9	ML	Clayey silt with sand	100	71	57	Raw data linear trendline values	16.7	30.7	19.9	22.5	
			400	263	193	Adjusted values w/ no cohesion	0	31.4	0	24	Excludes the 100kPa normal stress test
			800	489	348	Recommend values for assessment	0	31	0	23	
DS-1/2/3	SM	Silty Sand with gravel	100	90	79	Raw data linear trendline values	37.4	35.1	31.2	29.7	
			400	350	291	Adjusted values w/ no cohesion	0	36	0	32	Excludes the 100kPa and 400kPa normal stress tests
			800	587	505	Recommend values for assessment	0	35	0	30	
DS-4/5/6	SM	Silty Sand with clay and gravel	100	146	59	Raw data linear trendline values	68.5	33	0.5	30.3	
			400	306	234	Adjusted values w/ no cohesion	0	37	0	30.3	Excludes the 100kPa normal stress test
			800	597	468	Recommend values for assessment	0	35	0	30	
08-ROD-0B01	GM	Silty gravel with clay and sand	150	90		Raw data linear trendline values	13.5	26.7	-	-	Reconstituted sample with a low density
			350	187		Adjusted values w/ no cohesion	0	28			
			550	291		Recommend values for assessment	0	28			
Shel-01	CH	Clay, high plasticity	207	94	71	Raw data linear trendline values	45.4	14.9	31.1	11.1	Slide debris
			413	169	113	Adjusted values w/ no cohesion	0	18	0	14	Excludes the 207kPa and 620kPa normal stress tests
			620	204	153	Recommend values for assessment	0	15	0	11	
SWF-1C-003	CI	Clay, medium plasticity	207	106	86	Raw data linear trendline values	-6.5	28.1	-44.6	30.2	
			413	211	175	Adjusted values w/ no cohesion	0	27.5	0	26	
			620	327	326	Recommend values for assessment	0	28	0	26	
SWF-2C-002	CL	Clay, low plasticity	207	138	135	Raw data linear trendline values	90	16.9	117	9.9	
			413	246	225	Adjusted values w/ no cohesion	0	23	0	18.5	
			620	263	207	Recommend values for assessment	0	23	0	18	Angle adjusted higher due to the unusually cohesion value
SWF-4C-001	ML	Silt, low plasticity	207	163	142	Raw data linear trendline values	36.2	30.9	14.1	30.9	
			413	278	253	Adjusted values w/ no cohesion	0	34	0	32	
			620	411	389	Recommend values for assessment	0	33	0	31	Peak angle adjusted higher due to the high cohesion value, and higher residual angle
159755	SM	Silty Sand	100	83		Raw data linear trendline values	16	32.8	-	-	
			200	141		Adjusted values w/ no cohesion	0	35.6			
			300	212		Recommend values for assessment	0	33			

Table A1-3: Triaxial Test Results

		Peak friction angle data												Average peak friction angle (deg)	Residual Angle	
		Test 1				Test 2				Test 3						
Sample	USCS	Effective major stress at failure (kPa)	Effective minor stress at failure (kPa)	Stress Ratio at failure, σ'_1/σ'_3	**Friction angle (deg)**	Effective major stress at failure (kPa)	Effective minor stress at failure (kPa)	Stress Ratio at failure, σ'_1/σ'_3	**Friction angle (deg)**	Effective major stress at failure (kPa)	Effective minor stress at failure (kPa)	Stress Ratio at failure, σ'_1/σ'_3	**Friction angle (deg)**		Critical friction angle, α (deg) [1]	**Residual friction angle, ϕ (deg) [2]**
96477 (CU-1)	ML	646	157	4.11	**37.5**	1219	354	3.44	**33.4**	1913	399	4.79	**40.9**	**37**	24	**26**
159753 (CU-1)	CL	537	118	4.55	**39.8**	899	319	2.82	**28.4**	1409	531	2.65	**26.9**	**32**	20	**21**
159756 (CU-2)	SM	721	130	5.55	**44.0**	1398	291	4.80	**41.0**	2827	823	3.43	**33.3**	**39**	28	**32**
160022 (CU-3)	CL	517	168	3.08	**30.6**	865	500	1.73	**15.5**	1358	552	2.46	**25.0**	**24**	19	**20**

Notes

1. From q-p graph on test results
2. $\phi = \sin^{-1}(\tan \alpha)$

Table A1-4: Stength Properties by Soil Type

				Density (prior to testing)		Peak		Residual		
USCS	Description	Test ID	Plasticity Index	Bulk (Mg/m^3)	Dry (Mg/m^3)	Cohesion (kPa)	Friction Angle (°)	Cohesion (kPa)	Friction Angle (°)	Comments
Coarse Grained Test Summary										
GM	Silty gravel with clay and sand	08-ROD-OB01	-	1.9	1.6	0	28	-	-	Result is not likely to be representative of material. Surface sample collected from overburden dump area. The test was noted as being a reconstituted low density sample.
S	Coarse sand	RS-1	-	-	-	0	40	-	-	No test details or source location available. The USCS symbol is assumed based on soil description
S	Coarse sand	RS-2	-	-	-	0	39	-	-	
S	Coarse sand	RS-3	-	-	-	0	42	-	-	
S	Coarse sand	RS-4	-	-	-	0	*43*	*0*	41	
SM	Silty Sand with gravel	Clay-1	11	-	-	*0*	*31*	-	-	Results are not likey to be representative given the particle size distribution with significant gravel content (20-30%). No
SM	Silty Sand with gravel	Clay-2	12	-	-	*0*	*31*	-	-	
SM	Silty Sand with gravel	2187	-	2.0	1.8	0	36	-	-	Reconstituted sample compacted to 88% maximum dry density
SM	Sand, Silty, trace of clay and gravel	TP97-01	-	2.3	2.1	0	37	0	32	Angle are likley to be lower bound for the material given the large normal stresses that the materials were tested at. (800 to 2400 kPa)
SM	Silty Sand with gravel	DS-1	5	2.2	1.9	0	35	0	30	
SM	Silty Sand with clay and gravel, medium plasticity	DS-4	17	2.2	1.8	0	35	0	30	
SM	Silty sand	159756 (CU-2)	-	2.2	2.0	0	39	0	32	
SM	Silty sand	159755	-	1.6	1.4	*0*	*33*	-	-	Reconstituted sample to void ratio of 0.81 (likely lower density compared to insitu)
Silts										
ML	Silty, clayey,sandy, low plastic	TP97-02	-	2.1	1.9	0	30	0	26	
ML	Clayey silt with sand	DS-7	17	2.0	1.5	0	31	0	23	
ML	Silt - sandy, trace gravel	96477 (CU-1)	2	1.9	1.5	0	37	0	26	
ML	Silt, low plasticity	SWF-4C-001	8	2.0	1.6	0	33	0	31	
Clays										
CL	Clay, silty, some sand, gravel, low pla	160022 (CU-3)	24	2	1.7	0	24	0	20	
CL	Clay, low plasticity	SWF-2C-002	25	2.1	1.8	0	23	0	19	
CL	Clay, low plasticity	159753 (CU-1)	26	2	1.6	0	32	0	21	
CI	Clay, medium plasticity	Clay	34	1.9	1.6	*21*	*20*	-	-	Reconstituted sample to 95% standard proctor; not lab data was available.
CI	Clay, medium plasticity	SWF-1C-003	35	2.1	1.8	0	28	0	26	
Slide debris										
CH	Clay, high plasticity	SHEL-01	54	1.8	1.4	0	21	0	10	Shelby tube pushed into slide debris by hand. Samples collected from SWF at the 780 Bench
CL	Silty clay with low plasticity	SHEL-02A/B	25	1.9	1.3	0	15	0	11	Shelby tube pushed into slide debris by hand. Samples collected from SWF at the 780 Bench

Appendix K: Main Dam Stability Analysis

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Memo

To:	File	**Client:**	Minto Exploration Inc.
From:	Murray McGregor	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 18, 2014
Subject:	Main Dam Stability Analysis		

1 Introduction

This memo presents the stability analyses for the proposed Main Dam. The analyses were completed in accordance with the Canadian Dam Safety Guidelines (CDA 2007, Revised 2013). As a portion of the dam foundation contains ice-rich clays similar to those encountered in the south wall of the Main Pit and beneath the Dry Stack Tailings Storage Facility (DSTSF), additional analyses were completed to assess stability in the event that a shear zone is present in the warm permafrost clay foundation on which a portion of the dam will be constructed.

The Main Dam's location is shown in Figure 1. The dam is a rock fill structure with an internal geomembrane liner system and a fine-grained core that is keyed into fine grained overburden and/or bedrock. The dam has a crest width of 14.5 m, a 2.5H:1V upstream slope and a 3H:1V downstream slope. The dam has a crest elevation of 807 m, with a provision to raise the crest height to 812 m, if required. The key trench has a variable depth with a base width of 10.5 m and 2H:1V side-slopes. During operations, a tailings deposition strategy will be implemented to develop a tailings beach upstream of the dam.

A geotechnical field investigation was completed in 2013 to characterize the foundation soils (SRK, 2014a). The dam is located at the edge of a paleochannel, with overburden thicknesses ranging from 0 to 60 m along the southern abutment. Borehole logs have confirmed the presence of ice lenses and permafrost. A thick zone of unconsolidated fill (operationally referred to as the "Boulder Laydown Area") covers most of the southern abutment. This material was placed during pit development and consists of oversized waste rock and random fill. The fill material will be excavated prior to the dam construction. Further details on the foundation conditions are presented in the Preliminary Design Report (SRK 2014b).

2 Methodology

2.1 Assessment method

Stability modelling was completed using limit equilibrium software, Slide 6.0 (Rocscience 2012). Factor of safety (FOS) values were used as a metric for evaluating performance. The analyses focused on both surface failures (skin failures) and global mechanisms (slip surfaces greater than a 5 m depth).

2.2 Design Criteria

The geotechnical stability design criteria are in accordance with requirements stipulated in CDA (2007, Revised 2013) with the minimum FOS for static and seismic assessments provided in Tables 1 and 2.

Table 1: Factors of Safety for Slope Stability – Static Assessment (Table 6-2, CDA (2007, Revised 2013))

Loading Conditions	Minimum Factor of Safety	Slope
End of Construction before reservoir filling	1.3	Upstream and downstream
Long-term steady state	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3	Upstream
Note 1. Factor of safety is the factor required to reduce operational shear strength parameters to bring a potential sliding mass into a state of limiting equilibrium (using generally accepted methods of analysis). **Note 2.** Higher factors of safety may be required if drawdown occurs relatively frequently during normal operation.		

Table 2: Factors of Safety for Slope Stability – Seismic Assessment (Table 6-3, CDA (2007, Revised 2013))

Loading Conditions	Minimum Factor of Safety	Slope
Pseudo-static	1	Upstream and downstream
Post-earthquake	1.2 to 1.3	Upstream and downstream

Based on the preliminary dam hazard classification of VERY HIGH assigned to the Main Dam, the Annual Exceedance Probability earthquake design ground motion will be at least in the 1/5,000 year event (CDA 2007, Revised 2013).

2.3 Seismic Coefficients

An assessment of peak ground acceleration (PGA) was completed using the 2010 National Building Code Seismic Hazard Calculations (NRCC 2013), provided in Attachment 1. The 2010 seismic hazard calculations provides PGA and specific spectral acceleration values for seismic events with probability of exceedance ranging from 1/100 (0.01) to 1/2475 (0.000404). The seismic parameters were extrapolated for the 1/5,000 year event and are also included in Attachment 1. The seismic design parameters are presented in Table 3.

Table 3: Site Class C Ground Motion Parameters for 1/5,000 Event

Spectral Period (s) or Peak Parameters	Seismic Values (g)
0.2	0.285
0.5	0.175
1.0	0.100
2.0	0.060
PGA	0.145

Note: values Extrapolated from NBCSHC data as shown in Attachment 1.

The ground motions interpolated from the seismic hazard calculator (NRCC 2013) are for site Class C: Very Dense Soil and Soft Rock. Table 4 presents all site classes and properties used to define different classes.

Table 4: Site Classification for Seismic Site Response

Site Class	Ground Profile Name	Average Properties Top 30 m		
		Average Shear Wave Velocity, $\bar{V}_s$(m/s)	Average Standard Penetration Resistance, $\bar{N}_{60}$	Soil Undrained Shear Strength, s_u
A	Hard Rock(2)	$\bar{V}_s > 1500$	N/A	N/A
B	Rock(2)	$760 < \bar{V}_s \leq 1500$	N/A	N/A
C	Very Dense Soil and Soft Rock	$360 < \bar{V}_s \leq 760$	$\bar{N}_{60} > 50$	S_u > 100 kPa
D	Stiff Soil	$360 < \bar{V}_s \leq 760$	$\overline{15 \leq N}_{60} \leq 50$	50 kPa < s_u ≤ 100 kPa
E	Soft Soil	$\bar{V}_s < 180$	$\bar{N}_{60}$<15	S_u < 50 kPa
		Any soil with more than 3 m of soil with the following characteristics: Plasticity Index: PI ≥ 20. Moisture content: w ≥ 40%. Undrained shear strength: s_u < 25 kPa.		
F(1)	Other Soils	Other Soils include: Liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils, and other soils susceptible to failure or collapse under seismic loading. Peat and/or highly organic clays greater than 3 m thickness. Highly plastic clays (PI>75) more than 8 m thick. Soft to medium stiff clays more than 30 m thick.		

Source: Adapted from National Building Code of Canada 2010 Table 4.1.8.4A (NRCC 2012)

Notes:
1. Site specific evaluation required.
2. Site Classes A and B are not to be used if there is more than 3 m of softer materials between the rock and the underside of the footing or mat foundations. If more than 3 m of softer materials exist, the Site Class is determined based on the average properties of the softer materials.

Seismic coefficients used in the pseudo-static analyses are presented in Table 5. The coefficients were calculated using the method presented in Section 6.2.2 of the LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Reference Manual (FHWA 2011). As the coefficients are dependent on height of the slope, multiple model runs were completed to estimate the FOS depending on the critical slip surface depth. The slope

heights of 1 m, 5 m, and 10 m assumed to characterize surficial slip surfaces, slip surfaces within the core, and deep slip surfaces respectively.

Table 5: Design Seismic Coefficients

Critical Slip Surface Slope Height	Seismic coefficient
1 m	0.0701
5 m	0.0606
10 m	0.0487

No vertical seismic coefficients were used for the pseudo-static modelling. For most earthquakes the horizontal acceleration component is much greater than the vertical acceleration component; therefore, the vertical seismic coefficient is commonly assumed to be zero (Seed and Whitman 1970; FHWA 2011 and Anderson *et al*. 2008).

2.4 Dam Geometry

Three sections were analyzed for the Main Dam slope stability assessment. The first section (Section A) is located at the maximum height of the dam and assesses internal stability. The second section (Section B) is located where the least favourable foundation conditions are expected. The third section (Section C) is slightly rotated from Section B to assess the down-valley, deep seated stability. The location of each section is provided in Figure 1 and the section geometries are presented in Figures 2 and 3.

- In each section, the ground surfaces are based on topographic site surveys provided in January 2014 and the original ground surface provided by Minto. Bedrock contact was generated from the borehole database and the lithology is based on SRK's interpretations of borehole lithology. There is an area between the Main Pit and the DSTSF area that has limited soil data. This area was conservatively assumed to be fine grained overburden.

- It was assumed that all of the boulder laydown was excavated prior to the dam construction. In Section B, the overburden is stripped down to an elevation of 799 m in the key trench.

- The foundation stratigraphy at Section A is based on the maximum excavation surface provided by Minto.

- Section B is located near borehole 13-MPD-06 (SRK, 2014a). The overburden at this location is approximately 40 m thick and generally consists of frozen fine grained silts and clays with a 4 m sand and gravel layer located approximately 7 m below the original ground surface.

- A pre-mining geometry was developed as part of the evaluation of a potential deep continuous shear zone. The pre-mining surface was provided by Minto.

Two dam height geometries were evaluated: an 807 m crest height with a water surface elevation of 804 m, and an 812 m crest height with a water surface elevation of 809 m. All of Section A's analyses were completed using the 812 m crest height as this represents the more conservative

case. Section B was analyzed with both 807 m and 812 m dam heights. The 807 m dam geometries assumed water at full supply level with no tailings, while the 812 m dam geometry assumed tailings deposited on the upstream side of the dam with a water pond setback 100 m from the dam crest. The 812 m dam was only assessed with a tailings beach since it is a contingency for possible ice entrainment and water would not reach the full supply level without the formation of a tailings beach.

The geometry for the cases that considered a potential weak shear zone (Section C in Figure 1) is provided in Figure 4. The shear zone was modelled with a thickness of 5 m and located 6 m above the bedrock contact. This location and thickness was selected to be similar to the movements observed at the DSTSF and at the south wall failure of the Main Pit (SRK 2012).

2.5 Material Properties

Material properties used in the analyses are presented in Table 4. The foundation soils, waste rock, and tailings material properties were selected following a review of all geotechnical strength properties at the Minto Mine, the details of the review are provided in SRK (2014c). The analyses were completed using the residual shear strength values to be conservative.

No liner or geosynthetic products were included in the analysis. The dam core is assumed to be constructed of soils from the Reclamation Overburden Dump. Two direct shear tests were completed on one sample of this material (O8-ROD-OB01). The tests were completed with the samples compacted to 85% Standard Proctor and 90% Standard Proctor, with the resulting friction angle ranging from 27 to 28 degrees (EBA 2008). The average value of 27.5 degrees was used for this analysis.

Table 4: Material Properties

Material	Bulk Density (t/m^3)	Cohesion (kPa)	Friction Angle (°)
Clean coarse grained soils	1.9	0	38
Silty sands, residuum, weathered bedrock	1.9	0	32
Silts	1.8	0	26
Clays, including high plasticity, ice rich clays	1.8	0	19
South wall failure slide debris	1.8	0	10
Waste rock	2.1	0	37
DSTSF compacted tailings	1.9	0	32
Conventional tailings	1.3	0	8 (0)[1]
Dam core	1.8	0	27.5
Potential weak shear zone	1.8	Varies (See Table 5)	

Source: MintoMatProp_Memo_1CM002-018_PM_rev01
Note: Tailings is assumed to have friction angle of zero for pseudo static analyses.

In select analyses, high cohesion values were specified to represent frozen soils to force the failure surfaces to occur within unfrozen soil units. The extent of the unfrozen soil zones were determined from thermal modelling results presented in SRK (2014d).

Various back-analysis have been completed on the movements at the DSTSF and the 2009 south wall failure of the Main Pit to estimate the properties of the shear zone material. A summary of the back-calculated values is provided in in SRK (2014c). The results show a large degree of variance and as a result, no single set of parameters was recommended in the review of the geotechnical properties. Instead, a sensitivity analysis was recommended that uses a range of parameters. Table 5 presents the range of strength values used in the Main Dam stability analyses.

Table 5: Summary of Potential Weak Zone Material Properties

Case	Cohesion (kPa)	Friction Angle (°)	Pore Pressure Ratio (r_u)
Drained	0	10	Varied from 0.5 to 0.7
Undrained	Varied from 35 to 60	0	0

Note(s):

(1) The drained cases set the cohesion and friction angles based on laboratory test results of the south wall failure slide debris and varied the pore pressure ratio over a range of values that were back-calculated from other areas on site. SRK (2009) back-calculated the failure of the south wall of the Main Pit and found shear strength of ten degrees, and a r_u of zero. Norwest (2014a) and (2014b) back-calculated the south wall of the Main Pit and DSTSF and found that an approximate r_u of 0.6 was required to obtain a FOS of 1 with a ten degree friction angle.

(2) The undrained case properties represent conservative undrained shear strengths back-calculated using a limit equilibrium approach from the DSTSF and presented in Appendix A of SRK (2014e).

2.6 Pore Water Pressure

The water level in the pit was conservatively assumed to correspond with the spillway elevation (i.e. at elevation 804 m). The specified phreatic surfaces through the dam are shown in Figures 2 and 3.

- Drained pore water conditions assumed no pore pressures within the dam structure or foundation.

- Flooded pit cases assumed water behind the dam at the full supply level with a simplified phreatic surface through the dam that represents the zero pressure elevation; the water tables are consistent with those presented in the seepage analysis (SRK 2014f).

- A rapid drawdown analysis was completed for the 807 m dam scenario where the water table was reduced from 804 m to 786 m. The rapid drawdown analysis uses the Duncan *et al*. (1990) method for determining pore water conditions.

2.7 Cases Considered

Static and pseudo-static stability analyses were completed in the upstream and downstream movement direction for both Sections A and B under various pore water conditions and dam geometries. The modelled scenarios are presented in Table 6 and in Section 3.

Stability analyses that considered a potential weak shear zone were evaluated using the range of back-calculated material properties presented in Section 2.5. These failures were limited to deep

block searches through a potential shear zone assumed at a thickness and depth that has been observed elsewhere on site.

3 Results

3.1 Stability Analysis Results

The results of the stability analysis are presented in Table 6 below. Individual model geometries and critical slip surfaces are presented in Attachment 2A through 2F.

Table 6: Limit Equilibrium Factor of Safety Results

Model Run	Movement Direction	Seismic Hazard	Pore-Water Conditions	Dam Geometry	Minimum FOS	Calculated FOS
Section A – Maximum Height of Dam						
1	Upstream	Static	Drained	812 dam	1.3	1.4
2	Upstream	Static	Flooded Pit	812 dam	1.3	1.4
3	Upstream	Pseudo-Static	Drained	812 dam	1.0	1.2
4	Upstream	Pseudo-Static	Flooded Pit	812 dam	1.0	1.0
5	Downstream	Static	Drained	812 dam	1.3	2.1
6	Downstream	Pseudo-Static	Drained	812 dam	1.0	1.8
Section B – South Dam Abutment						
9	Upstream	Static	Drained	807 dam	1.3	1.5
10	Upstream	Static	Flooded Pit	807 dam	1.3	1.3[2]
11	Upstream	Pseudo-Static	Drained	807 dam	1.0	1.3
12	Upstream	Pseudo-Static	Flooded Pit	807 dam	1.0	1.0
13	Downstream	Static	Drained	807 dam	1.3	1.8
14	Downstream	Static	Flooded Pit	807 dam	1.5	1.5
15	Downstream	Pseudo-Static	Drained	807 dam	1.0	1.5
16	Downstream	Pseudo-Static	Flooded Pit	807 dam	1.0	1.0
17	Upstream	Static	Flooded Pit	812 dam /w tailings	1.3	5.6
18	Upstream	Pseudo-Static	Flooded Pit	812 dam /w tailings	1.3	4.3
19	Downstream	Static	Flooded Pit	812 dam /w tailings	1.5	1.6
20	Downstream	Pseudo-Static	Flooded Pit	812 dam /w tailings	1.0	1.3

Notes: 1. The 812 dam represents Section A geometry with either drained or ponded water cases while the 812 dam /w tailings case represents Section B with an 812 m dam and tailings beach developed on the upstream side.

2. Select upstream analyses indicate that geotextile reinforcement is needed within the dam core.

Figure 5 provides the critical failure surfaces at Section A. The internal stability assessed in Section A shows the upstream critical slip surface projecting through the fine grained core with an

approximate maximum depth of 5 m while the downstream critical slip surface is a shallow skin failure along the downstream surface.

Figure 6 provides the critical failure surface at Section B. The foundation stability, assessed in analyses of Section B, shows foundation controlled critical slip surfaces. The upstream critical slip surface projects through the fine grained core with an approximate maximum depth of 5 m while the downstream critical slip surface is a deep seated (> 10 m) rotational surface.

Under seismic conditions, the FOS in the pseudo-static analysis for overall dam stability was found to be above 1.0. Therefore displacements are expected to be within a tolerable range.

The rapid drawdown analyses resulted in a FOS of 1.3. This analysis was completed without a liner with the failure surface passing through the location where the liner would be and through the dam core. A liner with an interface friction angle greater than or equal to the dam core is recommended to ensure upstream and rapid drawdown stability.

Table 7 presents the results of the stability analyses of the potential deep shear zone scenario for a range of shear zone properties at Section C. The analysis was completed at three points in time: prior to mining, using the current ground surface, and following the construction of the Main Dam. The individual model geometries are presented in Attachment 2E and 2F along with their respective critical slip surfaces. The results show that there has been a significant improvement in the FOS since the mining began. This is due to the excavation of the Main Pit and due to construction of the Mill Valley Fill Buttress. The results also show that there is no significant difference in stability before and after construction of the Main Dam.

Table 7: Potential Shear Zone Factor of Safety Results

Cohesion (kPa)	Friction Angle (°)	Pore Pressure (r_u)	Calculated Factors of Safety		
			Pre-mining	Current	Dam Design
0	10	0.6	1.4	2.0	2.0
35	0	0	1.1	1.5	1.5
60	0	0	1.7	2.1	2.0

All calculated FOS meet or exceed the minimum FOS prescribed by the CDA guidelines.

3.2 Sensitivity Analysis Results

A sensitivity analysis was completed to assess the sensitivity of the FOS to changes in the dam and foundation material properties, as well as to changes in the dam design slope. The sensitivity plots completed are presented in Attachment 2G.

The internal stability sensitivity analysis shows that upstream FOS values can change from 1.2 to 1.6 when the friction angle is varied between 23.5 degrees to 31.5 degrees. The upstream FOS drops below the recommended guideline of 1.3 when the internal angle of friction of the dam core drops below 26.3. There are currently two direct shear tests characterizing the currently

assumed core material borrow source (ROD). Both direct shear tests indicate internal friction angles higher than 26.3 degrees.

The foundation material has a lesser influence on overall stability. The sensitivity was completed for the static analysis of the 807 m dam with flooded pit pore water conditions at Section B (model run 10). FOS values range from slightly more than 1.2 to nearly 1.4 when foundation material properties are varied from 22 to 30 degrees. The upstream FOS falls below the CDA recommended guidelines when the friction angle drops below 23 degrees.

The sensitivity of slope angles was completed using the same section as the foundation material sensitivity (model run 10). The sensitivity analysis indicated that the FOS values varied from about 1.2 to nearly 1.5 when varying the upstream slope angle from 18 to 24 degrees while downstream FOS ranged from 1.3 to 1.8 when varying the downstream slope angle from 16.5 to 22.5.

The sensitivity analyses of scenarios with a potential shear zone show as the pore pressure ratio varies from 0.5 to 0.7, the calculated FOS change from 2.3 to 1.7 respectively. Cohesion sensitivity analyses show FOS ranging from 1.5 to 2.0 with undrained shear strength ranging from 35 to 60 kPa. In all modelled cases for the current conditions and constructed dam design, the FOS is never below 1.5. The sensitivity of pore pressures and undrained shear strengths are plotted in Attachment 2H.

4 Discussion and Conclusions

All calculated factors of safety meet or exceed the minimum factors of safety prescribed by the CDA guidelines with the exception of the rapid drawdown analysis.

The strength properties of the dam core will need to be further confirmed prior to construction in the event that the dam design needs to be changed. If the strength properties are significantly lower than the values assumed in this analysis, changes to the dam design may be required. Possible design changes include reducing the dam slope angle or adding additional layers of geotextile within the core to increase the strength similar to a mechanically stabilized earth wall.

Stability results at Section B through the fine grained overburden were found to be strongly dependant on the foundation soil material properties. The silt foundation is currently assumed to have a friction angle of 26 degrees which meets the CDA guidelines for FOS of 1.3 or greater.

The upstream tailings beach significantly improves the dam stability. The added weight of tailings against the upstream slope further increases the upstream stability by acting as a buttress. The beach also moves the water further away from the dam surface providing lower water table and an increased seepage path.

A seismic event may reduce strength values from peak to residual in some areas; however, since all stability analyses were completed using the residual values, it can be concluded that the post-earthquake stability would be largely unchanged and meet the CDA minimum post-earthquake factor of safety requirements of 1.2 - 1.3.

Based on the results of the model, down-valley movement of the Main Dam is extremely unlikely. The stability analysis of a potential deep shear zone show that the FOS has greatly improved since the pre-mining conditions, primarily due to the construction of the Mill Valley Fill buttress. Further, the difference in FOS between the current conditions and the preliminary dam design is insignificant.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

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Figures

NOTES

1. Data presented in NAD 1983 UTM Zone 8N
2. Base Orthophoto provided by Minto Mine, August 2012

0 50 100 150 200 250
Scale in Metres

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FILE NAME: MainDamPreliminaryDesign_Settlement_Figure-1_rev3.dwg

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Minto Mine

Main Dam Stability

Site Layout and Critical Sections

DATE:	APPROVED:	FIGURE:
July 2014	MJM	1

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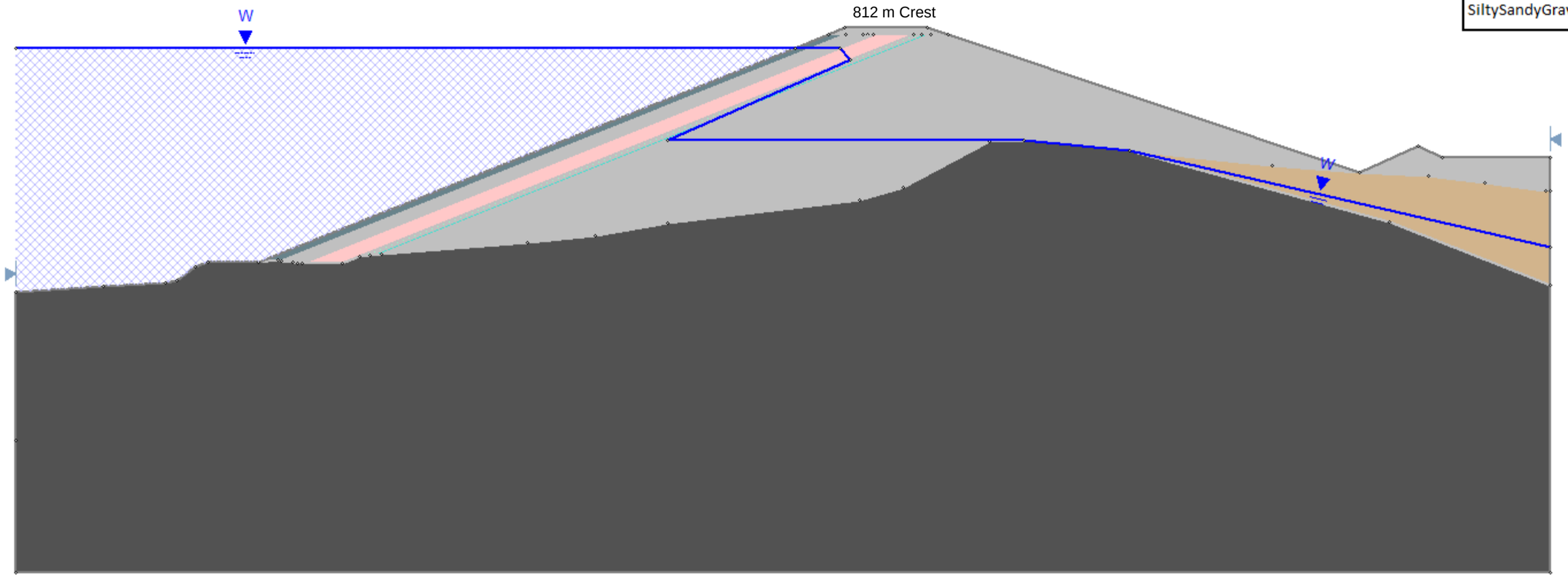
Material Name	Color
RipRap	
WasteRock	
Core	
Bedrock	
SiltySandyGravel	



Section A – 812 m dam with ponded water

Note: This section was used to assess internal stability only and does not consider foundation failures.

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Filename: Fig_DamSlope_11x17land_Rev00_PL_MJM.pptx

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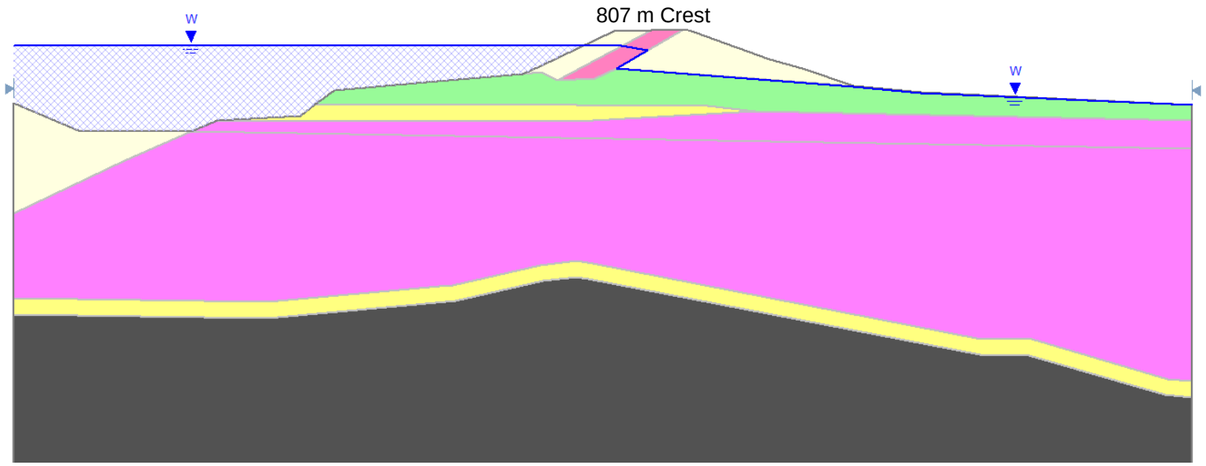
Minto Mine

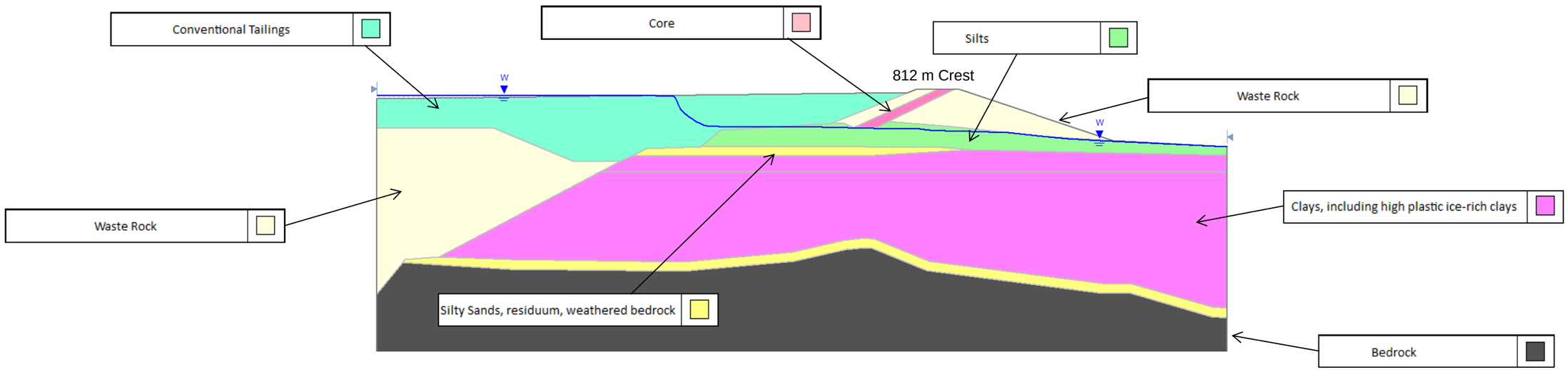
Main Dam Stability

Section A Dam Geometry

Date:	Approved:	Figure:
July 2014	MJM	**2**



Section B – 807 m dam with ponded water



Section B – 812 m dam with tailings beach

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Job No: 1CM002.018.0310

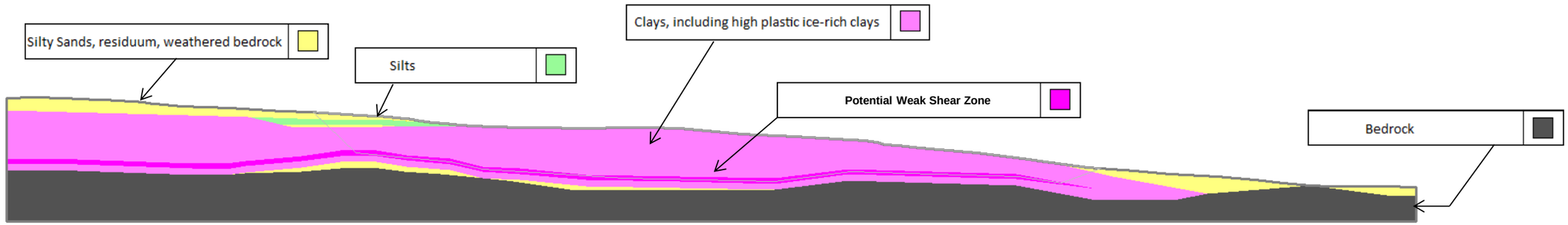
Filename: Fig_DamSlope_11x17land_Rev00_PL_MJM.pptx

Minto Mine

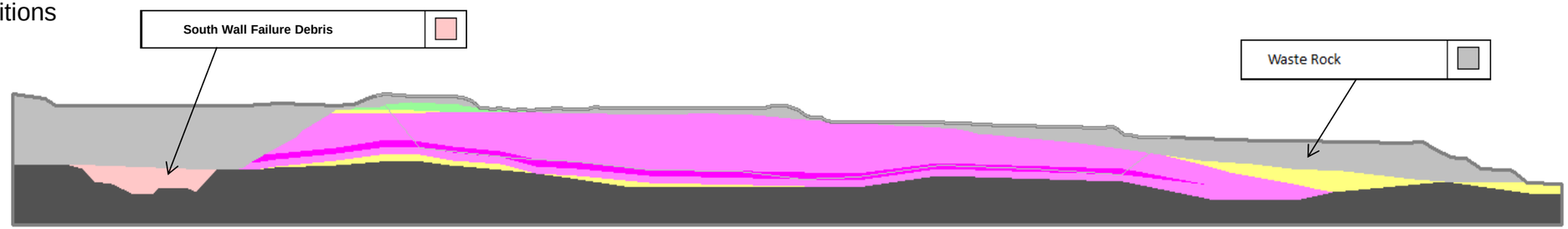
Section B Dam Geometry

Date:	Approved:	Figure:
July 2014	MJM	3

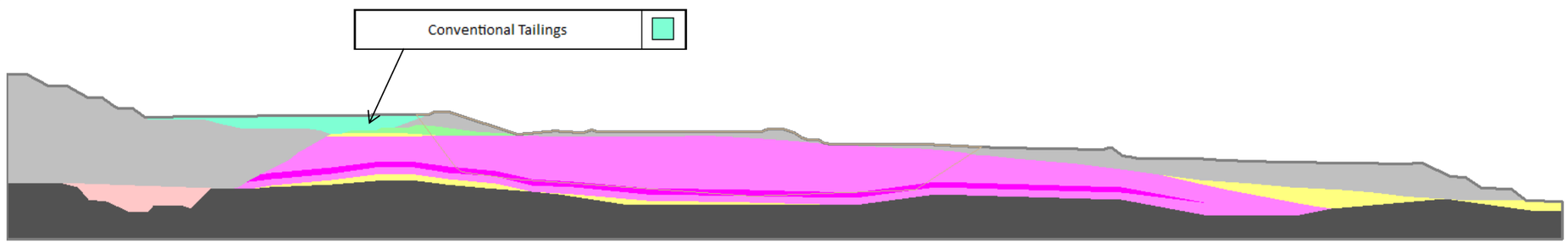
Natural Slope



Current Conditions



Tailings Facility Constructed



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Job No: 1CM002.018.0310

Filename: Fig_DamSlope_11x17land_Rev00_PL_MJM.pptx

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Minto Mine

Main Dam Stability

Section C - Deep Continuous Weak Zone Profile

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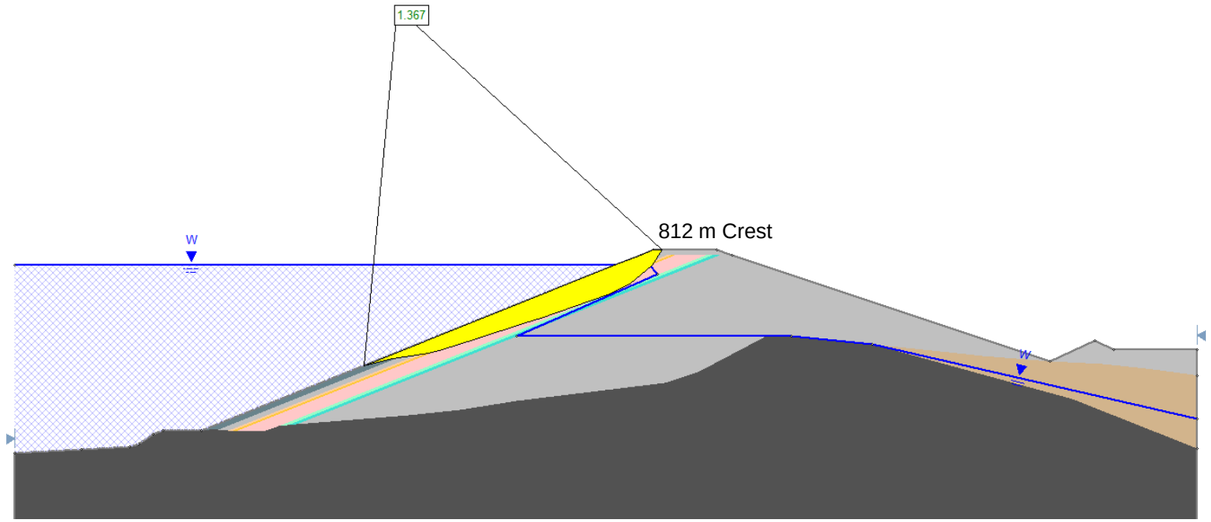
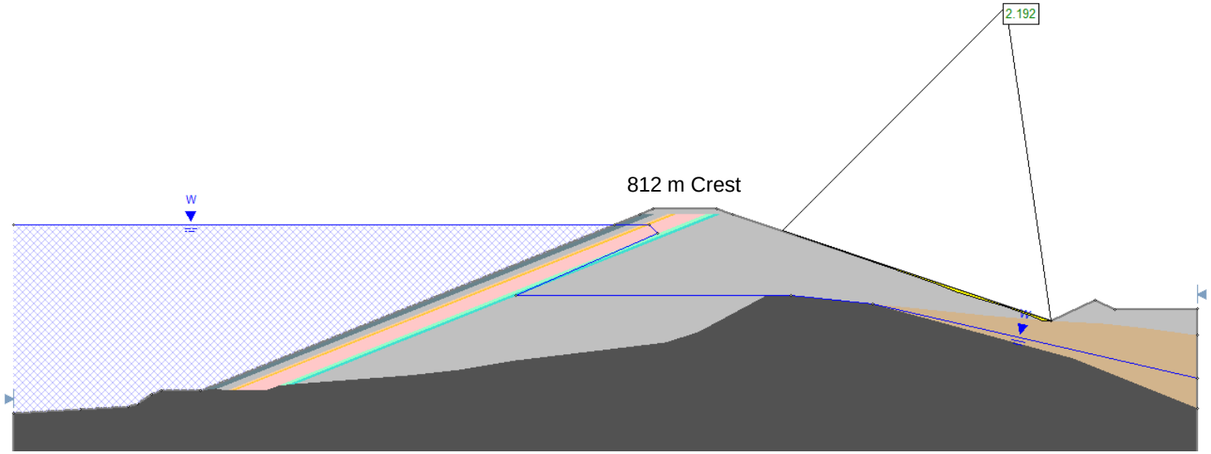
July 2014	MJM	**4**



Section A – Upstream Critical Slip Surface



Section A – Downstream Critical Slip Surface

Note: This section was used to assess internal stability only and does not consider foundation failures.

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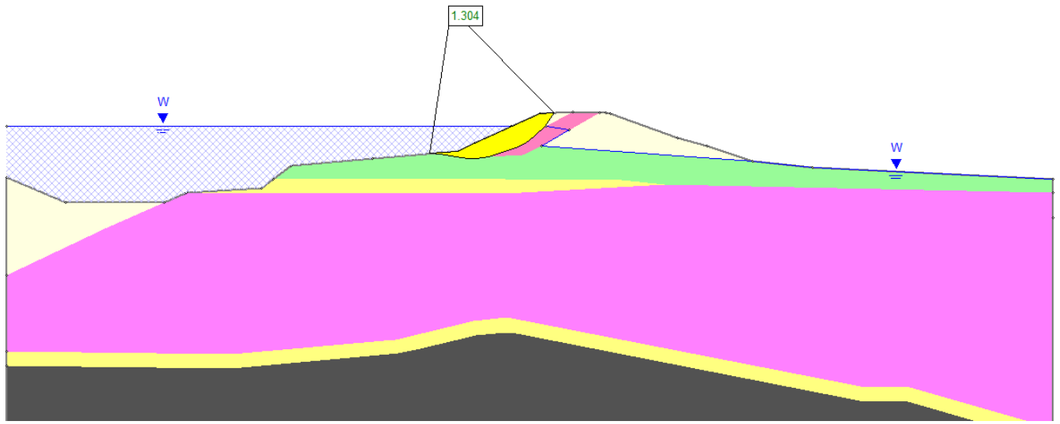
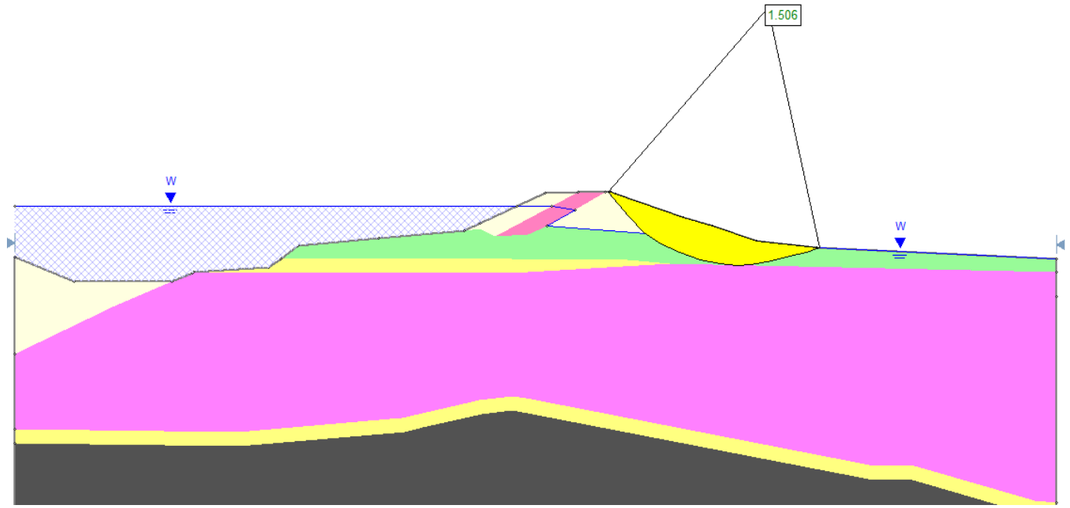
		Section A Critical Slip Surfaces
Job No: 1CM002.018.0310 Filename: Fig_DamSlope_11x17land_Rev00_PL_MJM.pptx	Minto Mine	Date: July 2014 / Approved: MJM / Figure: **5**



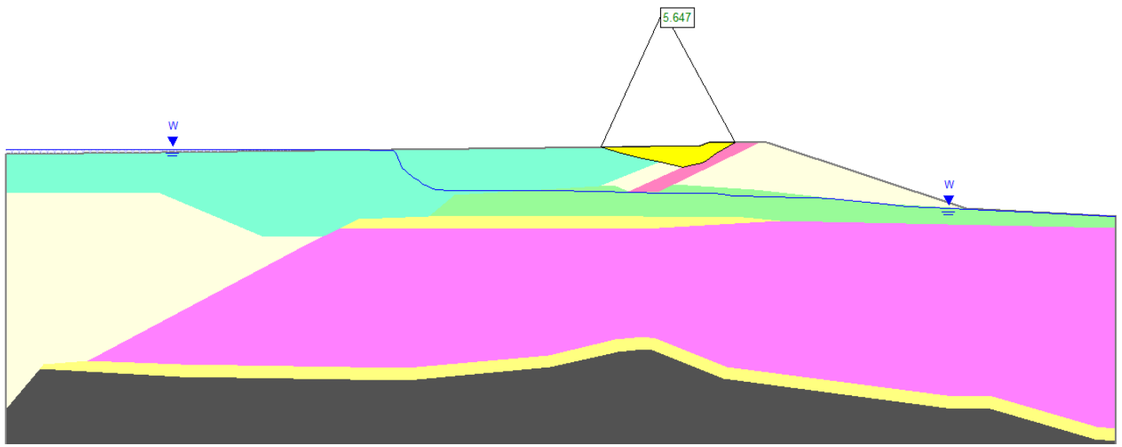
Section B – Upstream Critical Slip Surface

(807 m dam with ponded water)



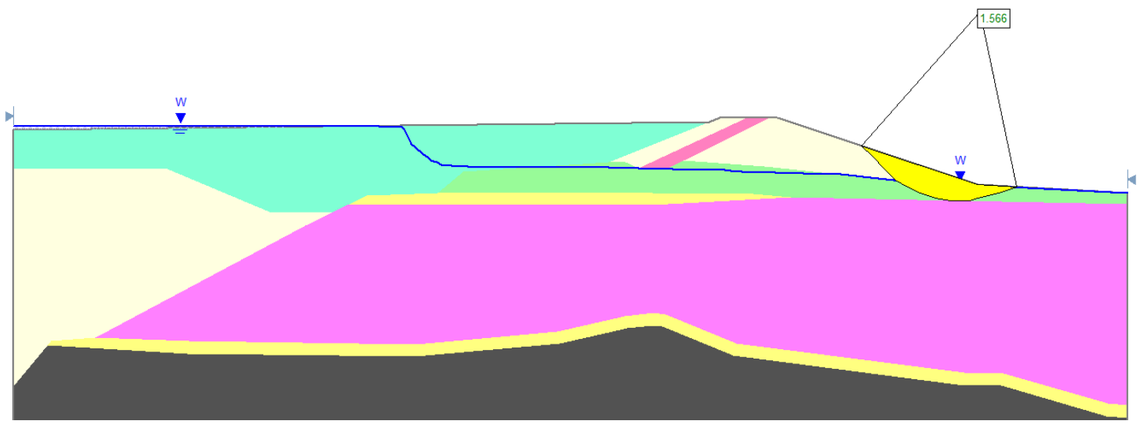
Section B – Downstream Critical Slip Surface

(807 m dam with ponded water)



Section B – Upstream Critical Slip Surface

(812 m dam with tailings beach)



Section B – Downstream Critical Slip Surface

(812 m dam with tailings beach)

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Job No: 1CM002.018.0310 Filename: Fig_DamSlope_11x17land_Rev00_PL_MJM.pptx	Minto Mine	**Section B Critical Slip Surfaces**		
		Date: July 2014	Approved: MJM	Figure: **6**

Attachment 1: Seismic Parameters

2010 National Building Code Seismic Hazard Calculation

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

Requested by: , SRK Consulting

August 02, 2013

Site Coordinates: 62.6064 North 137.2523 West

User File Reference: Minto Mine

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

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.213	0.132	0.077	0.048	0.110

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.051	0.104	0.144
Sa(0.5)	0.037	0.070	0.093
Sa(1.0)	0.025	0.045	0.057
Sa(2.0)	0.017	0.029	0.036
PGA	0.028	0.057	0.077

References

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

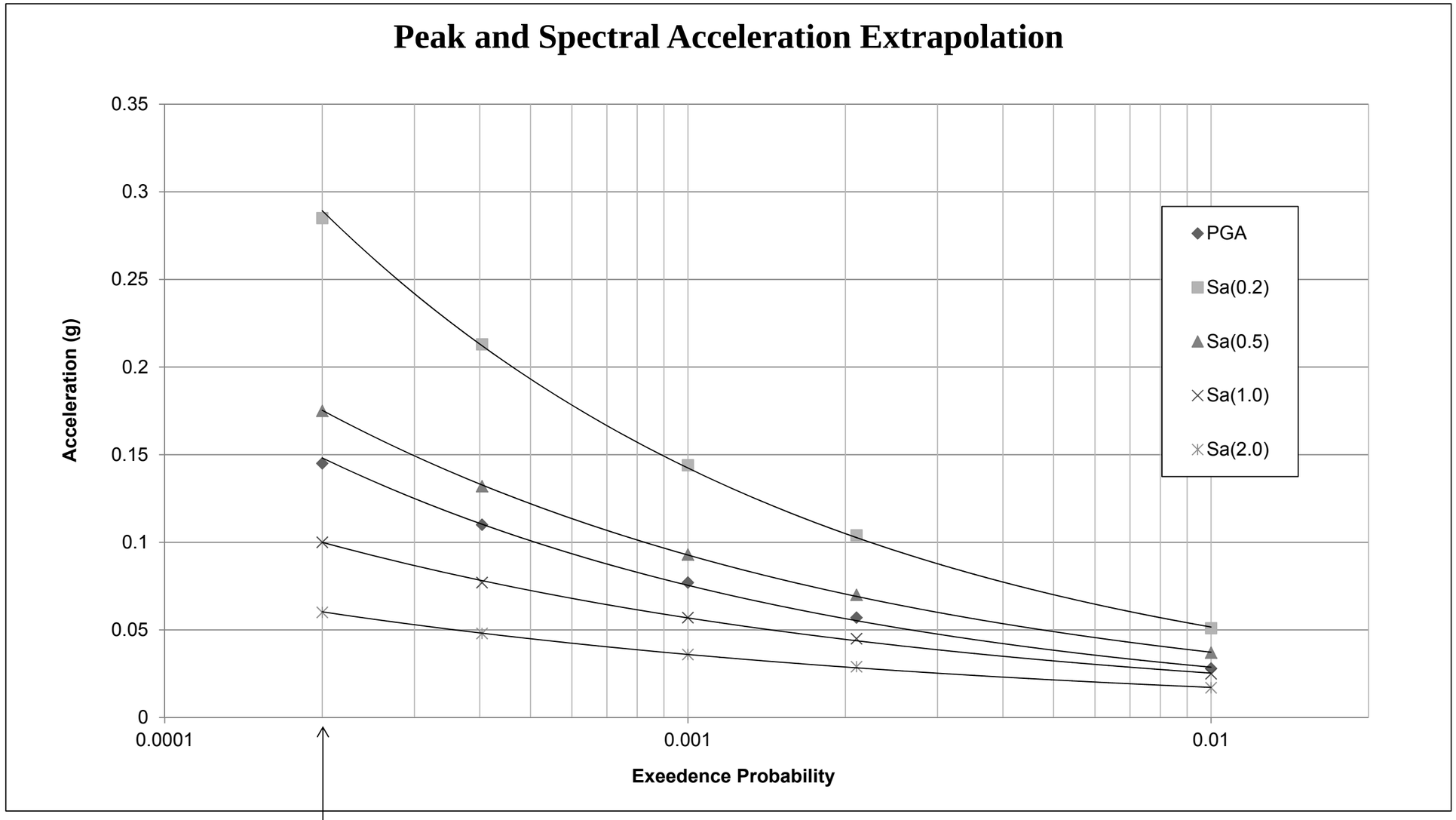
User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

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

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

Aussi disponible en français

Peak and Spectral Acceleration Extrapolation



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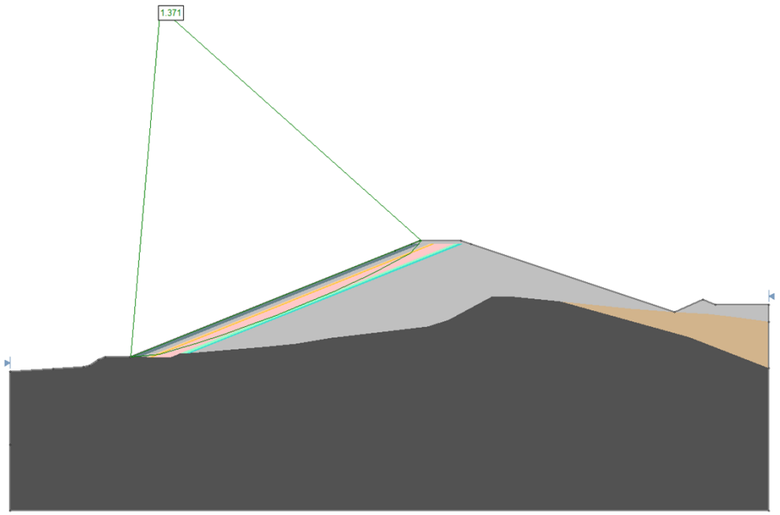
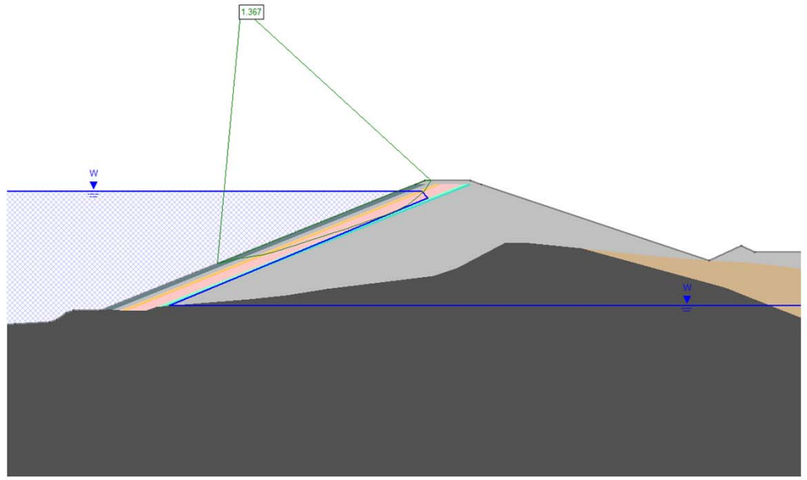
srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Stability
Job No: 1CM002.018.0310 Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx	Minto Mine	**Earthquake Ground Motion Parameter Selection**
		Date: May 2014 / Approved: MJM / Attachment: **1**

Attachment 2: Results



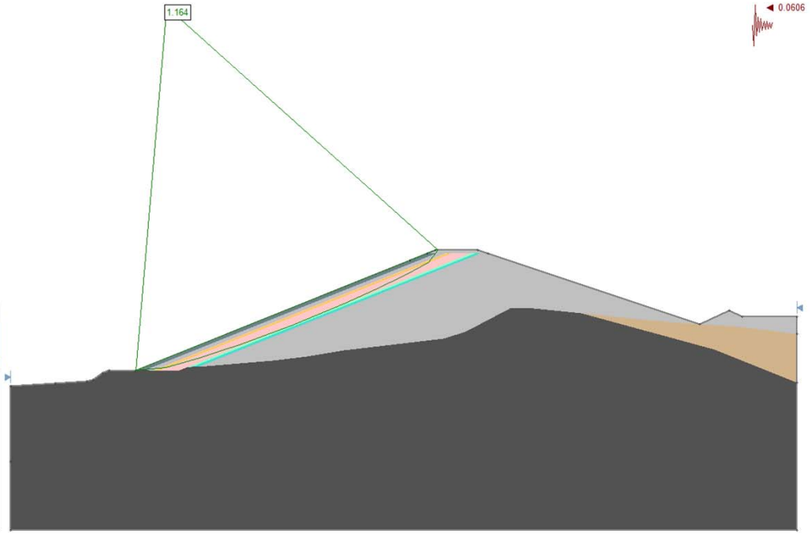
Model run 1: Upstream slide; Static seismic condition; Drained pore-water condition; 812 masl dam

Factor of Safety: 1.37



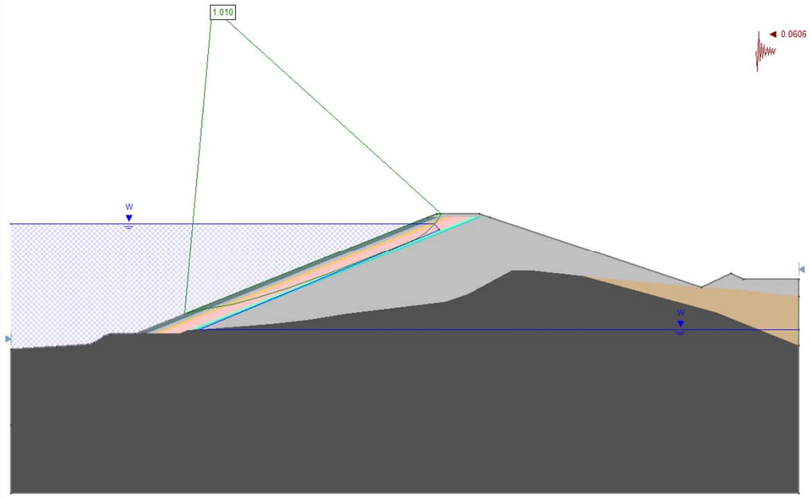
Model run 2: Upstream slide; Static seismic condition; Flooded Pit pore-water condition; 812 masl dam

Factor of Safety: 1.37



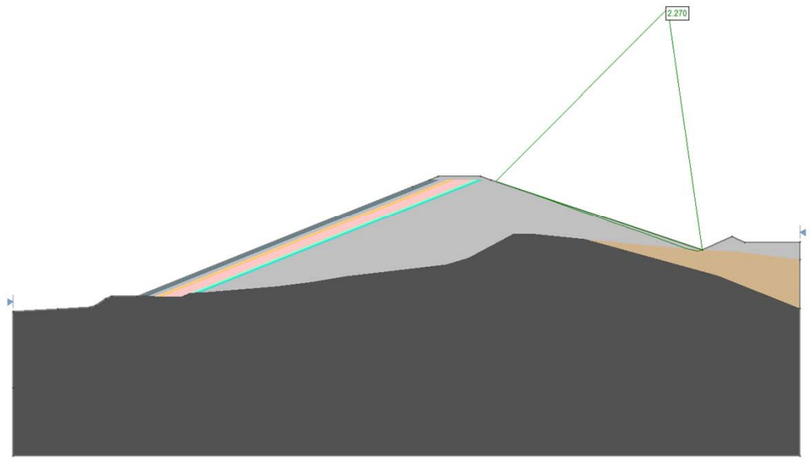
Model run 3: Upstream slide; Pseudo-Static seismic condition; Drained pore-water condition; 812 masl dam

Factor of Safety: 1.16



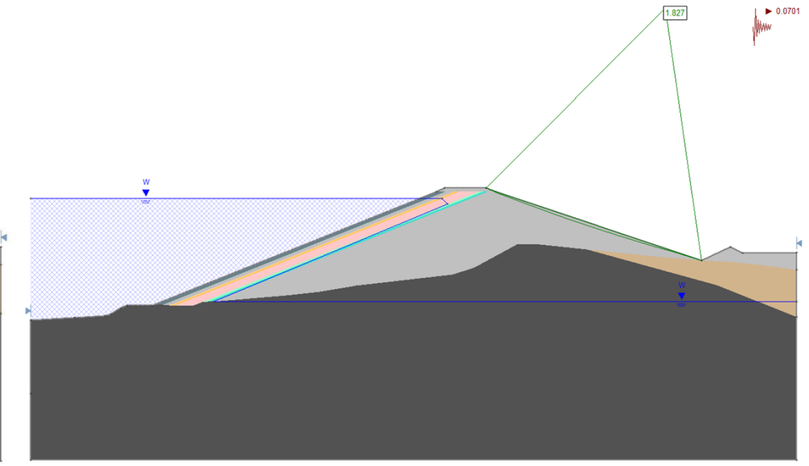
Model run 4: Upstream slide; Pseudo-Static seismic condition; Flooded Pit pore-water condition; 812 masl dam

Factor of Safety: 1.01



Model run 5; Downstream slide; Static seismic condition; Drained pore-water condition; 812 masl dam

Factor of Safety: 2.27



Model run 6; Downstream slide; Pseudo-Static seismic condition; Drained pore-water condition; 812 masl dam

Factor of Safety: 1.83

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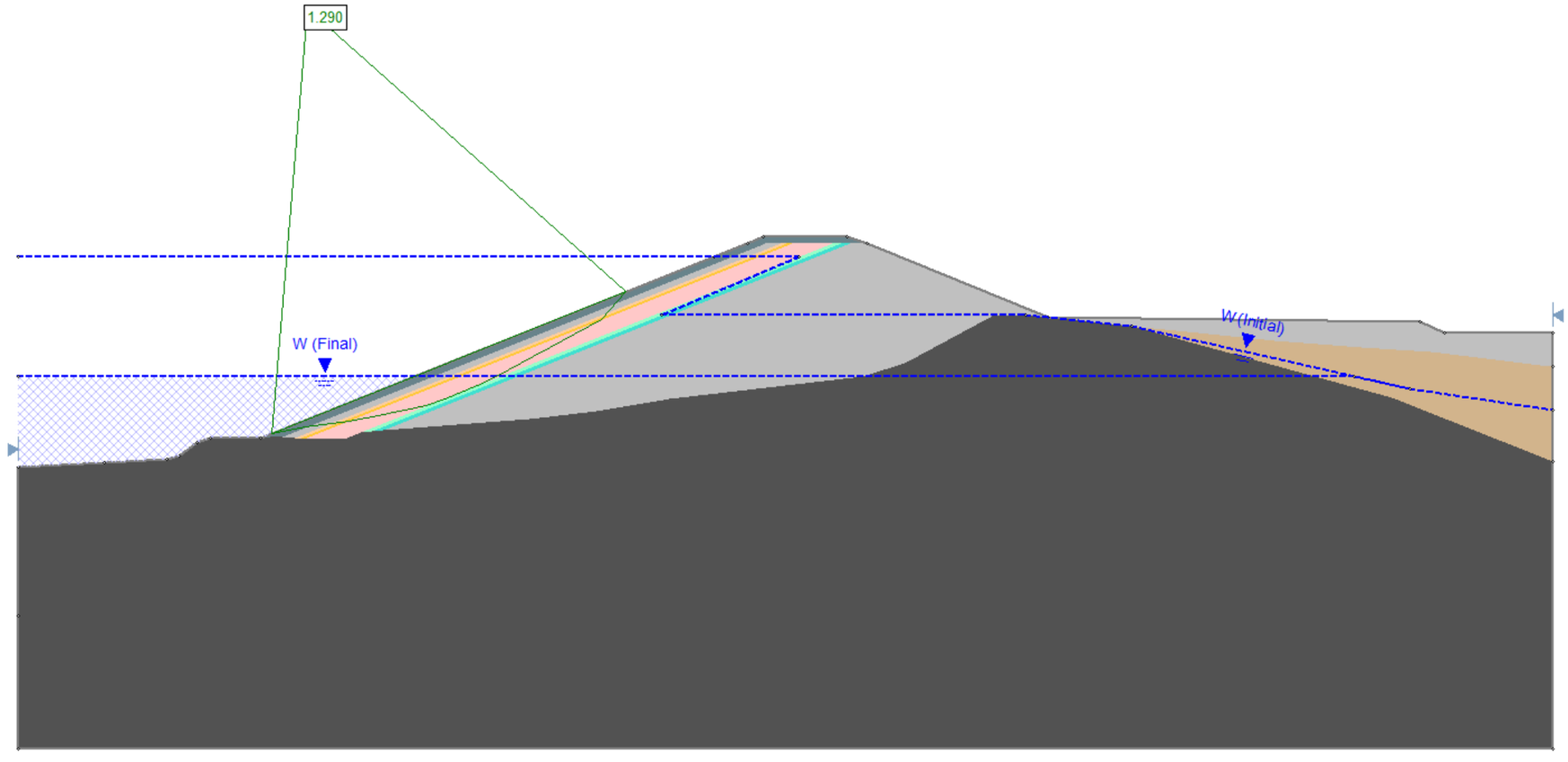
		Section A Stability Analyses
Job No: 1CM002.018.0310 Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx	Minto Mine	Date: May 2014 / Approved: MJM / Attachment: **2A**



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		Section A Rapid Drawdown Stability Analysis
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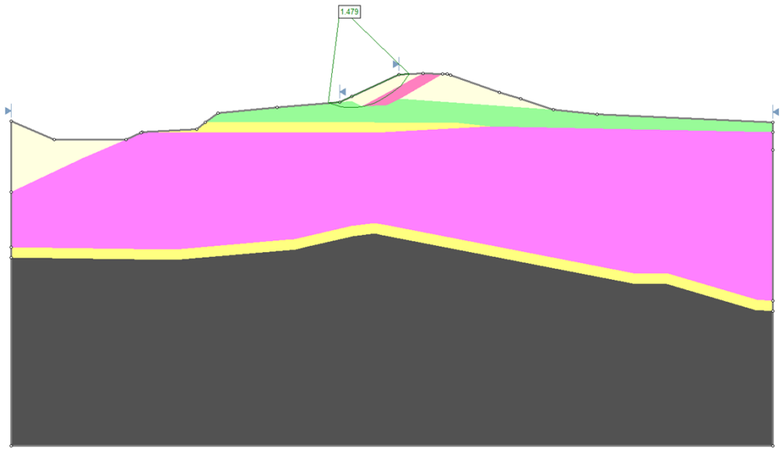
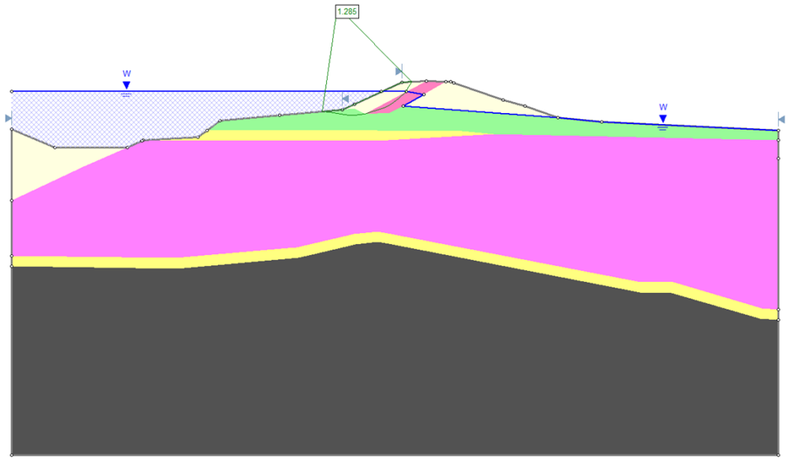


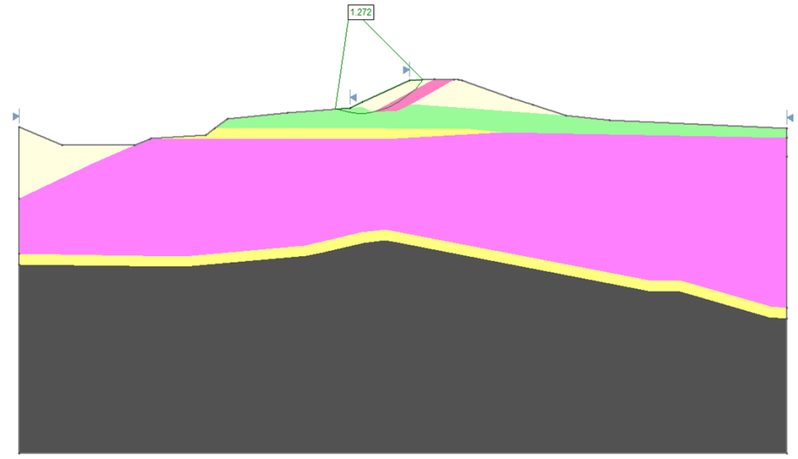
Model run 7; Upstream slide; Static seismic condition; Drained pore-water condition; 807 masl dam

Factor of Safety: 1.48



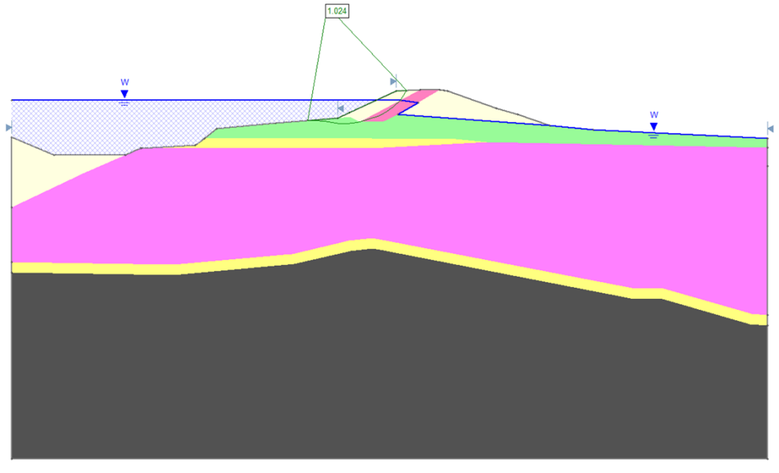
Model run 8; Upstream slide; Static seismic condition; Flooded Pit pore-water condition; 807 masl dam

Factor of Safety: 1.30



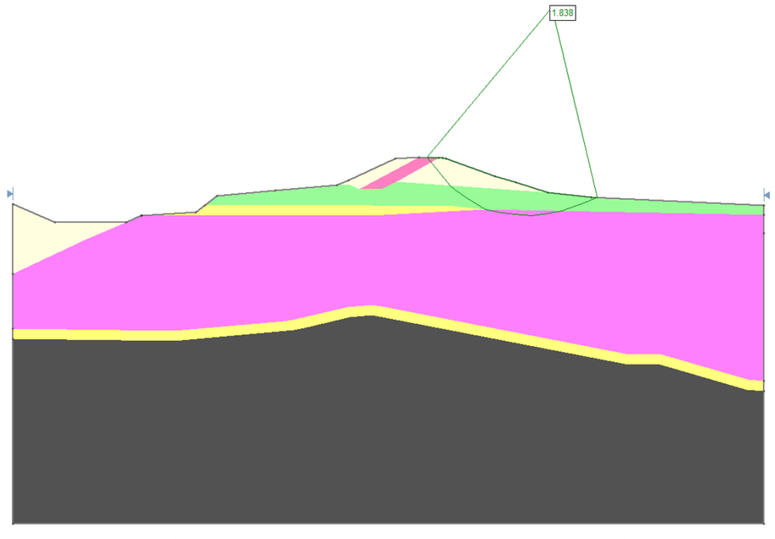
Model run 9; Upstream slide; Pseudo-Static seismic condition; Drained pore-water condition; 807 masl dam

Factor of Safety: 1.27



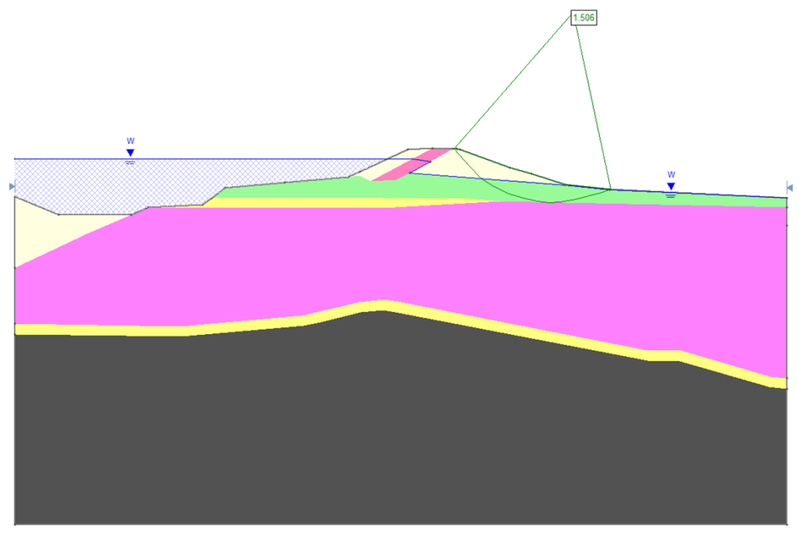
Model run 10; Upstream slide; Pseudo-Static seismic condition; Flooded Pit pore-water condition; 807 masl dam

Factor of Safety: 1.02



Model run 11; Downstream slide; Static seismic condition; Drained pore-water condition; 807 masl dam

Factor of Safety: 1.84



Model run 12; Downstream slide; Static seismic condition; Flooded Pit pore-water condition; 807 masl dam

Factor of Safety: 1.51

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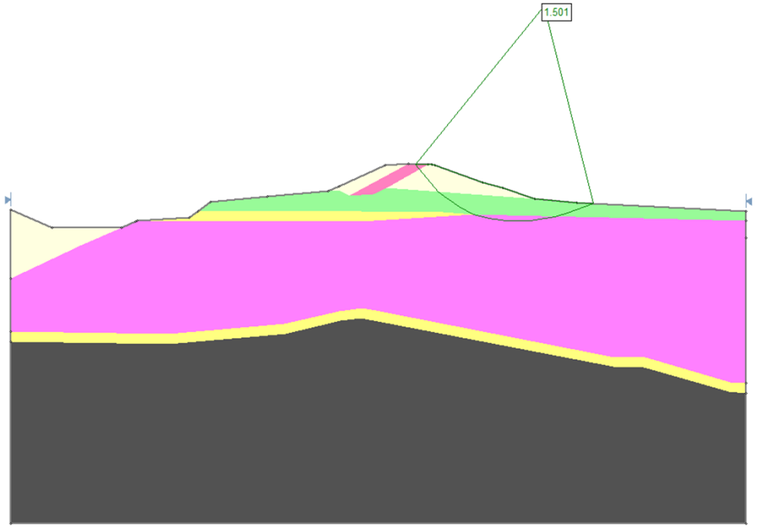
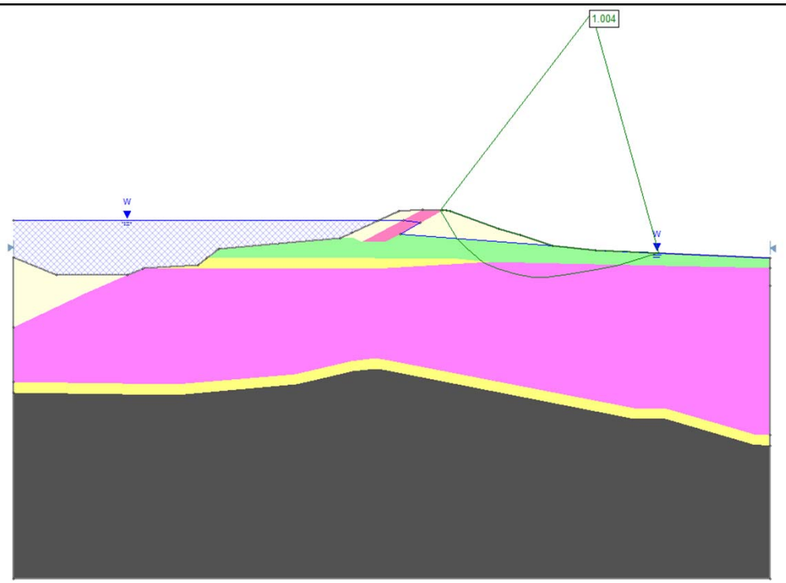
		Section B Stability Analyses
Job No: 1CM002.018.0310 Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx	Minto Mine	Date: May 2014 / Approved: MJM / Attachment: **2C**



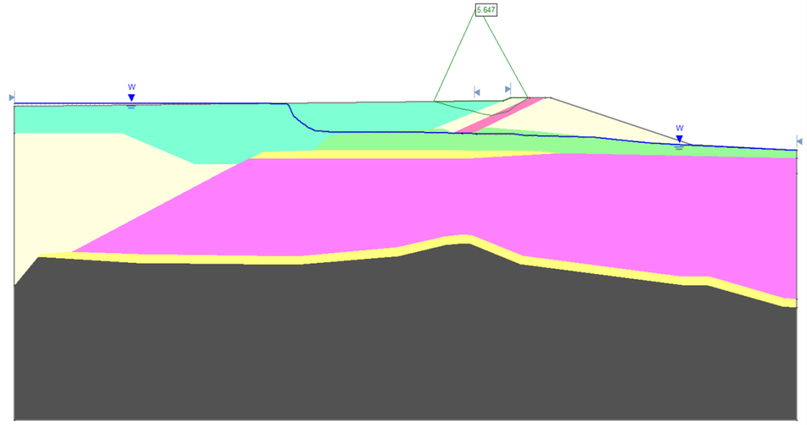
Model run 13; Downstream slide; Pseudo-Static Seismicity; Drained pore-water condition; 807 masl dam

Factor of Safety: 1.50



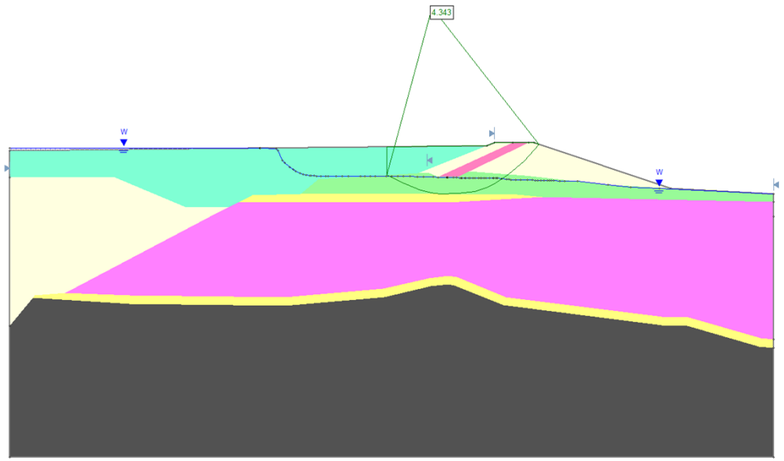
Model run 14; Downstream slide; Pseudo-Static Seismicity; Flooded Pit pore-water condition; 807 masl dam

Factor of Safety: 1.00



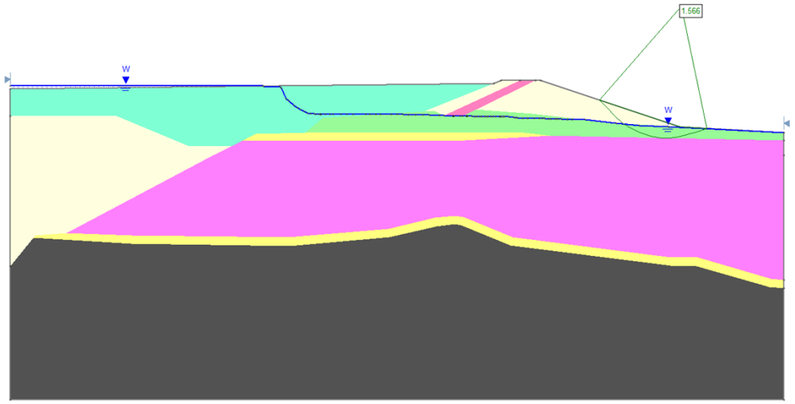
Model run 15; Upstream slide; Static Seismicity; Water Table pore-water condition; 812 masl dam with tailings

Factor of Safety: 5.65



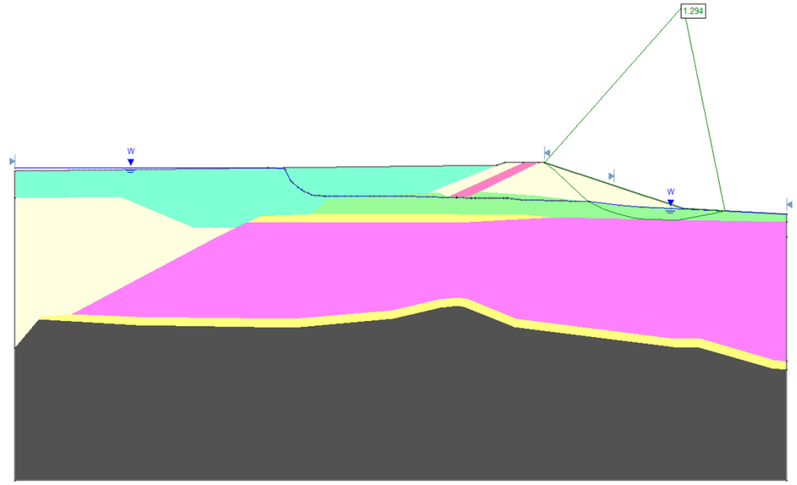
Model run 16; Upstream slide; Pseudo-Static Seismicity; Water Table pore-water condition; 812 masl dam with tailings

Factor of Safety: 4.34



Model run 17; Downstream slide; Static Seismicity; Water Table pore-water condition; 812 masl dam with tailings

Factor of Safety: 1.57



Model run 18; Downstream slide; Pseudo-Static Seismicity; Water Table pore-water condition; 812 masl dam with tailings

Factor of Safety: 1.29

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Job No: 1CM002.018.0310

Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx

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Minto Mine

Main Dam Stability

Section B Stability Analyses

Date:	Approved:	Attachment:
May 2014	MJM	**2D**

Natural Slope

FOS 1.48

Model:	Cohesion	Cu = 0
	Friction Angle	Φ = 10
	Pore-water Pressure Ratio	Ru = 0.60

As-built

FOS 2.02

Model:	Cohesion	Cu = 0
	Friction Angle	Φ = 10
	Pore-water Pressure Ratio	Ru = 0.60

Tailings

FOS 2.05

Model:	Cohesion	Cu = 0
	Friction Angle	Φ = 10
	Pore-water Pressure Ratio	Ru = 0.60

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Job No: 1CM002.018.0310

Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx

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Minto Mine

Main Dam Stability

Deep Continuous Weak Zone Shear Zone (Ru pore pressure)

Date:	Approved:	Attachment:
May 2014	MJM	**2E**

Natural Slope

FOS 1.44

Model:	Cohesion	Cu = 47.5
	Friction Angle	Φ = 0
	Pore-water Pressure Ratio	Ru = 0.60

As-built

FOS 1.81

Model:	Cohesion	Cu = 47.5
	Friction Angle	Φ = 0
	Pore-water Pressure Ratio	Ru = 0.60

Tailings

FOS 1.79

Model:	Cohesion	Cu = 47.5
	Friction Angle	Φ = 0
	Pore-water Pressure Ratio	Ru = 0.60

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Job No: 1CM002.018.0310

Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx

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Minto Mine

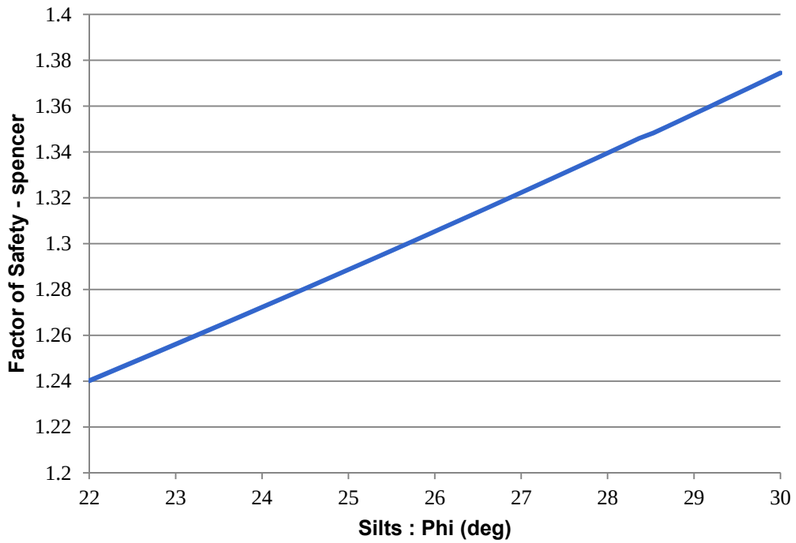
Main Dam Stability

Deep Continuous Weak Zone Shear Zone (Undrained)

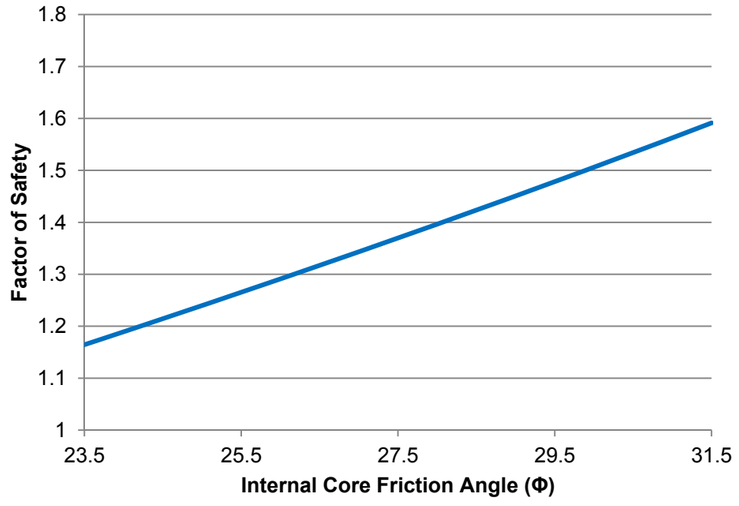
Date:	Approved:	Attachment:
May 2014	MJM	**2F**

Friction Angle Sensitivity (Φ)

Silt Foundation Material (Section B)

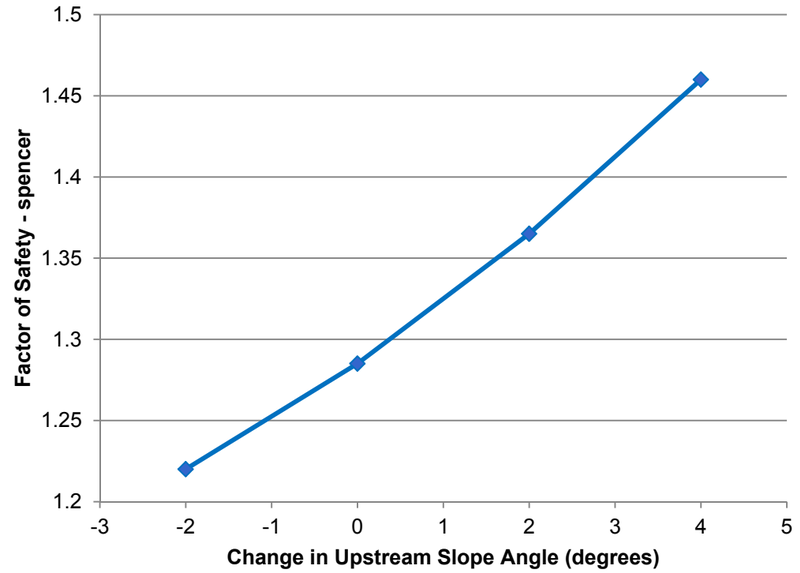


Internal Core Material (Section A)

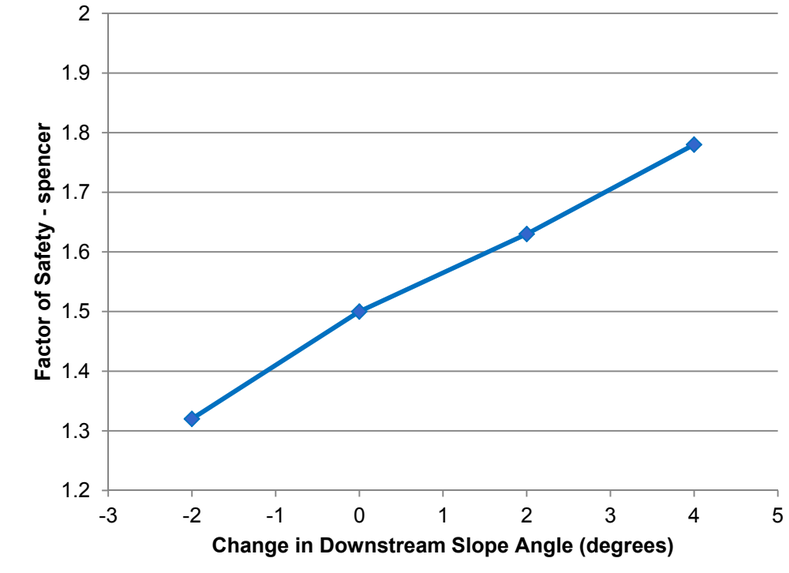


Slope Angle Sensitivity

Upstream Slope (Section B)



Downstream Slope (Section B)



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Job No: 1CM002.018.0310

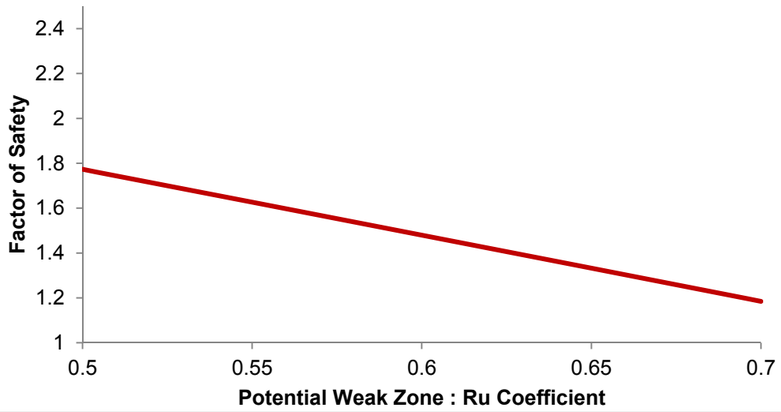
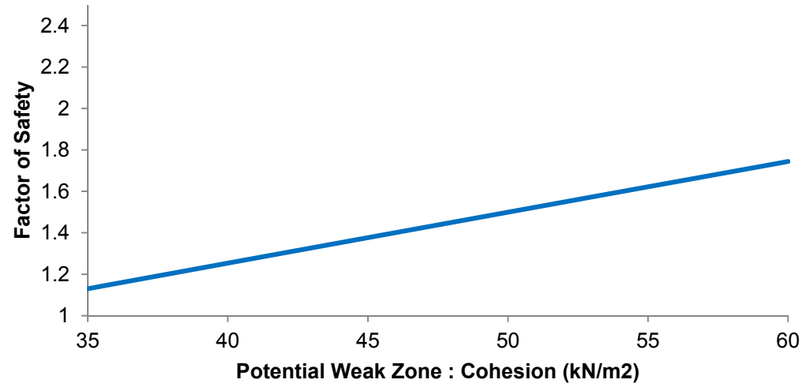
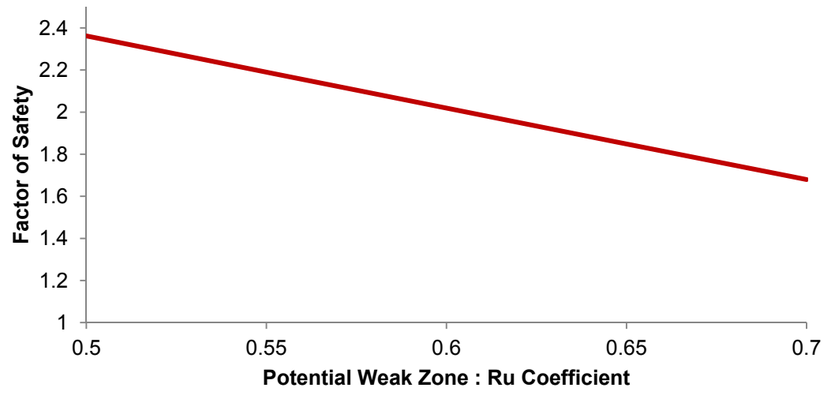
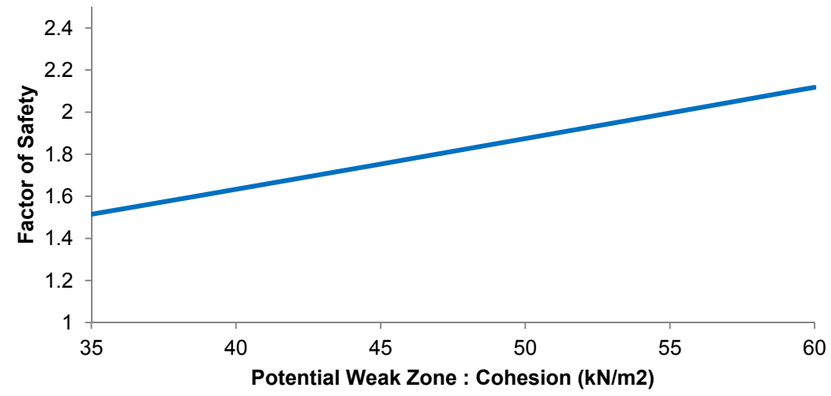
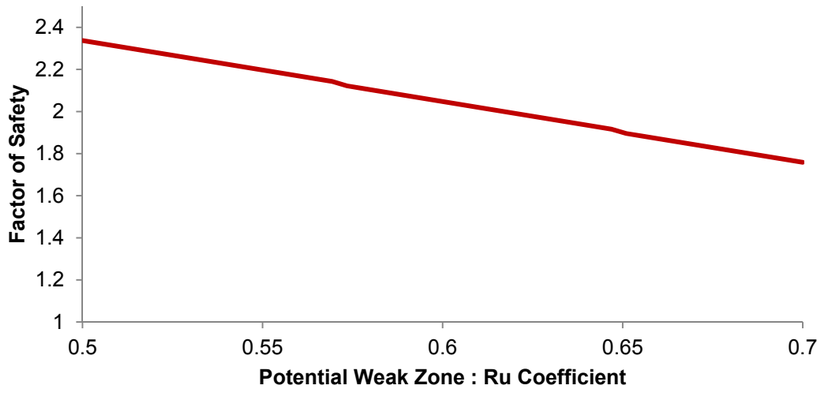
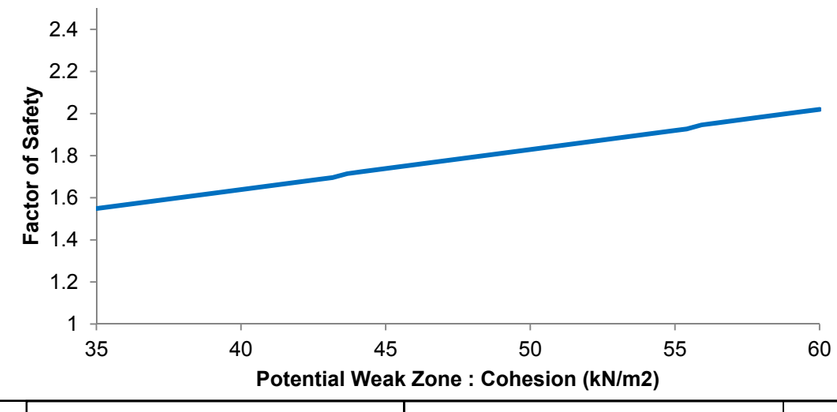
Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx

Minto Mine

Main Dam Stability

Stability Analyses Sensitivity Section A/B

Date:	Approved:	Attachment:
May 2014	MJM	**2G**

	Pore-water Pressure Ratio Sensitivity(Ru)	Cohesion Sensitivity (Cu)
Pre-mining slope		
Current slope		
Tailings slope		

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Job No: 1CM002.018.0310

Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx

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Minto Mine

Main Dam Stability

Stability Analyses Sensitivity Section C

Date:	Approved:	Figure:
May 2014	MJM	**2H**

Appendix L: Minto Main Dam – Assessment of Potential Impacts Associated wth Liquefaction

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www.srk.com

Memo

To:	File	**Client:**	Minto Exploration Inc.
From:	Murray McGregor	**Project No:**	1CM002.018
Cc:	Maritz Rykaart, SRK	**Date:**	July 15, 2014
Subject:	Main Dam – Assessment of Potential Impacts associated with Liquefaction		

1 Introduction

During discussions with stakeholders in relation to the preliminary design of the Main Pit Dam for tailings storage, the potential liquefaction of tailings was raised as an issue that should be considered in the dam design. It was agreed; therefore, that a preliminary assessment of the likelihood of, and potential impact from, tailings liquefaction would be undertaken.

This memo presents a preliminary assessment of the liquefaction potential of the tailings and the potential impacts of tailings liquefaction during operations and closure.

2 Liquefaction

2.1 Seismic Hazard

Based on the preliminary dam hazard classification of “very high”, the Annual Exceedance Probability earthquake design ground motion appropriate for use in the design of the Main Dam is the 1/5,000 year event (CDA 2007).

An assessment of peak ground acceleration (PGA) was completed using the 2010 National Building Code Seismic Hazard Calculations (NRCC) provided in Attachment 1. The 2010 NBCSHC provides PGA and specific spectral acceleration values for seismic events with probability of exceedance ranging from 1/100 (0.01) to 1/2,475 (0.000404). The seismic parameters were extrapolated for the 1/5,000 year event as shown in Attachment 2. The seismic design parameters are presented in Table 1.

Table 1: Site Class C Ground Motion Parameters for 1/5,000 Event

Spectral Period(s) or Peak Parameters	Seismic Values (g)
0.2	0.285
0.5	0.175
1.0	0.100
2.0	0.060
PGA	0.145

Note: The values shown above have been extrapolated from NBCSHC data as shown in Attachment 2.

2.2 Liquefaction Potential

2.2.1 Tailings During Operations

Note that, rather than undertaking a SHAKE analysis based on assumed tailings densities, the approach that has been taken for this assessment is to assume the tailings will liquefy in response to the design earthquake. Liquefied tailings would have a density higher than that of water. Regardless, the preliminary dam design conservatively assumes that water will be impounded against the dam, i.e. it is being designed as a water dam. Furthermore, the stability of the upstream slope with ponded water has been evaluated in the slope stability analysis memo (SRK 2014), and the results of that evaluation confirmed that tailings liquefaction will have no impact on the dam performance.

2.2.2 Tailings Post-closure

It is expected that the tailings will require some time before consolidation and close-out of the facility is completed. The current closure plan is still in draft and details on the necessity of a cover system are to be determined. However, the presence of the tailings closure cover (if one is constructed), would not prevent liquefaction from occurring in accordance with the design earthquake for closure. If an earthquake of significant magnitude occurs after the tailings facility has been closed out, the surface of the tailings facility can be expected to settle. Liquefaction of the tailings will almost certainly lead to the formation of boils on the surface. It is not uncommon for tailings boils to occur in covers post-closure. An adaptive management would be an appropriate way to deal with this occurrence. The tailings facility will have an increased density after an earthquake leaving it in a more stable configuration; therefore, the extent of boil liquefaction and boil generation would diminish after each major earthquake event.

3 Conclusions

There is currently insufficient data to assess the tailings liquefaction potential. For conservatism, it has been assumed that the tailings will liquefy. The consequences of tailings liquefaction are negligible since the dam has been designed to withstand a more conservative scenario of retaining water. Tailings liquefaction may lead to differential settlement of tailings and possible boil generation; however, these are considered to be closure issues and do not affect the overall performance of the tailings management facility. No further investigation is warranted at this stage.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

4 References

Canadian Dam Association [CDA], 2007. Canadian Dam Safety Guidelines.

[NRCC] National Research Council of Canada. 2013. 2010 National Building Code of Canada Seismic hazard calculator July 23, 2013. Available at: http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2010-eng.php (Accessed August 2, 2013).

SRK Consulting (Canada) Inc. 2014. Main Dam Stability Analysis. Prepared for Minto Explorations Ltd. SRK Project Number 1CM002.018. July 2014.

Attachment 1: 2010 National Building Code Seismic Hazard Calculation

2010 National Building Code Seismic Hazard Calculation

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

Requested by: , SRK Consulting

August 02, 2013

Site Coordinates: 62.6064 North 137.2523 West

User File Reference: Minto Mine

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

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.213	0.132	0.077	0.048	0.110

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.051	0.104	0.144
Sa(0.5)	0.037	0.070	0.093
Sa(1.0)	0.025	0.045	0.057
Sa(2.0)	0.017	0.029	0.036
PGA	0.028	0.057	0.077

References

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

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

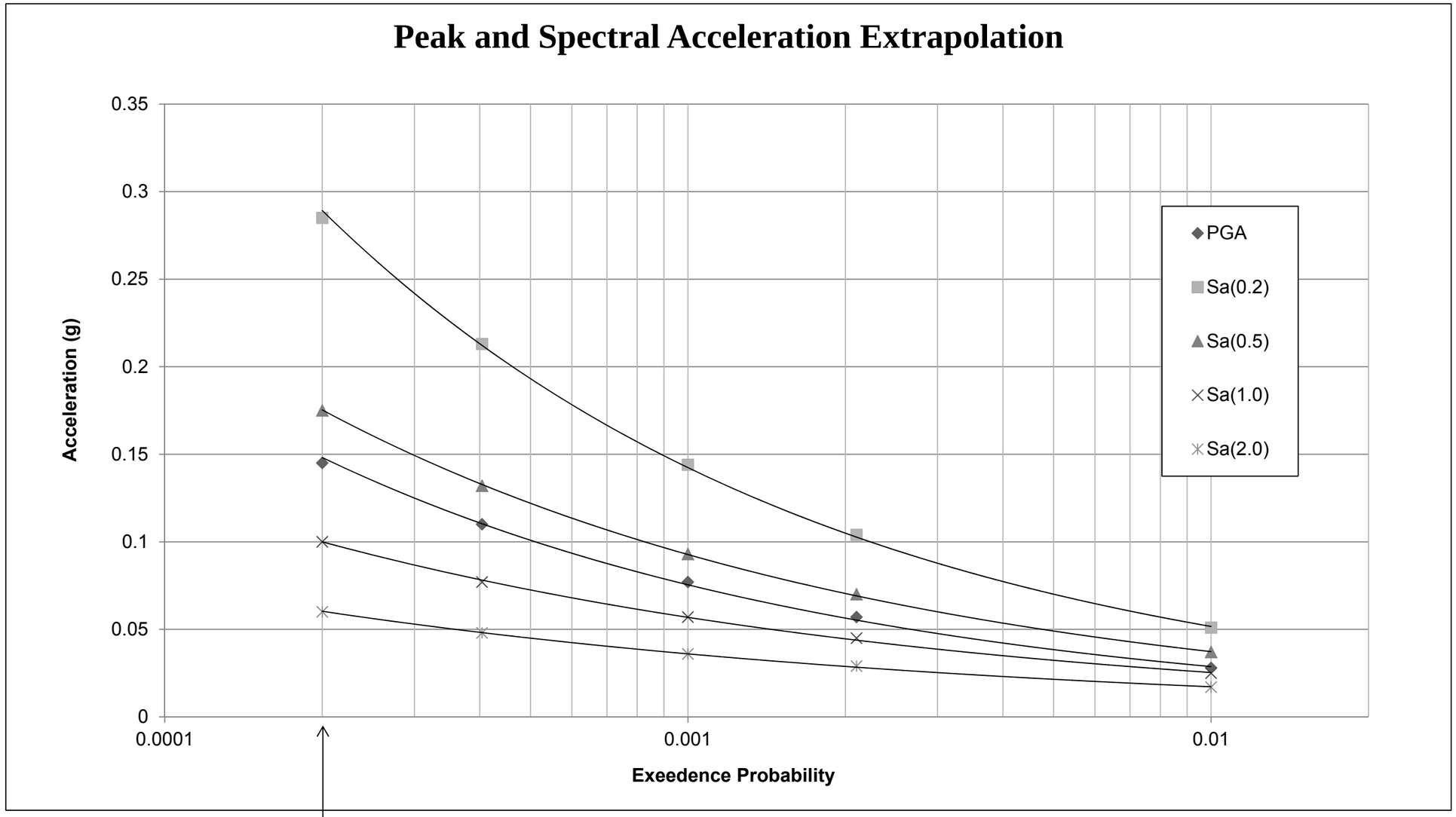
Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

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

Aussi disponible en français

Attachment 2: Peak and Spectral Acceleration Extrapolation

Peak and Spectral Acceleration Extrapolation



Source: J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\!080_Deliverables\Preliminary Dam Design Report\Appendicies\Slope Stability Memo\Attachments

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Stability
		Earthquake Ground Motion Parameter Selection
Job No: 1CM002.018.0310 Filename: Att_DamSlope_11x17land_Rev02_PL_MJM.pptx	Minto Mine	Date: May 2014 / Approved: MJM / Attachment: **2**

Appendix M: Minto Mine - Main Pit Tailings Consolidation

SRK Consulting (Canada) Inc.
2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

T: +1.604.681.4196
F: +1.604.687.5532

vancouver@srk.com
www.srk.com

Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Peter Luedke	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 15, 2014
Subject:	Main Pit Tailings Consolidation		

1 Introduction

The proposed Phase V/VI Main Pit Tailings Management Facility (TMF) would contain tailings up to a maximum depth of 93 m at a full storage capacity, with an 812 m Main Dam crest elevation, and an 809 m maximum tailings elevation. This memo presents the results of conservative preliminary consolidation modelling of tailings in the Main Pit. Ice entrainment was not considered in this analysis.

2 Tailings Consolidation Modeling

2.1 Method of Analysis

The analysis was completed with FSConsol, created by GWP Software Inc. This software uses finite strain consolidation theory to describe large strain consolidation problems with non-linear soil properties (GWPP, 2007). FSConsol also allows varied tailings discharge rates and staged-storage capacities. As FSConsol is a one-dimensional model, the maximum consolidation was calculated at the surface of the tailings where the depth is the greatest (i.e. 93 m).

The FSConsol model predicts consolidation of uniform tailings discharged continuously at a specified rate which can be varied over time. As FSConsol analyzes a single column of soil (or tailings), the deposition rate in this column has an inversely proportional relationship with the surface area of the tailings facility at a given elevation. Updating the model with actual filling rates would allow for an improved predictive model; however, this was not deemed necessary at this stage of design.

2.2 Model Inputs

The soil properties were based on consolidation test results completed by Klohn Crippen Berger (KCB) on samples from underflow tailings in December 2012. The consolidation tests were completed on two different samples, after a 55% and 60% solids content settling test, respectively. The base case tailings properties used in the FSConsol model were the 55% solids content consolidation data; however, a sensitivity analysis was run using the 60% solids content

data. This was deemed more conservative input, since with lower solids content, the tailings will likely settle more after discharging at a higher rate. The tailings test results are provided in Attachment 1.

A stage-storage capacity curve was generated based on a TMF model at five meter intervals. For the purposes of modelling, the stage-storage capacity curve was simplified by averaging the change in area over approximately 20 m intervals. The simplified capacity curve is presented in Figure 1. An assumed tailings deposition rate of 7,445 metric tons per day was specified based on a mill throughput of 4,200 dry metric tonnes and a tailings solids content of 55%.

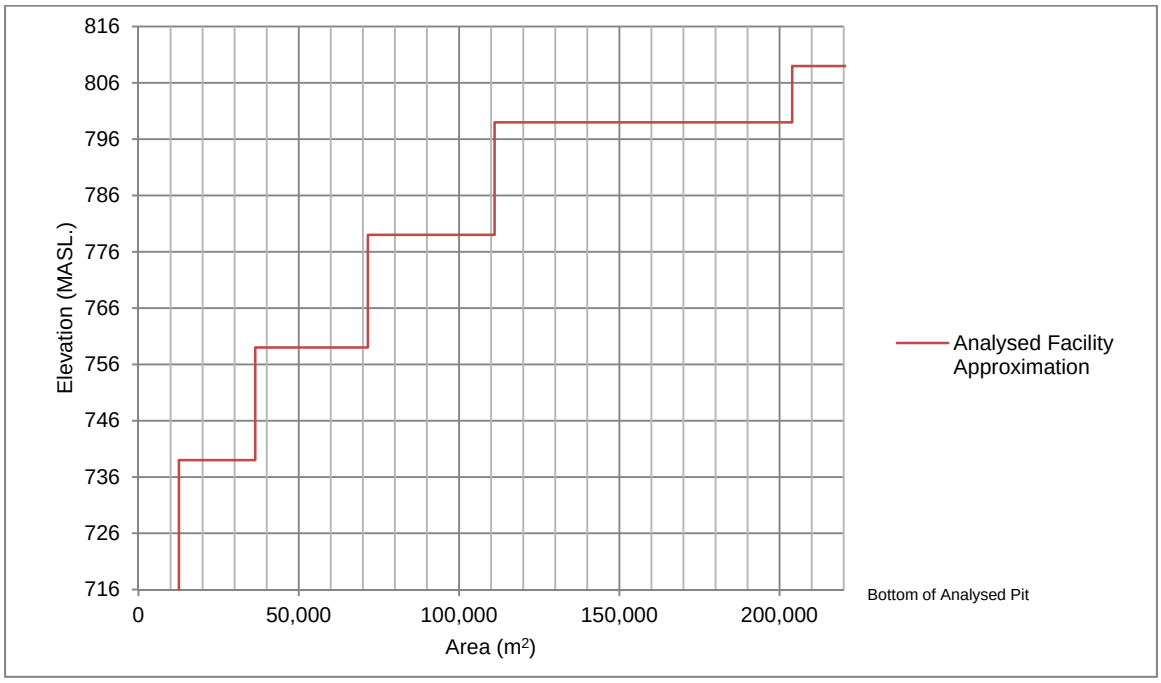


Figure 1: Approximated Stage-Storage Capacity Curve

Source: \\ van-svr0\Projects\01_SITES\Minto1CM002.018_MainDam_PrelimDesign\Stability_+_Seepage_Analysis\Tailings Consolidation\Minto_FSConsolCompiledData_1CM002.018_rev01_PL.xlsx

2.3 Results

The model results show the facility reaches an elevation of 809 m (maximum tailings storage capacity) in 6.5 years with a milling rate of 4,200 dry metric tonnes per day at 55% solids content. Deposition of hydraulic tailings was modelled to start on November 1, 2012. Following tailings deposition, the tailings are estimated to consolidate by about 3.7 m after 40 years (approximately 33 years after end of deposition) and 8.5 m after 100 years (approximately 93 years after end of deposition). The consolidation settlement is summarized in Table 1.

Table 1: Tailings Surface Elevation Over Time

Year	Change in Tailing Surface Elevation (m) @ 55% Solids Content	Change in Tailing Surface Elevation (m) @ 60% Solids Content
0	Deposition begins	Deposition begins
6.4	Deposition complete	Deposition complete
40	-3.7	-4.9
100	-8.5	-7.6

The settlement results listed in Table 1 are predicted at the deepest portion of the facility with a full tailings depth of 93 m. Settlement will be proportionally less in shallower zones of the facility. The modelled Main Pit TMF filling curve and the consolidation is shown in Figures 2 and 3 for both 55% and 60% solids content tailings.p

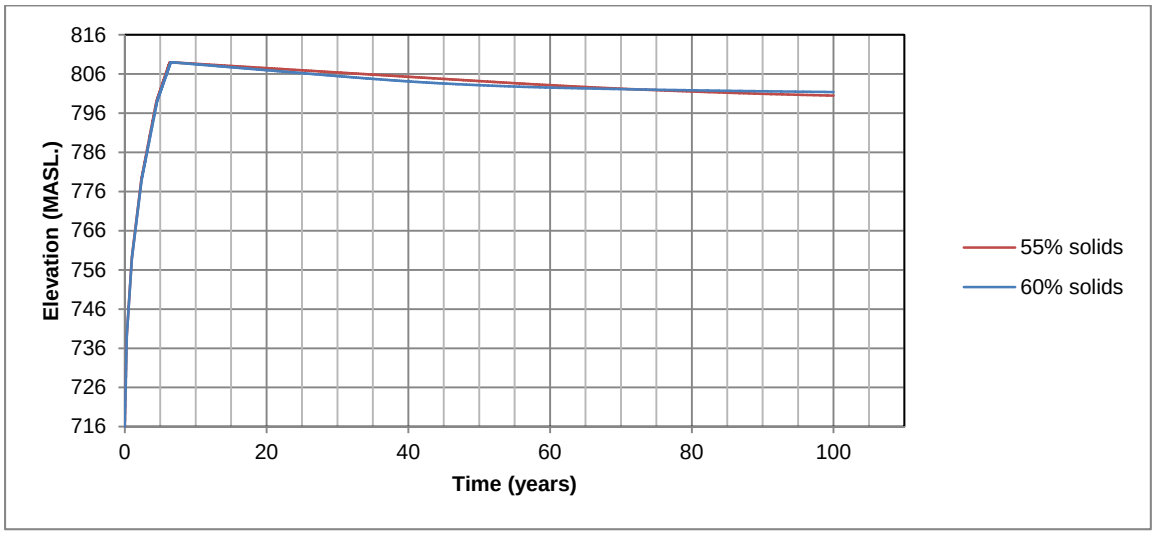


Figure 2: Predicted Tailings Surface Elevation

Source: \\VAN-SVR0\Projects\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Stability_+_Seepage_Analysis\Tailings Consolidation\FSConsol Output\Updated model_20140520\Minto_FSCdata_4200tpd_1CM002.018_rev02_20140520.xlsx

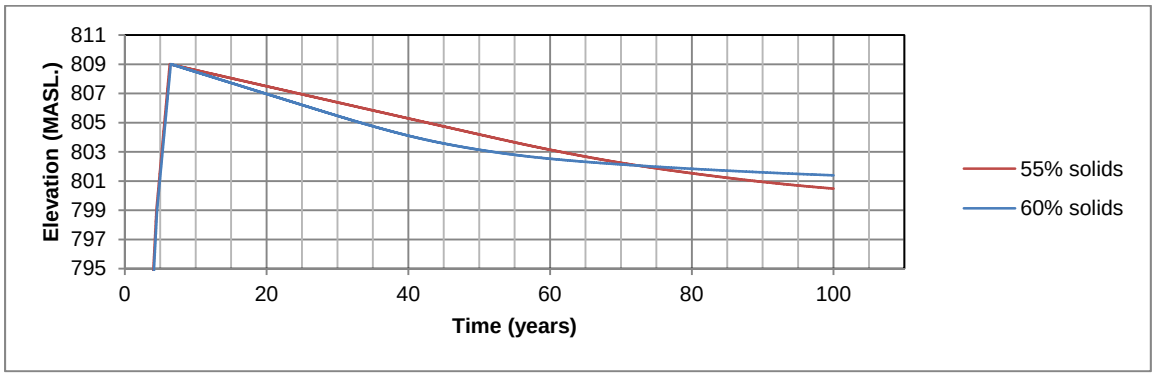


Figure 3: Predicted Tailings Surface Elevation (Close-up of Consolidation)

Source: \\VAN-SVR0\Projects\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\Stability_+_Seepage_Analysis\Tailings Consolidation\FSConsol Output\Updated model_20140520\Minto_FSCdata_4200tpd_1CM002.018_rev02_20140520.xlsx

3 Summary

The numeric assessment of total tailings consolidation for the Main Pit tailings impoundment suggests long-term consolidation settlement of between 7.6 and 8.5 m for the maximum depth of tailings deposited in Main Pit of 93 m. Although this analysis is overly simplistic and inherently conservative, it provides an early indication of how the tailings surface will deform over time. This is important in assessing the long-term tailings closure planning, as well as taking into consideration the long-term thaw settlement of the Main Dam.

Prior to closing of the facility, additional detailed tailings characterization should be completed, and this assessment should be updated and refined to take into consideration actual 3D geometry of the impoundment, variability of tailings properties, as well as ice entrainment.

Disclaimer—SRK Consulting (Canada) Inc. has prepared this document for Minto Explorations Ltd.. Any use or decisions by which a third party makes of this document are the responsibility of such third parties. In no circumstance does SRK accept any consequential liability arising from commercial decisions or actions resulting from the use of this report by a third party.

The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

4 References

GWP Software Inc. (2007). *FSConsol Slurry Consolidation Software Users Manual.* Edmonton, AB: GWP Software Inc.

Attachment 1 - Tailings Test Results

Klohn Crippen Berger

January 16, 2013

Capstone Mining Corp.
P.O. Box 33174
Whitehorse, Yukon
Y1A 6S1

Mr. Eamon Mauer
Minto Mine Geotech and Reclamation Engineer

Dear Mr. Mauer:

Minto Project
Tailings Lab Testing

Please find the results of the laboratory tailings testing attached to this cover letter. More specifically, this includes:

- Consolidation Test Results;
- Grain Size Distribution Results;
- Settling Test Results; and
- Specific Gravity of Soil Solids Results.

If you have any questions or comments about the attached laboratory tailings testing results, please feel free to contact me by phone (604-251-8438) or email (bchambers@klohn.com).

Thank you for the opportunity to complete the above scope of work, we look forward to continuing to provide our services to you in the future.

Yours truly,
KLOHN CRIPPEN BERGER LTD.

Robert W. Chambers, P.Eng.
Staff Consultant

DH:cd

130116L-Tailings Lab Testing .docx
M09638A03.732

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APPENDIX I

Consolidation Test Results

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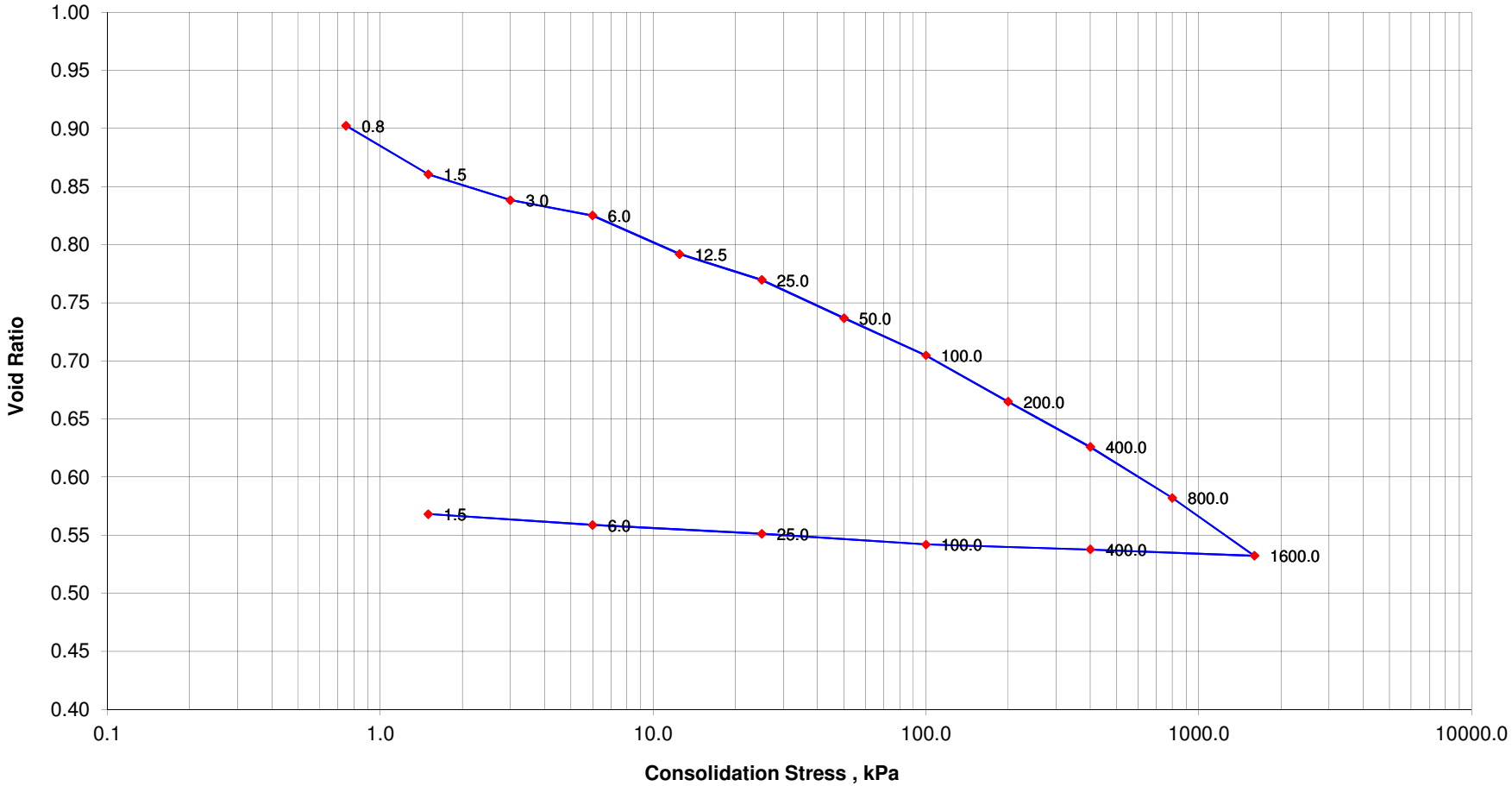
Consolidation Test - Tailings after 55% Solids Content Settling Test

e - log(p)



Klohn Crippen Berger	PROJECT NO.:	M09638A03		
	PROJECT:	Minto Mine		
	LOCATION:	YK	DATE TESTED:	15-Dec-2012
	SAMPLE NO.:	Underflow Tailings	DEPTH:	After 55% solids content settling test
	TESTED BY:	BY	CHECKED BY:	JG

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CONSOLIDATION

PROJECT NO.:	M09638A03
PROJECT:	Minto Mine
SAMPLE NO.:	Underflow Tailings
DETAILS	After 55% solids content settling test
	Supernatant removed
TEST NO.:	T1262CONS02R
LOADING MACHINE NO.:	OED1 / ID80

Test Specimen Information:

Initial water content:	59.15 % (based on trimmings)
Final water content:	23.17 % (based on final dry weight)
Dry mass:	84.01 g
Diameter	63.53 mm
Area	31.699 cm^2
Specific Gravity	2.72

Initial Specimen Height (mm):	25.33
Height of Solid (mm):	9.743
Initial void ratio:	1.600
Void Ratio Factor	0.1026

* Calibration to be done after test

** Estimated t_{50}

Pressure (kPa)		*Change in Height	Final	Change in	Change in	Void	t_{50**}	Cv	Mv	k	Cc
From	To	Corrected (mm)	Height (mm)	Void Ratio	Void Ratio Acc	Ratio	(min)	(cm^2/sec)	(cm^2/N)	(cm/sec)	
0.0	**0.8**	6.794	18.536	0.6973	0.6973	**0.902**					
0.8	**1.5**	0.407	18.129	0.0418	0.7391	**0.861**					
1.5	**3.0**	0.217	17.912	0.0223	0.7614	**0.838**					
3.0	**6.0**	0.129	17.783	0.0132	0.7746	**0.825**					
6.0	**12.5**	0.324	17.459	0.0333	0.8078	**0.792**	0.56	4.6E-03	6.1E-02	2.7E-06	**0.104**
12.5	**25.0**	0.217	17.242	0.0223	0.8301	**0.770**	0.20	1.2E-02	1.9E-02	2.3E-06	**0.074**
25.0	**50.0**	0.321	16.922	0.0329	0.8630	**0.737**	0.12	2.0E-02	1.5E-02	2.9E-06	**0.109**
50.0	**100.0**	0.312	16.610	0.0320	0.8950	**0.705**	0.09	2.6E-02	7.4E-03	1.9E-06	**0.106**
100.0	**200.0**	0.387	16.223	0.0397	0.9347	**0.665**	0.07	3.2E-02	4.7E-03	1.4E-06	**0.132**
200.0	**400.0**	0.382	15.841	0.0392	0.9739	**0.626**	0.05	4.2E-02	2.4E-03	9.7E-07	**0.130**
400.0	**800.0**	0.428	15.413	0.0439	1.0178	**0.582**	0.05	4.5E-02	1.3E-03	5.9E-07	**0.146**
800.0	**1600.0**	0.485	14.929	0.0497	1.0675	**0.532**	0.04	4.7E-02	7.9E-04	3.6E-07	**0.165**
1600.0	**400.0**	-0.052	14.981	-0.0054	1.0621	**0.538**					
400.0	**100.0**	-0.043	15.024	-0.0044	1.0577	**0.542**					
100.0	**25.0**	-0.089	15.113	-0.0091	1.0486	**0.551**					
25.0	**6.0**	-0.074	15.187	-0.0076	1.0410	**0.559**					
6.0	**1.5**	-0.092	15.279	-0.0094	1.0316	**0.568**					

Klohn Crippen Berger

PROJECT NO.:	M09638A03		
PROJECT:	Minto Mine		
LOCATION:	YK	DATE TESTED:	15-Dec-12
SAMPLE NO.:	Underflow Tailings	DETAILS	After 55% solids content settling test
TESTED BY:	BY	CHECKED BY:	JG

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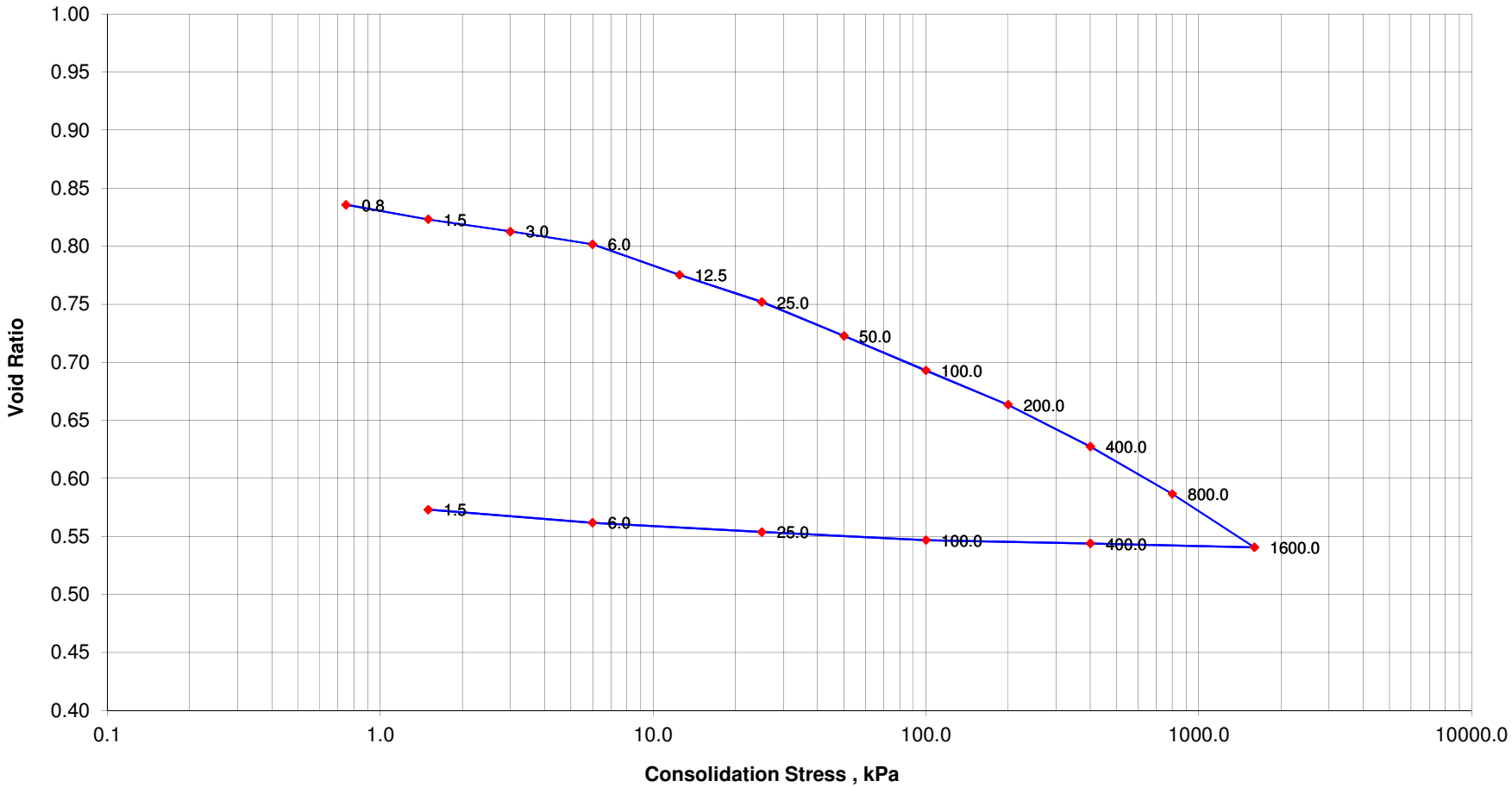
Consolidation Test - Tailings after 60% Solids Content Settling Test

e - log(p)



Klohn Crippen Berger	PROJECT NO.:	M09638A03		
	PROJECT:	Minto Mine		
	LOCATION:	YK	DATE TESTED:	7-Dec-2012
	SAMPLE NO.:	Underflow Tailings	DEPTH:	After 60% solids content settling test
	TESTED BY:	BY	CHECKED BY:	JG

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CONSOLIDATION

PROJECT NO.:	M09638A03
PROJECT:	Minto Mine
SAMPLE NO.:	Underflow Tailings
DETAILS	After 60% solids content settling test
	Supernatant removed
TEST NO.:	T1262CONS01
LOADING MACHINE NO.:	OED1 / ID80

Test Specimen Information:

Initial water content:	46.80 % (based on trimmings)
Final water content:	21.83 % (based on final dry weight)
Dry mass:	94.16 g
Diameter	63.53 mm
Area	31.699 cm^2
Specific Gravity	2.72

Initial Specimen Height (mm):	24.71
Height of Solid (mm):	10.921
Initial void ratio:	1.263
Void Ratio Factor	0.0916

* Calibration to be done after test

** Estimated t_{50}

Pressure (kPa)		*Change in Height	Final	Change in	Change in	Void	t_{50}**	Cv	Mv	k	Cc
From	To	Corrected (mm)	Height (mm)	Void Ratio	Void Ratio Acc	Ratio	(min)	(cm^2/sec)	(cm^2/N)	(cm/sec)	
0.0	**0.8**	4.664	20.047	0.4270	0.4270	**0.836**					
0.8	**1.5**	0.138	19.909	0.0126	0.4397	**0.823**					
1.5	**3.0**	0.113	19.796	0.0103	0.4500	**0.813**					
3.0	**6.0**	0.121	19.675	0.0111	0.4611	**0.802**					
6.0	**12.5**	0.287	19.388	0.0263	0.4874	**0.775**	0.40	7.8E-03	4.9E-02	3.7E-06	**0.082**
12.5	**25.0**	0.256	19.132	0.0234	0.5108	**0.752**	0.20	1.5E-02	2.0E-02	3.0E-06	**0.078**
25.0	**50.0**	0.320	18.812	0.0293	0.5401	**0.723**	0.14	2.1E-02	1.3E-02	2.8E-06	**0.097**
50.0	**100.0**	0.325	18.487	0.0297	0.5698	**0.693**	0.15	1.9E-02	6.9E-03	1.3E-06	**0.099**
100.0	**200.0**	0.323	18.164	0.0296	0.5994	**0.663**	0.08	3.7E-02	3.5E-03	1.3E-06	**0.098**
200.0	**400.0**	0.393	17.771	0.0360	0.6354	**0.627**	0.06	4.7E-02	2.2E-03	1.0E-06	**0.119**
400.0	**800.0**	0.446	17.326	0.0408	0.6762	**0.587**	0.05	5.5E-02	1.3E-03	6.8E-07	**0.136**
800.0	**1600.0**	0.503	16.823	0.0460	0.7222	**0.541**	0.05	5.3E-02	7.3E-04	3.8E-07	**0.153**
1600.0	**400.0**	-0.036	16.860	-0.0033	0.7188	**0.544**					
400.0	**100.0**	-0.032	16.892	-0.0029	0.7159	**0.547**					
100.0	**25.0**	-0.077	16.969	-0.0070	0.7089	**0.554**					
25.0	**6.0**	-0.086	17.055	-0.0079	0.7010	**0.562**					
6.0	**1.5**	-0.123	17.178	-0.0112	0.6897	**0.573**					

Klohn Crippen Berger

PROJECT NO.:	M09638A03		
PROJECT:	Minto Mine		
LOCATION:	YK	DATE TESTED:	7-Dec-12
SAMPLE NO.:	Underflow Tailings	DETAILS	After 60% solids content settling test
TESTED BY:	BY	CHECKED BY:	JG

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APPENDIX II

Grain Size Distribution Results

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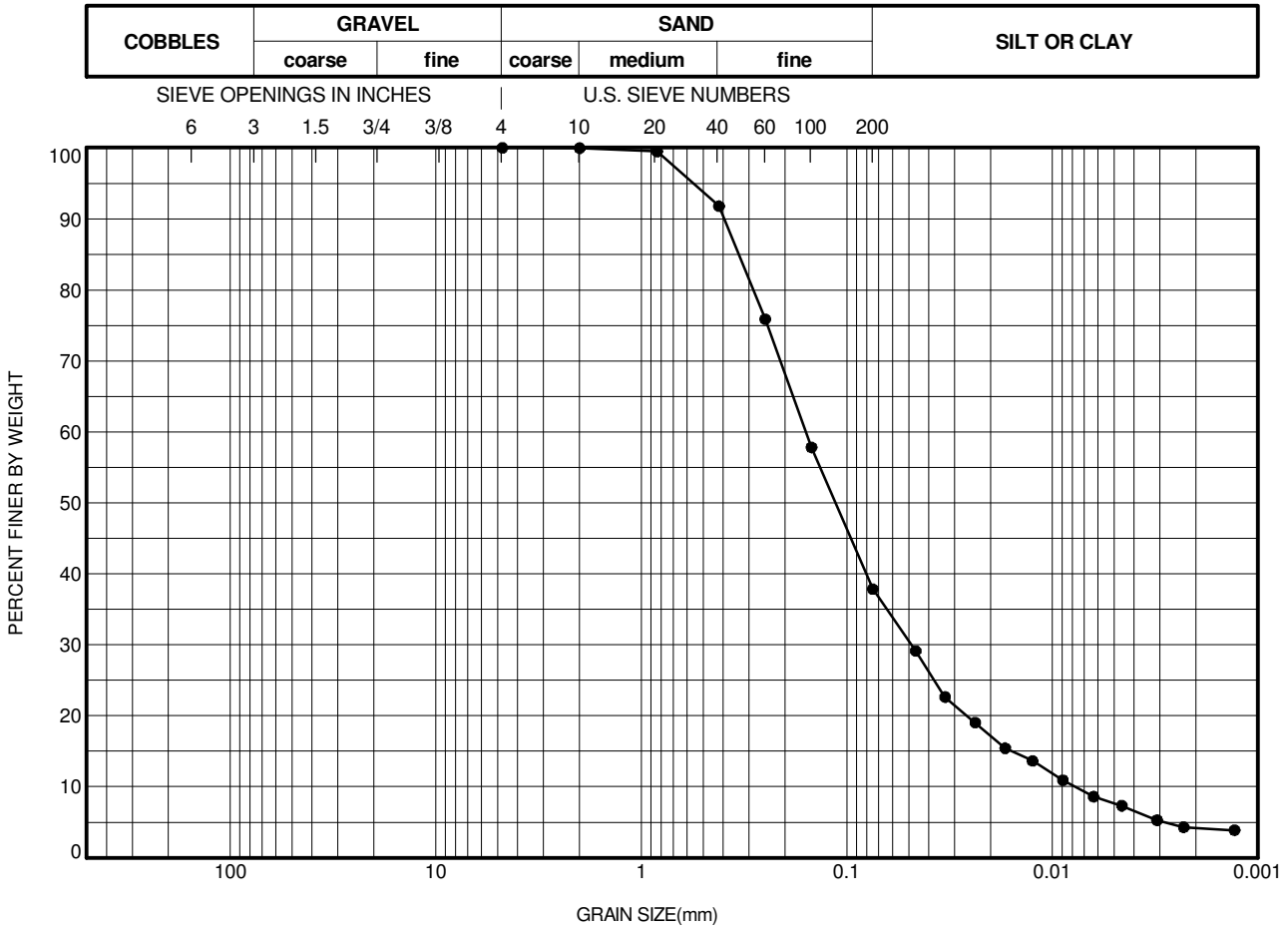
GRAIN SIZE DISTRIBUTION



	HOLE	DEPTH (m)	D85	D60	D50	D15	D10	CU	%GRAVEL	%SAND	%FINES
●	Tailings	0.00	0.337	0.159	0.114				0.0	62.1	37.9

	HOLE	SAMPLE	DEPTH (m)	W%	W_L	W_P	PI	REMARKS / SAMPLE DESCRIPTION
●	Tailings	Underflow	0.00					

CU = COEFFICIENT OF UNIFORMITY = D60/D10 **PARTICLE SIZES, e.g. D85, in mm** **Tested by Wet Sieving Method (ASTM D1140 & D422)**

Klohn Crippen Berger

PROJECT NO.: M09638A03
PROJECT: Minto Mine
LOCATION: Yukon
FIGURE:
DRAWN BY: FG **CHECKED BY:** BY

KCB_GRAIN_SIZE-SI M09638A03 - MINTO MINE T1262.GPJ SIEVE.GDT 12/18/12

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APPENDIX III

Setting Test Results

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SETTLING TEST (using 2L graduated standard beaker)

Beaker #	ST-20-2000
Beaker Tare (g)	509.66
Proposed Solid Content (%)	55%
Supernatant Density (g/cm^3)	1
Start Date	16-Dec-12
Start Time	4:25PM
Temperature (°C)	24.0

Initial Weight + Tare (g)	3718.37	Final Weight + Tare (g)	3715.98
Intial Dry weight (g)	1748.75	Final Dry weight (g)	1748.75
Initial Solid Volume (cm^3)	2091.25	Final Solid Volume (cm^3)	1613.14
Initial Solid Content (%)	54.50%	Final Solid Content (%)	64.04%
Initial Water Content (%)	83.49%	Final Water Content (%)	56.15%
Initial Dry Density (g/cm^3)	0.836	Final Dry Density (g/cm^3)	1.084
Initial Wet Density (g/cm^3)	1.534	Final Wet Density (g/cm^3)	1.693

		Readings											
Elapsed Time (min)	Temp. (°C)	Top Supernatant Readings	Top Solids Readings	Top Settlement Readings	*Height Supernatant (mm)	Volume Supernatant (cm^3)	Volume Supernatant Variation (cm^3)	*Height Solids (settlement) (mm)	Volume Solids (settlement) (cm^3)	Wet Density (g/cm^3)	Dry Density (g/cm^3)	Water Content (%)	Solid Content (%)
0	24.0	254.0	254.0	254.0	0.00	0.00	2091.25	164.77	2091.25	1.534	0.836	83.49%	54.50%
4		254.0	254.0	254.0	0.00	0.00	2091.25	164.77	2091.25	1.534	0.836	83.49%	54.50%
5		254.0	253.0	253.0	0.97	12.42	2078.83	163.80	2078.83	1.538	0.841	82.78%	54.71%
7		254.0	252.0	252.0	1.95	24.84	2066.41	162.82	2066.41	1.541	0.846	82.07%	54.93%
8		254.0	251.5	251.5	2.43	31.05	2060.20	162.34	2060.20	1.542	0.849	81.71%	55.03%
9		254.0	251.0	251.0	2.92	37.25	2053.99	161.85	2053.99	1.544	0.851	81.36%	55.14%
10		254.0	250.5	250.5	3.40	43.46	2047.78	161.37	2047.78	1.546	0.854	81.00%	55.25%
11		254.0	250.0	250.0	3.89	49.67	2041.57	160.88	2041.57	1.547	0.857	80.65%	55.36%
12		254.0	249.5	249.5	4.38	55.88	2035.36	160.39	2035.36	1.549	0.859	80.29%	55.47%
13		254.0	249.0	249.0	4.86	62.09	2029.15	159.91	2029.15	1.551	0.862	79.94%	55.58%
14		254.0	248.5	248.5	5.35	68.30	2022.94	159.42	2022.94	1.552	0.864	79.58%	55.69%
15		254.0	248.0	248.0	5.84	74.51	2016.74	158.93	2016.74	1.554	0.867	79.23%	55.80%
16		254.0	247.5	247.5	6.32	80.72	2010.53	158.45	2010.53	1.556	0.870	78.87%	55.91%
17		254.0	247.0	247.0	6.81	86.93	2004.32	157.96	2004.32	1.558	0.872	78.52%	56.02%
18		254.0	246.5	246.5	7.29	93.14	1998.11	157.48	1998.11	1.559	0.875	78.16%	56.13%
19		254.0	246.0	246.0	7.78	99.35	1991.90	156.99	1991.90	1.561	0.878	77.81%	56.24%
20		254.0	245.5	245.5	8.27	105.56	1985.69	156.50	1985.69	1.563	0.881	77.45%	56.35%
21		254.0	245.0	245.0	8.75	111.76	1979.48	156.02	1979.48	1.565	0.883	77.10%	56.47%
22		254.0	244.5	244.5	9.24	117.97	1973.27	155.53	1973.27	1.566	0.886	76.74%	56.58%
23		254.0	244.0	244.0	9.73	124.18	1967.06	155.04	1967.06	1.568	0.889	76.38%	56.69%
24		254.0	243.5	243.5	10.21	130.39	1960.85	154.56	1960.85	1.570	0.892	76.03%	56.81%
25		254.0	243.0	243.0	10.70	136.60	1954.64	154.07	1954.64	1.572	0.895	75.67%	56.92%
26		254.0	242.5	242.5	11.18	142.81	1948.43	153.59	1948.43	1.574	0.898	75.32%	57.04%
27		254.0	242.0	242.0	11.67	149.02	1942.23	153.10	1942.23	1.575	0.900	74.96%	57.15%
28		254.0	241.0	241.0	12.64	161.44	1929.81	152.13	1929.81	1.579	0.906	74.25%	57.39%
29		254.0	240.0	240.0	13.62	173.86	1917.39	151.15	1917.39	1.583	0.912	73.54%	57.62%
30		254.0	239.0	239.0	14.59	186.27	1904.97	150.18	1904.97	1.587	0.918	72.83%	57.86%
31		254.0	238.0	238.0	15.56	198.69	1892.55	149.21	1892.55	1.590	0.924	72.12%	58.10%
32		254.0	237.0	237.0	16.53	211.11	1880.13	148.24	1880.13	1.594	0.930	71.41%	58.34%
33		254.0	236.0	236.0	17.51	223.53	1867.72	147.26	1867.72	1.598	0.936	70.70%	58.58%
34		254.0	234.5	234.5	18.96	242.16	1849.09	145.81	1849.09	1.604	0.946	69.64%	58.95%
35		254.0	233.5	233.5	19.94	254.58	1836.67	144.83	1836.67	1.608	0.952	68.93%	59.20%
36		254.0	232.5	232.5	20.91	266.99	1824.25	143.86	1824.25	1.613	0.959	68.22%	59.45%
37		254.0	232.0	232.0	21.40	273.20	1818.04	143.37	1818.04	1.615	0.962	67.86%	59.57%
39		254.0	231.0	231.0	22.37	285.62	1805.62	142.40	1805.62	1.619	0.969	67.15%	59.83%
41		254.0	230.0	230.0	23.34	298.04	1793.21	141.43	1793.21	1.623	0.975	66.44%	60.08%
44		254.0	229.0	229.0	24.31	310.46	1780.79	140.46	1780.79	1.628	0.982	65.73%	60.34%
47		254.0	228.0	228.0	25.29	322.88	1768.37	139.48	1768.37	1.632	0.989	65.02%	60.60%
52		254.0	227.0	227.0	26.26	335.29	1755.95	138.51	1755.95	1.636	0.996	64.31%	60.86%
57		254.0	226.0	226.0	27.23	347.71	1743.53	137.54	1743.53	1.641	1.003	63.60%	61.12%
64		254.0	225.0	225.0	28.20	360.13	1731.11	136.57	1731.11	1.646	1.010	62.89%	61.39%
70		254.0	224.0	224.0	29.18	372.55	1718.70	135.59	1718.70	1.650	1.017	62.18%	61.66%
78		254.0	223.0	223.0	30.15	384.97	1706.28	134.62	1706.28	1.655	1.025	61.47%	61.93%
88		254.0	222.0	222.0	31.12	397.39	1693.86	133.65	1693.86	1.660	1.032	60.76%	62.20%
102		254.0	221.0	221.0	32.09	409.80	1681.44	132.68	1681.44	1.665	1.040	60.05%	62.48%
115		254.0	220.0	220.0	33.07	422.22	1669.02	131.70	1669.02	1.670	1.048	59.34%	62.76%
138		254.0	219.0	219.0	34.04	434.64	1656.60	130.73	1656.60	1.675	1.056	58.63%	63.04%
155		254.0	218.0	218.0	35.01	447.06	1644.19	129.76	1644.19	1.680	1.064	57.92%	63.32%
207		254.0	217.0	217.0	35.98	459.48	1631.77	128.79	1631.77	1.685	1.072	57.21%	63.61%
253		254.0	216.0	216.0	36.96	471.90	1619.35	127.81	1619.35	1.690	1.080	56.50%	63.90%
313		254.0	215.7	215.7	37.25	475.62	1615.62	127.52	1615.62	1.692	1.082	56.29%	63.98%
373		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%
480		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%
960		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%
1440		254.0	215.5	215.5	37.44	478.11	1613.14	127.33	1613.14	1.693	1.084	56.15%	64.04%

Klohn Crippen Berger

PROJECT NO.:	M9638A03	DATE:	16-Dec-12
PROJECT:	Minto Mine	LOCATION:	Yukon
DETAIL:	55% Solids Content		
DESCRIPTION:	Underflow Tailings		
TESTED BY:	FG	CHECKED BY:	BY

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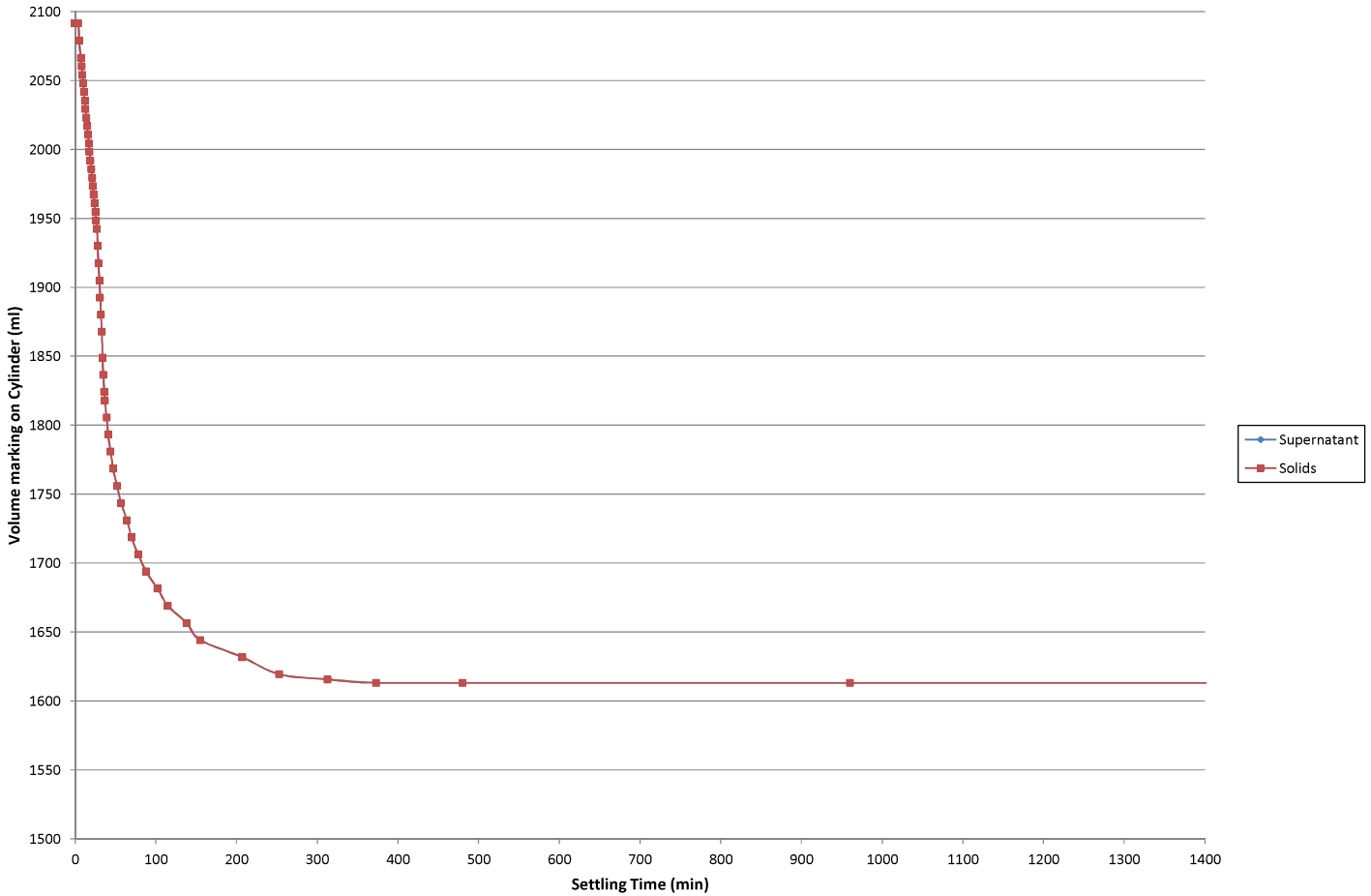
Settlement Test for Underflow Tailings at 55% Solids Content



Klohn Crippen Berger	PROJECT NO.:	M9638A03		DATE	16-Dec-12
	PROJECT:	Minto Mine		LOCATION:	Yukon
	DETAIL:	55% Solids Content			
	DESCRIPTION:	Underflow Tailings			
	TESTED BY:	FG	CHECKED BY BY	SHEET	2 of 2

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SETTLING TEST (using 2L graduated standard beaker)

Beaker #	ST-06-2000
Beaker Tare (g)	535.93
Proposed Solid Content (%)	60%
Supernatant Density (g/cm^3)	1
Start Date	3-Dec-12
Start Time	1:34 PM
Temperature (°C)	23.5

Initial Weight + Tare (g)	3650.82	Final Weight + Tare (g)	3644.20
Intial Dry weight (g)	1853.36	Final Dry weight (g)	1853.36
Initial Solid Volume (cm^3)	1976.18	Final Solid Volume (cm^3)	1521.47
Initial Solid Content (%)	59.50%	Final Solid Content (%)	69.67%
Initial Water Content (%)	68.07%	Final Water Content (%)	43.53%
Initial Dry Density (g/cm^3)	0.938	Final Dry Density (g/cm^3)	1.218
Initial Wet Density (g/cm^3)	1.576	Final Wet Density (g/cm^3)	1.748

		Readings											
Elapsed Time (min)	Temp. (°C)	Top Supernatant Readings	Top Solids Readings	Top Settlement Readings	*Height Supernatant (mm)	Volume Supernatant (cm^3)	Volume Supernatant Variation (cm^3)	*Height Solids (settlement) (mm)	Volume Solids (settlement) (cm^3)	Wet Density (g/cm^3)	Dry Density (g/cm^3)	Water Content (%)	Solid Content (%)
0	23.5	248.0	248.0	248.0	0.00	0.00	1976.18	157.38	1976.18	1.576	0.938	68.07%	59.50%
1		248.0	247.5	247.5	0.49	6.14	1970.04	156.89	1970.04	1.578	0.941	67.74%	59.62%
2		248.0	247.0	247.0	0.97	12.29	1963.89	156.41	1963.89	1.580	0.944	67.40%	59.74%
3		248.0	246.0	246.0	1.95	24.58	1951.60	155.43	1951.60	1.583	0.950	66.74%	59.97%
4		248.0	245.0	245.0	2.92	36.87	1939.31	154.46	1939.31	1.587	0.956	66.08%	60.21%
5		248.0	244.0	244.0	3.89	49.16	1927.02	153.49	1927.02	1.591	0.962	65.41%	60.45%
6		248.0	243.0	243.0	4.86	61.45	1914.74	152.52	1914.74	1.595	0.968	64.75%	60.70%
7		248.0	241.5	241.5	6.32	79.88	1896.30	151.06	1896.30	1.600	0.977	63.76%	61.07%
8		248.0	240.0	240.0	7.78	98.32	1877.87	149.60	1877.87	1.606	0.987	62.76%	61.44%
9		248.0	238.0	238.0	9.73	122.90	1853.29	147.65	1853.29	1.614	1.000	61.44%	61.94%
10		248.0	237.0	237.0	10.70	135.18	1841.00	146.68	1841.00	1.619	1.007	60.77%	62.20%
11		248.0	236.0	236.0	11.67	147.47	1828.71	145.71	1828.71	1.623	1.013	60.11%	62.46%
12		248.0	235.0	235.0	12.64	159.76	1816.42	144.74	1816.42	1.627	1.020	59.45%	62.72%
13		248.0	233.5	233.5	14.10	178.20	1797.98	143.28	1797.98	1.633	1.031	58.45%	63.11%
14		248.0	232.0	232.0	15.56	196.63	1779.55	141.82	1779.55	1.640	1.041	57.46%	63.51%
15		248.0	230.5	230.5	17.02	215.07	1761.12	140.36	1761.12	1.647	1.052	56.46%	63.91%
16		248.0	228.5	228.5	18.96	239.65	1736.54	138.42	1736.54	1.656	1.067	55.14%	64.46%
17		248.0	226.0	226.0	21.40	270.37	1705.81	135.98	1705.81	1.668	1.086	53.48%	65.16%
18		248.0	225.0	225.0	22.37	282.66	1693.52	135.01	1693.52	1.672	1.094	52.82%	65.44%
19		248.0	224.0	224.0	23.34	294.95	1681.23	134.04	1681.23	1.677	1.102	52.15%	65.72%
20		248.0	223.0	223.0	24.31	307.24	1668.94	133.07	1668.94	1.682	1.110	51.49%	66.01%
22		248.0	222.0	222.0	25.29	319.53	1656.65	132.09	1656.65	1.687	1.119	50.83%	66.30%
24		248.0	221.0	221.0	26.26	331.82	1644.37	131.12	1644.37	1.692	1.127	50.16%	66.59%
28		248.0	220.0	220.0	27.23	344.11	1632.08	130.15	1632.08	1.698	1.136	49.50%	66.89%
32		248.0	219.0	219.0	28.20	356.40	1619.79	129.18	1619.79	1.703	1.144	48.84%	67.19%
37		248.0	218.0	218.0	29.18	368.69	1607.50	128.20	1607.50	1.708	1.153	48.17%	67.49%
42		248.0	217.0	217.0	30.15	380.98	1595.21	127.23	1595.21	1.714	1.162	47.51%	67.79%
53		248.0	216.0	216.0	31.12	393.27	1582.92	126.26	1582.92	1.719	1.171	46.85%	68.10%
65		248.0	215.0	215.0	32.09	405.55	1570.63	125.29	1570.63	1.725	1.180	46.19%	68.41%
82		248.0	214.0	214.0	33.07	417.84	1558.34	124.31	1558.34	1.731	1.189	45.52%	68.72%
110		248.0	213.0	213.0	34.04	430.13	1546.05	123.34	1546.05	1.737	1.199	44.86%	69.03%
170		248.0	212.0	212.0	35.01	442.42	1533.76	122.37	1533.76	1.742	1.208	44.20%	69.35%
240		248.0	211.5	211.5	35.50	448.57	1527.61	121.88	1527.61	1.745	1.213	43.86%	69.51%
470		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1140		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1230		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1320		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1390		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%
1440		248.0	211.0	211.0	35.98	454.71	1521.47	121.40	1521.47	1.748	1.218	43.53%	69.67%

Klohn Crippen Berger

PROJECT NO:	M09638A03			DATE:	3-Dec-12
PROJECT:	Minto Mine			LOCATION:	Yukon
DETAIL:	60% Solids Content				
DESCRIPTION:	Underflow Tailings				
TESTED BY:	FG	CHECKED BY:	BY	SHEET	1 of 2

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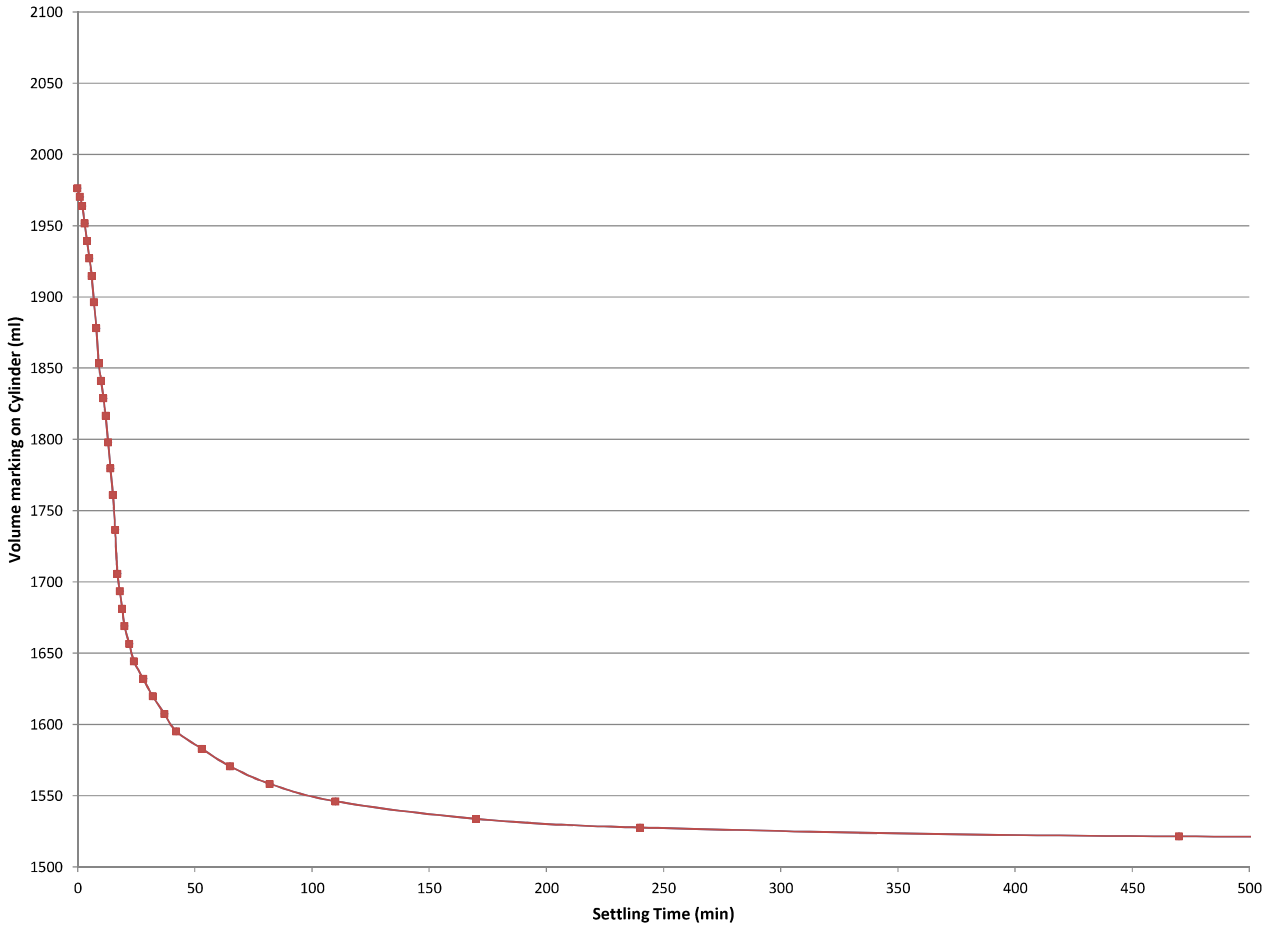
Settlement Test for Underflow Tailings at 60% Solids Content



Klohn Crippen Berger	PROJECT NO:	M09638A03		DATE	3-Dec-12
	PROJECT:	Minto Mine		LOCATION:	Yukon
	DETAIL:	60% Solids Content			
	DESCRIPTION:	Underflow Tailings			
	TESTED BY:	FG	CHECKED BY BY	SHEET	2 of 2

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APPENDIX IV

Specific Gravity

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SPECIFIC GRAVITY OF SOIL SOLIDS
(ASTM-D854)

Sample No.		Underflow Tailings					
Flask No.		KL-2	KL-3	KLA-4			
Volume of Flask @ 20° C	ml	500	500	500			
Method of Air removal		Boiling	Boiling	Boiling			
De-airing Period	hr	2	2	2			
Test temperature	° C	22.8	22.8	22.8			
Mass of Flask+Water (M_a)	g	675.48	676.08	678.54			
Mass of Flask+Water+Soil (M_b)	g	712.53	712.98	715.36			
Mass of Dish/Flask+Soil		235.75	235.77	238.34			
Mass of Dish/Flask		177.12	177.39	180.06			
Mass of Dry Soil (M_o)	g	58.63	58.38	58.28			
Correction factor (K) @ Test Temperature		0.9994	0.9994	0.9994			
Specific Gravity of Solids @ 20° C		2.715	2.716	2.714			
Average Specific Gravity of Solids @ 20° C		**2.72**					

Sample No.							
Flask No.							
Volume of Flask @ 20° C	ml						
Method of Air removal							
De-airing Period	hr						
Test temperature	° C						
Mass of Flask+Water (M_a)	g						
Mass of Flask+Water+Soil (M_b)	g						
Mass of Dish/Flask+Soil							
Mass of Dish/Flask							
Mass of Dry Soil (M_o)	g						
Correction factor (K) @ Test Temperature							
Specific Gravity of Solids @ 20° C							
Average Specific Gravity of Solids @ 20° C							

Specific Gravity of Solids @ 20° C = $(K \times M_o)/(M_o + M_a - M_b)$

Klohn Crippen Berger

PROJECT NO.:	M09638A03
PROJECT:	Minto Mine
LOCATION:	Yukon
DATE:	29-Nov-12
TESTED BY:	FG CHECKED BY BY

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Appendix N: Minto Main Dam Preliminary Design – Seepage Analysis

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vancouver@srk.com
www.srk.com

Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Emma Helmers	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 18, 2014
Subject:	Main Dam Seepage Analysis		

1 Introduction

This memo summarizes the results of the seepage analysis conducted for the preliminary design of the Main Dam.

The location of the proposed Main Dam and a typical section is provided in Figures 1 and 2. The dam has a crest width of 14.5 m, a 2.5H:1V upstream slope and a 3H:1V downstream slope. The dam has a crest elevation of 807 m, with a provision to raise the crest height to 812 m if required. The dam is a rock fill structure with an internal geomembrane liner and a fine-grained core system that is keyed into fine grained overburden and/or bedrock. The upstream face of the dam will include a 1.0 m thick riprap layer for erosion protection.

The liner-fine grained core system consists of a 2.25 m thick layer of low hydraulic conductivity material that is overlain by a LLDPE liner. A bedding layer is to be placed over the top of the liner followed by 2.5 m of waste rock. Two filter layers, each 0.5 m thick, are located downstream of the fine-grained core.

Tailings deposition will be sequenced such that a large tailings beach develops on the upstream side of the dam. The beach will be at a 1% slope and will reach a maximum elevation of 1 m below the top of the dam core. The pond will be maintained 100 m upstream from the dam such that the water elevation is a minimum of 2 m below the top of the dam core, and 3 m below the dam crest.

The foundation materials vary along the length of the dam. On the north end, the foundation consists entirely of bedrock. Towards the south end, the foundation consists of overburden that generally consists of a thin layer of silty sandy gravel overlying silty clay with pockets of clean sand. At the southernmost end of the dam, the overburden is up to 65 m thick. Thermal modelling (SRK 2014a) has shown that the majority of the silty clay material in the foundation will be frozen over the design life, especially when insulated by the tailings that will lie upstream of the dam.

The seepage criteria placed on the dam relate to both the amount and location of the seepage face at the toe of the dam. The design objective of the dam is to limit seepage through the toe of the dam during operations as much as possible to avoid the need for pumping. To ensure that seepage volumes are less than assumed in Phase V/VI water quality predictions, the total seepage volume through the dam should be limited to the upper bound calculated and used in the inputs to the water quality prediction model of 83 m^3/d (SRK 2013).

2 Seepage Analysis

2.1 Assessment Method

For this assessment, seepage volumes in m^3/day/m along the dam were estimated using Rocscience's Slide 6.0 (2012), a commercially available software package which has a 2D finite-element groundwater modelling capability. Seepage volumes were obtained from steady-state model solutions, which represent the long-term seepage volumes expected to occur after the dam is constructed, and tailings and/or water is deposited into the pit to its full storage capacity.

The model results were validated using Darcy's Equation. These hand-calculations were completed by assessing seepage through the core of the dam using the same material properties as used in the Slide model.

Both the Slide 6.0 models and the Darcy calculations did not include the LLDPE liner that is located on the upstream side of the dam core. To assess seepage through the lined dam, further hand calculations were completed to estimate the amount of leakage that could potentially occur through liner defects.

2.2 Model Setup

2.2.1 Sections

Forty-five cross-sections along the dam were cut from a three-dimensional Gemcom GEMS model of the dam design to evaluate the range of foundation conditions that exist along the dam. Six of these sections were selected to be representative of the different foundation conditions along the entire length of the dam and were used to develop the seepage estimates.

Table 1 lists the representative sections, along with the dam height, approximate chainage along the crest from the south end of the dam, foundation details, and the length of the dam represented by each section. The locations of each section are shown on Figure 1.

Table 1: Sections used in Seepage Modelling

Section	Model Section ID	Approximate Chainage along 812 m Dam (0+m)	Foundation Materials	Overburden Thickness under Dam Crest (m)	Length of 812 m Dam (m)	Length of 807 m Dam (m)
1	D5	0+50	Silty sandy gravel, silty clay	57	75	50
2	D14	0+140	Silty sandy gravel, silty clay	63	130	75
3	D22	0+220	Silty sandy gravel, silty clay, sand	34	80	100
4	D30	0+300	Bedrock, silty sandy gravel	0	50	50
5	D36	0+360	Bedrock	0	70	75
6[1]	D41	0+410	Bedrock	0	45	-

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Note:

1. D41 was not used in the analysis for the 807 m dam as the dam crest intersects with natural ground near this point. Therefore, the length of the dam represented by section D36 for the 807 m dam was increased

2.2.2 Boundary Conditions and Mesh

Typical boundary conditions specified in the model are presented in Figure 3. The boundaries on the upstream side of the dam were given a total head value corresponding to the water pond elevation in the pit (3 m below the dam crest). The dam crest and downstream boundaries were assigned unknown boundary conditions, which means they may act as either zero-nodal flow or zero pressure (seepage face) boundaries.

Dam seepage was measured with the addition of a flux section line from the dam crest to the base of the dam. A second flux section line was placed to measure groundwater flow, through the dam foundation, measured from the base of the dam to the top of the bedrock surface. The locations of flux section lines are provided in Figure 3.

For simplicity and to minimize model run times, the model sections were assigned meshes made up of 3-noded trianglular elements with finer meshes in areas where the section geometry was more detailed.

2.3 Material Properties

Material properties were assigned to each construction material based on commonly accepted values, empirical calculations based on other material properties, engineering judgement and, where available, laboratory test results. Table 2 displays the material properties used in the seepage modelling.

Table 2: Material Properties

Material	Saturated Hydraulic Conductivity, K_s (m/s)	Unsaturated Hydraulic Conductivity Estimation Method	Saturated Hydraulic Conductivity Estimation Technique
Bedrock	$8.0x10^{-8}$	Simple	Packer test results
Tailings	$1.0x10^{-7}$	Simple	Constant head test (EBA 2007)
Riprap	$5.0x10^{-3}$	Simple	Engineering judgement
Waste Rock	$1.0x10^{-3}$	Simple	Engineering judgement
Bedding 1	$1.0x10^{-4}$	Simple	Engineering judgement
Bedding 2	$5.0x10^{-5}$	Simple	Engineering judgement
Core	$2.0x10^{-7}$	Van Genuchten	Based on grain size distribution
Fine Filter	$1.0x10^{-5}$	Van Genuchten	Based on grain size distribution
Coarse Filter	$1.0x10^{-4}$	Van Genuchten	Based on grain size distribution
Sand	$5.0x10^{-6}$	Van Genuchten	Based on grain size distribution
Silty Clay	$2.0x10^{-7}$	Van Genuchten	Based on grain size distribution
Silty Sandy Gravel	$1.0x10^{-5}$	Simple	Based on grain size distribution
Frozen Foundation	$1.0x10^{-8}$	Van Genuchten	Based on grain size distribution

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Notes:

1. See Section 2.3.1 and 2.3.2 for details on the Van Genuchten and Simple unsaturated hydraulic conductivity estimation methods.
2. All soil materials are assumed to be unfrozen, except where noted.

The basis for the selection of the saturated hydraulic conductivity of the materials in the models is as follows:

- The bedrock was assigned a hydraulic conductivity using a geometric average of the packer test results from both the Main Dam Phase 1 Field Investigation (SRK 2014b) and from packer test results obtained by Golder Associates in their original field investigation in 1974 (Golder 1974).
- The tailings were assigned a hydraulic conductivity of $1x10^{-7}$ m/s based on a constant head test performed in 2007 during the design phase of the Dry Stack Tailings Storage Facility (EBA 2007).
- The dam core, fine and coarse filters, sand, and silty clay hydraulic conductivities were estimated using the Rosetta program, based on grain size distributions.
- The silty sandy gravel hydraulic conductivity was assigned to be the same as the fine filter, as it has a similar grain size distribution.
- The bedding material was assigned similar hydraulic conductivities to the fine and coarse filters as they will need to have similar properties.

- The waste rock was assigned a hydraulic conductivity one order of magnitude higher than the commonly accepted value for clean sand. This allows for some flow through any macropores that are not decreased in size by compaction, in order to be conservative.
- The riprap was assigned a larger hydraulic conductivity as it will contain little fines.
- The frozen foundation was assigned a low hydraulic conductivity as there is expected to be little flow through frozen soil.

Compaction of the dam materials can reduce the size of the pores and lower the hydraulic conductivity. It has been found that structures compacted in lifts exhibit anisotropy in hydraulic conductivity, with the horizontal hydraulic conductivity being larger than in the vertical direction. Compaction has been found to reduce the hydraulic conductivity of materials by up to 3 times (Stepniewski *et al*. 2011). Accordingly, the vertical hydraulic conductivity of the waste rock, bedding material, core and filter material was set to half of the horizontal hydraulic conductivity of each material.

Unsaturated conditions were modelled in Slide 6.0 by defining a soil water characteristic curve for each material. The soil water characteristic curve defines how hydraulic conductivity changes with matric suction (negative pore pressure). Two different methods, van Genuchten and Simple, were used to estimate these curves. The van Genuchten model predicts the hydraulic conductivity of unsaturated soils using a closed-form equation containing three independent parameters, derived from Mualem theory (van Genuchten, 1980). The Simple model is a built in feature in Slide 6.0 that estimates the unsaturated hydraulic conductivity based on the saturated hydraulic conductivity and typical literature functions for the assigned soil type.

The resulting unsaturated hydraulic conductivity curves are presented in Figure 4. The following sections describe each method in further detail.

2.3.1 Unsaturated Hydraulic Conductivity Estimation - Van Genuchten Method

Van Genuchten parameters were estimated using the Rosetta software program developed by the United States Department of Agriculture (Skaggs, 1999). Required inputs into the program are the percentage of sand, silt and clay in a material, the bulk density, and water retention parameters to perform its calculations.

Table 3 contains the input parameters adopted for the estimation of hydraulic parameters using Rosetta. The grain size distributions of the foundation materials were obtained from the results of the Main Dam Phase 1 Field Investigation (SRK 2014b). Grain size distributions for the core, fine and coarse filter materials were obtained from the Main Dam Filter Design (SRK 2014c).

The coarse filter material will likely contain a significant portion of gravel; however, the filter design requires the material to be well-graded and the fine portion of each material will likely have a larger impact on the hydraulic conductivity than the coarse portion. Therefore, the material was entered as 100% sand in the Rosetta model.

Table 3: Rosetta Material Property Inputs

				van Genuchten Parameters[1]	
Material	**% Sand**	**% Silt**	**% Clay**	**a (1/m)**	**n**
Sand	63	28	9	2.8	1.4
Silty Clay	30	48	22	0.66	1.6
Core	40	51	9	0.65	1.6
Fine Filter	50	50	0	1.4	1.5
Coarse Filter	100	0	0	2.5	1.6

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Notes:

1. van Genuchten parameters were inputs entered into Slide 6.0 to generate unsaturated hydraulic conductivity curves.

2.3.2 Unsaturated Hydraulic Conductivity Estimation - Simple Method

Unsaturated hydraulic conductivities that were not obtained using the Rosetta software and the van Genuchten method were assigned Simple unsaturated models in Slide 6.0. All materials were assigned to have a 'sand' type behavior, except for the tailings that were assigned 'silt' behavior.

Both waste rock and riprap were assigned sand for soil type because it has been found that flow through compacted waste rock containing fines behaves much like flow through soil (Martin *et al*., 2004).

2.4 Finite Element Modelling Cases

Multiple cases were analyzed using finite-element modelling in Slide 6.0 to represent a wide range of scenarios for seepage. Table 4 presents the cases considered. As mentioned in Section 2.1, all of these cases assume that the geosynthetic liner is not present.

The base case scenario is an 807 m crest elevation dam with a developed tailings beach and a frozen foundation (Case 6). Variants of the base case analyzed the case where the foundation is fully thawed and with no anisotropy of the dam fill materials (Cases 4 and 5). Each of these cases were also evaluated for a dam with a crest elevation of 812 m with tailings upstream of the dam due to the possibility that the dam height will need to be raised if there is significant ice entrainment during tailings deposition.

The worst case scenario is the 807 m crest dam with no tailings beach, i.e. the Main Pit fills with water up to the spillway elevation of 804 m immediately following construction. Seepage was evaluated for this scenario with a frozen foundation (Case 9) and a fully thawed foundation with and with anisotrophy of the dam fill materials (Cases 7 and 8).

Table 4: Cases Analyzed in Seepage Modelling

Case Number	Case Description
1	812 m dam, tailings beach, anisotropy
2	812 m dam, tailings beach, no anisotropy
3	812 m dam, tailings beach, anisotropy, frozen foundation
4	807 m dam, tailings beach, anisotropy
5	807 m dam, tailings beach, no anisotropy
6 – Base Case	**807 m dam, tailings beach, anisotropy, frozen foundation**
7	807 m dam, no tailings upstream of dam, anisotropy
8 [1]	807 m dam, no tailings upstream of dam, no anisotropy
9	807 m dam, no tailings upstream of dam, anisotropy, frozen foundation

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Notes:

1. Model results verified with Darcy's Equation hand calculations (see Attachment 2 of this memo).

2.5 Liner Leakage Calculations

Slide 6.0 does not have the capability to model geomembrane liners because flow through liner defects is a different flow mechanism than flow through soils. All liners can be expected to have small defects from the manufacturing process and further defects can and do occur during construction. Flow through the liner occurs as leakage through these discrete defects.

Booker and Rowe (1997) presented a semi-analytic solution to estimate leakage through a hole, (or perforation), in a geomembrane and an underlying low-permeability clay material. The amount of flow is dependent on the hydraulic conductivity and thickness of the material underlying the liner, the radius of the hole, and the hydraulic head on top of the liner. The following equation was used to calculate flow through the liner, adapted for the case where the radius of the membrane hole divided by the thickness of the material underneath the liner is approaching neither zero nor infinity:

$$Q = n * \left(4 + \left(2.455 + 0.685 \tanh\left(0.6 \ln\left(\frac{r_o}{D}\right)\right)\right)\right) * r_o * h_d * k$$

Where:

- n = number of holes across the considered membrane area
- r_o = the effective radii of the "hole"
- D = thickness of the low hydraulic conductivity material under the liner
- h_d = hydraulic head on top of the liner; and
- k = hydraulic conductivity of material under liner.

For conservatism, it was assumed that all holes occur in areas over wrinkles, which is defined as areas where the liner is not in direct contact with the clay layer. In this case, water passing through the liner hole is distributed over a larger contact area on the clay surface. As a result, the effective radius of the hole was defined as the approximate radius of the wrinkle.

The wrinkle radii of 0.1 and 1.0 m were assumed in this analysis. Giroud *et al*. (1994) published typical liner perforation diameters, but did not include typical wrinkle sizes. With strict construction quality control and quality assurance (QA/QC), it is considered to be unlikely that there will be many wrinkles with a radius larger than 10 cm. The 1.0 m radius wrinkle was considered in this analysis to determine the sensitivity of the parameter due to the potential for differential settlement or unequal compaction due to material variability.

Giroud and Bonaparte (1989) showed that there is one manufactured defect per acre (4,046 m^2) of geomembrane on average, and that the defects that occur during the construction process heavily depend on the construction QA/QC. With strict QA/QC measures, there may be as few as one to two defects per acre. The US Army Corps of Engineers (USACE) have reported the typical variation in defect density with construction quality, displayed in Table 5 (USACE 1997).

Table 5: Construction Defect Densities

Construction Quality	Defect Density (#/acre)	Frequency (%)
Excellent	Up to 1	10
Good	1 to 4	40
Fair	4 to 10	40
Poor	10 to 20 (old landfills)	10

A number of different liner leakage cases were considered. It was noted by the USACE that excellent construction quality is typically only achievable in a laboratory environment and that poor construction quality does not often occur with modern approaches to construction, hence the lower frequencies for these two cases. With this in mind, cases in which construction quality ranged from fair to good were considered with varying defect sizes. Cases analyzed are displayed in Table 6. Liner leakage was calculated for both the 807 m and 812 m crest elevations.

The most likely case is considered to be L5 in which there are small wrinkle diameters and the quality of the construction is good, and the worst cases are L4 and L8 in which the diameter of the wrinkles is large and the construction quality is fair.

Table 6: Cases Analyzed in Liner Leakage Calculations

Case	Dam Crest Elevation (m)	Construction Quality	Wrinkle Radius (m)
L1	812	Good	0.1
L2	812	Fair	0.1
L3	812	Good	1.0
L4	812	Fair	1.0
L5 – BASE CASE	**807**	**Good**	**0.1**
L6	807	Fair	0.1
L7	807	Good	1.0
L8	807	Fair	1.0

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2.6 Sensitivity Analysis

Sensitivity analyses were completed in Slide 6.0 on the following cases:

- For the 807 m and 812 m dams with tailings beaches, the hydraulic conductivity of both the dam core and the tailings were varied.
- For the 807 m dam without a tailings beach (i.e. with a Pit Lake against the upstream face of the dam), the hydraulic conductivity of the dam core was varied.
- For the liner leakage cases, the radii of the liner wrinkles were varied. It was not considered necessary to vary the hydraulic conductivity of the dam core for the liner leakage cases as, according to the equation in Section 2.5, it can be seen that seepage will vary linearly with core hydraulic conductivity.

Sensitivity analyses were completed on the dam materials hydraulic conductivities because there is a large degree of uncertainty of their values at this time. There are few lab hydraulic conductivity tests on material that would be suitable for a dam core at the Minto site, and there is only one constant head test result available for the tailings. The sensitivity analyses completed are listed in Table 7.

Table 7: Sensitivity Analyses

Sensitivity Analysis	Seepage Case	Parameter Varied
A	Case 1: 812 m dam, tailings beach, anisotropy due to compaction	Core hydraulic conductivity
B	Case 1: 812 m dam, tailings beach, anisotropy due to compaction	Tailings hydraulic conductivity
C	Case 4: 807 m dam, tailings beach, anisotropy due to compaction	Core hydraulic conductivity
D	Case 4: 807 m dam, tailings beach, anisotropy due to compaction	Tailings hydraulic conductivity
E	Case 7: 807 m dam, no tailings upstream of dam, anisotropy due to compaction	Core hydraulic conductivity
F	Liner leakage cases L1 to L8 [1]	Wrinkle radii

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Notes:

1. Refer to Table 6 for descriptions of liner seepage cases L1 through L8.

3 Results

3.1 Estimated Seepage

Seepage was calculated across the entire length of the dam for all cases. Table 8 displays the results of the seepage analysis in Slide 6.0 and using Darcy's Equation. Full results of the FEM models can be found in Attachment 1.

Table 8: Estimated Seepage and Groundwater Flow (No Liner)

Case	Dam Seepage (m^3/d)	Groundwater Flow (m^3/d)	Total Flow (m^3/d)
1	20	50	71
2	21	50	71
3	29	6	35
4	7	29	36
5	8	29	36
6 – Base Case	**11**	**11**	**22**
7	342	169	511
8	463	171	634
9	330	117	447

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Notes:

1. Numbers were rounded to the nearest whole number.

Darcy's Equation was used to confirm the results of the FEM modelling. Calculations were completed on Case 8 and resulted in a total estimated seepage through the dam of 700 m^3/day (compared to 463 m^3/day). The higher estimate using Darcy's Equation was expected as Darcy's Equation assumed that all water will flow through the dam (no groundwater flow through the foundation). In the southern portion of the dam where the foundation consists of overburden, water may preferentially flow through high conductivity portions of the foundation and enter the groundwater system as opposed to flowing through the dam. The 700 m^3/day estimate is similar to the total flow estimate (dam seepage plus groundwater flow) of 634 m^3/day for Case 8.

The Darcy's Equation calculations can be found in Attachment 2.

Table 9 presents a summary of the liner leakage calculations. Details of the calculations can be found in Attachment 3. For both the 807 m and 812 m dam, assuming a good construction quality rating, the dam seepage is estimated to be 1 m^3/day (Cases L1 and L5).

The groundwater flow estimates assumed that the foundation was completely thawed in all cases. The groundwater flow estimate of 36 m^3/day is similar to the total flow modelled results of Case 4 that consisted of a fully thawed foundation without a liner (36 m^3/day).

Table 9: Estimated Dam Seepage, Liner Leakage Calculations

Case	Dam Seepage (m^3/d)	Groundwater Flow (m^3/d)[1]	Total Flow (m^3/d)
L1	1	36	37
L2	2	36	39
L3	14	36	51
L4	29	36	65
L5 – Base Case	**1**	**36**	**37**
L6	1	36	37
L7	8	36	44
L8	13	36	49

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Notes:

1. Numbers were rounded to the nearest whole number.

   Models with the dam core having extremely low hydraulic conductivity (1×10^{-10} m/s) were run in Slide to get the expected seepage through the foundation in this table. It is assumed that the seepage through the foundation can be estimated as such and that it will be similar for all liner leakage cases.

3.2 Sensitivity Analysis

A summary of the sensitivity analysis results are provided in Table 10. The full results can be found in Attachment 4. In general, both the design and ultimate dam cases including tailings were not especially sensitive to the hydraulic conductivity of the core material; however, they were very sensitive to changes in the hydraulic conductivity of the tailings. This is due to the fact that water must pass through significantly more tailings than core material, so the tailings controls seepage to a higher degree than the core material. For the design water storage dam case, there are no tailings upstream of the dam so it was very sensitive to the hydraulic conductivity of the core material. For the liner leakage cases, changes in liner wrinkle radius will cause larger changes in seepage. The amount of liner leakage is also sensitive to the hydraulic conductivity of the core material.

Table 10: Sensitivity Analysis Results

Sensitivity Case	Parameter Varied	Sensitivity	Comments
A	Core K	Insensitive	Likely insensitive as thickness of core is so small in comparison to thickness of tailings
B	Tailings K	Sensitive	Large thickness of tailings, has largest effect on gradient and therefore flow, any changes in K will have large effect on seepage
C	Core K	Insensitive	Likely insensitive as thickness of core is so small in comparison to thickness of tailings
D	Tailings K	Sensitive	Large thickness of tailings, has largest effect on gradient and therefore flow, any changes in K will have large effect on seepage

Sensitivity Case	Parameter Varied	Sensitivity	Comments
E	Core K	Sensitive	No tailings in this case, core is lowest hydraulic conductivity material and seepage will be heavily affected by its properties. Less sensitive than sensitivity to tailings because of higher flow through foundation in the water storage dam case (less water reaches foundation through tailings)
F	Wrinkle Radii	Sensitive	Sensitivity is slightly higher with increasing wrinkle radii

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4 Recommendations and Final Remarks

For both the 807 m and 812 m crest elevation dams, seepage through the dam is expected to be far less than the predicted upper bound of 83 m^3/d used in the water quality prediction model (SRK 2013).

The base case scenario consisting of an 807 m dam with a tailings beach and frozen foundation estimates seepage through the dam to be 11 m^3/d without the presence of a geomembrane liner. Assuming the installation quality of the geomembrane is good with minimal defects, leakage through the dam will likely approach the 1 m^3/day value calculated in the liner leakage calculations for both the 807 m and 812 m dams.

The liner leakage calculations assumed the worst case scenario of no tailings beach development and water immediately on the upstream side of the dam. To achieve levels of seepage calculated in the liner leakage calculations, a strict construction QA/QC program is necessary to ensure that seepage through the dam is minimized.

Anisotropy in hydraulic conductivity due to compaction does not have a significant effect on dam seepage. In the cases of a completely thawed foundation, the seepage through the dam is reduced from the base case by up to 50%. This is because the frozen foundation is effectively impermeable and more water is then able to flow beneath the foundation. Based on the assumed material properties, the dam will meet the seepage objectives. However, the sensitivity analysis showed that the amount of seepage is very sensitive to the hydraulic conductivity of the tailings material and the dam core in the case of the pit lake scenario. Therefore, it is recommended that further testing be completed on these materials in order to have increased confidence in the seepage predictions.

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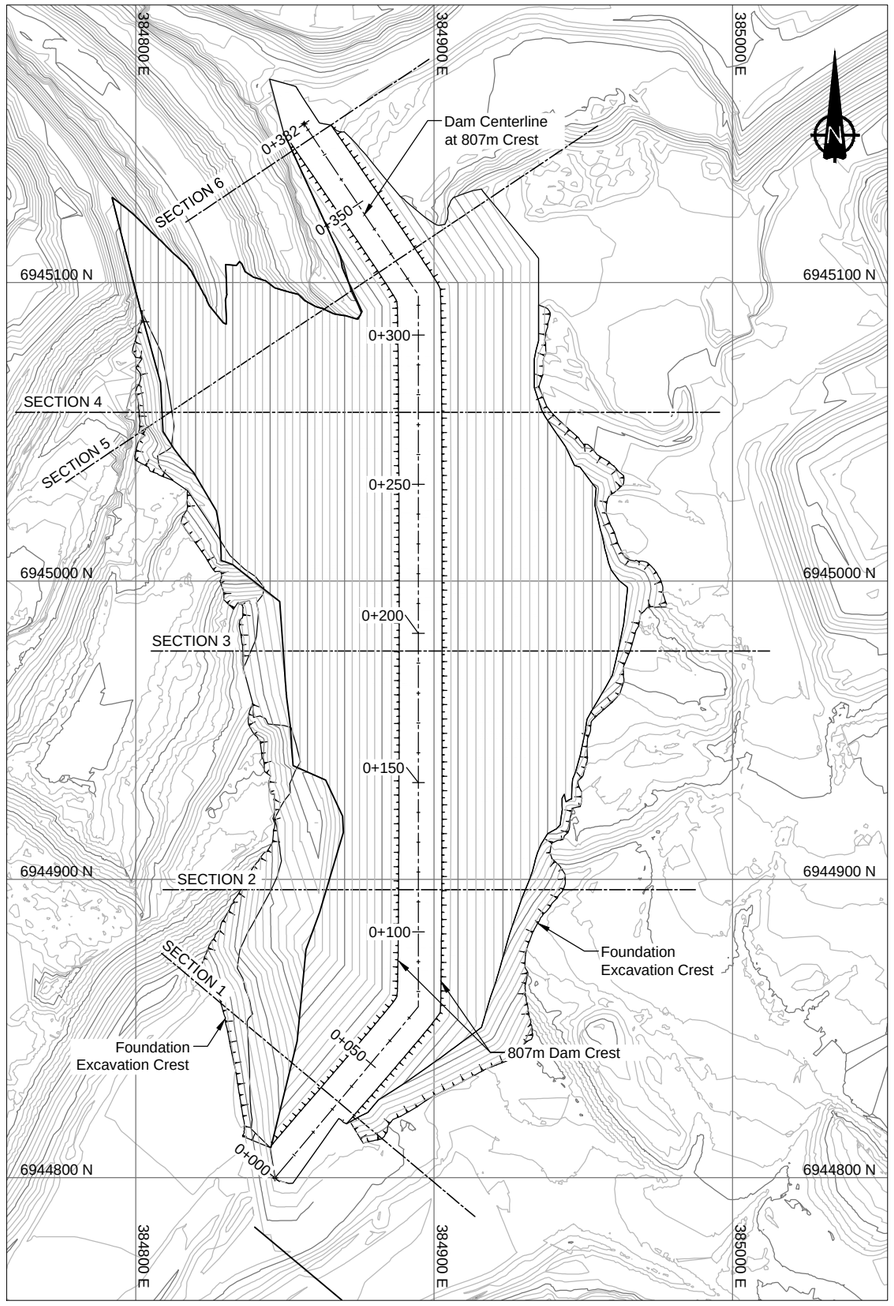
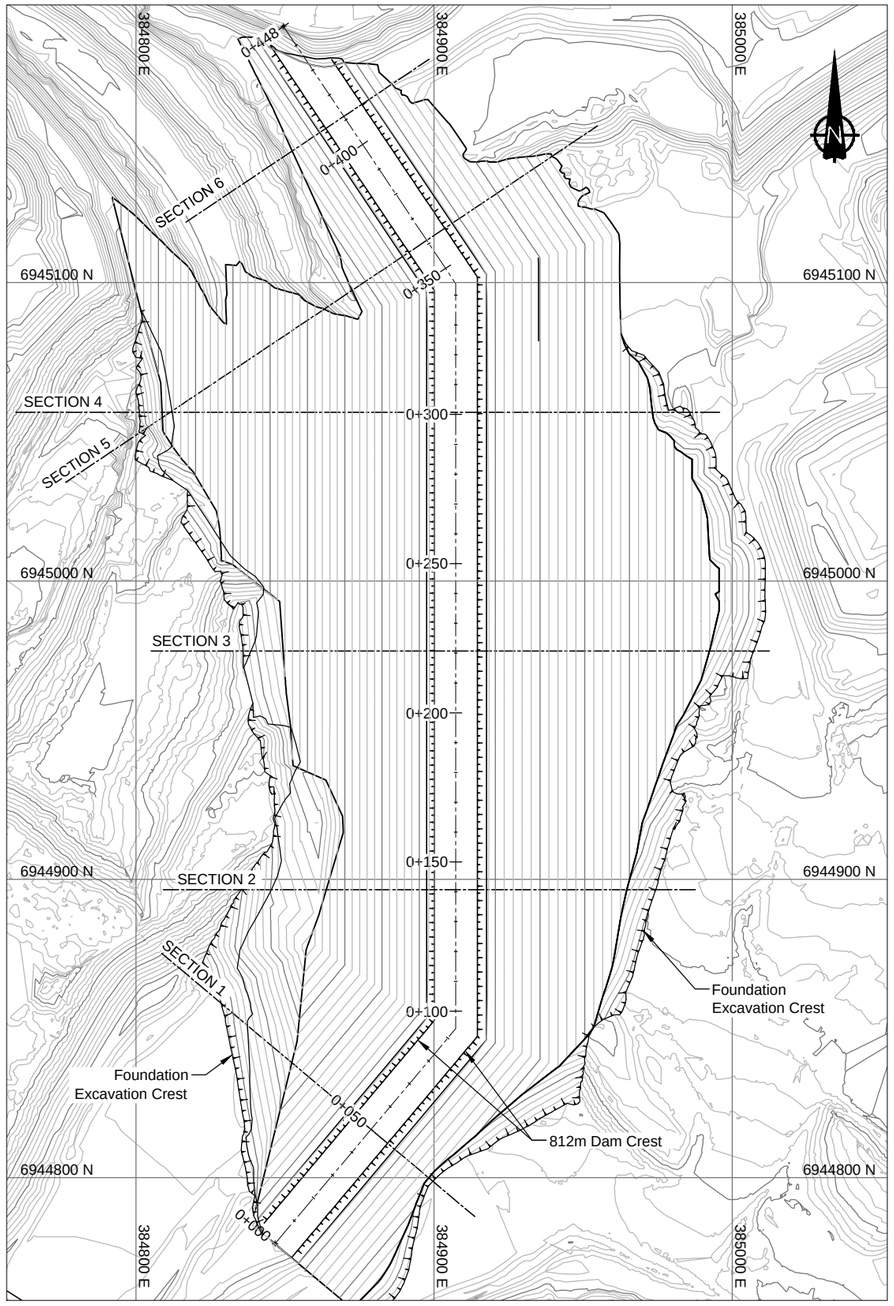
Figures



PLAN VIEW OF 807m DAM



PLAN VIEW OF 812m DAM

0 10 20 30 40 50
Scale in Metres

NOTES

1. Contour interval 1m.
2. Topography received from mine site.

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MINTO MINE

Main Dam Seepage Analysis

807m and 812m Dam Layouts

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July 2014	MJM	1

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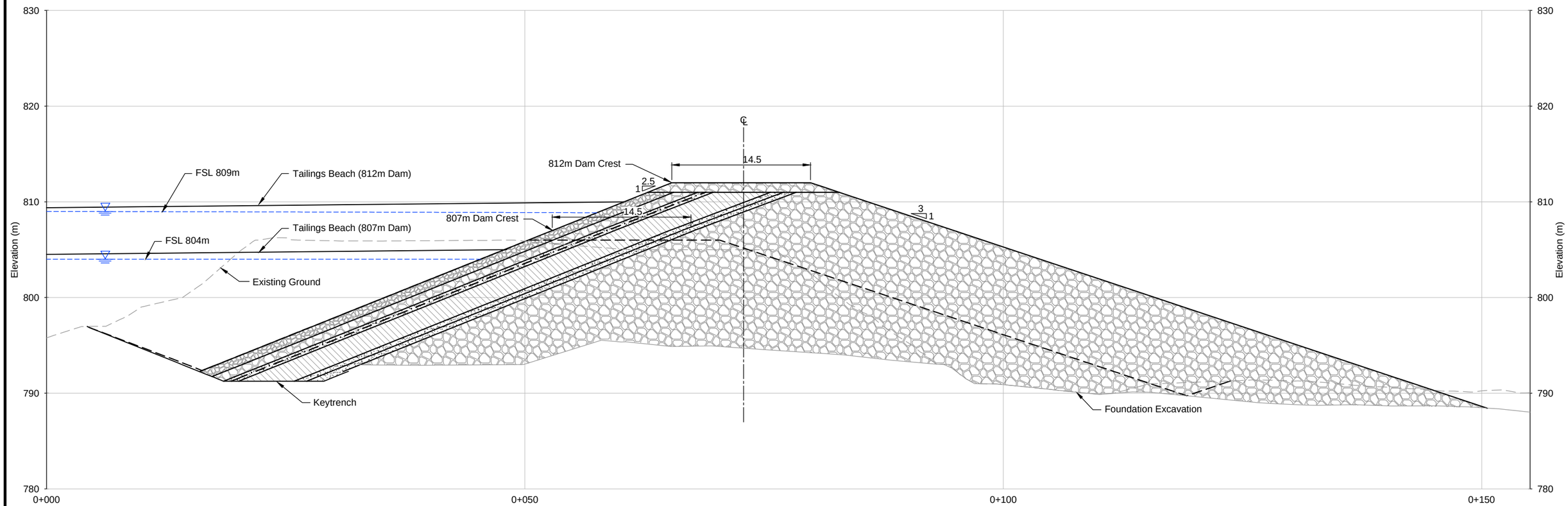

LEGEND

- Existing Ground
- Top of Bedrock
- 807m Dam Stage
- Liner
- Core Material
- Transition Material
- Waste Rock
- Rip Rap

0 2 4 6 8 10
Scale in Metres

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Main Dam Seepage Analysis

Typical Dam Section

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July 2014	MJM	2

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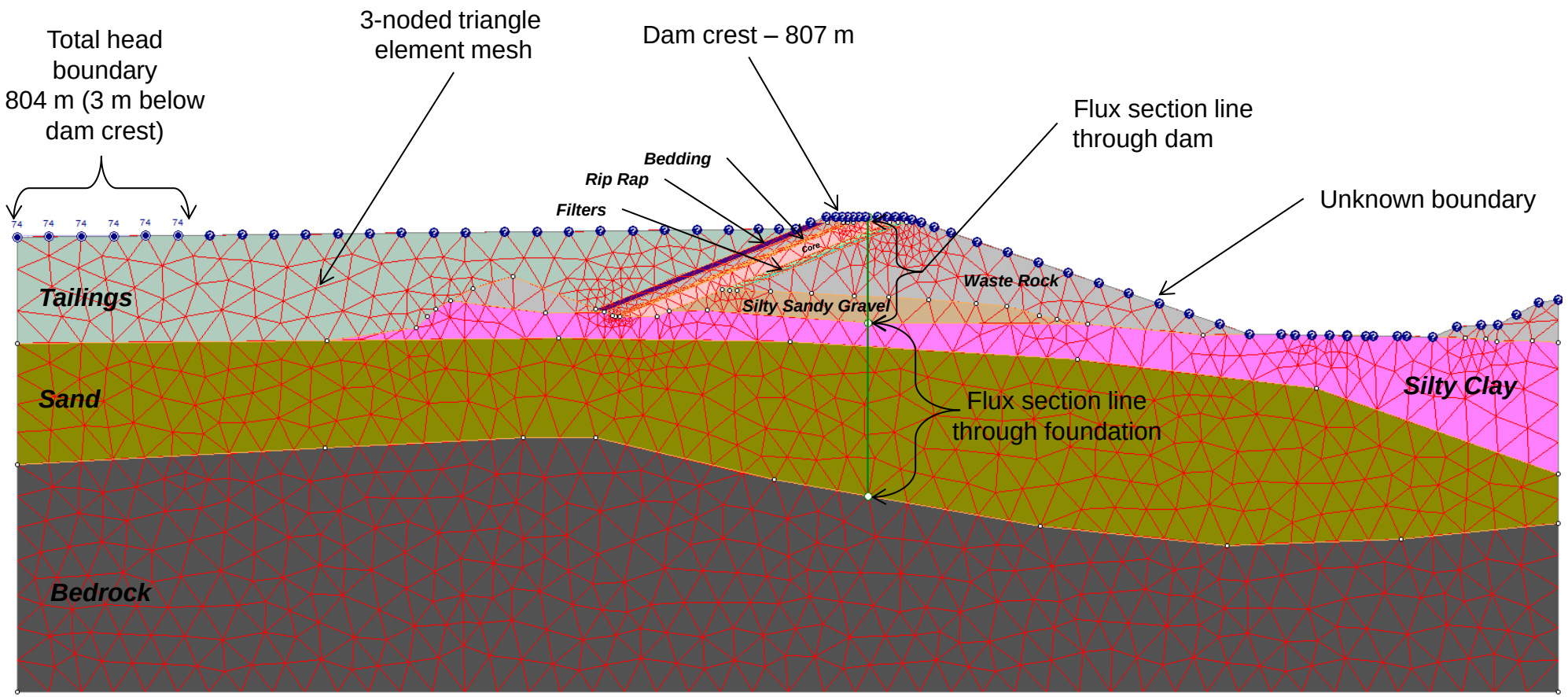

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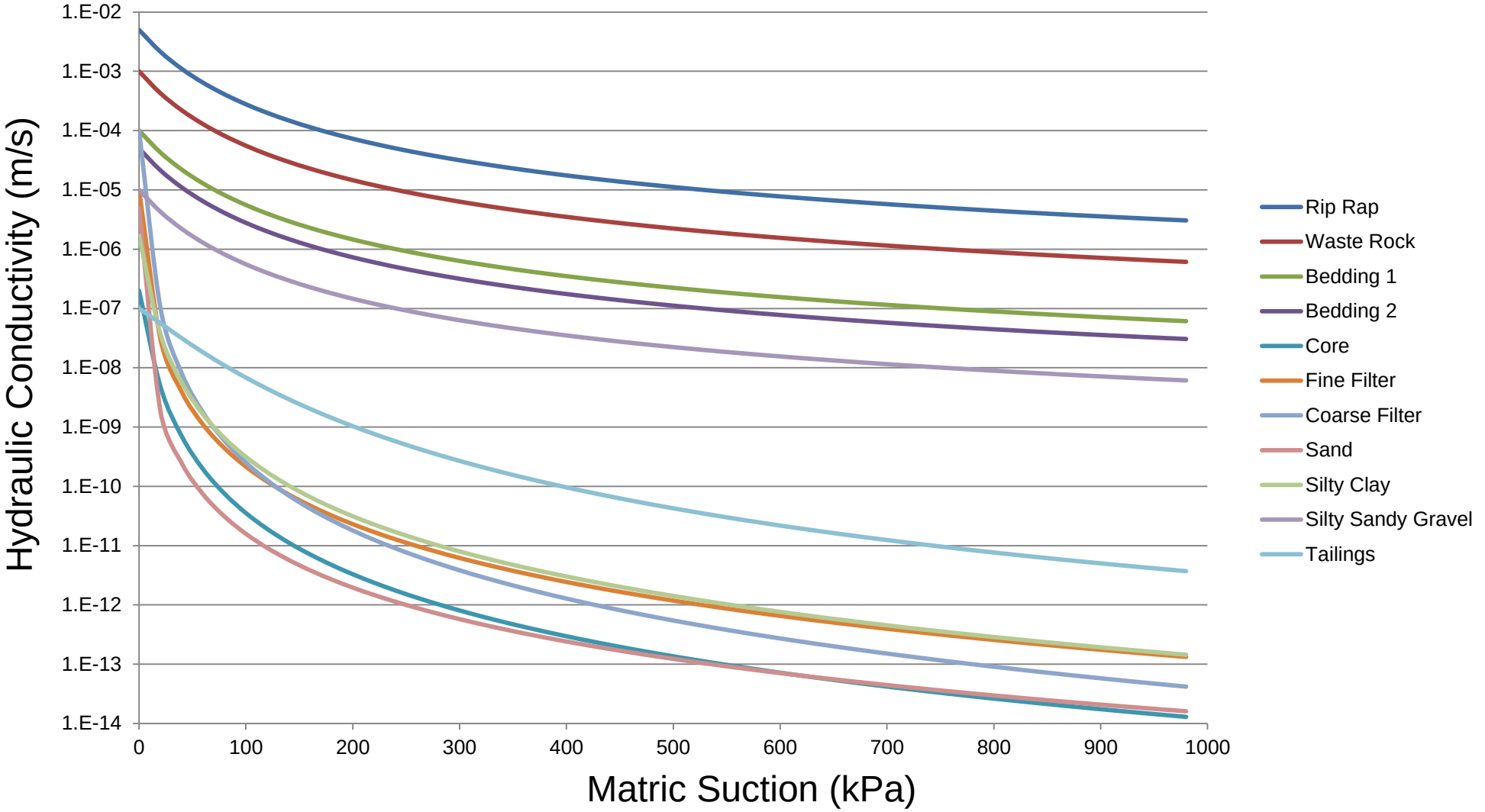
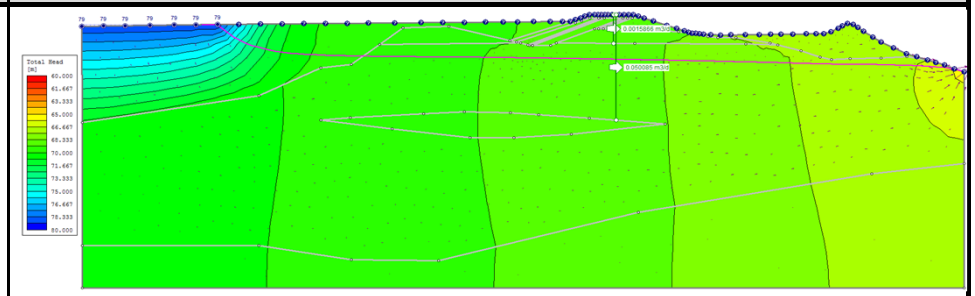
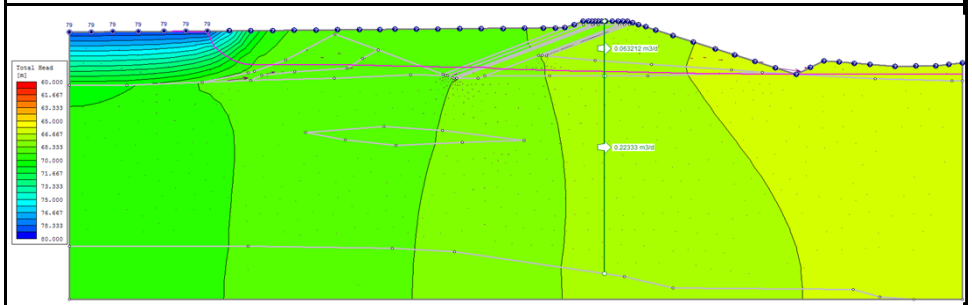
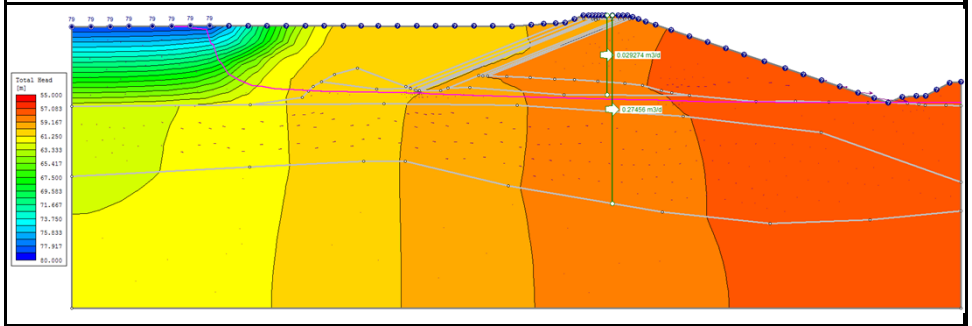
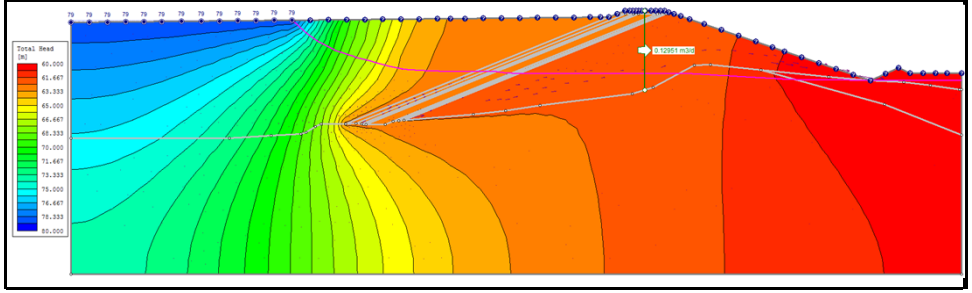
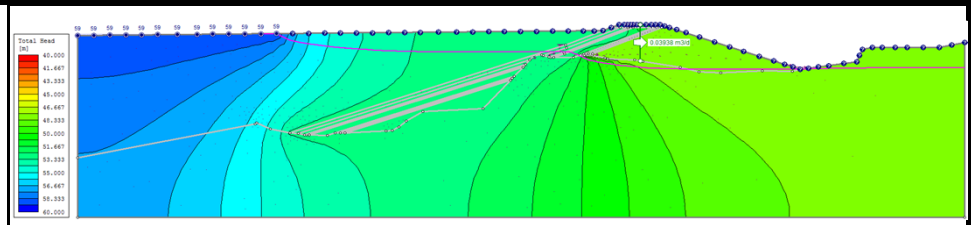
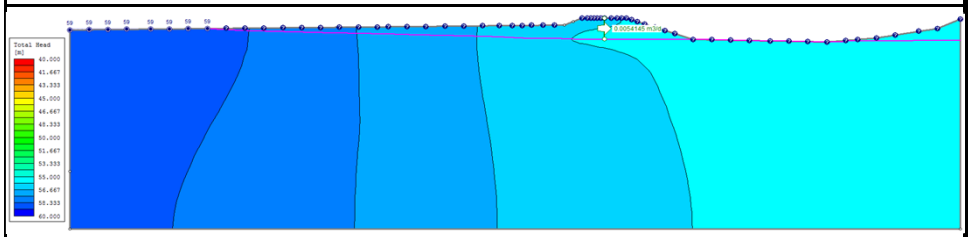
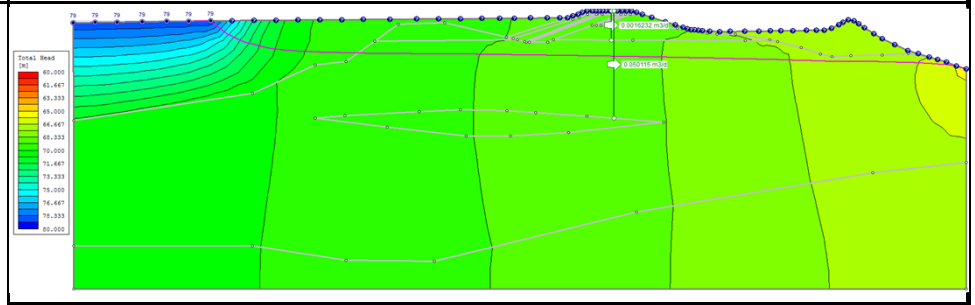
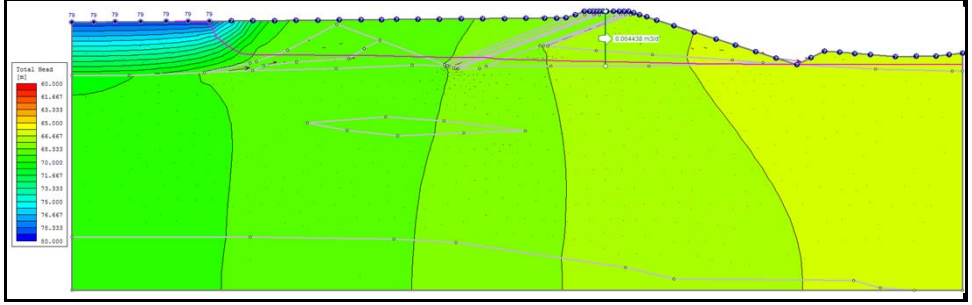
		Finite Element Model Setup		
Job No: 1CM002.018 Filename: Figure3_ModelSetup_1CM002.018_rev0_eh.pptx	Minto Mine	Date: July 2014	Approved: EH	Figure: **3**

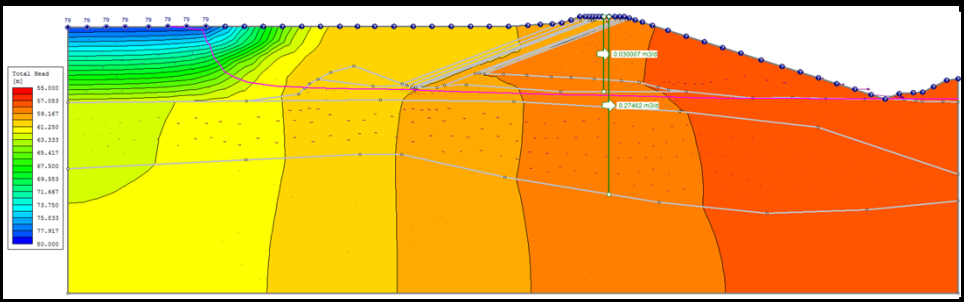
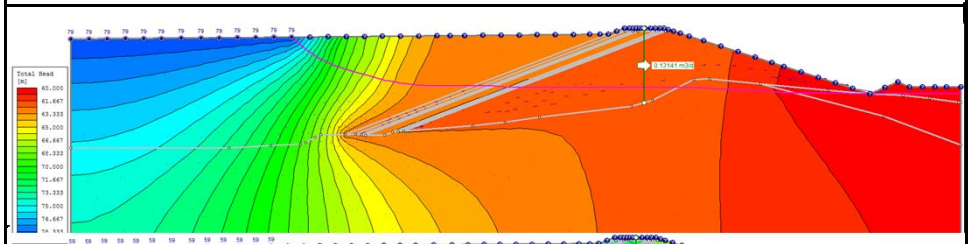
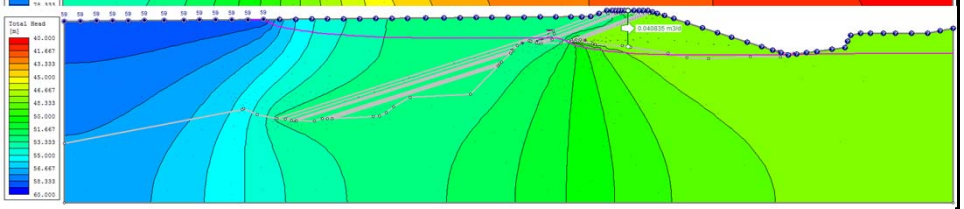
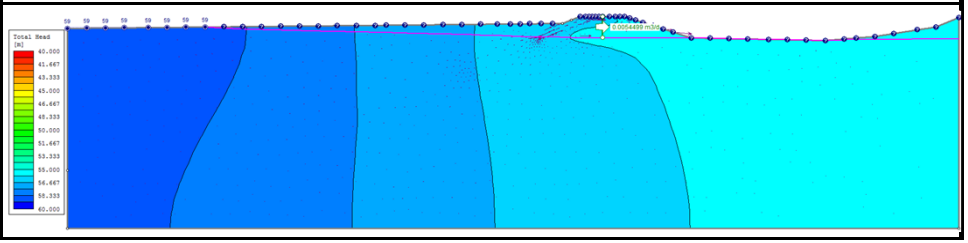


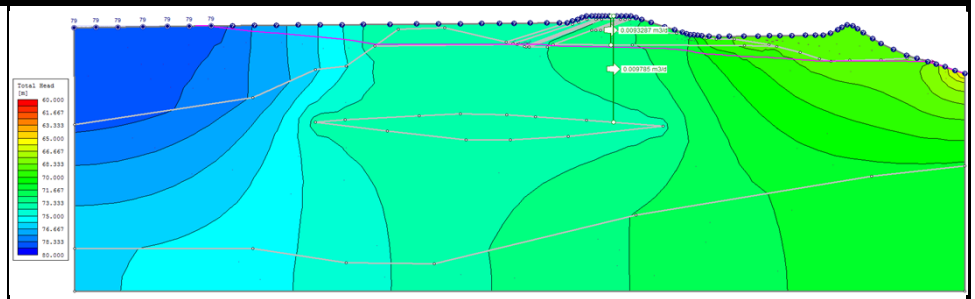
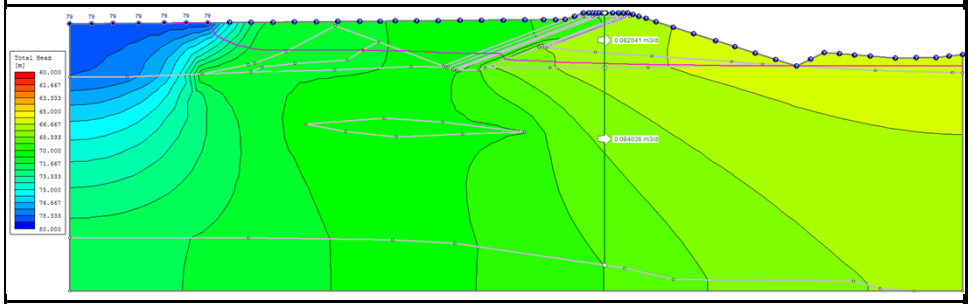
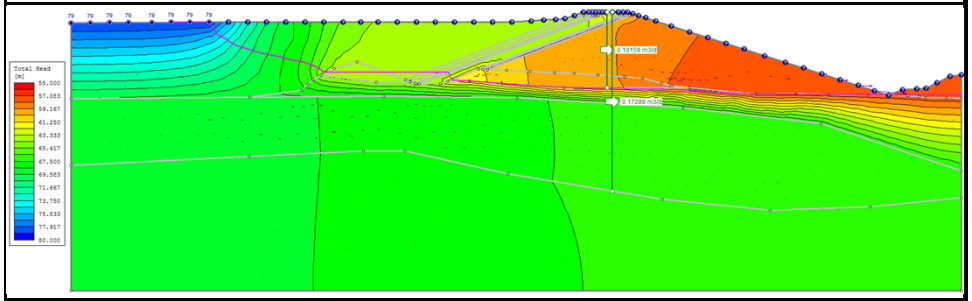
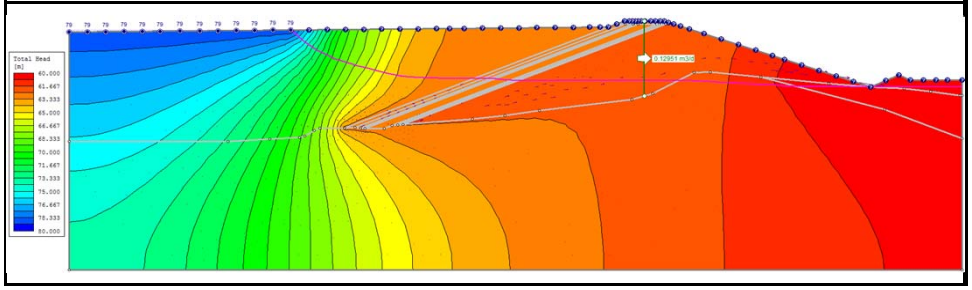
srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	Main Dam Seepage Analysis
		Hydraulic Conductivity Curves of Dam Materials
Job No: 1CM002.018 Filename: Figure3_Kcurves_1CM002.018_rev0_eh.pptx	Minto Mine	Date: July 2014 / Approved: EH / Figure: **4**

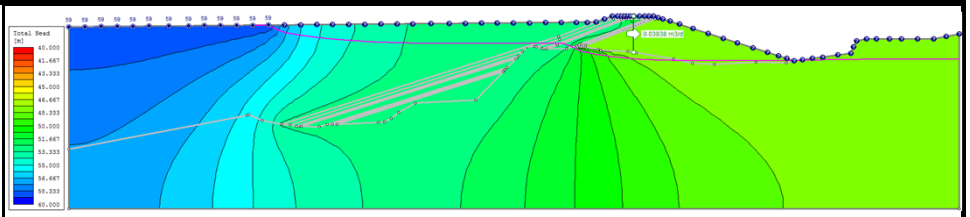
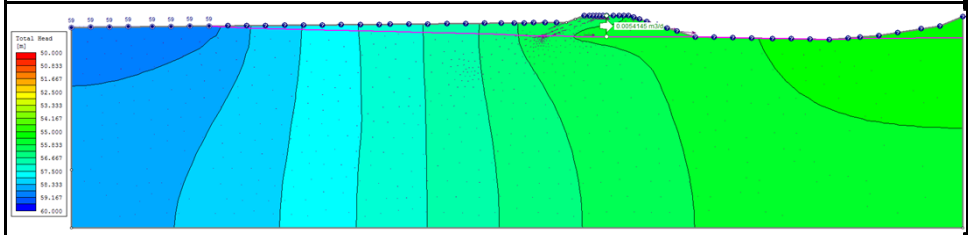
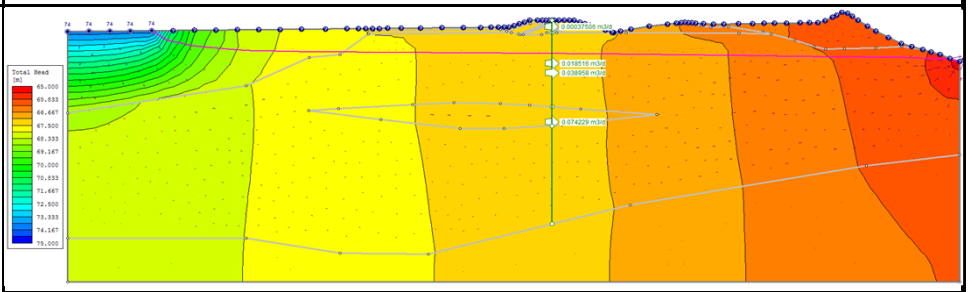
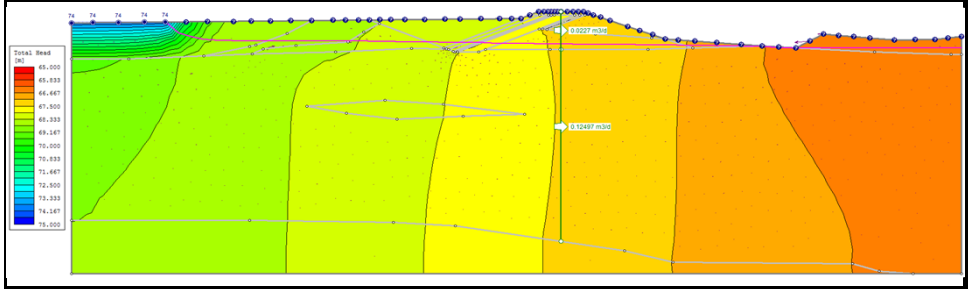
Attachment 1: FEM Model Results

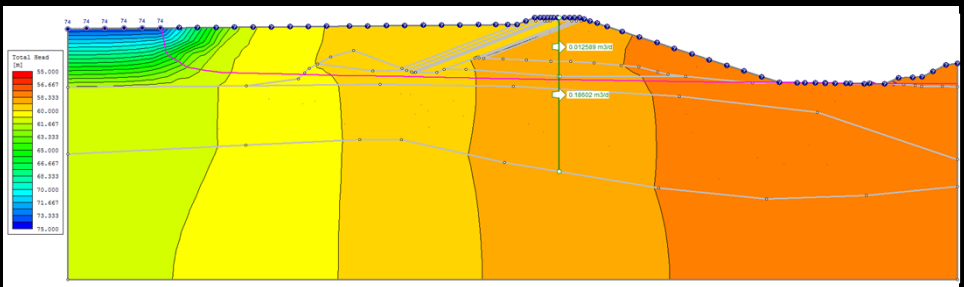
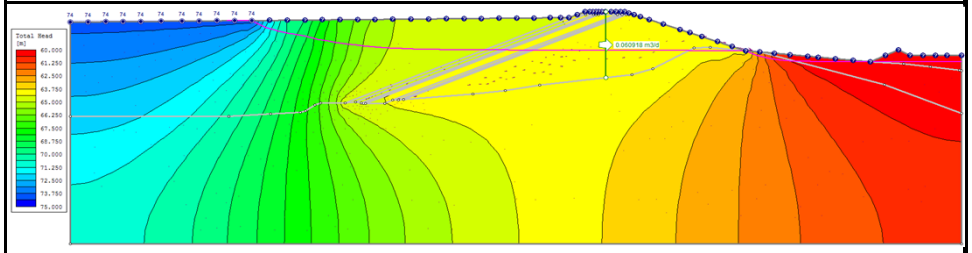
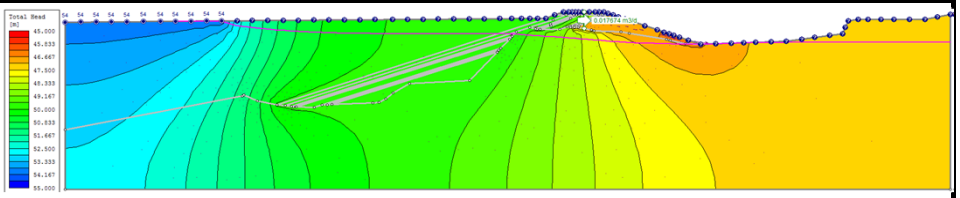
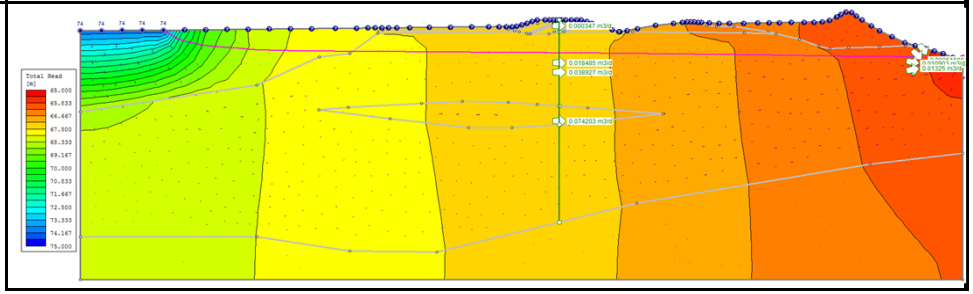
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
1	D5	0.12	20.21	
	D14	8.28		
	D22	2.34		
	D30	6.48		

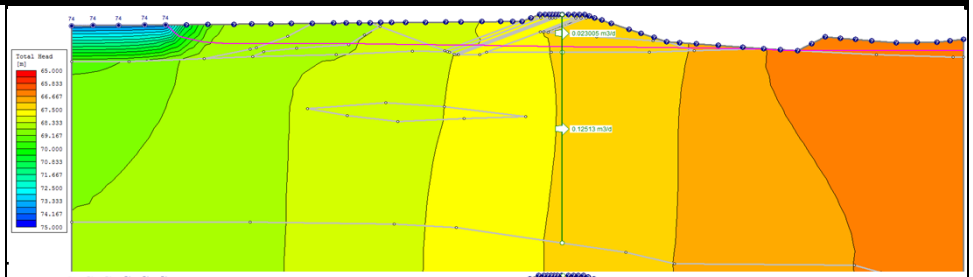
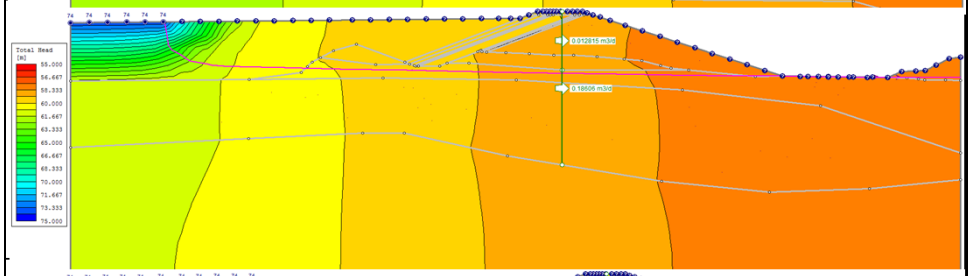
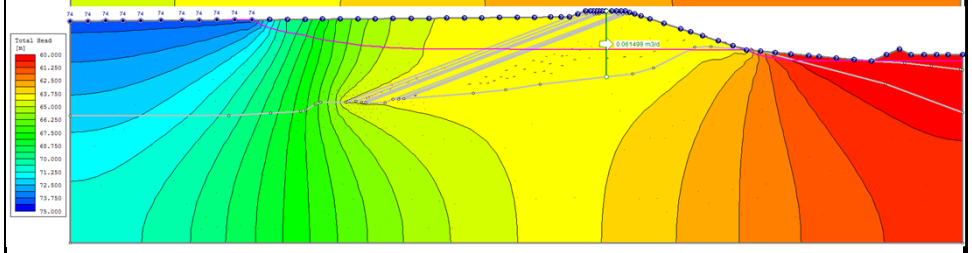
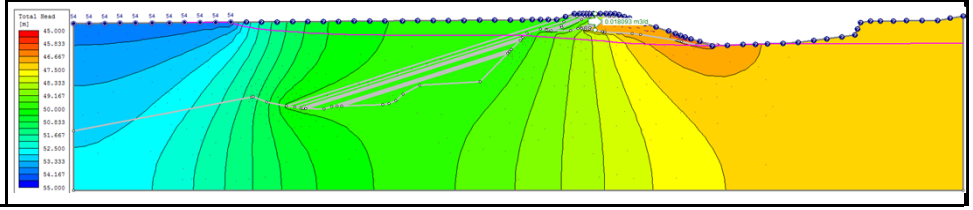
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
	D36	2.76		
	D41	0.24		
	D5	0.12		
	D14	8.38		

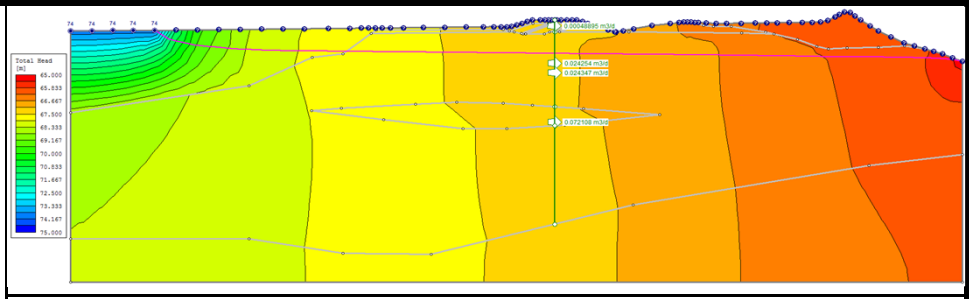
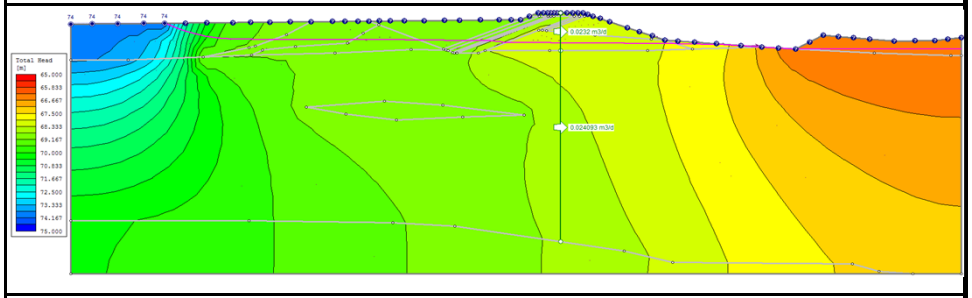
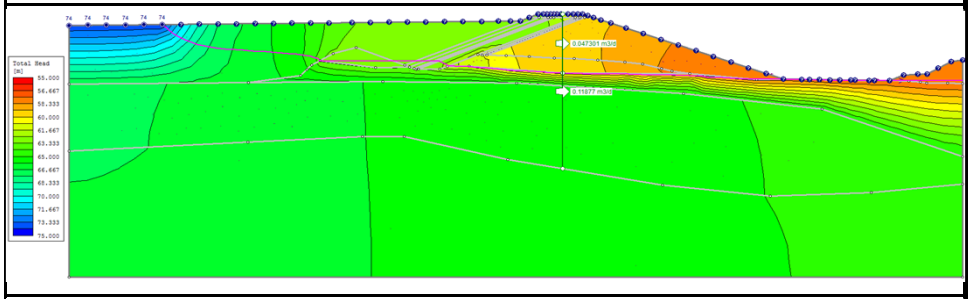
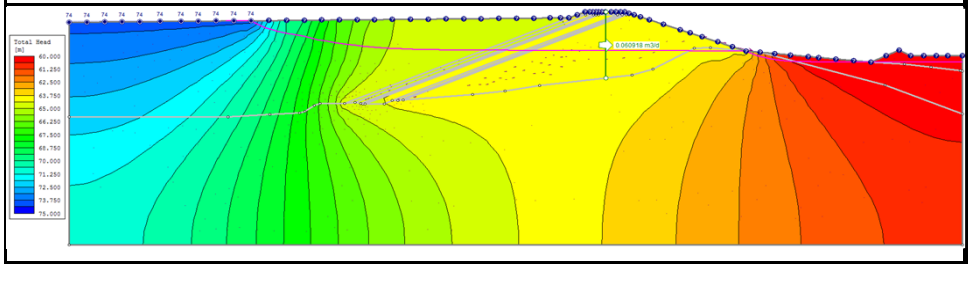
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
2	D22	2.40	20.57	
	D30	6.57		
	D36	2.86		
	D41	0.25		

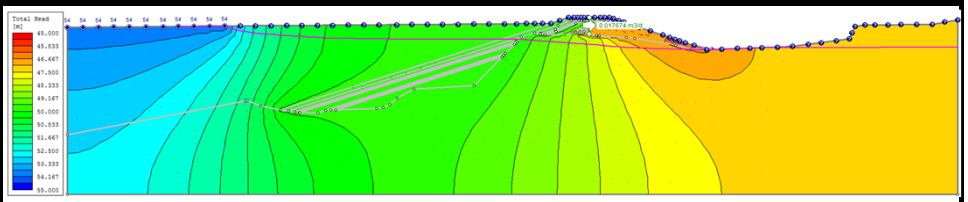
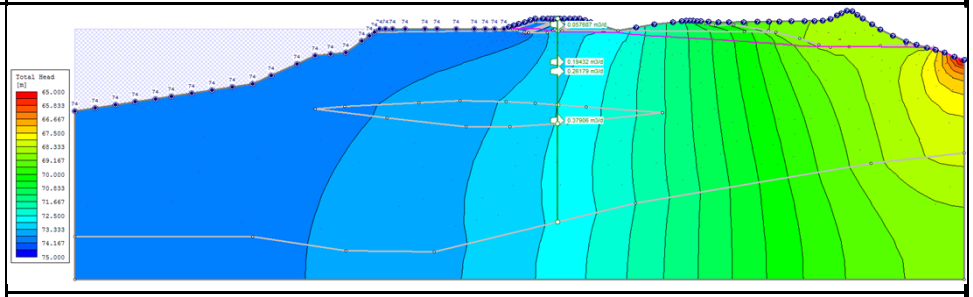
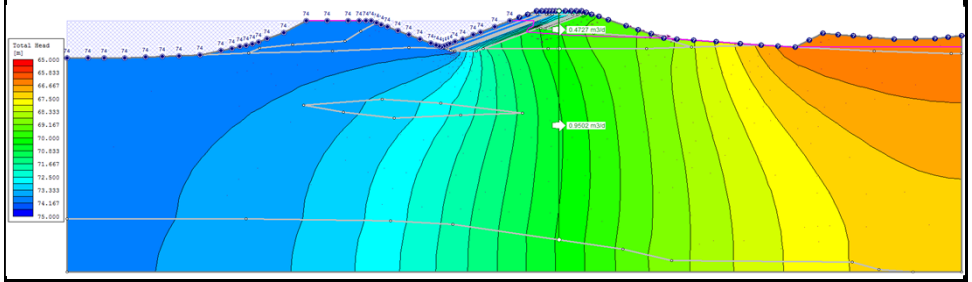
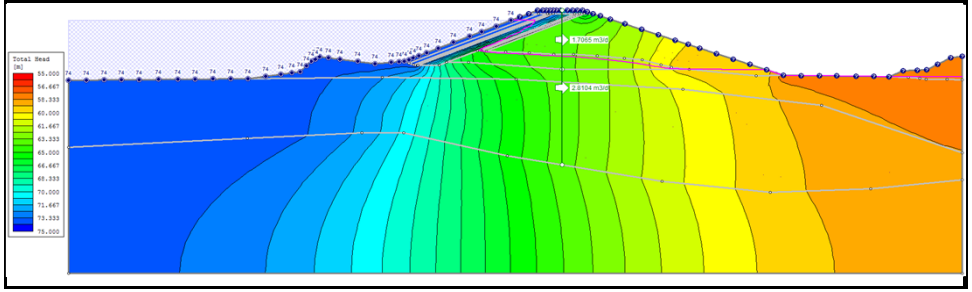
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
3	D5	0.70	28.97	
	D14	10.67		
	D22	8.13		
	D30	6.48		

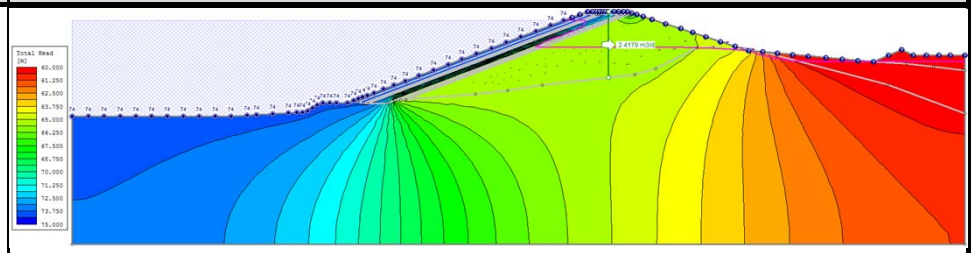
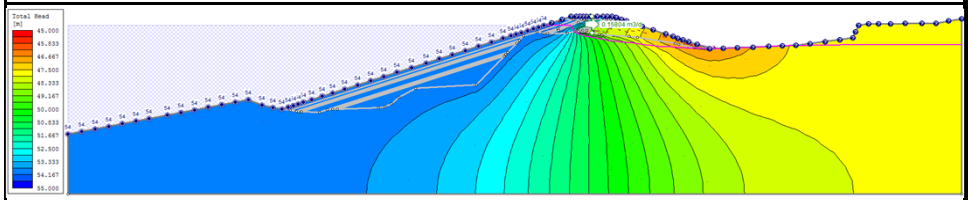
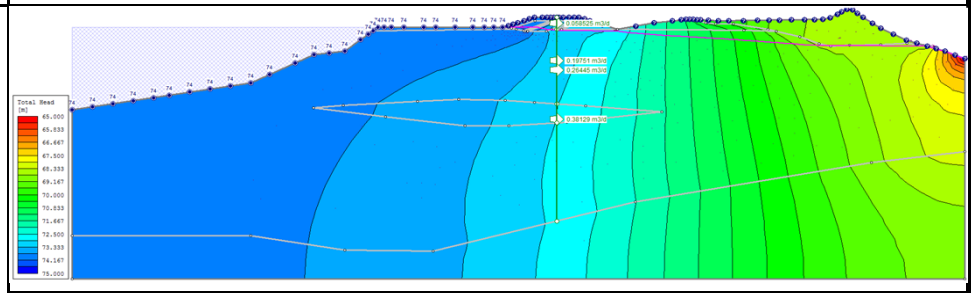
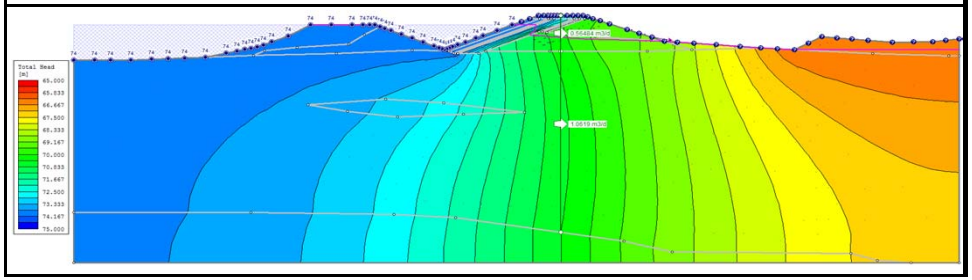
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
	D36	2.76		
	D41	0.24		
	D5	0.02		
	D14	1.70		

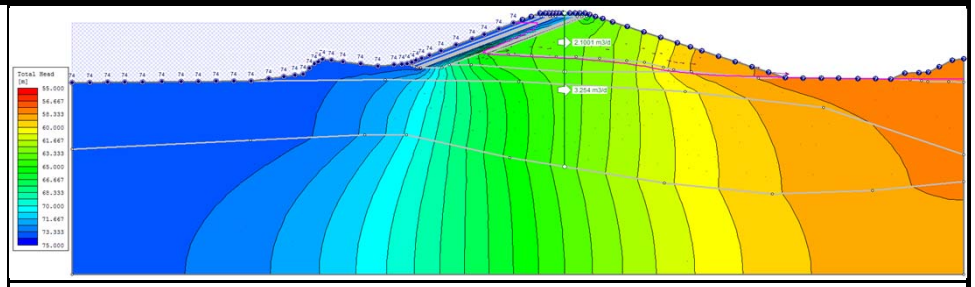
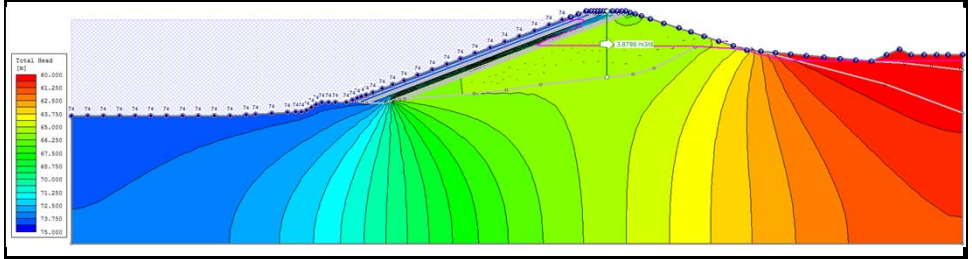
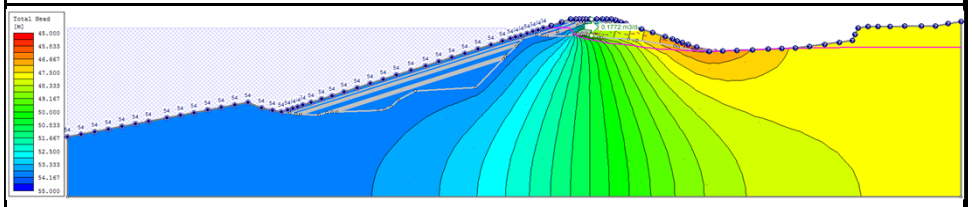
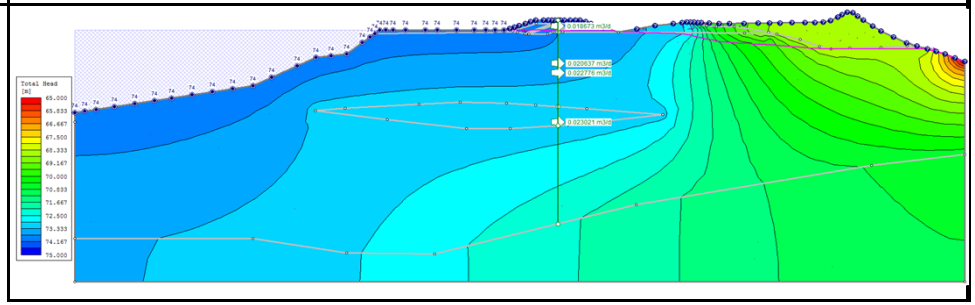
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
4	D22	1.26	7.35	
	D30	3.05		
	D36	1.33		
	D5	0.02		

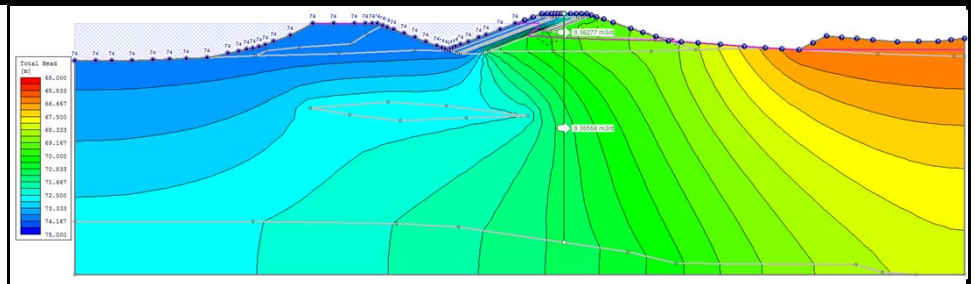
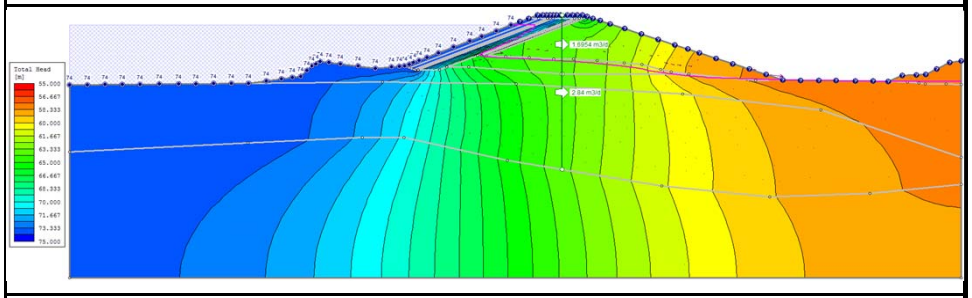
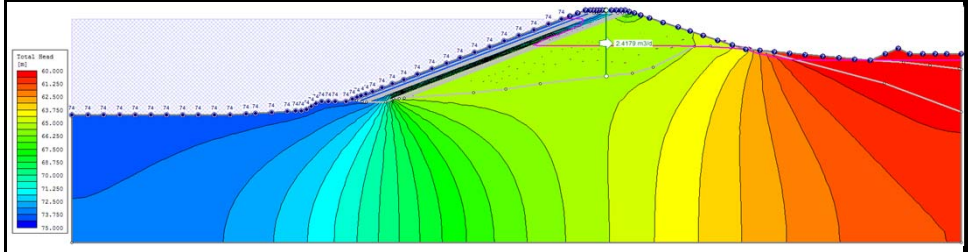
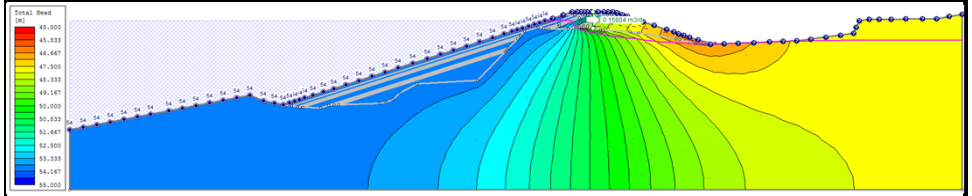
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
5	D14	1.73	7.46	
	D22	1.28		
	D30	3.07		
	D36	1.36		

Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
6	D5	0.02	10.86	
	D14	1.74		
	D22	4.73		
	D30	3.05		

Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
	D36	1.33		
7	D5	2.87	341.72	
	D14	35.45		
	D22	170.65		

Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
	D30	120.90		
	D36	11.85		
	D5	2.93		
	D14	42.36		

Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
8	D22	210.01	462.58	
	D30	193.99		
	D36	13.29		
	D5	0.93		

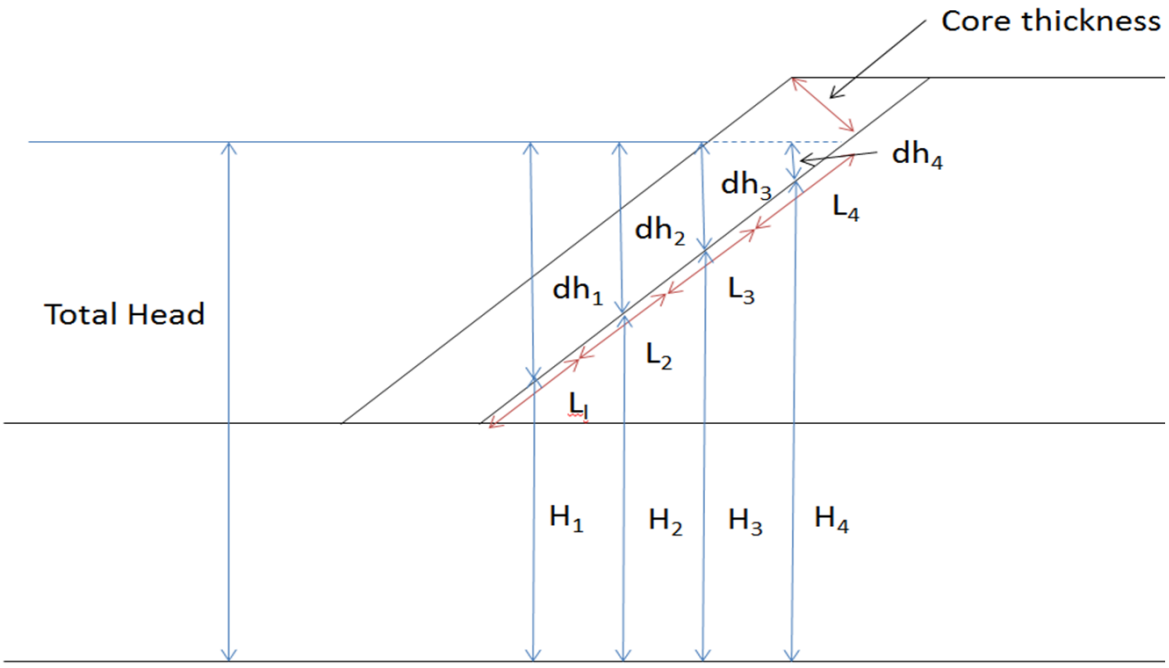
Case	Section	Seepage (through section), m^3/d	Total Seepage through Dam, m^3/d	Results (Total Head Contours)
9	D14	27.21	330.43	
	D22	169.54		
	D30	120.90		
	D36	11.85		

Attachment 2: Darcy Calculation Results

Results - Darcy Hand Calculation, Seepage through Dam Core
807m Water Storage Dam, No Anisotropy due to Compaction

Core K	2.00E-07	m/s
Core Thickness (dl)	2.3	dh
Total Head	74	m

Section	Core Section	Core Section Length (m)	Total Head (m)	Gradient (dh/dl)	Specific Discharge, q ($m^3/d/m$)	Dam Section Length (m)	Flow through Core section (m^3/d)	Total Flow (m^3/d)
D5	1	3.159	73.414	0.25	0.004	50	0.70	0.70
D14	1	5.934	66.254	3.37	0.35	75	26	59
	2	5.939	68.458	2.41	0.25		19	
	3	5.93	70.677	1.44	0.15		11	
	4	5.919	72.902	0.48	0.05		4	
D22	1	5.883	61.972	5.23	0.53	100	53	174
	2	5.895	64.158	4.28	0.44		44	
	3	5.895	66.348	3.33	0.34		34	
	4	5.888	68.535	2.38	0.24		24	
	5	5.892	70.723	1.42	0.15		15	
	6	5.876	72.908	0.47	0.05		5	
D30	1	9.214	48.344	11.15	1.78	50	89	379
	2	9.205	51.766	9.67	1.54		77	
	3	9.218	55.185	8.18	1.30		65	
	4	9.215	58.609	6.69	1.07		53	
	5	9.218	62.031	5.20	0.83		41	
	6	9.202	65.454	3.72	0.59		30	
	7	9.196	68.872	2.23	0.35		18	
	8	9.224	72.287	0.74	0.12		6	
D36	1	7.314	52.738	9.24	1.17	75	88	88
	Total							700



$$Q = k * i * A$$

$$i = dh/dl$$

$$A = area\ through\ which\ flow\ occurs$$

Attachment 3: Liner Leakage Calculation Results

Constant Inputs	
Isotropic Core K (m/s)	2.00E-07
Clay Thickness, D (m)	2.3
Area per liner slice, 812m dam (m^2)	4241
Area per liner slice, 807m dam (m^2)	2592

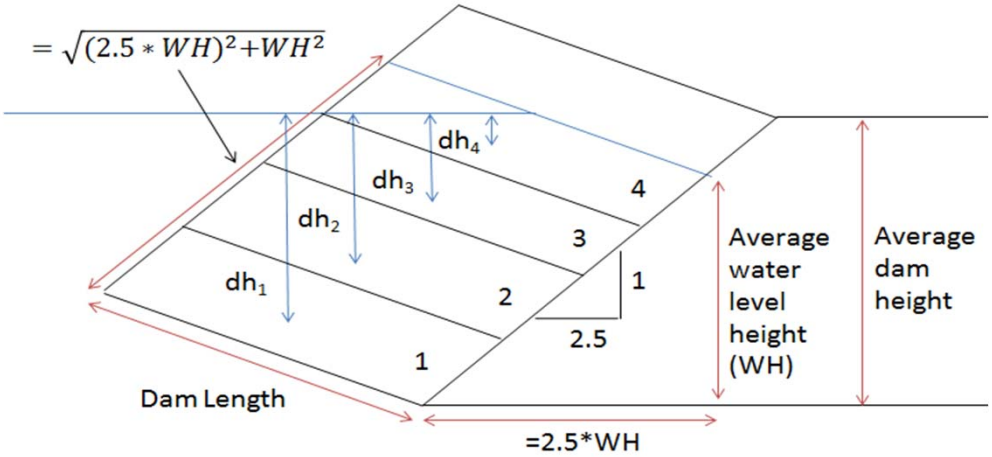
$$Q = n * \left(4 + F\left(\frac{r_o}{D}\right)\right) * r_o * h * k$$

$$F = 2.455 + 0.685 * \tanh(0.6 * \ln\left(\frac{r_o}{D}\right))$$

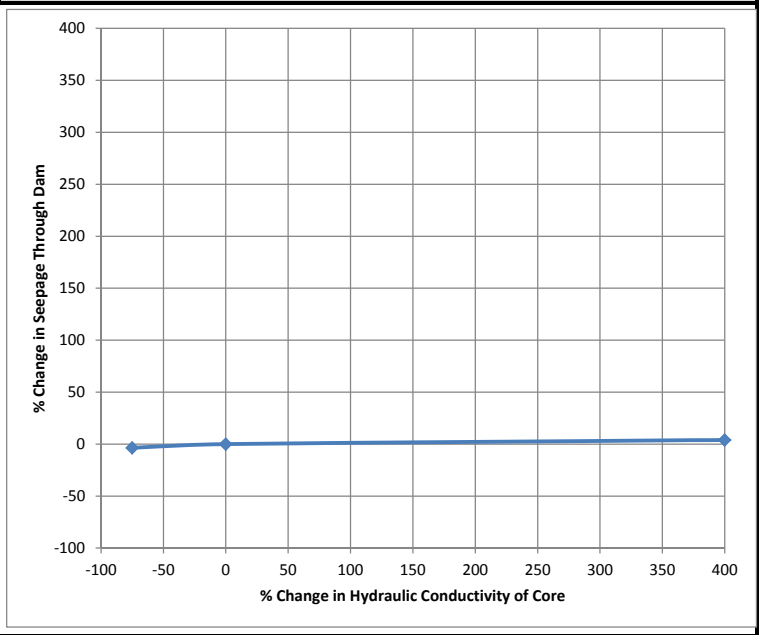
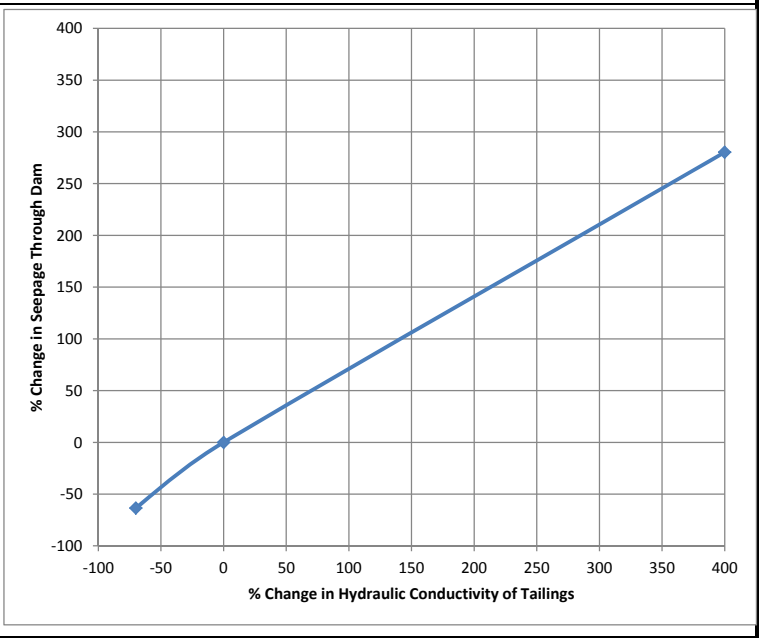
Source: (Rowe and Booker, 1997)

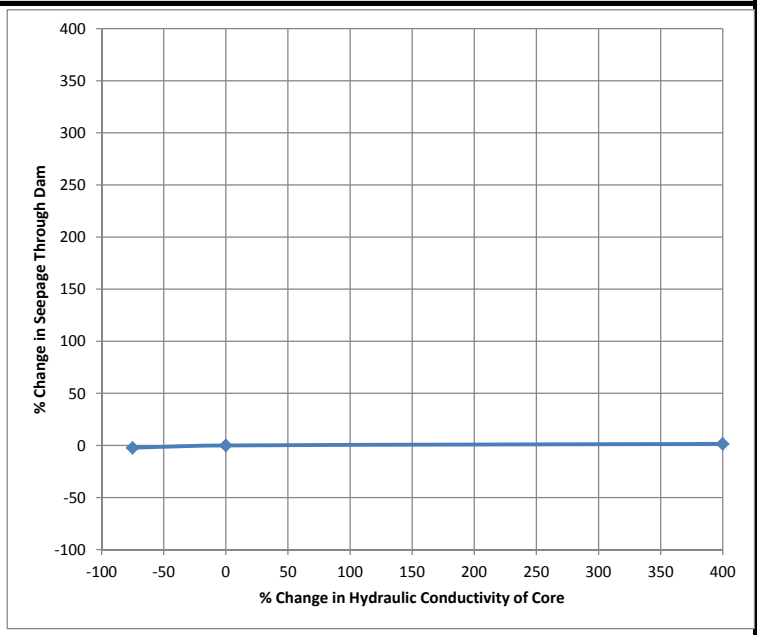
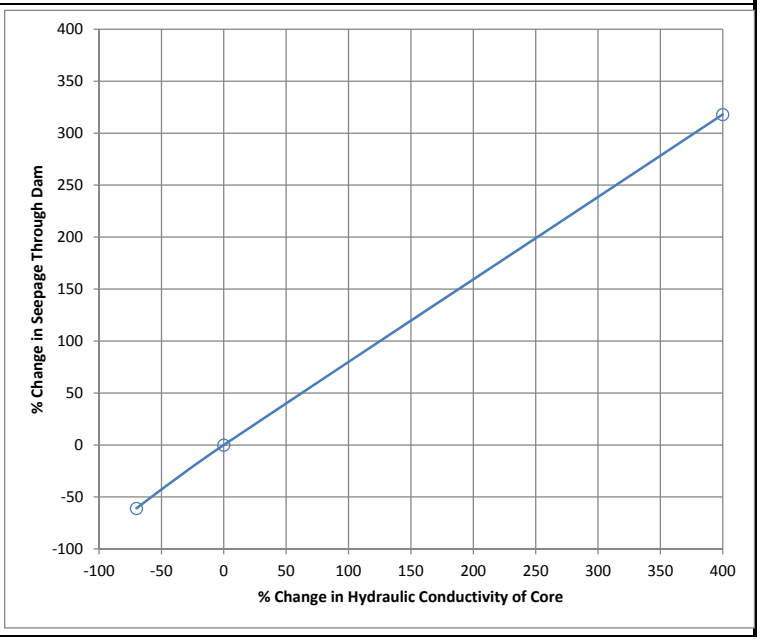
Variable Inputs - Point Defects				
Input	Value	Description	Value	Description
Defects per liner slice, n	5	4/acre good construction, 1/acre manufactured	11	10/acre fair construction, 1/acre manufactured
Defect radius, ro (m)	0.1	20cm dia. Wrinkle	1	2m diameter wrinkle

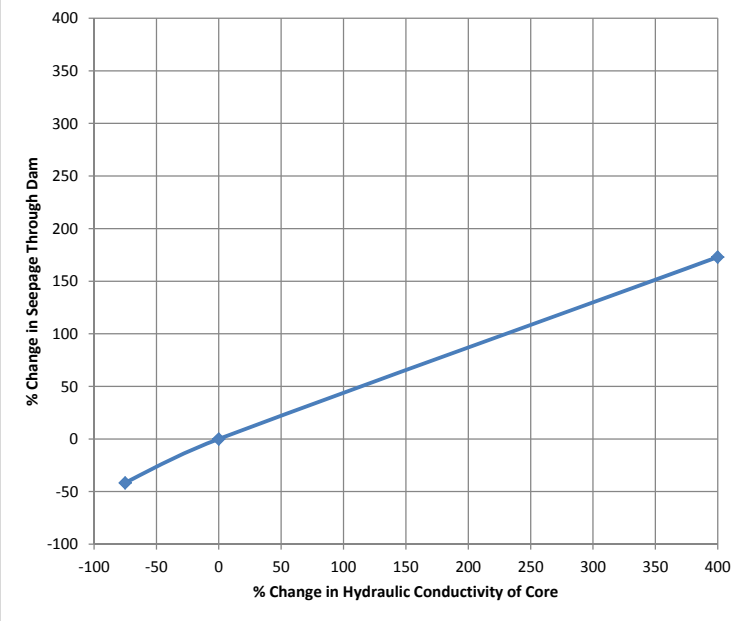
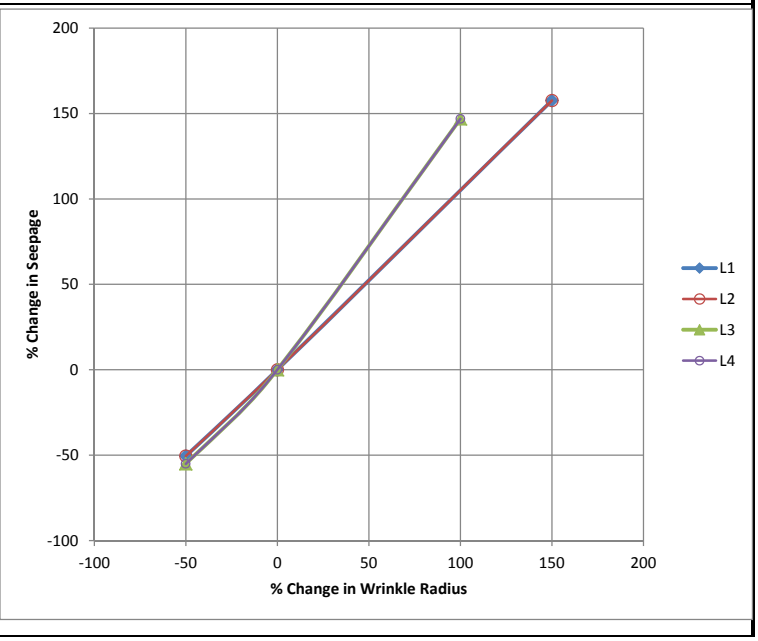
Case	Dam Crest Elevation (m)	Average Water Level Height (m)	Slice	Head (m)	Seepage (m3/d)	Total Seepage (m3/d)
L1	812	14	1	12.25	0.52	1.18
			2	8.75	0.37	
			3	5.25	0.22	
			4	1.75	0.07	
L2	812	14	1	12.25	1.04	2.37
			2	8.75	0.74	
			3	5.25	0.44	
			4	1.75	0.15	
L3	812	14	1	12.25	6.26	14.31
			2	8.75	4.47	
			3	5.25	2.68	
			4	1.75	0.89	
L4	812	14	1	12.25	12.52	28.62
			2	8.75	8.94	
			3	5.25	5.37	
			4	1.75	1.79	
L5	807	11	1	9.625	0.27	0.62
			2	6.875	0.19	
			3	4.125	0.12	
			4	1.375	0.04	
L6	807	11	1	9.625	0.47	1.09
			2	6.875	0.34	
			3	4.125	0.20	
			4	1.375	0.07	
L7	807	11	1	9.625	3.28	7.50
			2	6.875	2.34	
			3	4.125	1.41	
			4	1.375	0.47	
L8	807	11	1	9.625	5.74	13.12
			2	6.875	4.10	
			3	4.125	2.46	
			4	1.375	0.82	

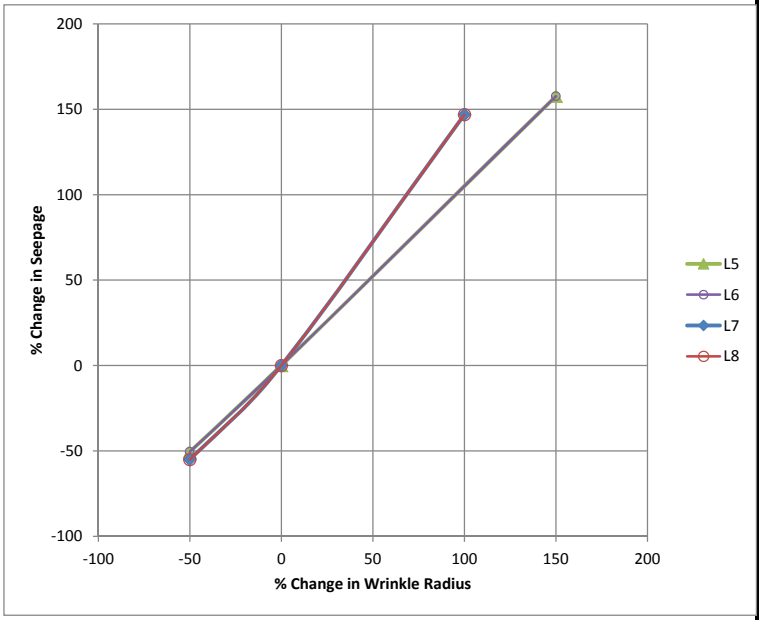


Attachment 4: Sensitivity Analysis Results

Sensitivity Case	Performed on Dam Case	Parameter Varied	Sensitivity Graph
A	1	Dam Core K	
B	1	Tailings K	

Sensitivity Case	Performed on Dam Case	Parameter Varied	Sensitivity Graph
C	4	Dam Core K	
D	4	Tailings K	

Sensitivity Case	Performed on Dam Case	Parameter Varied	Sensitivity Graph
E	7	Dam Core K	
F	L1-L4	Wrinkle radii	

Sensitivity Case	Performed on Dam Case	Parameter Varied	Sensitivity Graph
	L5-L8	Wrinkle radii	

Appendix O: Minto Main Dam Preliminary Design – Dam Instrumentation

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Memo

To:	File	**Client:**	Minto Explorations Ltd.
From:	Emma Helmers, Iozsef Miskolczi	**Project No:**	1CM002.018
Cc:	Maritz Rykaart	**Date:**	July 17, 2014
Subject:	Main Dam Instrumentation		

1 Introduction

This memo summarizes the recommended instrumentation to allow performance monitoring of the dam.

2 Instrumentation

2.1 Concepts

The dam foundation is subject to differential thaw and consolidation settlement (SRK 2014a; SRK 2014b). During the operational phase, this settlement must be within allowable tolerances to ensure adequate functioning of the geomembrane liner and to ensure that the design freeboard is maintained. At closure, the allowable freeboard must be maintained. It is therefore important to monitor the ground temperature conditions, ongoing settlement and deformation to provide a direct link to how the dam's performance may be affected.

Although the fine grained core is protected through the inclusion of an appropriate filter zone, it remains important that there is no undue build-up of pore pressures, and thus, monitoring of areas where these pressures may build-up is important.

The instrumentation plan described in the following sections is illustrated in Figures 1 to 3.

2.2 Inclinometers

Four proposed inclinometer installations are listed in Table 1. The inclinometer tubes will be installed through the dam core into the underlying bedrock. Since deformation of the bedrock is not expected, this will provide a reference point for deformations in the dam core due to thaw induced or other forms of lateral movement or creep.

Table 1: Proposed Inclinometer Locations

Inclinometer	Chainage Along Dam (0+m)	Location Description
MD-inc-1	0+130	Dam crest into bedrock
MD-inc-2	0+130	1/3 of the way down the downstream slope, into bedrock
MD-inc-3	0+170	Dam crest into bedrock
MD-inc-4	0+170	1/3 of the way down the downstream slope, into bedrock

Inclinometer readings need to be taken manually at frequencies not exceeding one month intervals. If the dam is raised to account for capacity loss due to ice entrainment, all inclinometer installations will have to be re-installed as raising and extending existing installations is not practical.

2.3 Vibrating Wire Piezometers

The proposed locations for vibrating wire piezometers to measure pore pressure build-up in the dam core are listed in Table 2. These locations are areas where, due to the geometry of the dam, problems are the most likely.

Table 2: Proposed Vibrating Wire Piezometer Locations

Piezometer	Chainage along Dam (0+m)	Location Description
MD-VW-1	0+150	Toe of the core, downstream side
MD-VW-2	0+280	Toe of the core, downstream side

The vibrating wire piezometers may be installed in steel tubes for protection from machinery and placed behind the core during construction. The cable attached to the piezometers can be run along the downstream side of the core and up to the crest of the dam where it can be attached to a data logger in order to enable automatic data recording.

2.4 Ground Temperature Cables

Ground temperature cables (thermistors) need to be installed as described in Table 3Table 3Each cable should contain between 10 and 20 beads and must be spread such that information is gathered both within the core, the permafrost foundation and into bedrock. All cables should be directed to a central data logger to allow for automatic data collection.

Table 3: Proposed Thermistor Locations

Thermistor	Chainage Along Dam (0+m)	Location Description
MD-TH-1	0+080	Dam crest down into bedrock
MD-TH-2	0+140	Dam crest down into bedrock
MD-TH-3	0+150	Upstream toe, from foundation excavation slope down into bedrock
MD-TH-4	0+160	Downstream toe, from foundation excavation slope down into bedrock
MD-TH-5	0+190	Dam crest down into bedrock

Ground temperature cables should be installed into boreholes backfilled with sand or similarly thermally conductive material.

2.5 Survey Prisms

Deformation and settlement should be directly monitored through frequent surveys using survey prisms (or basic settlement points) permanently installed along the crest and downstream slope of the dam. The proposed installation plan is as follows:

- Every 25 m along the crest of the dam on both the upstream and downstream sides of the crest from 0+00 to 0+250 m;
- Every 50 m along the crest of the dam on both the upstream and downstream sides of the crest from 0+250 to 0+450 m;
- Every 25 m along the downstream slope of the dam, half way down the slope from the crest, from 0+00 to 0+250 m; and
- Every 50 m along the downstream slope of the dam, half way down the slope from the crest, from 0+250 to 0+450.

The survey prisms should be embedded in large boulders which are in turn embedded into the waste rock forming the crest and downstream slopes. This will ensure that the prisms themselves are subject to minimal movement so that differential settlement measurements are as accurate as possible.

3 Monitoring

3.1 During Construction

Where practical, instrumentation will be installed as construction occurs and manual readings will need to be taken regularly in order to ensure they are in proper working order.

3.2 Post-Construction

All instrumentation installed in the dam with the exception of the survey prisms and inclinometers will be connected to data loggers in order to enable automatic recordings. The data loggers should be downloaded at regular intervals of one month or less.

Surveys and inclinometer readings need to be carried out at monthly intervals until alternate monitoring instructions are provided by the Engineer-of-Record.

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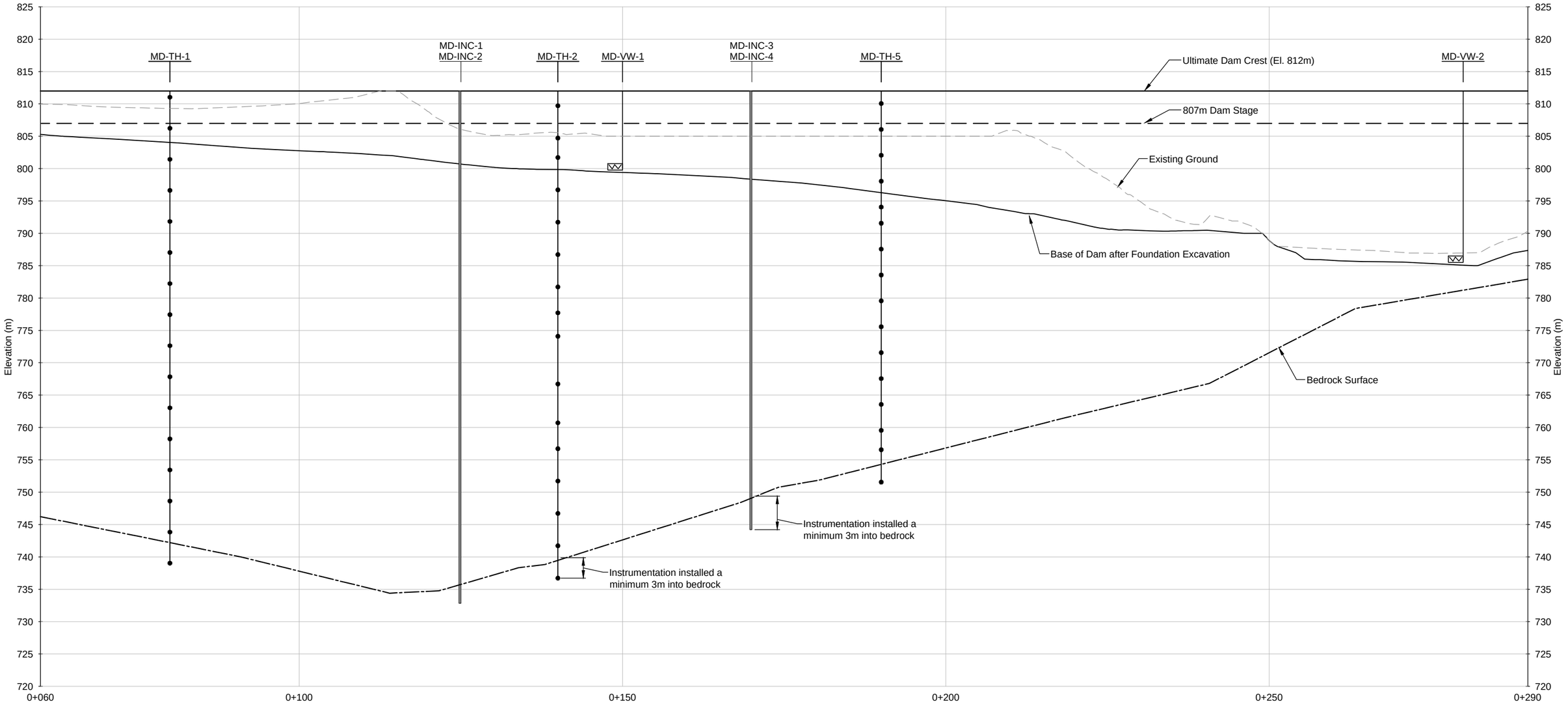
The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

4 References

SRK (2014a). Minto Main Dam Thermal Analysis and Settlement Predictions. Project Number: 1CM002.018. Submitted to Minto Explorations Ltd. July 2014.

SRK (2014b). Minto Main Dam Consolidation Settlement Assessment. Project Number: 1CM002.018. Submitted to Minto Explorations Ltd. July 2014.

Figures



INSTRUMENTATION SHOWN ALONG DAM CENTERLINE PROFILE

0 2 4 6 8 10
Scale in Metres

LEGEND

- Existing Ground
- Top of Bedrock
- 807m Dam Stage

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SRK JOB NO.: 1CM002-018
FILE NAME: 1CM002_018-1.dwg

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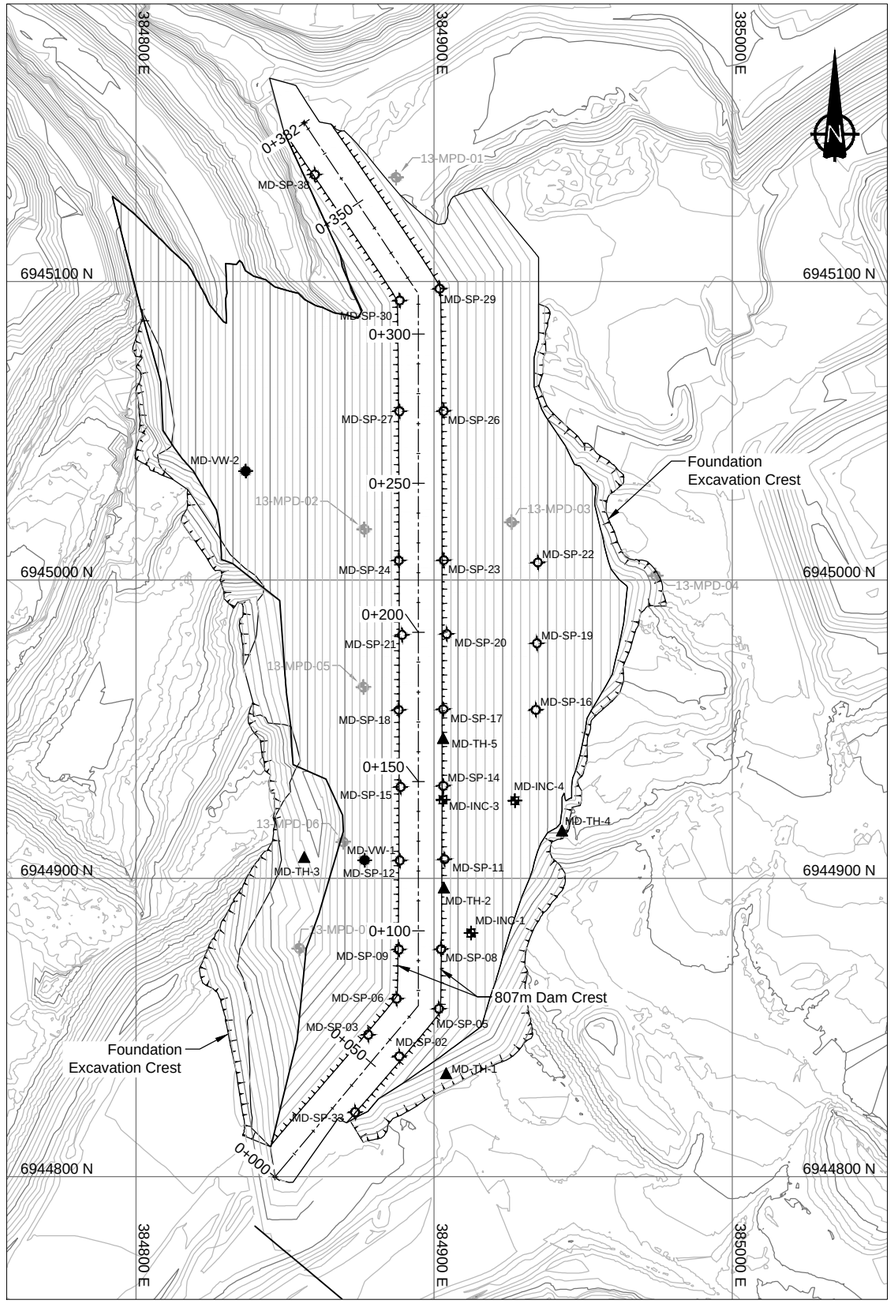
MINTO MINE

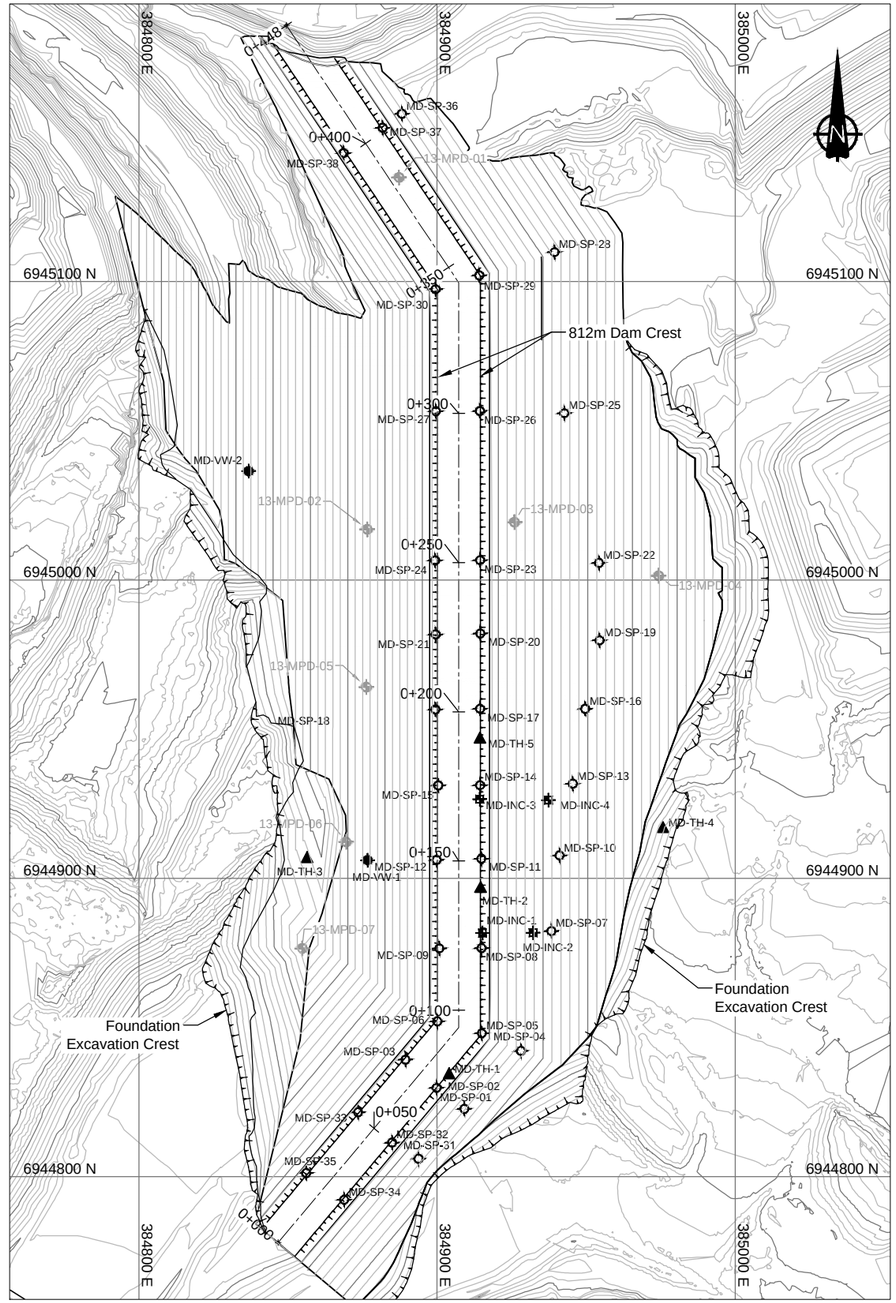
Main Dam Instrumentation

Instrumentation Shown on Dam Centerline Profile

DATE:	APPROVED:	FIGURE:
May 2014	MJM	1



PLAN VIEW OF 807m DAM



PLAN VIEW OF 812m DAM

0 10 20 30 40 50
Scale in Metres

LEGEND

- Surface Monitoring Location
- Vibrating Wire Piezometer
- Inclinometer Location
- Thermistor String Location
- Borehole Location

NOTES

1. Contour interval 1m.
2. Topography received from mine site.

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FILE NAME: 1CM002_018-1.dwg

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MINTO MINE

Main Dam Instrumentation

807m and 812m Main Dam Instrumentation

DATE:	APPROVED:	FIGURE:
May 2014	MJM	2

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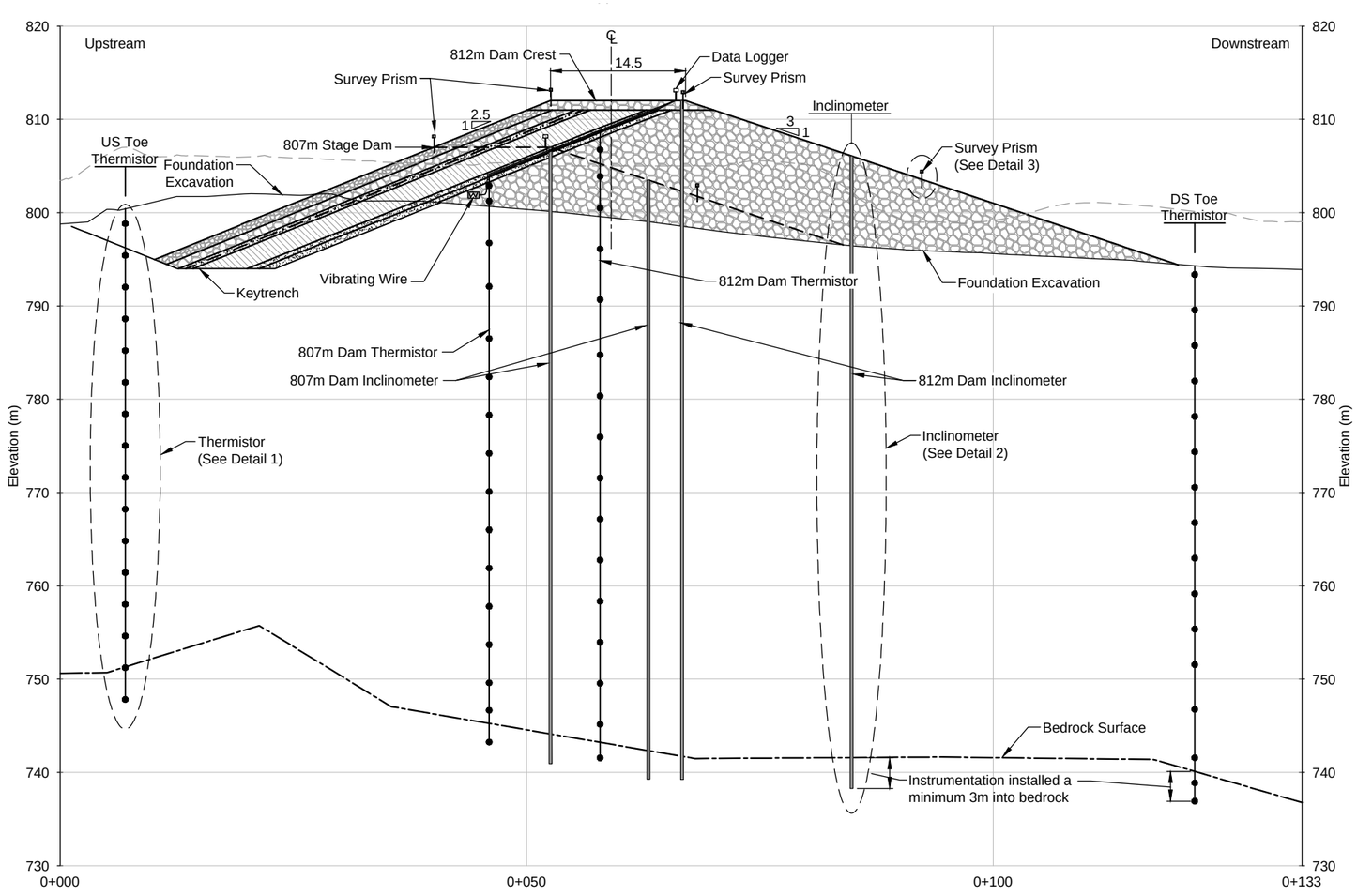
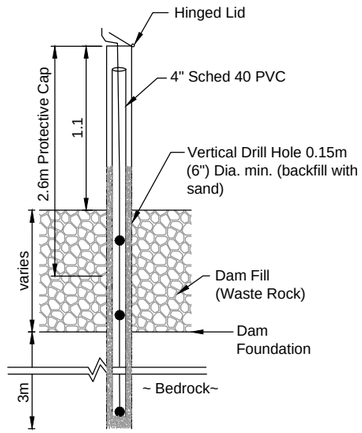


INSTRUMENTATION SECTION

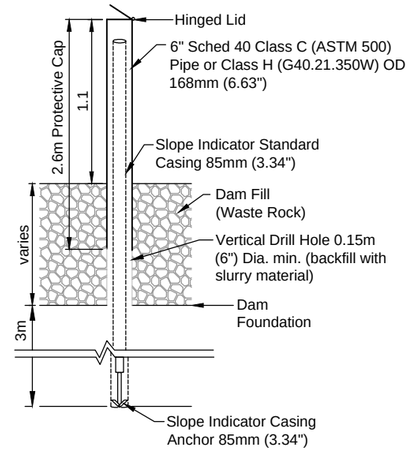
0 2 4 6 8 10
Scale in Metres

LEGEND

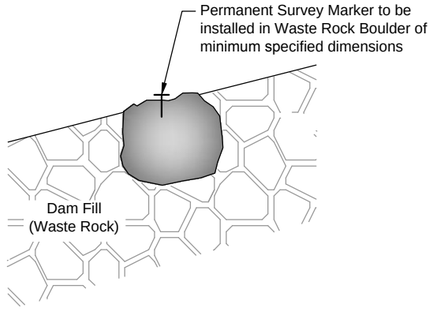
- Existing Ground
- Top of Bedrock
- 807m Dam Stage
- Liner
- Core Material
- Transition Material
- Waste Rock
- Rip Rap



DETAIL 1
1 THERMISTOR INSTALLATION
Not to Scale



DETAIL 2
2 INCLINOMETER INSTALLATION
Not to Scale



DETAIL 3
3 SURFICIAL SURVEY POINT INSTALLATION
Not to Scale

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SRK JOB NO.: 1CM002-018		Instrumentation Section and Details	
FILE NAME: 1CM002_018-1.dwg	MINTO MINE	DATE: May 2014	APPROVED: MJM / FIGURE: 3

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Appendix P: Preliminary Technical Specifications – Main Dam

Preliminary Technical Specifications - Main Dam, Minto Mine

Revision B – Issued for Licensing

Prepared for

Minto Explorations Ltd.

Prepared by

SRK Consulting (Canada) Inc.
1CM002.018
July 2014

Preliminary Technical Specifications - Main Dam, Minto Mine

Revision B – Issued for Licensing

July 2014

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Project No: 1CM002.018

File Name: MainDamTechSpec_Report_1CM002-018_AT_PM_EMR_20140718_FNL

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1 General Requirement

1.1 Part 1 – General

1.1.1 Documents

1. This section of the Specifications forms part of the Contract Documents and are to be read, interpreted and coordinated with all other parts.

1.1.2 Revision Summary

1. Table 1-1 provides a summary of the revision history of these Technical Specifications.

Table 1-1: Revision History of these Technical Specifications

Revision	Status	Date	Major Changes
A	Issued for Review	May, 2014	First Issue
B	Issued for Licensing	July, 2014	Editorial Changes

1.1.3 Definitions

2. The following definitions and interpretations shall apply to these Technical Specifications:

   (a) PROJECT means the total Main Dam Project contemplated, of which the Works described in the Document may be the whole or part.

   (b) WORKS is defined as the entire complete construction as defined by this Document, or the various separately identifiable parts thereof.

   (c) CONTRACT DOCUMENTS are defined as the agreement, addenda, Contractors bid when attached as an exhibit to the agreement, the bonds, the general conditions, the supplementary conditions, these Specifications, the Drawings, together with all Modifications and approved change orders issued after the execution of the agreement.

   (d) SPECIFICATIONS are defined as this document of Specifications prepared by SRK Consulting (Canada) Inc. on behalf of the Owner. These Specifications are to be read, interpreted and coordinated with all Drawings, Modifications, approved change orders, or any other relevant documents produced by the Engineer and Owner.

   (e) DRAWINGS are defined as all engineering drawings, plans, sketches and maps issued with these Specifications, or subsequently, as approved and signed by the Engineer.

   (f) MODIFICATIONS are defined as changes made to the Specifications and/or Drawings, which have been approved and signed by the Engineer. These Modifications can be issued at any time, including after issuance of these Specifications and any accompanying Drawings and/or other Modifications.

   (g) SUBMITTALS as defined as any documentation, as outlined in this Document that is used as formal means of communications and record during execution of the Works, and originated by any of the Responsible Parties.

(h) RESPONSIBLE PARTIES:

i. OWNER is defined as Minto Explorations Ltd.

ii. ENGINEER (also, ENGINEER-OF-RECORD) is defined as a representative appointed and authorized by the Owner of the Works described in this Document.

iii. CONTRACTOR is defined as the party or appointed representative of the party that has an agreement with the Owner to execute the Works defined in this Document.

iv. SUB-CONTRACTOR is defined as the party or appointed representative of the party that has an agreement with the Contractor to execute specialized components of the Works defined in this Document that cannot be carried out by the Contractor.

v. ENVIRONMENTAL MONTIOR is defined as the party or appointed representative of the party that has an agreement with the Owner to act as Environmental Monitor for the Project.

vi. SURVEYOR is defined as the party or appointed representative of the party that has an agreement with the Contractor to act as Project Surveyor for the execution of the Work defined in this Document. The Surveyor shall be a licensed with the Association of Canada Land Surveyor and has equipment and means on site to carry out horizontal and vertical ground surveys with accuracy of ±2 mm. The Surveyor shall also have the equipment and means to prepare Digital Terrain Models (DTM) and drawings on site that is compatible with AutoCAD 2012 or later. The Surveyor reports to the Contractor, but will be available for use by the Engineer as required.

vii. QUALITY CONTROL is defined as tests and protocols performed on and off site to ensure the quality of products and materials meeting the Specifications herein prior to use in the Project.

viii. QUALITY ASSURANCE is defined as tests and protocols performed on in-situ products and materials, after placement and deployment, to ensure their quality meet the Specifications herein.

(i) ON-SITE MATERIAL is defined as borrow materials obtained from within designated on-site borrow or stockpiles.

(j) OFF-SITE MATERIAL is defined as material obtained from sources other than on-site.

(k) RECORD DOCUMENTS are defined as the documents prepared and certified by a Land Surveyor, Material Testing Technician, Quality Control and Assurance Personnel, Specialist Professionals, or any other parties documenting any aspect of the Works.

(l) PRODUCTS are defined as processed fill material, geosynthetics, materials, machines, components, equipment, fixtures, and instruments forming the Works. Products may also include existing material or components that are approved for reuse.

(m) SLOPES are defined in all instances in these Specifications and Drawings in terms of horizontal distances to vertical distances (e.g. 2H:1V, shall be read 2 horizontal units to 1 vertical unit).

(n) EQUIPMENT means all construction mobile equipment that will be used for the Works.

(o) NATURAL GROUND is defined as undisturbed, natural in-situ soil and bedrock that are not placed from previous mine works or construction.

(p) EXISTING GROUND is defined as current condition of the ground conditions as confirmed by the initial survey prior to Work. The existing ground could be a combination of natural ground and fill placed from mine works or other construction activities.

1.1.4 Summary of Works

1. The Contractor will be responsible for ensuring that all Works defined in this Document be executed in accordance with all appropriate permits and approvals. Furthermore, the Contractor is responsible for ensuring that all the Works are carried out in accordance with all relevant Territorial and Federal Occupational Health and Safety Regulations and the Minto Mine Health and Safety protocols.

2. The Works covered by these Specifications include, but are not limited to the following:

   (a) Implementation, operation, maintenance and removal of construction water management, and temporary construction sediment and runoff control.

   (b) Initial survey of existing ground prior to Works for Owner's approval, ongoing progress survey during execution of the Works and as-built survey on all construction components for the Engineer's approval.

   (c) Clearing, stripping, and excavation in identified areas.

   (d) Development, management, operation and closure of borrow areas and borrow access roads.

   (e) Excavation of key trench and foundation preparation to receive construction materials specified herein and on the Drawings.

   (f) Removal of woody debris and other organic material found within excavation limits and key trench areas.

   (g) Procession and placement of construction materials in the Main Dam specified herein and on the Drawings.

   (h) Installation of geosynthetics specified herein and on the Drawings.

   (i) Excavation and construction of the spillway.

   (j) Supply and installation of all monitoring instrumentation as specified on the Drawings.

(k) Removal of all temporary structures and access used during construction of the Works and clean-up of the construction areas, borrows, and stockpile areas.

(l) Provide all as-built and quality control document to the Engineer and Owner as specified herein.

1.1.5 Contradictions

1. Should any contradiction, either implied or real, be encountered between the Specifications and the Drawings, the Contractor shall:

   (a) Notify the Owner and Engineer within 24 hours of realization; and

   (b) Stop all Works that concern the contradiction until the contradiction is remedied or clarified by the Engineer.

2. The decision of the Engineer is final.

1.1.6 Owner's Responsibilities

1. The Owner, in the context of the Works defined in this Document, shall:

   (a) Be the formal liaison between all parties.

   (b) Become familiar with and acquire all relevant permits, approvals and any other administrative matters which may impact the Works. The Owner's designated representative will ensure that all appropriate approvals have been obtained and that all contractual conditions have been satisfied.

   (c) Before proceeding with the Works, examine all Drawings and report to the Engineer any apparent discrepancies or interferences. The Engineer shall at all times retain the right to make revisions to the Specifications and Drawings.

   (d) Ensure the mining operation routes and general surface infrastructure are adjusted so that they do not interfere with the Works.

   (e) Ensure an appropriate work space, necessary facilities and communication equipment are available to the Engineer and his representatives to perform their duties on site.

   (f) Ensure that the Engineer and his representatives receive appropriate site specific health and safety training and/or orientation whilst on site.

1.1.7 Contractor's Responsibilities

1. The Contractor, in the context of the Works defined in this Document shall:

   (a) Comply with all Federal, Territorial and Owner specific work safe rules and regulations, and any other relevant required health and safety regulations.

   (b) Provide the Owner with copy of their health and safety plan, which has been specifically prepared for this Project.

(c) Become familiar with the relevant regional and site-specific conditions that deviate from the Specifications and Drawings, permits and inform the Owner when a delay is anticipated or the Engineer if a technical issue arises.

(d) Be responsible for making their own measurements, Quality Control and installation of the Works to fit the conditions encountered.

(e) Compile, maintain and update a construction schedule for the Works.

(f) Before proceeding with the Works, examine all Drawings, reports and Specifications herein for any apparent discrepancies or interferences to the Engineer. The Engineer shall at all times retain the right to make revisions to the Specifications and Drawings.

(g) Carry out Quality Control as specified in this Document.

(h) Perform construction water management around Work areas.

1.1.8 Engineer's Responsibilities

1. The Engineer, in the context of the Works defined in the Document, shall:

   (a) Provide the Owner and Contractor with Specifications and Drawings, including revisions and modifications, to be able to conduct the Work defined herein.

   (b) Provide the Owner and Contractor with digital Drawings files to facilitate layout of the Work defined herein.

   (c) Provide full-time site Engineer(s) or qualified representative(s) during construction of the Works as defined herein. The Engineer will monitor construction activities to ensure that the Works are constructed in accordance with the Specifications and Drawings.

   (d) Ensure timely technical response as defined herein, to Submittals presented by the Contractor and Owner.

   (e) Carry out Quality Assurance as specified herein.

1.1.9 Codes and Standards

1. The Quality Assurance and Quality Control programs described herein for soil materials shall use testing procedures from the list of American Society of Testing and Material Standards in Table 1-2.

Table 1-2: List of Testing Codes and Standards

Test Standard	Test Description
ASTM D287	Classification of Soils for Engineering Purposes
ASTM D2216	Water (Moisture) Content in Soil and Rock
ASTM D422	Standard Test Method for Particle-Size Analysis of Soils
ASTM D854	Specific Gravity of Soils
ASTM D698-12e1	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort
ASTM D6938-10	Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)
ASTM D5312	Standard Test Method for Evaluation of Durability of Rock for Erosion Control Under Freezing and Thawing Conditions

2. The Quality Assurance and Quality Control testing for geosynthetics material is described in associated sections herein.

1.1.10 Survey

1. All site survey shall be conducted by a licensed surveyor or a technician under their direct supervision.
2. Prior to any work, an initial survey shall be completed and signed off by all parties as existing ground reference. The Engineer and Contractor shall compare the existing ground surveys to the Drawings to ensure there are no major deviations that would impact the design.
3. As-built surveys shall be conducted periodically as the work progresses, and at completion of designated components as shown on the Drawings for Engineer's approval and documentation. The Engineer reserves the right to request an as-built survey on any specific Work for checks and reviews with a 24 hour notice.

1.1.11 Quality Control

1. The Contractor will carry out Quality Control (QC) for the Works defined herein, and will undertake testing at a frequency, and at the locations specified in the various sections of these Specifications and Drawings, or as defined in their approved QC Program.
2. The Contractor shall submit a copy of his QC Program for review by the Engineer and Owner.
3. All QC or other test data, survey data or the like, collected and compiled by the Contractor shall be made available to the Engineer and the Owner upon request.
4. The Contractor shall provide all the necessary equipment and technicians for material and product testing required to execute the QC Program.
5. QC shall be done continuously, as specified herein, to ensure the quality of the products prior to deployment and workmanship quality.
6. The Contractor's QC shall be done independently from the Engineer's Quality Assurance.

7. Quality Assurance, or any other form of performance testing by the Engineer or Owner, shall in no way relieve the Contractor of their responsibility for completing the QC in accordance with the specified requirements.

1.1.12 Quality Assurance

1. The Engineer will carry out Quality Assurance (QA) for the Works defined herein, and will undertake testing at a frequency, and in the Work areas specified in the various sections of the Specifications and Drawings. The Engineer may undertake any additional testing which they deem necessary on any part of the Works.
2. All QA or other test data, collected by the Engineer, shall be made available to the Owner and Contractor upon request.
3. The Contractor shall render such assistance as is necessary to enable QA sampling and testing to be carried out expeditiously, and provide all the necessary equipment.
4. The Engineer's QA shall be done independently from the Contractor's QC.

1.1.13 Submittals

1. Contractor shall submit their QC Program for review and approval 21 days prior to start of work.
2. Contractor shall conduct the initial existing ground survey prior to any Work and submit to Owner and Engineer for sign off 7 days prior to Work.
3. Contractor shall conduct as-built survey on completed components of the Work and submit for Engineer's review and sign off 2 days prior to advancing to next phase of construction activity.
4. All recorded QC data and records shall be made available to Engineer and Owner within 24 hours upon request.
5. The Contractor shall submit information as specified and requested from the Engineer. Any request for information (RFI) shall be submitted via submittals direct to the Owner and copied to the Engineer 7 days prior to scheduled commencement of Work.
6. The Engineer has the right to request a submittal, of any information, deemed necessary throughout execution of the Works. This includes information not currently defined as submittal information on the Specifications and Drawings. The processing time for these submittals shall be completed within 10 days of the request.
7. The Contractor shall submit the proposed construction schedule to the Owner and copying the Engineer 14 days prior to Work. Updates to the schedule shall be done weekly and shall be submitted to the Owner for review within 24 hours of the weekly updates.
8. The Contractor shall submit a construction water management plan for the Work areas including methodologies and procedures to manage groundwater from excavations and surficial water from runoff and run-ons. The plan shall be submitted 21 days prior to start of Work for Owner's review and approval.

9. Work shall not start until applicable approvals are obtained from the Owner in writing.

10. Approval of submittals shall not relieve the Contractor of their responsibility to construct the Works in accordance with specified requirements.

1.1.14 Construction Specifications and Drawings

1. Specifications and Drawings will be issued by the Engineer specific to construction needs prior to the commencement of the Works. The Specifications and Drawings shall be reviewed by the Owner and Contractor to ensure all aspects of the construction needs are met, and report to the Engineer any discrepancies and interferences.

2. Only Specifications and Drawings specifically marked with the following words are considered acceptable for Construction: ISSUED FOR CONSTRUCTION, or IFC. Furthermore, only the most recent issued revision of the IFC shall be considered for the Works.

----- End of Section 1-----

2 Clearing, Grubbing and Stripping

2.1 Part 1 – General

2.1.1 Documents

1. This section of the Specifications form part of the Contract Documents and are to be read, interpreted and coordinated with all other parts.

2.1.2 Definitions

1. The following words and terms, unless the context otherwise requires herein, shall have the definition below:

   (a) CLEARING means work involved in removal of debris, snow and ice, or any other deleterious materials which would interfere with the Works to expose competent inert natural overburden material to the satisfaction of the Engineer.

   (b) GRUBBING means removal of all tree stumps, roots, organics, vegetation and other organic debris to expose competent inert natural overburden material to the satisfaction of the Engineer.

   (c) STRIPPING means Works involving excavating and removal of unsuitable material including but not limited to organic, topsoil, ice-rich, contaminated, and over saturated material to expose competent inert natural overburden material to the satisfaction of the Engineer.

2.1.3 Descriptions

1. The Works covered by this section consists of supplying all labour, materials equipment, and performing all Works necessary for clearing, grubbing and stripping.

2. The Contractor shall clear, grub, strip, or any combination of clear, grub or strip, the indicated work areas including, but not limited to the following, or inferred herein and as directed by the Engineer:

   (a) Borrow areas.

   (b) Disposal areas.

   (c) Stockpiles areas.

   (d) Laydown areas.

   (e) Water management areas.

   (f) Main Dam footprint.

   (g) Spillway footprint.

   (h) Between individual lifts of placed construction materials.

   (i) Access between work areas.

3. Clearing, grubbing and stripping in all areas shall be surveyed and require approval by the Engineer prior to work commencement.

4. Contractor shall submit plans and procedures to acquire permission for stockpiling and storage of material removed through the process of clearing, grubbing and stripping.

2.1.4 Submittal

1. The Contractor shall submit to the Owner, for approval, a Clearing, Grubbing and Stripping Work Plan 7 days prior to commencement of work. The plan shall describe the schedule, work locations, extent of the works, storage and stockpile locations, and water management plan.

2. An initial survey shall be conducted over the work areas with a minimum 10 m extension beyond the propose work limits. The Contractor shall submit the initial survey to the Owner for review and approval.

3. An as-built survey shall be conducted over the completed work area for reference and documentation. The Contractor shall submit the as-built survey to the Owner for review and approval.

2.1.5 Permits and Regulations

1. The Contractor shall conduct their work in accordance with the Owner's and all applicable Federal and Territorial Regulations and Permits regarding the disposal of material from clearing, grubbing and stripping.

2. It is the Owner's responsibility to acquire all necessary permits and licenses for clearing, grubbing and stripping for the Work areas.

3. It is the Contractor's responsibility to be familiar with all said regulations, conditions and permits.

2.1.6 Protection

1. Unless otherwise instructed, the Contractor is to take all necessary precaution to prevent damage to natural and man-made features, including but not limited to monuments, survey marks, mining equipment and instruments, power lines, road and slopes.

2. The Contractor may not perform any Works outside of the permitted and pre-approved constructions area without additional approval by the Owner.

2.2 Part 2 – Execution

2.2.1 Preparation

1. The Contractor shall confirm the clearing, grubbing and stripping limits by survey prior to commencement of the Work. The Owner will inspect these demarcated areas and approve all limits prior to allowance to proceed.

2. The Contractor shall inspect the work sites and verify with the Engineer and the Owner for any restrictions within or adjacent to the sites.

2.2.2 Clearing

1. Snow and ice shall be removed from all construction footprint areas, prior to undertaking any work in that area, with a maximum tolerance of 3 cm of snow and ice left over construction surfaces, or otherwise approved by the Engineer.
2. Should snow fall on previously prepared and approved construction surfaces, including between individual lifts of fill placement, the Contractor will carry out additional clearing to the Specification herein and as approved by the Engineer.
3. Debris encountered below existing ground, covered by mine works shall be removed and disposed accordingly.
4. The Contractor shall take all necessary precautions to prevent damage to previously approved subgrades, unless otherwise specified by the Engineer.

2.2.3 Grubbing

1. Trees, stumps, shrubs, grasses, including their complete root system shall be removed from all construction footprint areas prior to undertaking any work in the area.
2. Merchantable trees shall be fell and stacked aside according to the approved plan.

2.2.4 Stripping

1. Areas subjected to clearing and grubbing shall undergo stripping to the depth necessary to remove all organic and topsoil to expose competent, inert, organic free soil material as approved by the Engineer.

2.2.5 Finished Surface

1. The Contractor shall leave the cleared, grubbed and stripped surfaced clear, free draining and debris free, in a condition suitable for inspection by the Engineer.

2.2.6 Disposal

1. Snow and ice cleared off the construction area shall be stockpiled downstream and outside of the construction area where it will not affect the construction, constructed elements or mining operation during thaw. The stockpile area shall be proposed by the Contractor and approved by the Owner. An Owner approved water management plan, prepared by the Contractor, must be in place prior to stockpiling of snow and ice in the specific area.
2. Tree, shrubs, grasses, roots and other organic debris cleared or grubbed from the areas shall be stockpiled in designated areas approved by the Owner.
3. The Contractor shall provide appropriate sediment control in accordance with the water management plan.
4. Conduct field surveys, and submit as-built Drawings, in electronic format of the worked areas for documentation and as requested by Owner.

2.3 Part 3 – Quality Control

1. Physically and visually demarcate, for confirmation and approval by the Owner, the Work areas that will be cleared, grubbed and stripped using proper survey control. Within this zone, clearly identify natural and man-made features that require protection as defined herein.
2. Implement measures, including spotters as needed, to allow visual inspection of clearing grubbing and stripping activities during execution to ensure it is done in accordance with these Specifications.
3. Conduct field surveys, and submit as-built drawings, in electronic format worked areas, as requested by the Owner and Engineer.
4. Inspect and maintain water management controls to ensure they are functioning as intended and as directed by the Engineer.

2.4 Part 4 – Quality Assurance

1. Owner to confirm that all permits and governmental approvals are in place prior to commencing any work.
2. Review the Contractor's Clearing, Grubbing and Stripping Work Plan as defined herein and submit review comment back to the Contractor as needed.
3. Visually inspect the demarcated zone prepared by the Contractor for Work and inform the Contractor if any changes are required.
4. Review the as-built drawings submitted by the Contractor for the Work and inform the Contractor if any changes are required.

----- End of Section 2 -----

3 Excavations

3.1 Part – General

3.1.1 Documents

1. This section of the Specifications forms part of the Contract Documents and are to be read, interpreted and coordinated with all other parts.

3.1.2 Definitions

1. The following words and terms, unless the context otherwise stated herein, shall have the meanings set out below:

   (a) SOIL and OVERBURDEN meaning is interchangeable and means general overburden material including sands, gravels, silts, clays, till, topsoil, and any combination of these material, which can be used in part as fill when they meet the specifications herein and as approved by the Engineer.

   (b) RIPRAP means a geochemically compatible, mechanically durable, angular, competent boulder material excavated from a designated stockpile.

   (c) UNSUITABLE MATERIAL means any potential construction materials that do not meet the Specifications herein which contain, but is not limited to, organics, contaminants, hazardous waste, garbage, decomposable or man-made debris, snow, ice, and oversaturated material.

   (d) BORROW means the natural area that suitable material can be excavated from for use in execution of the Works.

   (e) STOCKPILE means the mass of previously placed or sorted suitable material that can be used in the execution of the Works.

   (f) NEAT LINE means the final line or grade to which excavation is to be performed as described herein and as shown on the Drawings.

   (g) COMMON EXCAVATION means excavation of all material, including soil, riprap, fill, and unsuitable material by mechanical means.

3.1.3 Description

1. The Contractor shall perform excavation in the indicated work areas including, but not limited to the following, or inferred herein and as directed by the Engineer:

   (a) Borrow areas.

   (b) Stockpiles areas.

   (c) Water management areas.

   (d) Main Dam footprint

(e) Main Dam key trench.

(f) Spillway footprint.

2. The excavation Works entails excavation of soil and other material below existing ground to neat line and grades as indicated herein and on the Drawings.

3. The Works to be done under this section consists of furnishing all labour, materials, plant, equipment and water management facilities necessary to carry out the excavations.

4. The Works shall also include the disposal of all excavated material which are deemed by the Engineer to be unsuitable, surplus, or does not meet specifications as finished construction material, and the loading, transportation, stockpiling of rejected materials at locations for stockpiling or re-handling to meet specifications.

5. The Contractor is responsible to backfill over excavation or replacing rejected materials with acceptable materials to neat lines.

6. The Contractor will be responsible to locate suitable temporary stockpile locations for any excavated material. The Owner will however have the right to reject any identified sites, if in their opinion it may interfere with any of the current or future mining operations or works.

7. The Owner will designate a permanent storage location for unsuitable materials.

3.1.4 Engineering Designs

1. The details of the surface excavation shown on the Drawings and described herein represent an engineered design based on particular assumed conditions. Variations in site conditions may require adjustments to excavation shape, grade and slope configurations under Engineer's discretion.

2. If, in a specific area, a plan that has been previously adopted does not fit the site conditions in accordance with the Specifications, the Engineer shall request a revised plan from the Contractor prior to continuing excavation in the identified areas.

3. It is the Contractor's responsibility to make his own measurements to fit the engineering design grades, slopes, shapes and dimensions into existing ground conditions using survey methods. The excavation areas shall be demarcated subjected to Engineer's inspections.

3.1.5 Exclusions

1. The Contractor is responsible for borrow development and stockpile operation management. The Engineer does however reserve the right to request modifications to borrow development plan if the material produced does not meet Specifications.

2. The Contractor is responsible to implement, maintain and remove temporary water management in all excavated work areas.

3.1.6 Submittals

1. The Contractor shall submit a detailed Excavation Plan to the Owner outlining their intended methods for work within specific area at least 14 days prior to commencement of the works,

including, but not limited to the following details. The excavation plan must be approved by the Owner prior to commencement of work.

(a) Survey controls.

(b) Typical equipment deployment.

(c) Bench height and width.

(d) Working face slope angles.

(e) Access methods and roads.

(f) Typical excavation/load methods.

(g) Sediment and runoff control around the intended work areas.

(h) Water control and dewatering plan for works where groundwater or surface water runoff and run-on could occur.

(i) Contingency work plan for sudden change of weather conditions.

(j) Borrow areas and stockpiles development and close-out plans.

(k) Excavation/load methods in permafrost.

(l) Temporary excavation slope safety protocol.

(m) Daily work plan template.

2. The Contractor shall submit the initial existing conditions survey 2 days prior to work for documentation and sign off by Owner.

3. The Contractor shall submit an as-built survey of excavation 2 days prior to backfill or subsequent construction work for approval by the Engineer.

4. Approval of the submitted plan shall not relieve the Contractor of its sole responsibility to construct the Works in accordance with specified requirement herein.

3.2 Part 2 – Execution

3.2.1 Preparation

1. Prior to commencement of an excavation operation in any area, all necessary clearing, grubbing and stripping in that area shall be done in according to the specifications herein and as approved by the Engineer.

2. The Contractor shall satisfy themselves as to the properties, quantities, immediate and nearby ground conditions, and distribution of all the materials to be excavated.

3. The Contractor shall have a contingency plan for sudden unforeseeable change of weather conditions in place prior to commencement of excavations. The Contractor shall have a daily work plan in relation to the weather conditions, equipment and operation availability, area of work and schedule.

4. The Contractor shall have management controls in place for sediment, water runoff and run-on around the excavation area prior to work and to ensure there is minimal additional impact on the existing state of surrounding environment in accordance to the all issued regulations, licenses and permits.

5. The Contractor is responsible for all dewatering operations where necessary.

3.2.2 Control of Water

1. Surface water run-on flows shall be direct away from the Works by means of diversion berms, ditches or other acceptable means and, in any case, all surface flows on the work areas shall be satisfactory controlled, to the environmental standard specified in the license and permits.

2. The Contractor shall control the grade and excavation of surfaces to expedite surface drainage to the sumps and minimize infiltration into the natural ground.

3. Any inflow of groundwater, surface runoff, or melt water into the excavation must be controlled using suitably placed and sized sumps and pumps.

4. Any water collected in the sumps must be discharged in an approved manner to a designated area away from the construction activities. A pump and discharge contingency plan shall be submitted to the Owner for approval prior to construction.

5. The construction, operation and maintenance of the sumps and pumps are the responsibility of the Contractor.

3.2.3 Common Foundation Excavation Methods

1. Common excavation shall be performed as indicated on Drawings, as described herein, or as directed by the Engineer to lines, grades, elevations, and shall be finished to a reasonable smooth and uniform surface.

2. Should the Contractor, through carelessness or other fault, excavate beyond the designed neat lines, they shall replace the excavation with approved fill, in accordance with the Specifications herein or as directed by the Engineer.

3. All excavated material determined unsuitable by the Engineer shall be disposed according to approved plan.

4. At all times during construction, the Contractor shall adopt excavation procedures such that at no time shall the stability of any slopes be impaired. The Engineer reserves the right to stop work if they deem the conditions to be unsafe or compromise nearby infrastructures.

3.2.4 Key Trench Excavation

1. The key trench shall be excavated to the neat lines as shown on the Drawings and as directed by the Engineer.

2. The key trench shall be found in natural in-situ fine grained soil that is not disturbed by mining activities. The final excavation surface shall be a firm, smooth, relatively dry and competent surface that is ready to receive fill as specified herein. The Engineer must approve the final key trench excavation surface prior to placement of fill within.

3. If permafrost is encountered within the excavation, the Contractor shall manage the slope for safety according to specifications herein and manage the melt water to prevent further degradation of permafrost and foundation conditions. The Engineer must approve the final key trench excavation within permafrost prior to receiving fill.

3.2.5 Spillway Excavation

1. The spillway shall be excavated to the neat lines as shown on the Drawings and as directed by the Engineer.

2. The spillway shall be found in competent material that is free of snow, ice and other deleterious debris. The final excavation surface shall be a firm, smooth, relatively dry and competent that is ready to receive fill as specified herein. The Engineer must approve the final spillway excavation surface prior to placement of fill within.

3.2.6 Slope Stability and Safety

1. Immediately following excavation and at any time during the works, all loose material that appear to be unsafe, or that is deemed by the Owner or Engineer to be unsafe, on slopes shall be removed.

2. The Contractor is responsible for the slope stability measures that are considered incidental to the work, and shall be carried out subsequent to the excavation. The Engineer reserves the right to inspect the slope and suspend the work if they deem safety is compromised.

3.3 Part 3 – Quality Control

1. Physically demarcate the work areas that will be excavated using proper survey control for review and approval of the Engineer.

2. Implement measures, including spotters and frequent survey control as needed, to allow visual inspection of excavation activities to ensure they are done with to the neat lines shown on the Drawings and in accordance specification herein.

3. Implement measures to ensure adequate water management and dewatering as necessary.

4. Advise the Engineer when an excavation has been completed and is ready for inspection for approval. Interim survey control and as-built survey may be request by the Engineer to confirm lines and grades prior to final approval.

5. Conduct a field survey and submit as-built drawings, in electronic format of any excavated areas for submittal.

3.4 Part 4 – Quality Assurance

1. Review the Excavation Plan and provide comment, as necessary, back to the Contractor.

2. Visually inspect the demarcated zones, and any associated survey data prepared by the Contractor for excavation, and inform the Contractor any changes are required.

3. Visually inspect the final key trench excavated surfaces to determine their suitability for fill placement and advise the Contractor if further work is required.

4. Visually inspect the water management and dewatering effort and inform the Contractor if changes are required.

5. Review as-built drawings and inform the Contractor if changes are required.

----- End of Section 3 -----

4 Fill Material Specifications

4.1 Part 1 – General

4.1.1 Documents

1. This section of the Specifications forms part of the Contract Documents and is to be read, interpreted and coordinated with all other parts.

4.1.2 Definition

1. All construction fills are categorized according to the standards specified herein. Fill material shall be QC tested and approved by Engineer before use.
2. All construction fill shall be non-acid generating, non-contaminated, free of organics or similar deleterious impurities, free of excess water, as well as snow and ice.
3. The following words and terms, unless the context otherwise stated herein, shall have the meanings set out below:
   (a) FILL means any natural materials that are QC tested using standards specified herein and approved for construction use by the Engineer.

4.1.3 Description

1. The sources and borrow areas of all fill are shown in the Drawings or as designated by the Engineer. The material types required for construction of the work are:
   (a) Riprap.
   (b) Transition Material 1.
   (c) Transition Material 2.
   (d) Bedding Material.
   (e) Core Material.
   (f) Run-of-Mine (ROM) Material.
2. The Contractor is responsible for supplying, installing, operating and maintaining all the necessary plant, equipment, material, labour, QC technicians and supervision to produce and QC test the suitability of the specified construction fill material on site.
3. The Contractor is responsible for any process required for the fill to meet gradation, moisture, elimination of impurities specified herein.

4.1.4 Submittals

1. The Contractor shall submit a fill process plan including equipment, plant, capacity and schedule, process and handling procedures to meet the Specifications herein for the Engineer's approval 21 days prior to commencement of work.

2. The Contractor shall submit the information requested in the Quality Control Program for Fill Material to the Engineer for approval 21 days prior to commencement of work.
3. Work shall not start until applicable approvals are obtained from the Engineer in writing.
4. The Contractor shall submit all exploration records of proposed borrow areas and stockpiles for intended use as construction material within 7 days of work completion.
5. Approval of submittal shall not relieve the Contractor of its sole responsibility to construct the Works in accordance with the specified requirements herein.

4.2 Part 2 – Products

4.2.1 General

1. Fill shall be obtained and processed by the Contractor from designated borrow areas and stockpiles as shown on the Drawings.
2. Unsuitable material from excavation for the Works shall be disposed of in a designated onsite disposal area as directed by the Owner.
3. If the Contractor proposes to obtain fill from an area not within the designated areas shown on the Drawings, they shall communicate their intent to the Owner. The Contractor then shall first obtain the necessary approval and permits to carry out subsurface investigation. The Contractor shall draft accurate exploration records of any test pits, trenches or drill holes from the subsurface investigations. Soil samples shall be collected from the investigation and tested for construction suitability. All subsurface investigation field logs, survey data, and testing results shall be submitted to Engineer for review and approval within 7 days of the completion of exploration work.
4. The Contractor shall give the Owner a minimum of 8 weeks' of notice of their intention to develop any potential borrow areas that are not shown on the Drawings.
5. The Contractor shall make his own determination of the adequacy of any borrow source he intends to exploit. All exploration works are considered incidental to fill procurement and are not additional work.

4.2.2 Riprap

1. Riprap material shall be mechanically durable, geochemically suitable, and angular rock sourced from a designated stockpile which is free of organics or similar deletorious impurities, snow and ice.
2. Riprap shall be uniform sorted material with no fine grained material and a minimum boulder size 150 mm, maximum size of 500 mm and an average (D_{50}) size of 300 mm.
3. Basic mechanical screening and manual selection are expected to achieve the desired gradations.

4.2.3 Transition Material 1 (Fine Filter)

1. The Transition Material 1 shall consist of competent non-contaminated sand and gravel material sourced from the designated borrow areas, and that is free of organics or similar deleterious impurities, snow and ice.

2. The Transition Material 1 shall have the particle size distribution within the fine and coarse limits presented in Table 4-1.

Table 4-1: Transition Material 1 Particle Size Distribution Limits

Particle Size (mm)	Coarse Limit Percentage Passing	Fine Limit Percentage Passing
9.500	100	100
4.750	91	100
2.000	75	100
0.850	58	100
0.250	24	100
0.075	0	78
0.040	0	59
0.010	0	21
0.005	0	5

3. Mechanical screening and selective excavation are expected to meet the Specification above.

4.2.4 Transition Material 2 (Coarse Filter)

1. The Transition Material 2 shall consist of competent non-contaminated sand and gravel material sourced from the designated borrow areas, and that is free of organics or similar deleterious impurities, snow and ice.

2. The Transition Material 2 shall have the particle size distribution within the fine and coarse limits presented in Table 4-2.

Table 4-2: Transition Material 2 Particle Size Distribution Limits

Particle Size (mm)	Coarse Limit Percentage Passing	Fine Limit Percentage Passing
51.000	100	100
25.000	91	100
9.500	73	100
4.750	59	100
2.000	34	100
0.850	11	100
0.250	0	81
0.075	0	45
0.040	0	28
0.020	0	9

3. Mechanical screening and selective excavation are expected to meet the Specification above.

4.2.5 Bedding Material

1. The Bedding Material shall consist of competent non-contaminated sand and gravel material sourced from the designated borrow areas, and that is free of organics or similar deleterious impurities, snow and ice.

2. The Bedding Material shall have the particle size distribution within the fine and coarse limits presented in Table 4-3.

Table 4-3: Bedding Material Particle Size Distribution Limits

Particle Size (mm)	Coarse Limit Percentage Passing	Fine Limit Percentage Passing
51.000	100	100
25.000	90	100
9.500	73	100
4.750	59	100
0.850	36	83
0.250	24	61
0.075	14	42
0.040	10	36
0.010	5	22
0.005	0	16
0.002	0	10
0.001	0	7

3. Mechanical screening and selective excavation are expected to meet the Specification above.

4.2.6 Core Material

1. The Core Material shall consist of competent non-contaminated sand and gravel material sourced from the designated borrow areas, and that is free of organics or similar deleterious impurities, snow and ice.
2. The Core Material shall be handled in such a way to maintain within 2% of optimum moisture content as determined by QC testing.
3. The Core Material shall have the particle size distribution within the fine and coarse limits presented in Table 4-4.

Table 4-4: Core Material Particle Size Distribution Limits

Particle Size (mm)	Coarse Limit Percentage Passing	Fine Limit Percentage Passing
51.0000	100	100
25.0000	94	100
9.5000	85	97
2.0000	57	85
0.4250	36	71
0.0750	21	56
0.0400	17	46
0.0100	11	29
0.0050	8	23
0.0020	5	14
0.0015	4	12

4. Selective excavation and manual coarse rejection are expected to meet the Specification above.

4.2.7 Run-of-Mine (ROM) Material

1. The ROM Material shall consist of competent non-contaminated, geochemically suitable rock sourced from the designated stockpiles, and that is free of organics or similar deleterious impurities, snow and ice.
2. ROM Material shall be uniform sorted material with no fine grained material and a maximum size of 500 mm.
3. Manual sorting and visual inspection is expected to meet the Specification above.

4.3 Part 3 – Quality Control

1. The Contractor shall carry out Quality Control testing prior to utilization of the source as borrow for construction, and during the production of fill as outlined in Table 4-5.
2. The number of tests shown in Table 4-5 represents the minimum. The Engineer reserves the right to request additional tests to confirm the quality of fill material.
3. All samples shall be representative grab samples from the source or borrow.
4. Turnaround time for laboratory tests shall be less than 24 hours upon collection of samples.

Table 4-5: Minimum QC Testing Schedule for Construction Material

Material Type	Sample Location	Testing Standard	Test Frequency
Riprap	At stockpile	Visual Particle Size Control	• Ongoing
Transition Material 1	At borrow	Particle Size Analysis (ASTM D422)	• 3 at the source prior to development • One per 500 m^3 during production
Transition Material 2	At borrow	Particle Size Analysis (ASTM D422)	• 3 at the source prior to development • One per 500 m^3 during production
Bedding Material	At borrow	Particle Size Analysis (ASTM D422)	• 3 at the source prior to development • One per 500 m^3 during production
Core Material	At borrow	Particle Size Analysis (ASTM D422) Water Content in Soil (ASTM D2216)	• 3 at the source prior to development • One per 1,000 m^3 during production
ROM Material	At stockpile	Visual Particle Size Control	• Ongoing

4.4 Part 4 – Quality Assurance

1. The Engineer shall carry out Quality Assurance testing during the production and placement of construction material as outlined in Table 4-6.
2. The number of tests shown in Table 4-6 represents the minimum. The Engineer reserves the right to sample and test as much as needed to confirm the quality of the fill.
3. All samples shall be representative grab samples from borrow and in-situ placed material.
4. Turnaround time for laboratory tests shall be less than 5 days.

Table 4-6: Minimum QA Testing Schedule for Construction Material

Material Type	Sample Location	Testing Standard	Test Frequency
Riprap	At stockpile	Visual Particle Size Control Rock Durability (ASTM D5312)	• Ongoing
Transition Material 1	At borrow	Particle Size Analysis (ASTM D422) Moisture Content (ASTM D2216)	• 3 at the source prior to development • One per 1,000 m^3 during production
Transition Material 2	At borrow	Particle Size Analysis (ASTM D422) Moisture Content (ASTM D2216)	• 3 at the source prior to development • One per 1,000 m^3 during production
Bedding Material	At borrow	Particle Size Analysis (ASTM D422) Moisture Content (ASTM D2216)	• 3 at the source prior to development • One per 1,000 m^3 during production
Core Material	At borrow	Particle Size Analysis (ASTM D422) Water Content in Soil (ASTM D2216)	• 3 at the source prior to development • One per 2,000 m^3 during production
ROM Rock	At stockpile	Visual Particle Size Control	• Ongoing

----- End of Section 4 -----

5 Fill Placement

5.1 Part – General

5.1.1 Documents

1. This section of the Specifications form part of the Contract Documents and are to be read, interpreted and coordinated with all other parts.

5.1.2 Description

1. The Works specified herein includes furnishing all supervision, labour, materials, tools, equipment and Quality Control (QC) technician for placement of fill material to the lines and grades shown on the Drawings and Specifications herein.
2. The works shall include, but are not limited to the following:
   (a) Foundation preparation to receive fill.
   (b) The supply, load, haul, dump, spread and compact of the specified fill material as shown on the Drawings.
   (c) All related survey for layout and control of the Works.
   (d) The Contractor shall assist the Engineer, when necessary while the Engineer is performing Quality Assurance (QA) testing. In addition, the Contractor shall submit the copies of Contractor's QC results.
   (e) Maintenance of construction haul roads (as applicable) including surficial runoff, snow and ice.
   (f) The development, maintenance and restoration of fill material borrow areas.
   (g) Water management during fill material placement.
   (h) Removal and disposal of unsuitable and rejected materials.
3. Fill material required to be replaced included, but are not limited to the following and not constructed in the following order:
   (a) Prepare the key trench and overall Main Dam footprint foundation surfaces to receive fill.
   (b) Load, haul and place riprap in the upstream portion of the Main Dam.
   (c) Load, haul, dump, spread and compact Transition Material 1 in the upstream portion of the Main Dam.
   (d) Load, haul, dump, spread and compact Transition Material 2 in the downstream portion of the Main Dam.
   (e) Load, haul, dump, spread and compact Bedding Material above and below the geomembrane in the Main Dam.

(f) Load, haul, dump, spread and compact Core Material in the Main Dam.

(g) Load, haul, dump, spread and compact ROM Material in the downstream portion of the Main Dam.

(h) Load, haul, dump, spread and compact ROM Material as cap on top of the Main Dam.

5.1.3 Submittals

1. The Contractor shall submit a fill placement plan to the Engineer for review and approval 21 days prior to any fill placement. The plan shall include typical equipment, schedule, compaction methodology on slopes and water management.
2. The Contractor shall submit the information requested in the QC Program for fill placement to the Engineer for approval 21 days prior to commencement of work.
3. Work shall not start until applicable approvals are obtained from the Engineer in writing.
4. Approval of submittal shall not relieve the Contractor of its sole responsibility to construct the Works in accordance with the specified requirements herein.

5.2 Part 2 – Execution

5.2.1 Compaction Equipment

1. The compaction equipment shall be of the appropriate size and type to achieve the specified densities of the respective fill material. The core material may require a “sheep foot” compactor to achieve optimum compaction.
2. Where compaction procedures (lift thickness, number of passes, or compactor type) are specified, the Contractor shall provide compactors that meet or exceed those described herein.
3. A compaction pass specified herein is considered as 1 drive through in one-direction over a given section of material. A back and forth drive through is considered 2 passes.
4. The Engineer reserves the right to request additional compacting effort, re-handling, or larger equipment if specified densities are not achieved.
5. The Contractor shall submit a special provision to compact the final upstream slope of the core prior to geomembrane deployment to the Engineer for review 7 days prior to geomembrane deployment. The special provision shall have alternative methodology to allow compaction equipment to work the relative steep upstream face of the core.

5.2.2 Snow Removal Equipment

1. The Contractor shall provide all supervision, labour and equipment to clear snow and ice off the fill surface prior to subsequent fill placement. The snow clearing shall be done according to the Specifications herein.
2. Care shall be taken when clearing snow above or adjacent to previously placed and compacted material to avoid ripping and subsequent damage to approve surfaces. The

Contractor shall replace or rework any material that has been damaged as deemed by the Engineer.

3. If deemed necessary by the Engineer, the Contractor shall use manual labour to clear snow off small critical areas.

5.2.3 Foundation Preparation

1. The Contractor shall prepare an acceptable foundation surface to receive fill material. An acceptable foundation surface is a surface that is clean, sound, firm, and does not contain any loose or soften material, ponded water, organic material or similar impurities as determined by the Engineer.

2. All fill material shall be placed in accordance to the neat lines shown the Drawings, compacted in lifts according to Specifications herein, and be free of snow and ice or any other loose and deleterious material between lifts.

3. Where depressions or holes exist in the foundation or excavation surfaces, approved fill shall be placed and compacted in depressions as specified herein. Special techniques, handwork, and the like shall be done as necessary to meet Specifications.

4. Fill shall not be place on any surfaces or excavations until they have been inspected and approved by the Engineer.

5.2.4 General Fill Placement for All Products

1. Construction must be performed in accordance with the best modern practice and with equipment best adapted to the work being performed. Material must be placed so that each zone is homogenous, free of stratifications, lenses or pockets of isolated coarse particles, ruts, and layers of material with different texture or grading not confirming to the requirements stated herein.

2. No fill material shall be over existing fill surfaces until they have been prepared as specified herein and approved by the Engineer.

3. The placement of fill must conform to the lines, grades, and elevations as shown on the Drawings, as specified herein or as directed by the Engineer.

4. Fill placement must be conducted in such a manner to avoid mixing with other fill types in the adjacent zones.

5. Stockpiling, loading transporting, dumping and spreading of all materials shall be carried out is such a manner to avoid segregation or any other condition that compromise the homogeneity of the fill. Segregated fill must be re-handled, or removed and replaced with fill meeting the Specifications herein and as directed by the Engineer.

6. The compaction operations for the fill shall be conducted within the same work day to provide a smooth compact surface. Adjacent individual passes of compactor shall overlap by approximately 1/3 of the width of the compactor’s drum. New fill placed in the same elevation adjacent to previously compacted fill shall observe the 1/3 compaction overlap into the approved fill to ensure specified densities are achieved at the fill contact.

7. The Contractor is responsible to repair all damages on unfinished and unapproved work from the previous work day.

8. The Engineer shall inspect and approved each horizontal lift of fill prior to placement of subsequent lifts.

9. Unless otherwise specified, construction material maximum lift thickness and compaction requirement shall conform to the Specifications herein.

5.2.5 Riprap Material Placement

1. The riprap material must be placed and spread in horizontal lifts parallel to the long axis of the Dam and within the spillway or Dam lines as shown on the Drawings. The lifts shall provide an even surface for traffic, or otherwise directed by the Engineer.

2. Maximum lift after initial spread is 1.0 m.

5.2.6 Transition and Bedding Materials Placement

1. The fill materials must be placed in horizontal lifts parallel to the long axis of the Dam within the lines and grade, as shown on the Drawings and as directed by the Engineer.

2. The minimum final as-built thickness of the Transition Materials is 0.5 m and the Bedding Material is 0.3 m as shown on the Drawings, measured perpendicular to the slope. The Contractor shall have survey control to ensure this minimum thickness is achieved through the horizontal fill placement.

3. The Contractor shall have procedures to compact the thin bedding material, placed directly over the geomembrane, to the specified density without damaging it. This procedure shall be included in the fill placement plan, and demonstration trials shall be conducted to ensure the quality of work to the Engineer prior to execution. The Engineer reserve the right to request alternative methodology to achieve specified compaction if they deemed the proposed procedure might damage the geomembrane.

4. The Contractor is responsible to repair any damages done to the geomembrane from the fill placement over and adjacent to it. The Engineer reserves the right to remove the placed fill material to examine the geomembrane integrity. The examination is considered incidental to the fill placement work.

5. Maximum loose lift after initial spread is 0.3 m. The placement method must ensure no areas of fill exceed this maximum prior to compaction.

6. The Transition and Bedding Materials shall be compacted to achieve an equivalent of 95% Standard Proctor compaction.

5.2.7 Core Material

1. The Core Material must be placed in horizontal lifts parallel to the long axis of the Dam within the lines and grade as shown on the Drawings and as directed by the Engineer.

2. The final as-built slope of the core shall be 2.5H:1V. The Contractor shall have survey control to ensure this final slope is achieved through the horizontal fill placement.

3. The relatively steep upstream slope could be difficult to attain using conventional compactor operations. The Contractor shall have a provisional procedure to compact the loose material that is exposed at the final upstream slope surface of the core. The finished surface shall be smooth, firm, free of protruding particles and compacted to the density specified herein. This procedure shall be included in the fill placement and demonstration trials shall be conduct to ensure the quality of work to the Engineer prior to execution.

4. Maximum loose lift after initial spread is 0.3 m. The placement method must ensure no areas of fill exceed this maximum prior to compaction.

5. The Core Material shall be compacted to achieve an equivalent of 100% Standard Proctor compaction.

5.2.8 ROM Material

1. The ROM Material must be placed and spread in horizontal lifts parallel to the long axis of the Dam within the lines shown on the Drawings. The lifts shall provide an even surface for traffic, or as otherwise directed by the Engineer.

2. Maximum lift after initial spread is 1.0 m.

5.2.9 Compaction Trials

1. Compaction trials shall be performed upon production of fill material to determine site specific parameters such as density and compaction standards. The trials can be carried out as part of the fill placement operation or separate as the work dictates.

2. The Contractor shall periodically conduct field trials to optimize moisture conditioning, lift thickness, compaction effort and methodology.

3. The compaction trials on the material in question shall be done using a survey method according to the general procedure listed below, and as specified by the Engineer.

   (a) A pad made with the approved material of approximately 7 m by 20 m with specified thickness and placed according to the method specified herein.

   (b) A set of survey points with vertical and horizontal accuracy ± 5 mm shall be laid out in a 5 m by 5 m grid pattern or as directed by the Engineer.

   (c) The elevation of each survey point shall be recorded immediately after placement and after each compaction pass.

   (d) The compaction shall be done upward of 15 passes with survey record on each point after each pass.

   (e) The results from the trial shall be determined by the Engineer to a specified minimum number of passes required to achieve optimum compaction.

4. The compaction trial shall be repeated to simulate different construction methodology and material variance, and as directed by the Engineer.

5. The Engineer reserves the right to modify the Specifications to suit site conditions.

5.2.10 Restrictions Due to Weather and Suspension of Operations

1. The Contractor shall not place Transition, Bedding and Core Materials when results for such operations are unsatisfactory due to sustained rainfall, snowfall, and extraordinary freezing conditions, or any other reasons determined by the Engineer.
2. Where operations have been discontinued by the Contractor, or suspended by the Owner or Engineer, the effects of adverse conditions shall be assessed by the Engineer and adversely affected fills and subgrade shall be reworked, or replaced and repaired to the satisfaction of the Engineer before resumption of fill placement.
3. Before suspension of operation each day, each construction shift or before foreseeable adverse weather conditions, the placed fill shall be shaped and compacted to drain excess surface runoff and limit infiltration.

5.2.11 Sediment and Runoff Control

1. In general, the Contractor shall shape the work surfaces toward collection channels or water management structures,
2. The Contractor is responsible to revise, maintain, and operate temporary water management structures as specified in their approved Water Management Plan and specified herein. The structures shall prevent discharge of fines from construction areas from entering natural water courses and other parts of the mine. The Contractor shall adjust existing structures or install new controls if sediment runoff is found outside of the construction areas.

5.3 Part 3 – Quality Control

1. The Contractor shall be responsible for the quality of placed fill as described herein.
2. The Contractor shall collect representative approved samples of fill materials to conduct Proctor Compaction Density Tests to determined density associated with each fill.
3. The Contractor shall conduct regular topographic surveys to demonstrate the placement of fill are within specified lines, levels, grades and slopes according to the Drawings and specified herein.
4. The Contractor shall carry out QC Program testing during fill placement as outlined in Table 5-1.

Table 5-1: QC Testing During Placement of Construction Materials

Fill Type	Maximum Horizontal Lift Thickness (Tolerance)	Compaction Specifications (Tolerance)	QC Standard	QC Frequency
Riprap	1 m (± 100 mm)	N/A	Visual Inspection	Ongoing
Transition and Bedding Materials	0.3 m (± 50 mm)	95% Standard Proctor Density	Soil Density using Stand Effort (ASTM D698-12e1) Compaction Effort from Results of Compaction Trials	Minimum 12 for all specified material for project. Ongoing
Core Material	0.3 m (± 50 mm)	100% Standard Proctor Density	Soil Density using Stand Effort (ASTM D698-12e1) Compaction Effort from Results of Compaction Trials	Minimum 12 for all specified material for project. Ongoing
ROM Material	1 m (± 100 mm)	N/A	Visual Inspection	Ongoing

5.4 Part 4 – Quality Assurance

1. QA testing shall be carried out by the Engineer in placed and compacted fill materials.
2. The Engineer may request intermediate surveys to ensure the fills are placed within specified lines as shown on the Drawings and as described herein. The Contractor shall conduct these surveys upon request if existing information is not available and report the results to the Engineer within 24 hours.
3. Final acceptance of earthworks will be made only after as-built surveys, QC and QA test results are received and approved by the Engineer.
4. If the results from the sampling and testing, or in the opinion of the Engineer in the absence of test results, of the placed fill do not meet the Specifications herein, such fill shall be removed and replace with conforming material for rework. Rejection of fill by the Engineer may be made at source, in transport vehicle or in place.
5. The Engineer can re-inspect previously approved areas for damages or degradation and instruct the Contractor to repair the said damages specified herein.
6. The Engineer shall carry out QA testing during fill placement as outlined in Table 5-2. Additional testing may be conducted at the discretion of the Engineer.

Table 5-2: QA Testing During Placement of Fill Material

Fill Type	Maximum Horizontal Lift Thickness (Tolerance)	Compaction Specifications (Tolerance)	QC Standard	QC Frequency
Riprap	1 m (± 100 mm)	N/A	Visual Inspection	Ongoing
Transition and Bedding Materials	0.3 m (± 50 mm)	95% Standard Proctor Density	Soil Density using Stand Effort (ASTM D698-12e1) Density Test using Nuclear Method (ASTM D6938-10)	Minimum 12 for all specified material for project. Minimum 12 per lift of placed materials
Core Material	0.3 m (± 50 mm)	100% Standard Proctor Density	Soil Density using Stand Effort (ASTM D698-12e1) Density Test using Nuclear Method (ASTM D6938-10)	Minimum 12 for all specified material for project. Minimum 12 per lift of placed materials
ROM Material	1 m (± 100 mm)	N/A	Visual Inspection	Ongoing

----- End of Section 5 -----

6 Geosynthetics

6.1 Part 1 – General

6.1.1 Document

1. This section of Specifications form part of the Contract Documents and is to be read, interpreted and coordinated with all other parts.

6.1.2 Definition

1. The following words and terms, unless the context otherwise stated herein, shall have the meanings set out below:

   (a) GEOSYNTHETICS includes all man-made geosynthetics products utilized in the work, including all geotextile, geomembrane, weld rods, all other consumable supplies and components required for installation.

   (b) GEOMEMBRANE means a very low permeability doubled-sided textured synthetic membrane as specified herein.

   (c) WELD RODS means a synthetic consumable with comparable resin properties as the geomembrane to be used in seaming.

   (d) GEOTEXTILE means non-woven high strength geotextile as specified herein.

   (e) BENTONITE CLAY POWDER means a sodium based bentonite clay product as specified herein.

6.1.3 Description

1. The Works to be done under this section consist of furnishing all labour, material, supplies equipment and QC protocols necessary to deploy and install geosynthetics as shown on the Drawings and as specified herein, which will include but not limited to:

   (a) Geomembrane as primary barrier upstream of the dam core.

   (b) Geotextile as separation layer underneath riprap and the dam cap.

   (c) Spread bentonite powder at geomembrane tie-in as sealant.

2. The Contractor shall provide a qualified and experienced team or subcontractor for the geosynthetics installation, QC and provide warranty of the installation as specified in the Contract Documents.

6.1.4 Codes and Standards

1. The latest version of the standards shown in Table 6.1 shall apply for the QC testing during the work.

Table 6-1: QC Standard for Geosynthetics Installations

Test Standard	Test Description
ASTM D638	Standard Test Method for Tensile Properties of Plastics
ASTM D1004	Standard Test Method for Initial Tear Resistance of Plastic Film and Sheeting
ASTM D1204	Standard Test Method for Linear Dimensional Changes of Non-Rigid Thermoplastic Sheeting
ASTM D1505	Standard Test Method for Density of Plastics by Density-Gradient Technique
ASTM D1603	Standard Test Method for Carbon Black in Olefin Plastics
ASTM D1790	Standard Test Method for Brittleness Temperature of Plastic Sheeting by Impact
ASTM D4437	Standard Practice for Non-Destructive Testing for Determining Integrity of Field Seams Used in Jointing Flexible Polymeric Sheet Geomembrane
ASTM D4833	Standard Test Method for Index Puncture Resistance of Geotextile, Geomembrane and Related Products
ASTM D5641	Standard Practice for Geomembrane Seam Evaluation by Vacuum Chamber

6.1.5 Submittals

1. The Contractor shall submit a geosynthetics deployment plan for the Engineer's approval 14 days prior to work, which shall include the material storage and handling, panel layout, schedule, deployment methodologies, field seaming methodologies, subgrade preparation, ballast and anchorage. The subgrade preparation section of the plan shall include a template certification between the Contractor and their specialist geosynthetics installation subcontractors on the demarcated area for warrantied geosynthetics deployment.
2. The Contractor shall submit certifications of the specialist technicians and equipment for the work 14 days prior to commencement of the work.
3. The Contractor shall survey the as-built of the approved geosynthetics, as-built panel installation, QC document of installations, and product certifications for Engineer's approval 2 days prior to covering the geosynthetics with fill material. The geosynthetics shall not be covered until approval is given.

6.2 Part 2 – Products

6.2.1 Submittals

1. The Contractor shall submit the following information at least 14 days prior to material arrival on site for Engineer's review:
   (a) Geosynthetics specifications including product information, dimensions and properties.
   (b) Geosynthetics products serial identifications.

(c) Manufacturer's factory QC certifications of geosynthetics.

(d) Date of manufacture and QC certifications of resin.

(e) Bentonite origin and mineral content.

(f) Shipping itineraries of the geosynthetics.

6.2.2 Shipping and Handling

1. The Contractor must ensure all geosynthetics are delivered and handled according to manufacturer's recommendations.
2. The Engineer may perform inventory and visual inspection for product information, defects and damages of geosynthetics delivered to site.
3. The Contractor shall provide storage areas for the geosynthetics to prevent damages to geosynthetics, including ultraviolet light, puncture, mechanical abrasion, excessive heat and cold and other source of damages.
4. The Contractor is responsible to replace any rejected or severely damage materials as deemed by the Engineer.

6.2.3 Geomembrane

1. The geomembrane to be installed in the Main Pit Dam shall be a 1.5 mm (60 mil) Agru Linear Low Density Polyethylene (LLDPE) MicroSpike Liner, or equivalent, that meet or exceed the criteria as shown in Table 6-2.
2. The Contractor shall procure manufacturer recommended welding rods for the specified geomembrane that are made from the same material type and are compatible.

Table 6-2: Double Sided MircoSpike, 1.5 mm (60 mil) LLDPE Liner Properties

Property	Test Method	Values
Nominal Thickness	ASTM D5994	1.50 mm
Minimum Average Thickness		1.43 mm
Lowest Individual Thickness		1.28 mm
Minimum Average Asperity Height	ASTM D7466	0.41 mm
Density	ASTM D792	0.939 g/cm^3
Peak Strength at Break	ASTM D6693 at 50 mm/min	29.4 N/mm
Elongation at Break		400%
Tear Resistance	ASTM D1004	160 N
Puncture Resistance	ASTM D4833	310 N
Minimum Axi-Symmetric Break Resistance Strain	ASTM D5617	30%
Carbon Black Content	ASTM D5596	2%
2% Secant Modulus	ASTM D5352	630 N/mm

6.2.4 Geotextile

1. The geotextile to be installed in the project shall be a 405 g (12 oz.) non-woven geotextile, or equivalent, that meet or exceed the following criteria shown in Table 6-3.

Table 6-3: 405 g Non-Woven Geotextile Properties

Property	Test Method	Value
Mass per Unit Area	ASTM D5261	405 g/m^2
Grab Tensile Strength	ASTM D4632	1,420 N
Grab Elongation		50%
CBR Puncture Strength	ASTM D6240	4,116 N
Trapezoidal Tear Strength	ASTM D4533	555 N
Apparent Opening Size	ASTM D4751	0.15 mm
Permittivity	ASTM D4491	0.8 sec^{-1}
Water Flow Rate		2,440 l/mm/m^2
UV Resistance	ASTM D4355 (after 500 hours)	70% retained

6.2.5 Bentonite Powder

1. The bentonite powder to be used as a sealant shall be the EnviroGel 10 MSW, or equivalent, that meet or exceed the following criteria shown in Table 6-4.

Table 6-4: Bentonite Powder Properties

Property	Value
Bentonite	High swelling sodium bentonite
Dry fineness	98% minimum passing 2 mm 20% maximum passing 0.075 mm
High swelling	2 g of dry product swell to an apparent volume of 16 cm^3 with 100 cm^3 of water
Average Density	1,057 kg/m^3

6.3 Part 3 - Execution

6.3.1 Subgrade Preparation

1. Prior to geosynthetics deployment, all subgrade surfaces shall be rolled and compacted to provide a smooth, firm surface that is free of protruding debris and particles, and ponding water. All subgrade must be signed off by the specialist geosynthetics installer for warranty purposes. The Engineer must approve the subgrade prior to deployment.

2. The Contractor shall provide, where necessary, additional proof rolling or compaction, manual coverage of exposed coarse particles with fine sand or combination of them to meet subgrade specifications.

3. All subgrade damaged by construction equipment and deemed unsuitable for installation shall be repaired prior to deployment of geosynthetics. All repairs shall be inspected and approved by the Engineer.

4. A 1 m wide by 1 m deep anchor trench shall be excavated by the Contractor at the termination edges of the geomembrane, as shown on the Drawings and as approved by the Engineer, prior to deployment. The Contractor shall ensure the trenches are free of standing water at the time of deployment and suitable safety precautions working around open ground.

6.3.2 Geomembrane Installation

1. The geomembrane shall be installed to the lines as shown on the Drawings and as directed by the Engineer.

2. Sufficient ballast weights shall be available to hold the deployed geomembrane in place.

3. Seams shall be welded in accordance with manufacturer's recommendations.

4. The seams shall be orientated parallel to the dip direction of the maximum slope. In corners and odd shaped geometric locations, the number of seams should be minimized.

5. Horizontal seams shall have a shingle type overlap toward the down slope direction.

6. Geomembrane panel placement and seaming shall not occur when the ambient temperature area is below freezing, during precipitation, presence of excessive dew, or in high winds.

7. Geomembrane shall be protected from displacement, damage, or deterioration before, during and after placement. Typically, only the quantity of geomembrane that will be seamed and anchored in one work day shall be deployed.

8. Damaged, torn, permanently creased or permanently twisted panels shall be rejected or repaired to the satisfaction of the Engineer.

9. Geomembrane shall not be in tension, or "trampoline" over voids or changes in slope gradients. The geomembrane shall rest in intimate contact with the subgrade immediate prior to fill placement.

10. Excessive wrinkles cause by thermal expansion or fault in placement shall be remediated according to manufacturer's recommendation prior to fill placement.

11. Minor tears, punctures and gaps in the geomembrane shall be repaired by patching a circular shape piece of intact geomembrane, welded in place according to manufacturer's recommendations and QC tested.

6.3.3 Geotextile Deployment

1. Sufficient ballast weights shall be available to hold the deployed geomembrane in place.

2. The seams shall be orientated parallel to the dip direction of the maximum slope. In corners and odd shaped geometric locations, the number of seams should be minimized.

3. Horizontal seams shall have a shingle type overlap toward the down slope direction.

4. The seams shall have a minimum 150 mm overlap and shall be heat bonded together according to manufacturer's recommendations. The Engineer will inspect the bonds to ensure the quality of the Work. If the seams come undone during fill placement, the Contractor must repair the aperture prior to continuation of work.

5. Tear, gaps and large punctures in the geotextile shall be repaired by patching with minimum 150 mm overlap around the damaged area.

6.3.4 Bentonite Powder Placement

1. The bentonite shall be deployed in layer in the geomembrane tie-in at the bottom of the key trench as shown on the Drawings and as directed by the Engineer.

2. A minimum 150 mm thick layer of bentonite powder shall be placed at the bottom of the anchor trench, covering its entirety. Another layer of 150 mm of bentonite powder shall be placed over geomembrane once the installation is approved by the Engineer.

3. The anchor trench shall remain relative dry, free of ponding water during bentonite powder placement.

4. The Contractor is responsible to remove any expose hydrated bentonite and replace them with approved dry product.

6.4 Part 4 – Quality Control

1. The Contractor is responsible to repair all defective areas, sections and welds to specifications herein and approved by the Engineer.

2. The Contractor shall have a dedicated technician to conduct QC protocols for geosynthetics installations including mark panel identifications, conduct QC inspection and tests, complete as-built documents and assist the Engineer's inspection and QA.

3. The Engineer may observe QC testing protocol and request retesting as they deem fit. All air pressure tests and fusion welds shall be field inspected and signed off by the Engineer for approval. The air pressure test is considered failed without the Engineer's signoff.

4. The QC technician shall inspect all geosynthetics installations and mark all defects found for repairs. The QC inspection shall be done independently from the Engineer's QA inspections.

5. The Contractor shall carry out QC Program testing during fill placement as outlined in Table 6-5.

Table 6-5: QC Requirements for Geosynthetics Installations

Geosynthetics Material	Test Description	Test Standard	Value	Frequency
Geomembrane	Vacuum tests on extrusion welds	ASTM D5641	Maintain 35 kPa for 5 seconds	All extrusion welds
	Air pressure tests on fusion weld seams	ASTM D5820	Maintain 200 kPa for 2 minutes	All fusion weld seams
	Destructive tests on welds	ASTM 6392, D4437	Bonded Seam Strength = 13 N/m Peel Seam Strength = 9 N/m	Once every 400 m of weld seams
Geotextile	Heat bonds and patch repair	Visual	N/A	Ongoing
Bentonite	Placement thickness	Visual	No less than 150 mm nominal thickness over a 1 m^2	Ongoing

6. If any seam failed the QC requirement stated in Table 6-5, the Contractor shall do one or a combination of the following:

   (a) Reconstruct the seam between any two passed test locations.

   (b) Extrude weld or cap the failed section to tie into passed sections.

7. The Contractor shall submit an as-built drawing, in digital format, showing in-situ panel layout, seam length and locations, and repairs. The as-built geosynthetics drawings shall be submitted for the Engineer's approval within 7 days of final installation works.

6.5 Part 5 – Quality Assurance

1. The Engineer shall review and approve all material certifications and submittals.

2. The Engineer shall conduct their own inspection on all geosynthetics installation, independent of Contractor's QC.

3. The Engineer shall inspect and signoff on all air pressure tests. The Engineer will observe and inspect all other QC protocol for QA purposes.

----- End of Section 6 -----

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