Mill Valley Fill Extension Stage 2 Final Design Report

Prepared for

Minto Explorations Ltd.

Prepared by

SRK Consulting (Canada) Inc.
1CM002.040
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1 Introduction

1.1 Background

The Minto Mine is located in the Yukon, approximately 240 km north of Whitehorse. Mining and mineral processing started in 2007, with tailings storage occurring as a constructed stack of dewatered (“dry”) tailings. The location of the Dry Stack Tailings Storage Facility (DSTSF) is shown on the plan in Drawing 1. Tailings deposition at the DSTSF finished on November 1, 2012, at which point slurry tailings deposition commenced in the Main Pit.

The maximum tailings thickness at the DSTSF is approximately 25 m. The facility is largely founded on a deep soil deposit characterized as warm permafrost, with the deposit thickness reaching up to 85 m below original ground surface. This soil deposit consists mainly of clay and silt, much of which is ice rich, with occasional sand and gravel lenses.

Ground movement has been observed in the DSTSF for approximately the past five years. Documents and monitoring data reviewed by SRK indicate movements were first identified early in 2009 (SRK 2012). Furthermore, SRK concluded that the area of movement appears to be limited to within or near the edges of the DSTSF facility itself. The movements are occurring at depth within the relatively deep permafrost soil foundation. The available data indicates that a deep shear zone acting as a sliding surface is relatively well defined at depths of 28 to 64 m below the original ground surface and approximately 7 m above the bedrock contact (SRK 2012).

In September 2010, EBA Engineering Consultants Ltd. recommended the construction of a valley-fill buttress, called the Mill Valley Fill Extension (MVFE), downslope (north) of the DSTSF as a measure to arrest the movements. The construction of the MVFE (later referred to as the MVFE Stage 1) began in January 2012 and was completed by late 2013 (Minto 2013). The survey hubs that are being used to monitor rates of DSTSF movement have shown a deceleration ranging between 20 and 60% since the start of the MVFE placement.

Since the movement was originally detected, monitoring instrumentation consisting of inclinometers, survey hubs, thermistors, and piezometers have been installed at the facility during the course of a number of geotechnical investigations. Due to the relatively large magnitude of the displacements and continued mining activities, in most cases instrumentation became inoperable relatively quickly. The most recent suite of instrumentation (ground temperatures cables, vibrating wire piezometers, and inclinometers) was installed during a detailed geotechnical drilling investigation completed in April 2013 (SRK 2013b).

Data collected from the instrumentation installed in 2013 was used to update the understanding of the foundation soils and the movement geometry, as well as used to develop a conceptual design of an additional extension of the MVFE (referred to as the MVFE Stage 2) to incrementally slow and, ultimately, arrest the movements. This conceptual design was presented in SRK (2014a).

In parallel to the development of the conceptual design, Minto and Selkirk First Nation (SFN) appointed Dr. Richard Dawson of Norwest Corporation (with support from Dr. Dave Sego, Professor Emeritus at the University of Alberta) as an independent third party reviewer to review geotechnical conditions at the mine site and provide recommendations to improve stability of various structures, including the DSTSF. One of the reviewer's recommendations for the DSTSF was to utilize a safety factor approach that improves the current safety factor by 30 to 50% (Norwest 2014a). For the conceptual design of the MVFE Stage 2 (SRK 2014a), the 50% improvement to the current factor of safety was adopted as a design criterion and resulted in a three-tiered buttress.

A preliminary design of the MVFE Stage 2 was completed in June 2014 (SRK 2014b) that included a more detailed stability assessment and considered the design and construction recommendations provided in Norwest (2014a). The Minto Mine Independent Third Party Review Summary presentation (Norwest 2014b) presented to Minto and Selkirk First Nations in Whitehorse on September 18, 2014 confirms that these recommendations have been incorporated in the SRK preliminary design.

1.2 Scope of Work

SRK Consulting (Canada) Inc. was retained by Minto to complete the final design of the MVFE Stage 2 to allow for submission of the engineering drawings to the regulatory authorities to allow construction to proceed. Following on the preliminary design described above, the scope of work presented in this report includes:

- Development of engineering drawings and specifications that incorporate the recommendations provided in Norwest (2014) and include construction requirements for:
  - foundation preparation;
  - material specifications;
  - construction methodology; and
  - construction sequencing.
- Development of an instrumentation plan including performance monitoring requirements that will allow for assessments to be completed of the effectiveness of the MVFE Stage 2 in arresting movement of the DSTSF.

The analysis protocol for verification of the MVFE Stage 2 performance, including thermal analysis will be included in a separate scope of work.

2 Site Description

2.1 Surface Features

Local topography around the Minto Mine site consists of rolling hills and ridges with topographic relief ranging from 700 m near the Water Storage Pond to nearly 1,000 m at the highest elevation on the property. The last glaciation to directly affect this site occurred around 200,000 years ago. The most recent regional glaciation (10,000-20,000 years ago) terminated ~50 km to the southeast of the mine site (Duk-Rodkin 1999; EBA 2006); during the latest glacial period, periglacial conditions and events related to deglaciation likely affected the Minto site.

The area of the proposed MVFE Stage 2 is located in the Minto Creek valley, upstream (west) of the Water Storage Pond (Drawing 1). The area is bounded on the south and north by valley slopes, while to the west the boundary consists of the current mine development and facilities. The MVFE Stage 2 will be built by raising MVFE Stage 1 and expanding onto new ground east of the MVFE Stage 1. The valley floor in this area has a gentle 2.5 degree grade down-valley to the east; the valley sides that will contact the Stage 2 extension are sloping at 1.5H:1V on the north slope and 4.5H:1V on the south slope. The face of the existing MVFE has a slope of approximately 2H:1V, and the top surface of the existing MVFE has an average 1.5 degrees grade down-valley.

2.2 Subsurface Features

Discontinuous, warm permafrost, with temperatures generally warmer than -2°C, extending to a depth of about 56 m from the surface is present in the Minto Mine area. A wide range of ice distribution has been observed from drilled core samples collected by SRK (2013a), with some ice-rich strata presenting ice lenses and layers as thick as 1.1 m while others exhibited little excess ice. The north side of Minto Creek in the MVFE Stage 2 area is free of permafrost. Site investigations confirmed the presence of permafrost on the south side of the valley. The northern-most extent of permafrost is not known exactly, but is expected to be roughly in line with the original valley bottom.

The stratigraphy from the borehole logs indicates the existence of a distinct ice-rich clay zone in the lower part of the overburden. The maximum thickness of this zone is approximately 20 m beneath the DSTSF toe, with the thickness decreasing towards the south and the north. The ice-rich clay zone is overlain by interbedded layers of sand, silt, and clay which (in this report) will be referred to as mixed overburden.

The bedrock surface is generally parallel to the surface topography, except for the reach of Minto Creek immediately east and south of the MVFE. In this zone, the deepest portion of the bedrock is offset from the creek alignment to the south by about 200 m. Combined with the surface topography rinsing to the south, this creates an overburden pocket about 200 to 300 m wide with maximum thickness exceeding 80 m. The interpreted bedrock surface and overburden isopach are provided in Appendix C.

2.3 Surface Hydrology

The Minto Mine is located within the Upper Minto Creek watershed, which covers an area of approximately 1,065 ha. The Upper Minto Creek catchment is currently divided by a series of water diversion structures that report to the Main Pit or the Water Storage Pond.

Similar to the existing MVFE Stage 1, MVFE Stage 2 will be situated within the Minto Creek valley west (upgradient) of the Water Storage Pond, and its construction will require decommissioning of the Minto Creek Detention Structure and construction of a new collection sump between the eastern limit of the MVFE Stage 2 footprint and the Water Storage Pond.

2.4 Seismic Hazard

The tectonics and seismicity of southwestern Yukon are influenced primarily by the Pacific and North American lithospheric plate margins. In the Yukon’s St. Elias region, northwest British Columbia and southeast Alaska, the boundary of the two lithospheric plates changes from right lateral transform to subductive. Instead of sliding past each other, the Pacific Plate is forced beneath the stable North American Plate resulting in the St. Elias region being uplifted. This transfer of force along the fault into uplift or mountain building dissipates tectonic energy, reducing seismic effects on the region northeast of and across the fault (SRK 2007).

An assessment of peak ground acceleration was performed for the Minto Mine area using the 2010 National Building Code Seismic Hazard Calculation (Appendix A). The BC Mine Waste Rock Pile Research Committee (1991) outlined that a 10% probability of exceedance in 50 years or the 1:475 event is the appropriate design seismic event for design. The corresponding peak ground acceleration in the Minto project area is approximately 0.057 g.

3 Design Criteria

3.1.1 Stability Criteria

The primary purpose of the MVFE Stage 2 is to provide additional buttressing of the DSTSF to resist the currently observed movement. As mentioned in Section 1.1, an increase in the factor of safety (FOS) values of 50% over the existing buttress (MVFE Stage 1) was selected as an appropriate target for design. At the same time, as per the Yukon's requirements, the MVFE Stage 2 must meet the minimum FOS design criteria recommended in the "Mined Rock and Overburden Piles Investigation and Design Manual" (BC Mine Waste Rock Pile Research Committee 1991), provided in Table 1.

Table 1: BC Mined Rock and Overburden Pile Minimum Factor of Safety Guidelines

Stability Condition	Suggested Minimum Design Values for FOS	
	Case A	**Case B**
Stability of Dump Surface		
Short-term (during construction)	1.0	1.0
Long-term (reclamation – abandonment)	1.2	1.1
Overall Stability (Deep Seated Stability)		
Short-term (static)	1.3 – 1.5	1.1 – 1.3
Long-term (static)	1.5	1.3
Pseudo-static (earthquake)	1.1 – 1.3	1.0

1. Case A
- Low level of confidence in critical analysis parameters
- Possibly unconservative interpretation of conditions or assumptions
- Severe consequence of failure
- Simplified stability analysis method (charts, simplified method of slices, etc...)
- Stability analysis method poorly simulates physical conditions
- Poor understanding of potential failure mechanism(s)

2. Case B
- High level of confidence in critical analysis parameters
- Conservative interpretation of conditions, assumptions
- Minimal consequence of failure
- Rigorous stability analysis method
- Stability analysis method simulates physical conditions well
- High level of confidence in critical failure mechanism(s)

Ranges of suggested minimum design values are presented in Table 1 to reflect different levels of confidence in understanding site conditions, material parameters, and consequences of instability. Although numerous geotechnical characterization studies have been completed around the MVFE Stage 2 area, Case A is considered to be appropriate for the MVFE Stage 2 due to the observed movement and that its design criteria were used to guide the analyses. For pseudo-static (earthquake) analyses, the BC Mine Waste Rock Pile Research Committee (1991)

specifies peak ground accelerations with a 10% probability of exceedance in 50 years.
As mentioned in Section 2.4, the peak ground acceleration of 0.057 g was used in this analysis.

3.1.2 Operational and Closure Considerations

During operations, the northern extent of the MVFE Stage 2 is limited by the need to maintain the existing site access road. The eastern extent is also limited by the maximum footprint of the current Water Storage Pond, and by the need to allow space for the construction of a new collection sump to capture contact water.

The construction material is to consist of run-of-mine waste rock. The bottom portion of the MVFE Stage 2 over previously undisturbed areas must consist of coarse rock fill in order to ensure drainage conditions are maintained in the overlying bulk waste rock.

For closure, the size and geometry must be able to accommodate the other closure elements required or being considered such as the site access road, re-establishment of the Minto Creek channel, and reclamation soil covers. Although designs for these final closure elements remain to be developed, SRK took these elements into consideration in this design.

A soil cover will be constructed on the MVFE during closure. As such, the maximum slope grade was designed to not exceed 3H:1V.

4 MVFE Stage 2 Design Overview

The design of the MVFE Stage 2 is presented in Drawing 2. The total volume of the Stage 2 extension is 1.39 Mm^3, of which about 93,000 m^3 represent the coarse rock drainage base.

The MVFE Stage 2 consists of three tiers that were determined by FOS requirements for the DSTSF, constructed at elevations of 766 m, 776 m, and 781 m. The faces of each of the tiers will be at an overall 3H:1V. The eastern limit of the Stage 2 extension is designed such that the toe is 75 m upstream from the Water Storage Pond at the maximum operational pond elevation of 716.3 m.

The initial lift of the lowest tier of the extension (on original ground) shall be constructed of coarse waste rock to a minimum thickness of 8 m. Vegetation and topsoil from the all previously undisturbed areas are to be stripped prior to fill placement.

The top surface of each tier will be graded in north-south direction to create a minimum 3% grade toward the north (in the direction of the realigned Minto Creek at closure). An access road will be built on the face of the MVFE Stage 2 to allow for site access post-closure.

5 Stability Analysis

For the physical stability assessment, FOS values were utilized as the primary indices for evaluating performance. The assessment focused on mechanisms that drive overall slope failure, i.e. failures near the toe and deep seated failures along the inferred overburden shear zones. Small skin or surficial bench face failures (less than 5 m in depth) were not deemed critical to general stability and thus were not investigated in detail.

The stability was evaluated using a two dimensional slope stability software package, Slide 6.0 (Rocscience 2012), as the primary assessment program. The geometry of all sections is shown in Appendix B-1, while the results of the analyses using Slide 6.0 can be found in Appendix B-2.

A separate stability analysis has been completed on the proposed Main Dam and on the tailings impoundment located at the Main Pit. That analysis considered the potential of a shear zone to be continuous between the 2009 south wall failure of the Main Pit and the DSTSF along the paleochannel containing warm permafrost clays. The analysis found that the dam and impounded tailings would have a negligible impact on the MVFE. Details of the Main Dam stability analysis are provided in SRK (2014c).

5.1 Methodology

5.1.1 General

The method used to define the elevations of the three tiers of the MVFE Stage 2 was the same as that used in the conceptual design (SRK 2014a). The analysis was completed at the original sections (A, B, and C) in the conceptual design as well as at five additional sections (A1, A2, B1, B2 and C1) to confirm that the 50% improvement was achieved. The methodology for the stability analysis at each cross-valley section was as follows:

1. An analysis was first completed with only the DSTSF in place, i.e. no buttress. The undrained shear strength (c_u) of the ice-rich clay was back-calculated at limit equilibrium conditions (i.e. FOS = 1.0).

2. The second analysis was then completed under current site conditions, i.e. with the existing MVFE Stage 1 in place, using the c_u value calculated in Step 1. The FOS value obtained in this step was the reference value used to evaluate the FOS improvement for the Stage 2 design.

3. The third stability analysis was completed using with conceptual design of the MVFE Stage 2 with the tier elevations adjusted to ensure a minimum 50% FOS increase.

Following the completion of the cross-valley analysis, an assessment was completed in the down-valley stability analysis to ensure that the recommended minimum FOS values (see Table 1) were achieved. The lowest back-calculated undrained shear strength value found during the cross-valley analysis was used in this assessment.

Failure mechanisms for the cross-valley stability analyses considered the shear surface identified by inclinometer measurements. Where no instrumentation exists to confirm the presence of a shear zone, a shear plane was specified in the model with the similar characteristics of the known zone (i.e. located in ice-rich clays approximately five to ten meters above the bedrock surface). Since no potential shear surface has been identified in the down-valley direction (Section E), an auto-refine method was used to search for the critical failure surface in this direction.

5.1.2 Geometry

In addition to the sections completed in the conceptual design, an additional five representative cross sections (A1, A2, B1, B2 and C1) were developed for the cross-valley stability analysis. These sections were 50 m to 150 m apart and were oriented approximately in the direction of the identified movement (north-south). Two cross sections (D and E) were developed to evaluate the down-valley (east-west) stability. The section locations can be found in Drawing 2 while Appendix B-1 presents the stratigraphic sections.

The overburden stratigraphy was generated based on the information obtained from previous drilling programs. The locations of the boreholes used to develop the stratigraphy are provided in Drawing 1. The overburden was divided into the three major units:

- A fine-grained clayey layer of variable thickness located near the surface.
- A thick layer of ice-rich clay situated above the bedrock surface. It was assumed that this layer of clay with variable thickness was in the undrained condition.
- The rest of the overburden materials including silt, sands, gravels and cobbles were categorized as a layer of mixed overburden.

The tailings, original ground, and current ground surface topography were provided by Minto. The three-dimensional bedrock surface model was generated based on drillhole information, as detailed in Appendix C.

5.1.3 Material Properties

The material properties used in the analysis are presented in Table 2. These properties were selected following a review of all geotechnical laboratory strength tests at the Minto Mine site detailed in Appendix D. The analyses were completed using the residual strength values to be conservative. Given the similar strength properties for the silt, sand, and residuum materials, these were lumped together as one stratigraphic unit of “mixed overburden” for this analysis.

In the cross-valley stability analysis, the undrained shear strength (c_u) for the ice-rich clay was back-calculated assuming that the situation prior to the placement of the MVFE Stage 1 was at limit equilibrium conditions, i.e. minimum FOS equal to 1.0. The resulting c_u values were found to range from 35 to 70 kPa. In the down-valley stability analysis, the lowest c_u value (35 kPa) was assigned to the ice-rich clay material.

Table 2: Material Properties used in Stability Analysis

Material	Unit Weight (kN/m^3)	Undrained Shear Strength, c_u (kPa)	Friction Angle (°)
Mixed overburden	18.3	0	32
Fine-grained clayey overburden (drained)	17.7	0	23
Ice-rich clay (undrained)	17.7	varies	0
Upper overburden	18.1	0	29
DSTSF compacted tailings	18.6	0	32
Waste rock fill	20.6	0	37

5.1.4 Pore Water Pressures

The MVFE Stage 2 is to be constructed of run-of-mine waste rock with an initial toe blanket layer consisting of coarse waste rock with a minimum thickness of 8 m. Due to the relatively coarse nature of these materials; build-up of pore water pressures is not anticipated.

A phreatic surface was not included in the cross-valley stability analyses as the ground is frozen. As the shear zone material strengths were back-calculated, the resulting strengths incorporate the potential for excess pore water pressure along the shear zone. The stability analyses with the MVFE Stage 2 buttress also did not include a phreatic surface to provide an appropriate comparison in between the two cases. The down-valley stability analyses that did not include back-calculated shear zone properties included a water table that corresponded to the approximate pre-mining ground surface. Additional models were completed in the down-valley analysis to assess an elevated water table that could occur in the event of ice build-up as a result of glaciation at the downstream toe during winter conditions.

5.2 Cross-Valley Stability Results

Table 3 summarizes the results of the stability analysis on the selected eight cross sections in the north-south direction. It presents the elevations of the MVFE Stage 2 in each cross section and the increase of FOS compared to the current conditions. The calculated FOS and the c_u values used in the analysis for each cross section are also included.

Table 3: Cross-Valley Stability Analysis Results

Section	Conditions	Back-Calculated c_u (kPa)	FOS	MVFE Stage 2 Elevation (m)	Percent Improvement in FOS
A1	DSTSF only	50	1.04	-	-
	DSTSF and MVFE Stage 1		1.88	-	-
	DSTSF and MVFE Stage 2		3.36	781	79%
A	DSTSF only	40	1.02	-	-
	DSTSF and MVFE Stage 1		2.10	781	-
	DSTSF and MVFE Stage 2		5.22	781	149%
A2	DSTSF only	65	1.00	-	-
	DSTSF and MVFE Stage 1		2.07	-	-
	DSTSF and MVFE Stage 2		4.60	781	122%
B1	DSTSF only	60	1.05	-	-
	DSTSF and MVFE Stage 1		2.11	-	-
	DSTSF and MVFE Stage 2		3.08	776	46%
B	DSTSF only	70	1.02	-	-
	DSTSF and MVFE Stage 1		1.63	-	-
	DSTSF and MVFE Stage 2		6.22	776	282%
B2	DSTSF and MVFE Stage 1	45	1.02	-	-
	DSTSF and MVFE Stage 2		1.96	766	92%
C1	DSTSF and MVFE Stage 1	40	1.00	-	-
	DSTSF and MVFE Stage 2		1.91	766	91%
C	DSTSF and MVFE Stage 1	35	1.00	-	-
	DSTSF and MVFE Stage 2		1.14	N/A[1]	14%

Source File: Minto_MVFEStage2_SlideModelsResults_Rev05_KK.xlsx

Note(s):

(1) Section C intersects the front face of the MVFE Stage 2 buttress (see Drawing 2)

The MVFE Stage 2 design increases the FOS compared to the current MVFE by over 50% in all sections except sections B1 and C. For Section B1, the increase of FOS is slightly below 50%; however, it remains within the 30-50% range suggested by Norwest (2014). The 2D back-analysis at Section C was deemed to be invalid – the section line is located at the very edge of the DSTSF and intersects the toe slope of the MVFE Stage 2 rather than the bulk of the buttress. Section C was included in this analysis only for completeness – the section was analysed as part of the conceptual design (SRK 2014a) because it contains geotechnical instrumentation that was used to define the movement. Instrumentation data shows that the movement is less at Section C compared to Sections A and B. The weaker back-calculated c_u value at Section C compared to Sections A and B supports the conclusion that a 2D analysis is not appropriate for this section. The neighboring Section C1 shows a 91% percent improvement in the FOS and based on this result, the elevation of the MVFE Stage 2 buttress at the down-valley end is considered to be appropriate.

Detailed stability results and failure mechanisms can be found in Appendix B-2.

5.3 Down-Valley Stability Results

Table 4 presents a summary of the down-valley stability results that are evaluated against the minimum FOS design criteria recommended in the "Mined Rock and Overburden Piles Investigation and Design Manual" (BC Mine Waste Rock Pile Research Committee 1991). All stability results meet or exceed the minimum required FOS.

Table 4: Summary of Down-Valley Slope Stability Results

Condition	Description	Required Factory of Safety	Minimum Calculated FOS
1	Short-term (construction) –Buttress Surface Failure	1.0	2.1
2	Short-term (construction) – Deep Seated Failure	1.1-1.3	2.5
3	Long-term – Buttress Surface Failure	1.1	2.3
4	Long-term – Deep Seated Failure	1.3	2.3
5	Pseudo-static (earthquake) – Deep Seated Failure	1.0	1.5

Section D was also assessed to evaluate the deep-seated stability due to the continuation of the paleochannel that contains similar ice-rich clay material that could be susceptible to movement in the down-valley direction. Results of the assessment are provided in Table 5.

Table 5: Summary of Slope Stability Results above Deep Clay Pockets

Condition	Description	Required Factory of Safety	Minimum Calculated FOS
4	Long-term – Deep Seated Failure	1.3	3.6
4	Long-term (Earthquake) – Deep Seated Failure	1.0	1.5

A sensitivity analysis was also completed to evaluate the effect of a five meter rise in the water table along Section E that could potentially result due to ice build-up of the seepage at the downstream toe. Results show a small decrease in the calculated FOS with all values remaining higher than the recommended values. The results of the analysis are presented in Table 6.

Table 6: Summary of Down-Valley Slope Stability Results (with Water Table)

Condition	Description	FOS with Water Table at Original Ground	FOS with Water Table 5 m above Original Ground
2	Short-term (construction) – Deep Seated Failure	1.9	1.8
4	Long-term – Deep Seated Failure	2.0	1.9
5	Pseudo-static (earthquake) – Deep Seated Failure	1.4	1.4

6 Settlement Assessment

The addition of the MVFE Stage 2 fill will increase loading on the overburden foundation causing consolidation of the soils (if unfrozen) and associated settlement. This section discusses the expected settlement as a result of this loading and its implications for closure.

The total expected settlement of the buttress is made up of three different components:

- Settlement due to consolidation of the overburden in the thawed active zone (approximately the top five meters);
- Settlement due to long-term thawing of the permafrost and melting of the excess ice in the overburden foundation; and
- Settlement due to consolidation of the thawed permafrost overburden foundation.

The magnitude of the first component is small relative to the other two components and is likely to occur largely during construction or within the year following completion. This settlement component is expected to be negligible in the area overlying the existing MVFE Stage 1 fill.

Excess ice melting in the foundation will result in the largest settlement (in the order of meters), especially in the thick ice-rich clay underlying the south edge of the buttress. Thaw and concurrent settlement are expected to occur over decades to centuries, in parallel with thaw and related settlement under the DSTSF and across the site in general.

Differential settlement will undoubtedly occur due to the unevenly distributed load and the variable thickness of the fine-grained materials in the overburden foundation. Thawing of the ice-rich overburden foundation zones may cause larger settlement south of Minto Creek, compared to the north side where overburden thicknesses are lower and where permafrost is largely absent.

The magnitude of the differential settlement will increase with time; however, the risk posed to the MVFE Stage 2 is low as there are no settlement-sensitive design features present (such as a clay core or impermeable membranes). Risks arising from differential settlement to other site features are low, as no buildings or sensitive infrastructure are planned to be constructed on this facility.

Over the long term, risks posed by differential settlement can be mitigated if required by regrading the surface (if small incremental settlement present) or by placement of additional fill.

7 Construction Requirements

Construction requirements are summarized below:

- Prior to construction, vegetation and topsoil in undisturbed areas within the MVFE Stage 2 footprint is to be stripped and a new seepage collection system (to replace the current Minto Creek Detention Structure) is to be constructed down-valley of the MVFE Stage 2. Details of the MVFE Stage 2 collection system is provided in SRK (2015).
- Existing slope inclinometers, piezometers, and ground temperature cables within the MVFE Stage 1 and Stage 2 footprints are to be preserved. Details of the instrumentation are provided in Section 9.
- The 700 mm vertical culverts at water monitoring stations W8 and W8A are to be preserved and extended to the MVFE Stage 2 final ground surface. Details of the culverts are provided on Drawing 3.
- The initial toe blanket layer of coarse waste rock is to be constructed over previously undisturbed areas using run of mine waste rock. This layer is to be constructed using the same methodology as was used during the construction of the existing MVFE (Minto 2013), with dumping taking place from a height of 10 m above the valley floor, and relying on the natural segregation that arises in high bench faces to ensure that coarse materials will line the bottom of the valley and not impede the flow of water.
- Subsequent lifts are to consist of run-of-mine waste rock placed in an ascending (bottom up) construction methodology with maximum 5 m lifts. Each lift will be stepped in to create a face of 3H:1V final overall grade.
- Waste rock is to be end-dumped and rough graded using a dozer for access, traffic compaction and grade control prior to placement of the next lift.
- Snow accumulation of significant thickness (greater than 0.3 m) should not be allowed to build up between horizontal lifts. If thicker accumulations of snow develop, these should be removed before additional waste rock is placed over that area.

8 Surface Water Management

The site grading design and the surface drainage system for the MVFE Stage 2 will be part of the site surface water management plans (Minto 2014a and subsequent revisions). Further surface water management recommendations, outlined below, can be considered to minimize the risk of developing elevated pore water pressures within the buttress.

- The rough surface grade of each lift should have a minimum 0.5% overall grade sloped toward the north to promote runoff and to avoid surface ponding.
- Localized erosion of interim end-dumped bench faces is expected and is not a concern for the overall stability of the buttress. Any areas of consistent and notable localized erosion, specifically those that cause significant material transport or are greater than 1 to 2 m in depth should be remediated, for example by pushing coarser rock into the erosion gullies and by reducing/diverting flow paths in the eroded area. To assist with long-term erosion control, the final reclamation surface will be constructed with slopes of 3H:1V or gentler.
- The MVFE Stage 2 should be tied-in at the top of the buttress to the existing access roads and other infrastructure, in a manner that avoids water accumulation.

9 Geotechnical Instrumentation

Drawing 3 provides details and locations of the existing and proposed instrumentation to monitor the MVFE Stage 2 performance. New instrumentation consists of survey hubs and vibrating wire piezometers (with temperature sensors). All existing instrumentation in the MVFE Stage 2 footprint is to be preserved as much as practical. Replacement of damaged instrumentation as well as additional instrumentation to monitor post-construction conditions will be assessed following construction.

The following section describes the instrumentation installation requirements for the DSTSF and MVFE Stages 1 and 2.

Survey Hubs

There are five survey hubs located within the MVFE footprint that will be destroyed as a result of the MVFE Stage 2 construction (DSSH06, DSSH10, DSSH18, DSSH19, DSSH20). These hubs are to be replaced following construction to allow for comparisons of the movement rates pre- and post-construction. Hubs DSSH19 and DSSH20 are located beneath sloped areas of the MVFE Stage 2 and should may be relocated to a nearby crest.

Five new survey hubs (DSSH26 to DSSH30) are proposed to be installed following completion of construction at the locations indicated on Drawing 3.

Inclinometers

No new inclinometers are proposed at this time to monitor movements as the movement rates are more readily able to be measured using the survey hubs.

Piezometers

Table 7 provides details of the proposed vibrating wire piezometers installations within the MVFE Stage 2 foundation materials to measure pore water pressures. Two of these locations (15-DSP-7 and 15-DSP-8) are located in areas where high plastic clays may be present. Multiple sensors are proposed at these locations to monitor for potential excess pore pressure which may develop due to long term thawing. All sensors will also include the capability to monitor ground temperature.

The piezometers are to be installed prior to construction to allow for measurement of pore pressures during initial placement and operations. The piezometers leads are to be routed from the fill to read-out locations outside of the MVFE Stage 2 footprint. Typical details of the piezometer leads and read-outs are provided on Drawing 3.

Table 7: Proposed Vibrating Wire Piezometers

ID	Northing (m)	Easting (m)	Current Ground Elevation (m)	Number of Sensors	Sensor Depths (m)	Lead Length (m)
15-DSP-7	6,944,967.10	385,804.61	766.90	9	15, 20, 25, 35, 45, 50, 55, 60, 65	2
15-DSP-8	6,945,606.04	385,873.37	754.01	6	5, 10, 15, 20, 25, 30	70
15-DSP-9	6,945,165.38	385.836.24	732.68	1	5	75
15-DSP-10	6,945,225.28	385,944.40	722.15	1	5	75

Ground Temperature Cables

No function ground temperature cable will be destroyed as part of the MVFE Stage 2 construction. No new stand-alone ground temperature cables are proposed as the proposed piezometer cables will also have the ability to measure temperatures.

10 Performance Monitoring

10.1 Visual Inspections

Routine monitoring on a monthly basis should also be completed by Minto staff and include regular inspection of:

- Fill slopes for any signs of distress;
- Crest of each tier for any signs of cracking;
- Toes of each of the tiers for any signs of sloughing, deformation, seepage, or ice-build-up;
- All observed seepage or seeps should be noted and monitored;
- Ice build-up at the downstream toe should be monitored during the wintered months; and
- The existence of potential bulges in the toe areas downstream of the dump toe should also be checked.

Equipment operators should inspect the crest of the bench they working on as part of their regular field level risk assessment and inspect for any signs of cracking at or near the dump crest or look for any areas where toes appear to be ‘bulging’.

Any areas of concern or apparent/rapid changes should be brought to the attention of the site engineer and engineer-of-record for further evaluation.

An annual visual physical inspection of the MVFE Stage 2 should be completed by a qualified geotechnical engineer. Following these inspections, site inspection reports should be completed to outline any findings and observations, to include recommendations for maintenance, and to modify the monitoring program or the design if/as appropriate.

10.2 Surveying Requirements

The crest and toes of the MVFE Stage 2 should be surveyed at the completion of each construction phase to compare the as-built geometry to the design surface and to monitor for any deformations within previous lifts. Any significant deviations that could affect dump stability should be brought to the attention of the managing site engineer for further evaluation. To ensure early warnings or areas of slow movement are not missed, the crests and toes should be resurveyed and reviewed annually.

If any areas of continued movement are noted, then additional slope stability monitoring instrumentation (e.g., inclinometers or fixed survey monuments) should be installed to better estimate rates of deformation and to pick up potential accelerations of movement, which can be precursors to large failures. Routine monitoring requirements are discussed in the following section.

10.3 Instrumentation Monitoring

Instrumentation monitoring during construction and operation of the MVFE Stage 2 is to be routinely completed in accordance with the Physical Monitoring Plan (Minto 2014b). This document describes the inspection and instrumentation data collection frequencies, installation details, as well as the data collection procedures. Threshold triggers and actions for each instrumentation type are presented in the Operational Adaptive Management Plan (Minto 2014c).

The initial monitoring frequencies at the time of design are summarized in Table 8. The frequencies may be altered over the course of construction and operations according to the adaptive management plan. All subsequent revisions of the Minto (2014b) and Minto (2014c) are to supersede this document. Monitoring requirements should be re-assessed at closure.

Table 8: Instrumentation Monitoring Frequencies

Instrument Type	Reading Frequency
Inclinometer	Bi-weekly
Survey hubs	Weekly
Piezometers	Monthly
Ground temperature cables	Monthly

All instrumentation data is (e.g. survey hubs, inclinometer, piezometers and ground temperature cables) to be reviewed by a qualified geotechnical engineer annually, or as determined by the adaptive management plan.

This final report, "Mill Valley Fill Extension Stage 2 Preliminary Design Report", was prepared by SRK Consulting (Canada) Inc.

ORIGINAL SIGNED AND STAMPED

Peter Mikes, MEng. PEng
Senior Consultant

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Iozsef Miskolczi, MASc. PEng
Senior Consultant

and reviewed by

ORIGINAL SIGNED BY

Cam Scott, MEng, PEng
Practice Leader

All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

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.

11 References

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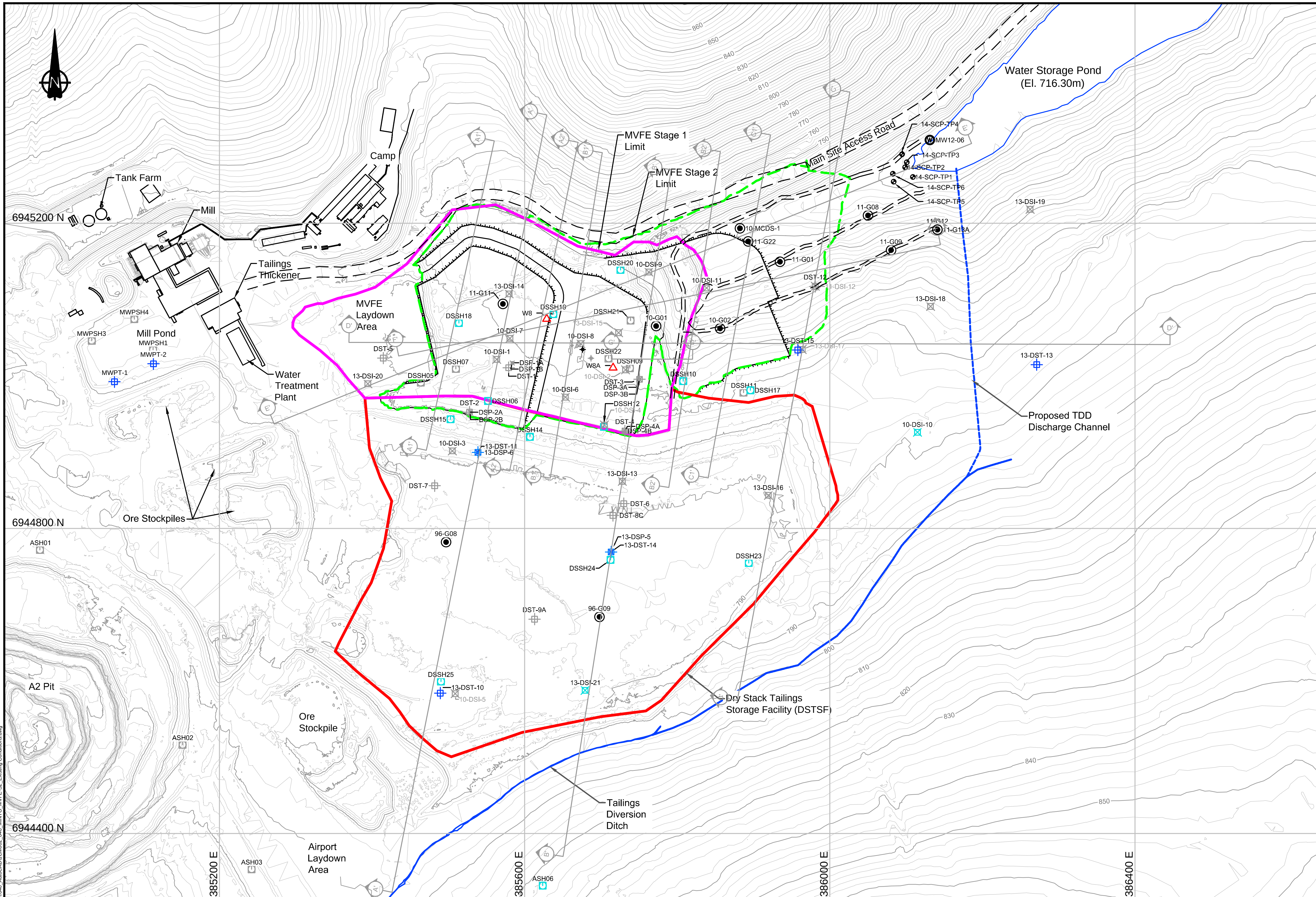
Drawings



LEGEND

- Test pits
- Geotechnical Borehole
- Monitoring Well
- Water Monitoring Station
- Ground Temperature Cable - Active
- Ground Temperature Cable - Destroyed
- Survey Hubs - Active
- Survey Hubs - Destroyed
- Slope Inclinometer - Active
- Slope Inclinometer - Destroyed
- Vibrating Wire Piezometer - Active
- Vibrating Wire Piezometer - Destroyed
- DSTF Limits
- MVFE Stage 1 Limits
- MVFE Stage 2 Limits
- Diversion Ditch

NOTES

1. Contour Interval is 2m.
2. Ground Contours provided by Minto Mine, September 2014.

0 40 80 120 160 200
Scale in Metres

NO.	DESCRIPTION	CHK'D	APP'D	DATE
0	Issued for Construction	PM	EMR	15Sept15
A	Issued for Review	PM	CS	31Aug15

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FILE NAME: 1CM002_040_MINTO_MVFE-S2_Existing Conditions.dwg

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Minto Mine

SRK JOB NO.: 1CM002.040

MVFE Stage 2 Final Design

DRAWING TITLE: Plan View - Existing Conditions

DRAWING NO. 1 | SHEET 1 of 10 | REVISION NO. 0

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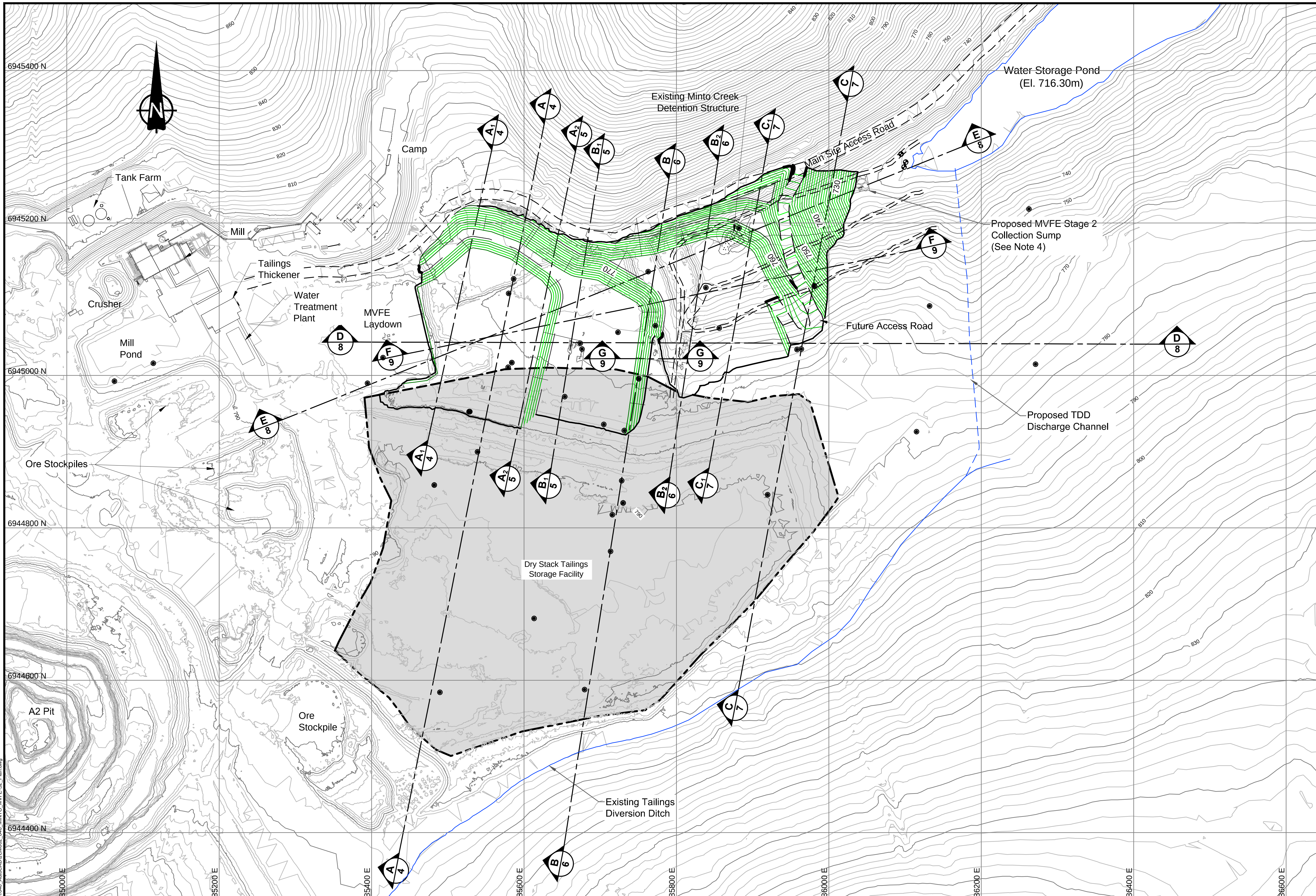

LEGEND

- Boreholes
- Test pits
- Existing Major Contour (10m)
- Existing Minor Contour (2m)
- Proposed Major Contour (5m)
- Proposed Minor Contour (1m)
- Road Lines
- Water Storage Pond (Max. operating level 716.3m)
- DSTSF

NOTES

1. Contour interval is 1m.
2. Coordinate system shown is UTM Zone 8, NAD 83.
3. Topography received from Minto Mine September 2014.
4. Details of the MVFE 2 Collection Sump provided in SRK memo entitled "Design for the MVFE Stage 2 Collection Sump" dated June 11, 2015.
5. Details of the proposed TDD Discharge Channel provided in SRK Memo entitled "Upgraded Tailings Diversion Ditch Design" dated September 24, 2014.
6. All functional instrumentation within the MVFE Stage 2 footprint are to be preserved.
7. All vegetation and topsoil within undisturbed areas within the MVFE Stage 2 footprint is to be stripped and stockpiled.
8. All fill material is to consist of construction grade waste rock sourced from the Minto North.
9. A toe blanket is to be constructed over previously undisturbed areas in the Minto Creek valley by end-dumping from a minimum lift height of 10 m above the valley floor.
10. Subsequent lifts are to be placed in a bottom-up construction methodology in maximum 5 meter lifts.

0 20 40 60 80 100
Scale in Metres

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MVFE Stage 2 Final Design

DRAWING TITLE:
Plan View - MVFE Stage 2

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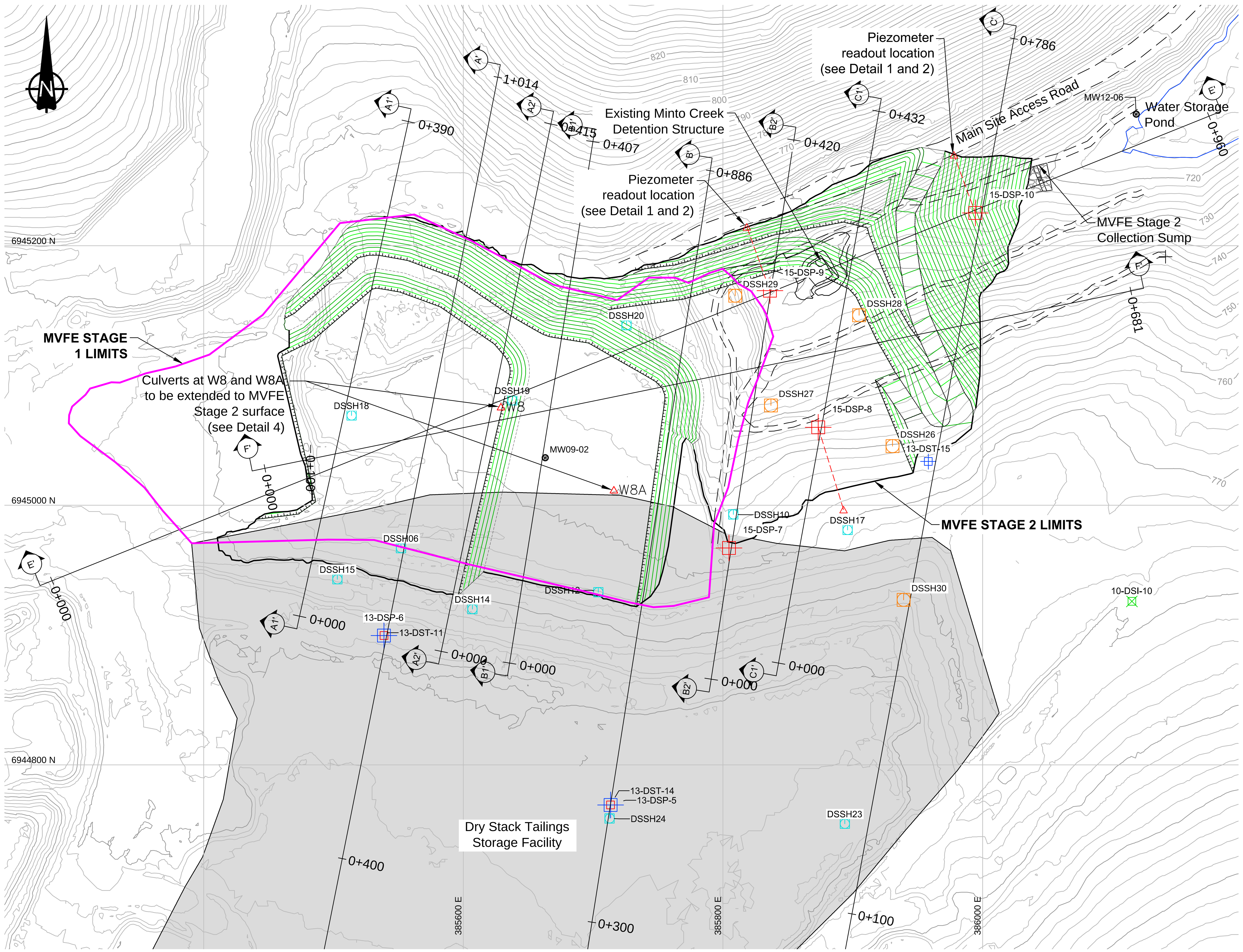

PLAN

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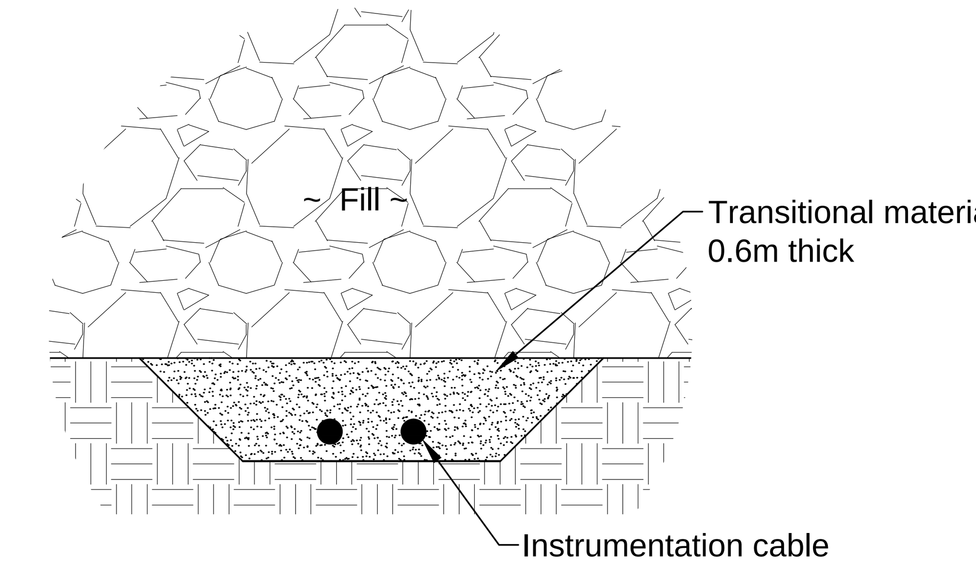
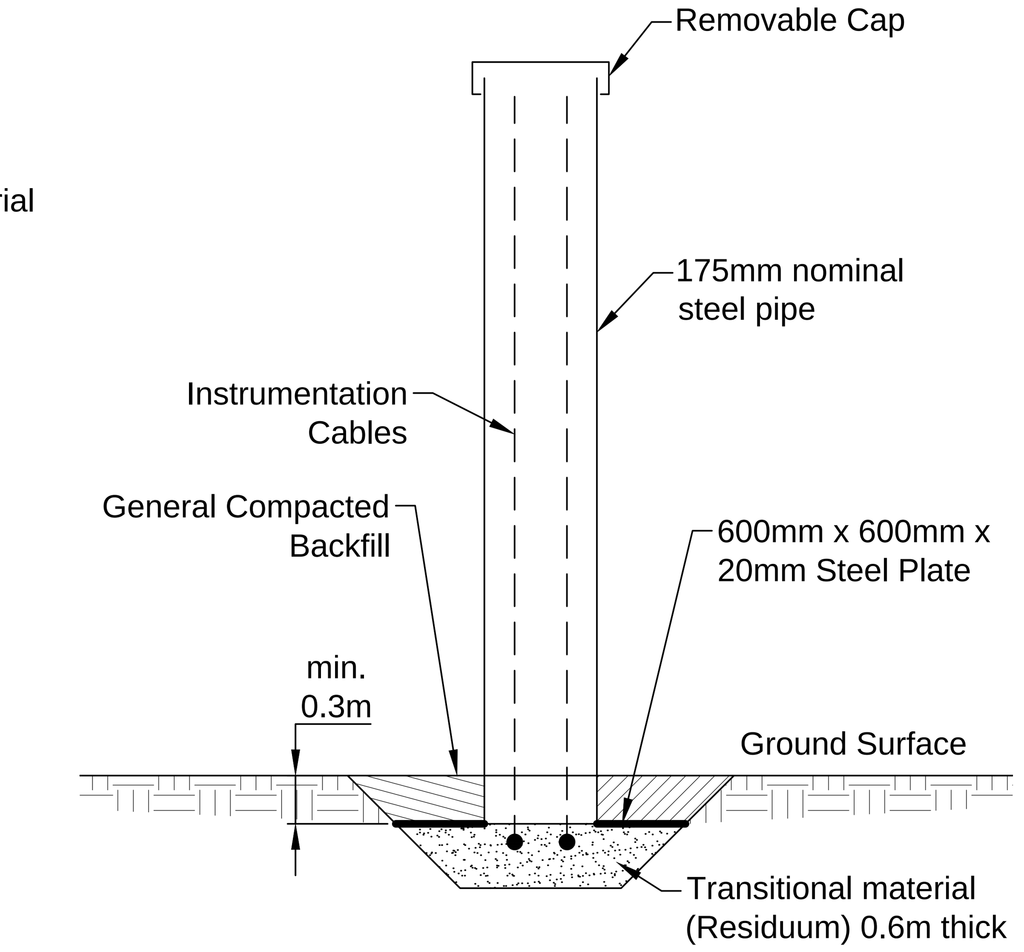
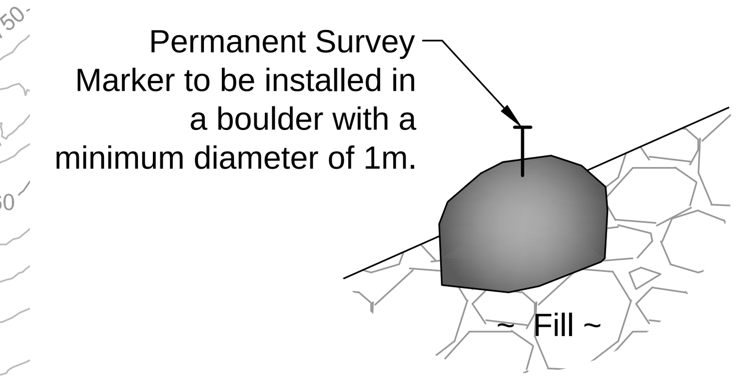


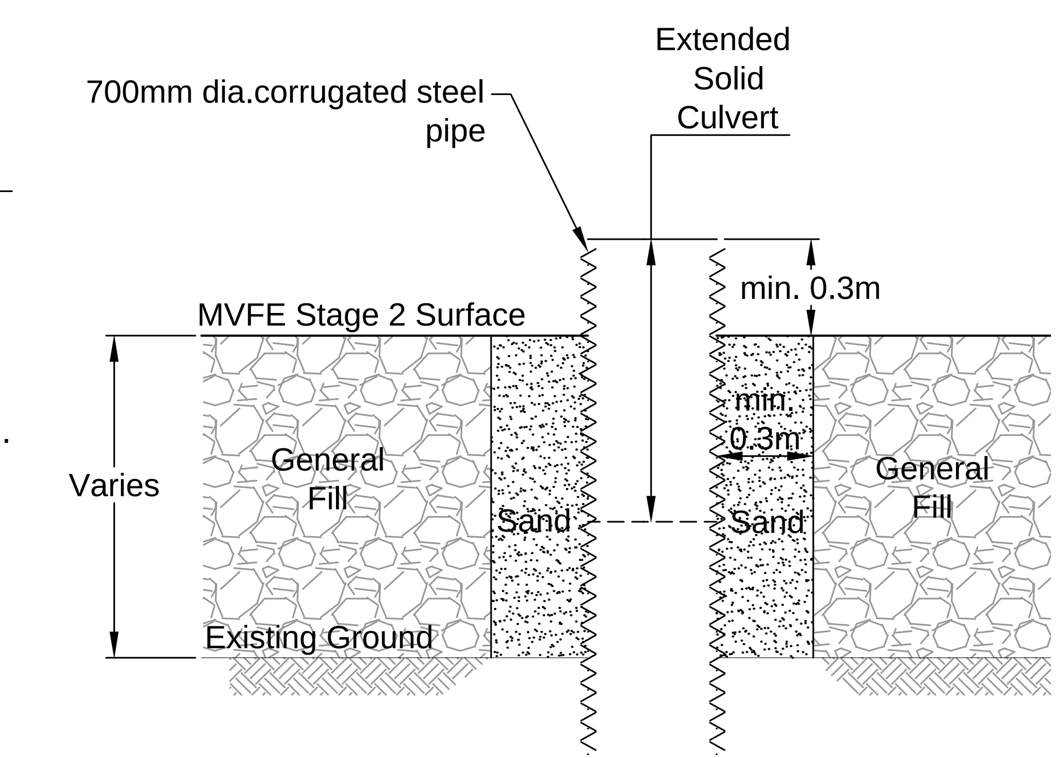
DETAIL 1
INSTRUMENTATION CABLES ALONG SURFACE



DETAIL 2
INSTRUMENTATION READ OUT LOCATIONS



DETAIL 3
SURFICIAL SURVEY HUB INSTALLATION



DETAIL 4
WATER MONITORING STATION EXTENTIONS

NOTES

1. All sharp objects or protruding materials that could damage the instrumentation cables are to be removed prior to installing cables on surface.
2. Cables are to be covered with a minimum 0.6m of transitional material (residuum) as protection prior to general fill placement.
3. All monitoring points are to be clearly marked for easy identification.
4. All instrumentation to be installed under the supervision of a technician or engineer with experience in such work.
5. All piezometers to be installed in boreholes that are fully grouted to surface with a cement-bentonite grout.
6. 13-DSP-7 and 13-DPS-8 are to consist of multi-point piezometer stings connected on a single cable.
7. All piezometers to consist sensors are to have a pressure rating of 700kPa and be equipped with a thermistor with a 0.1°C resolution.

Proposed Instrumentation Details

Instrument Type	Instrument ID	Northing	Easting	Elevation (m)	Sensor Depths (m)	Lead Length (m)
Piezometer	15-DSP-7	6944967.0960	385804.61	766.90	15, 20, 25, 35, 45, 50, 55, 60, 65	2
Piezometer	15-DSP-8	6945060.0400	385873.37	754.01	5, 10, 15, 20, 25, 35	70
Piezometer	15-DSP-9	6945165.3780	385836.24	732.68	5	75
Piezometer	15-DSP-10	6945225.2760	385994.40	722.15	5	75
Survey Hub	DSSH26	6945045.6090	385930.60	-		
Survey Hub	DSSH27	6945076.9130	385836.99	-		
Survey Hub	DSSH28	6945146.4470	385904.85	-		
Survey Hub	DSSH29	6945161.4540	385809.45	-		
Survey Hub	DSSH30	6944927.1370	385939.17	-		

LEGEND

- Boreholes
- Test pits
- Monitoring Well
- Proposed Piezometer Location
- Proposed Piezometer Read Out Location
- Water Monitoring Station
- Survey Hubs - Active
- Proposed Survey Hubs
- Ground Temperature Cable - Active
- Slope Inclinometer - Active
- Vibrating Wire Piezometer - Active
- Existing Major Contour (10m)
- Existing Minor Contour (2m)
- DSTSF
- Proposed Major Contour (5m)
- Proposed Minor Contour (1m)
- Road Lines
- Water Storage Pond (Max. operating level 716.3m)
- MVFE Stage 1 Limits
- MVFE Stage 2 Limits

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DRAWING TITLE:
Instrumentation Plan

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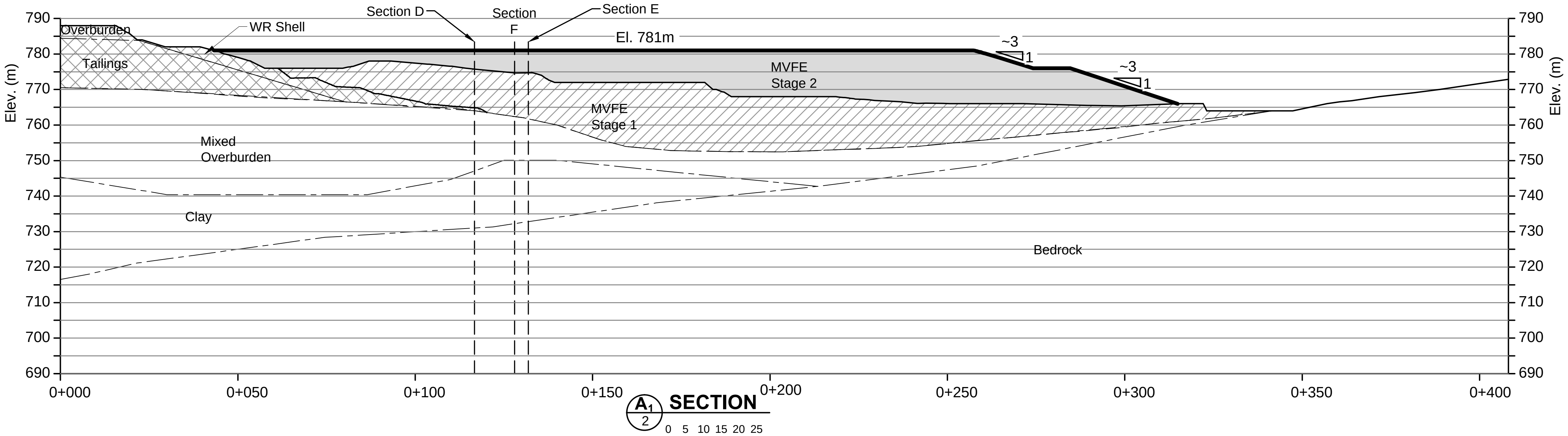

A1 / 2 SECTION

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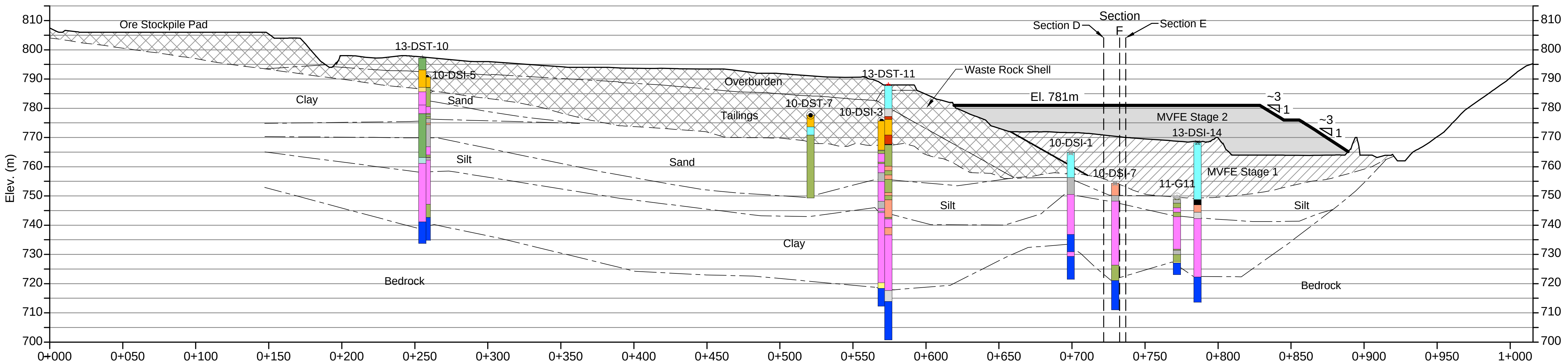

A / 2 SECTION

2x Vertical Exaggeration

0 10 20 30 40 50
Scale in Metres

LEGEND

- MVFE Stage 2
- Existing Ground Surface
- Pre-Mining Ground Surface
- Inferred Stratigraphy
- Mill Valley Fill Extension Stage 2
- Mill Valley Fill Extension Stage 1
- Tailings Storage Facility (includes tailings, WR shell and overburden cover)
- Coarse Rock Fill

Borehole Stratigraphy

- Waste Rock
- Tailings
- Organics
- Gravel
- Sand
- Clay
- Silt
- Residuum
- Bedrock

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MVFE Stage 2 Final Design

DRAWING TITLE: Section A1 - A1' and A - A'

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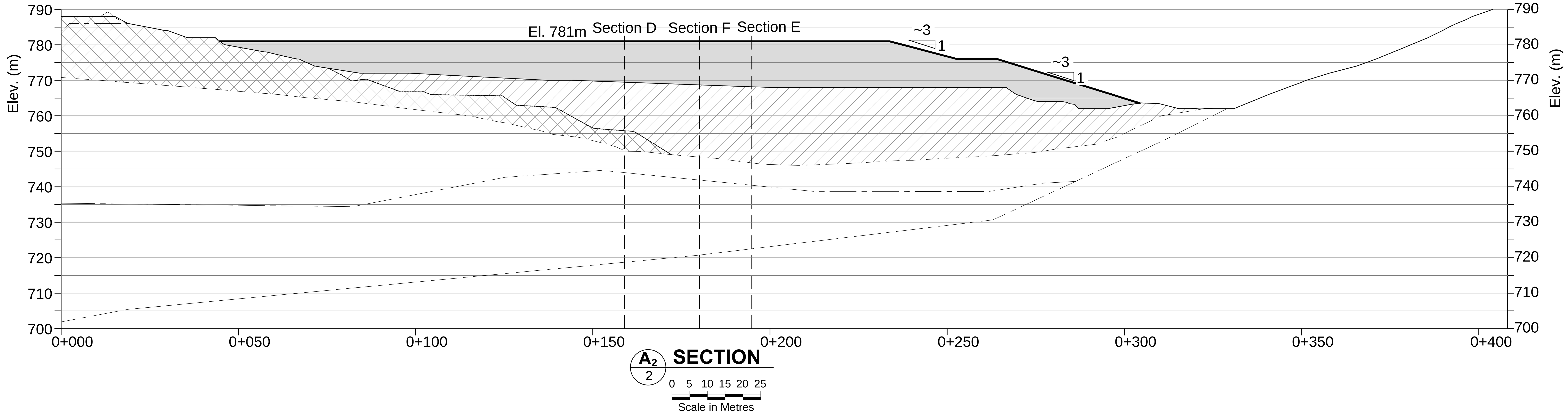

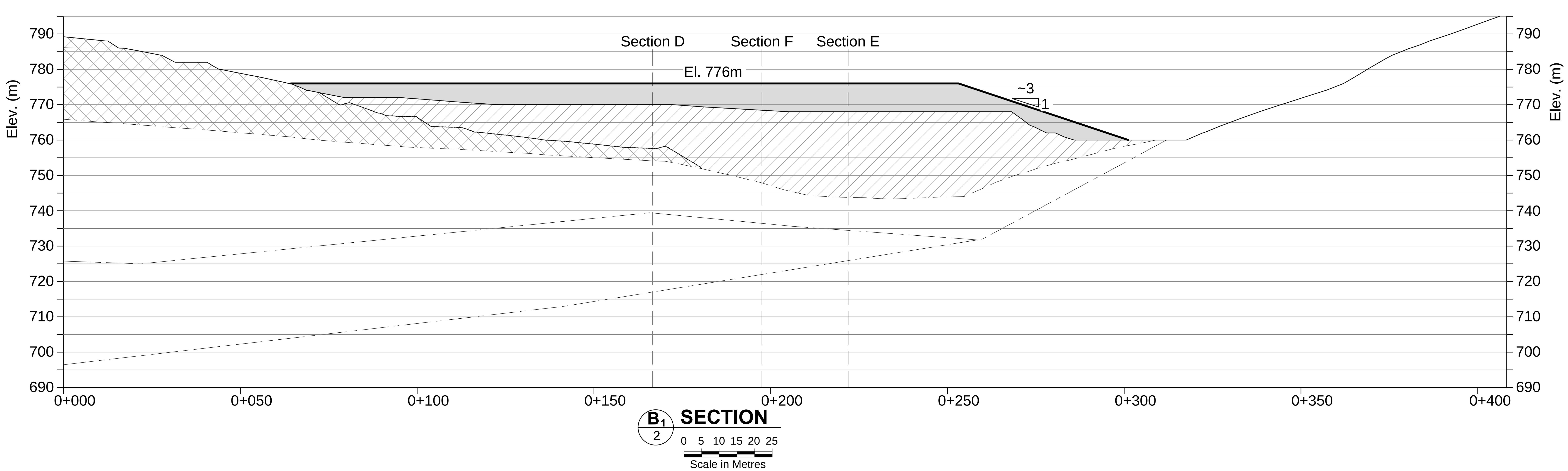

LEGEND

- MVFE Stage 2
- Existing Ground Surface
- Pre-Mining Ground Surface
- Inferred Stratigraphy
- Mill Valley Fill Extension Stage 2
- Mill Valley Fill Extension Stage 1
- Tailings Storage Facility (includes tailings, WR shell and overburden cover)
- Coarse Rock Fill

Borehole Stratigraphy

- Waste Rock
- Tailings
- Organics
- Gravel
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- Clay
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NO.	DESCRIPTION	CHK'D	APP'D	DATE
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DRAWING NO.: 5 | SHEET: 5 of 10 | REVISION NO.: 0

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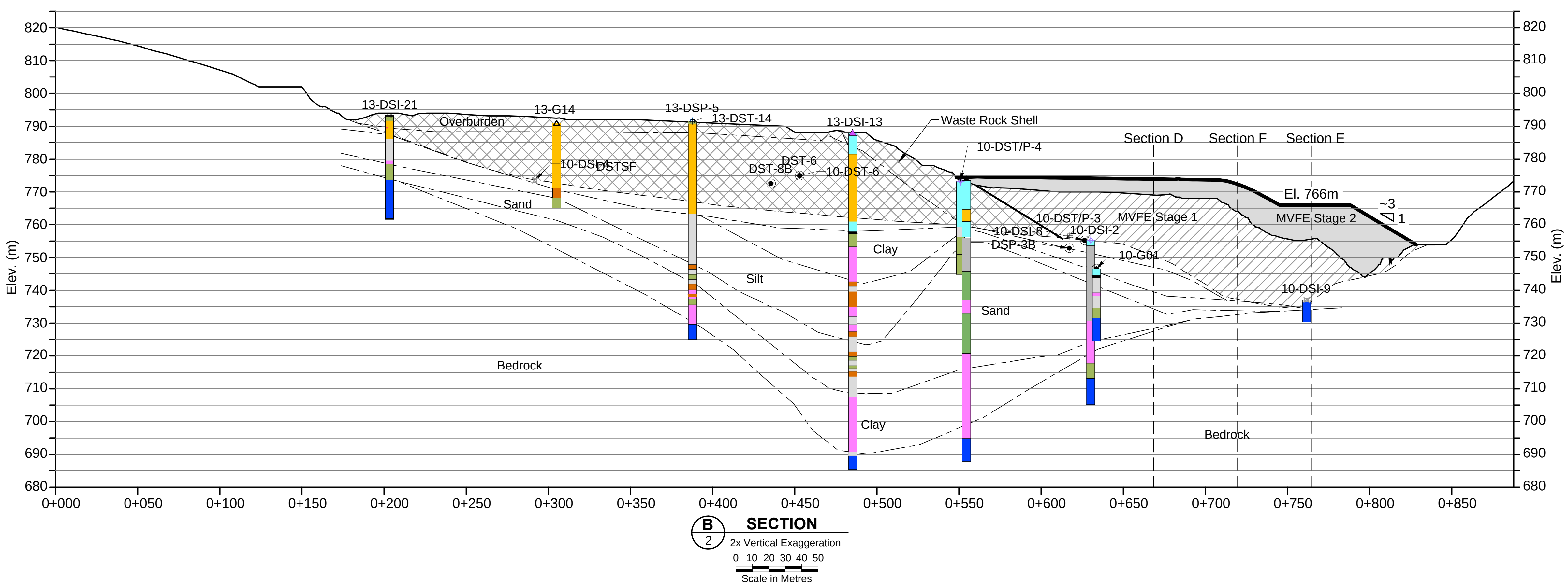

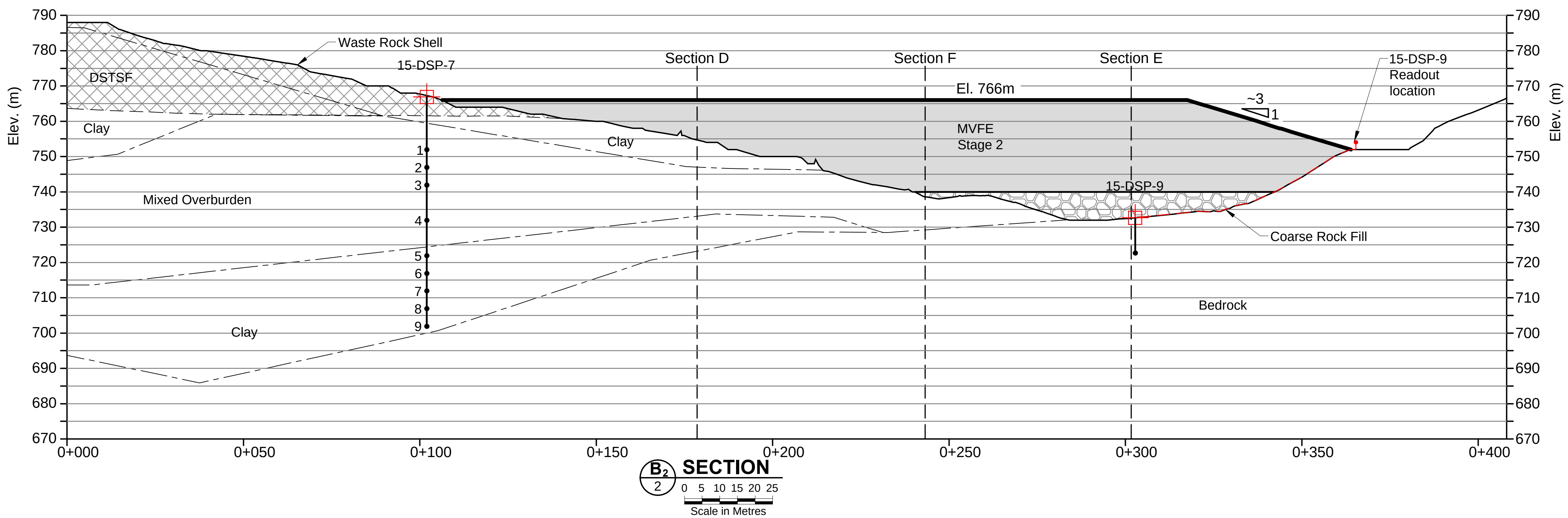

LEGEND

- MVFE Stage 2
- Existing Ground Surface
- Pre-Mining Ground Surface
- Inferred Stratigraphy
- Mill Valley Fill Extension Stage 2
- Mill Valley Fill Extension Stage 1
- Tailings Storage Facility (includes tailings, WR shell and overburden cover)
- Coarse Rock Fill
- Proposed Piezometer

Borehole Stratigraphy

- Waste Rock
- Tailings
- Organics
- Gravel
- Sand
- Clay
- Silt
- Residuum
- Bedrock

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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Minto Mine

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MVFE Stage 2 Final Design

DRAWING TITLE:
Section B - B' and B2 - B2'

DRAWING NO. 6 | SHEET 6 of 10 | REVISION NO. 0

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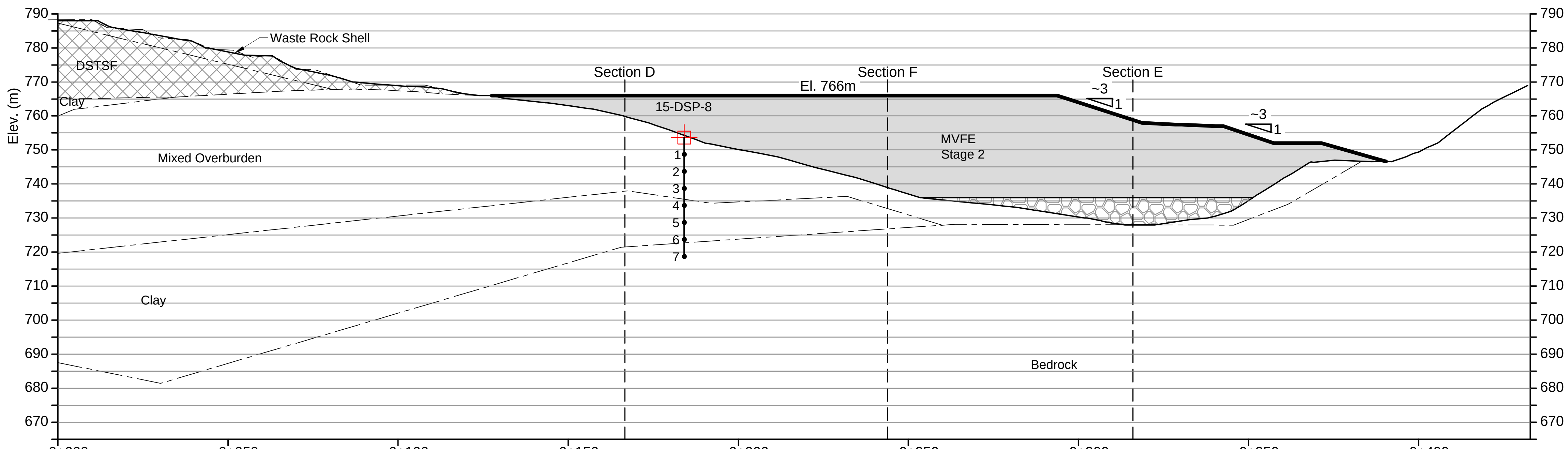
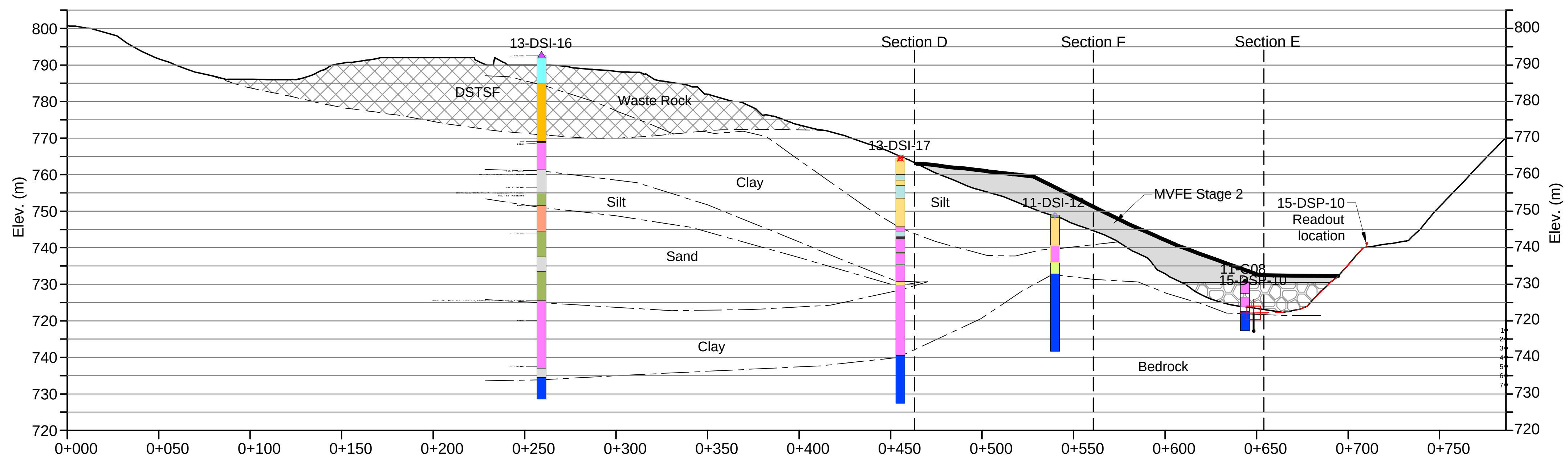


C1 / 2 **SECTION**

0 5 10 15 20 25

Scale in Metres



C / 2 **SECTION**

2x Vertical Exaggeration

0 10 20 30 40 50

Scale in Metres

LEGEND

- MVFE Stage 2
- Existing Ground Surface
- Pre-Mining Ground Surface
- Inferred Stratigraphy
- Mill Valley Fill Extension Stage 2
- Mill Valley Fill Extension Stage 1
- Tailings Storage Facility (includes tailings, WR shell and overburden cover)
- Coarse Rock Fill
- Proposed Piezometer

Borehole Stratigraphy

- Waste Rock
- Tailings
- Organics
- Gravel
- Sand
- Clay
- Silt
- Residuum
- Bedrock

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Minto Mine

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DRAWING TITLE: Section C1 - C1' and C - C'

DRAWING NO. 7 | SHEET 7 of 10 | REVISION NO. 0

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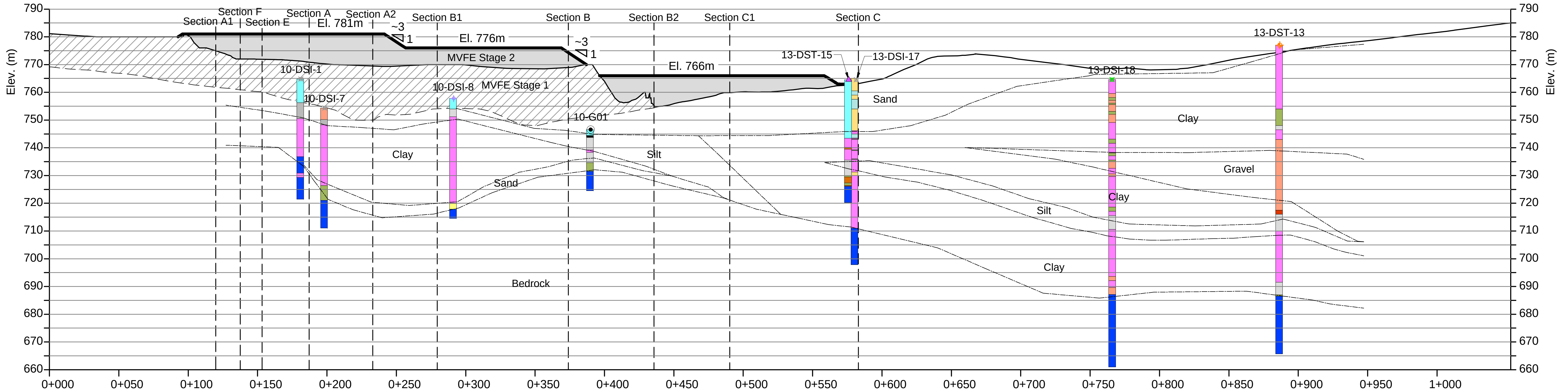

D SECTION 2
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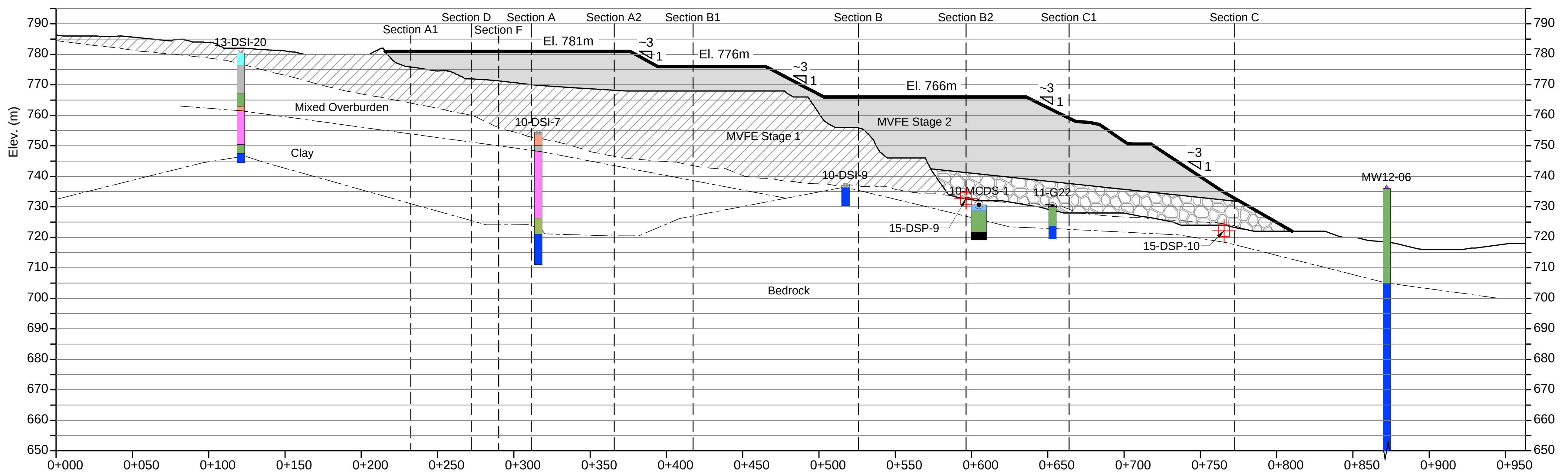
Scale in Metres



E SECTION 2
2x Vertical Exaggeration
0 10 20 30 40 50
Scale in Metres

LEGEND

- MVFE Stage 2
- Existing Ground Surface
- Pre-Mining Ground Surface
- Inferred Stratigraphy
- Mill Valley Fill Extension Stage 2
- Mill Valley Fill Extension Stage 1
- Tailings Storage Facility (includes tailings, WR shell and overburden cover)
- Coarse Rock Fill
- Proposed Piezometer

Borehole Stratigraphy

- Waste Rock
- Tailings
- Organics
- Gravel
- Sand
- Clay
- Silt
- Residuum
- Bedrock

NO.	DESCRIPTION	CHK'D	APP'D	DATE
0	Issued for Construction	PM	EMR	15Sept15
A	Issued for Review	PM	CS	31Aug15

REVISIONS

DRAWING NO. | DRAWING TITLE | DRAWING NO. | DRAWING TITLE

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srk consulting

DESIGN: IM | DRAWN: DJ | REVIEWED: PM
CHECKED: PM | APPROVED: EMR | DATE: September 15, 2015

FILE NAME: 1CM002_040_MINTO_MVFE-S2_Sections.dwg

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Minto Mine

SRK JOB NO.: 1CM002.040

MVFE Stage 2 Final Design

DRAWING TITLE:
Section D - D' and E - D'

DRAWING NO. 8 | SHEET 8 of 10 | REVISION NO. 0

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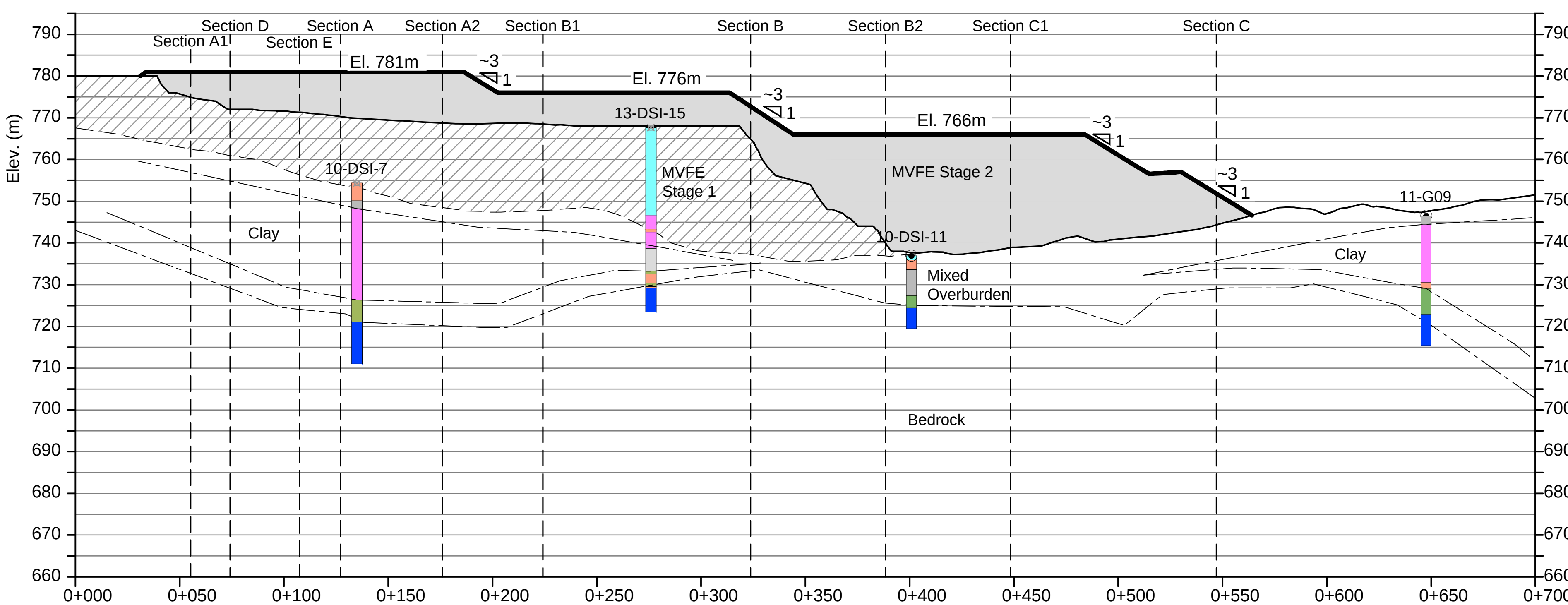

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2
SECTION
2x Vertical Exaggeration
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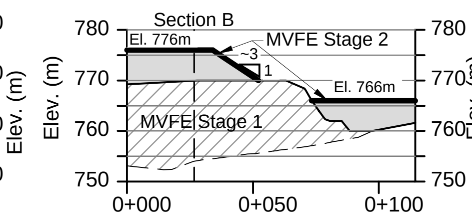
Scale in Metres



G
2
SECTION
2x Vertical Exaggeration
0 10 20 30 40 50
Scale in Metres

LEGEND

- MVFE Stage 2
- Existing Ground Surface
- Pre-Mining Ground Surface
- Inferred Stratigraphy
- Mill Valley Fill Extension Stage 2
- Mill Valley Fill Extension Stage 1
- Tailings Storage Facility (includes tailings, WR shell and overburden cover)
- Coarse Rock Fill

Borehole Stratigraphy

- Waste Rock
- Tailings
- Organics
- Gravel
- Sand
- Clay
- Silt
- Residuum
- Bedrock

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A	Issued for Review	PM	CS	31Aug15

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CHECKED: PM	APPROVED: EMR	DATE: September 15, 2015

FILE NAME: 1CM002_040_MINTO_MVFE-S2_Sections.dwg

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Minto Mine

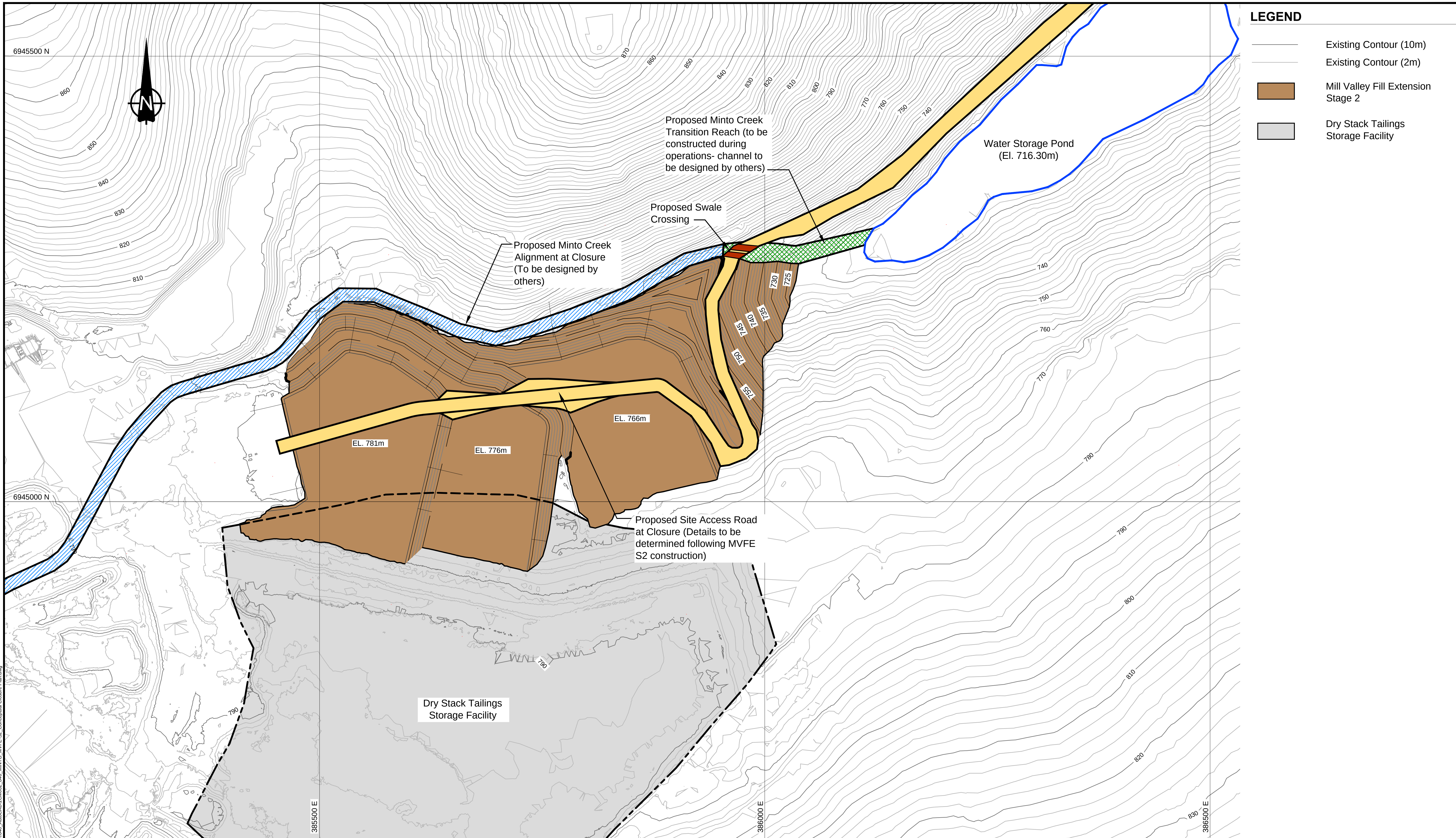
SRK JOB NO.: 1CM002.040

MVFE Stage 2 Final Design

DRAWING TITLE:

Section F - F' and G - G'

DRAWING NO.	SHEET	REVISION NO.
9	9 of 10	0



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REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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A	Issued for Review	PM	CS	31Aug15

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DESIGN: IM	DRAWN: DJ	REVIEWED: PM
CHECKED: PM	APPROVED: PM	DATE: September 15, 2015

FILE NAME: 1CM002_040_MINTO_MVFE-S2_Conceptual Closure Plan.dwg

Minto Mine

SRK JOB NO.: 1CM002.040

MVFE Stage 2 Final Design

DRAWING TITLE: Conceptual Closure Plan

DRAWING NO.	SHEET	REVISION NO.
10	10 of 10	0

Appendices

Appendix A: Site Seismic Hazard Evaluation

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

April 11, 2014

Site Coordinates: 62.6194 North 137.2504 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	**0.133**	**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)	0.038	0.070	0.093
Sa(1.0)	0.025	0.045	0.057
Sa(2.0)	0.017	0.029	0.036
PGA	0.029	0.057	0.078

References

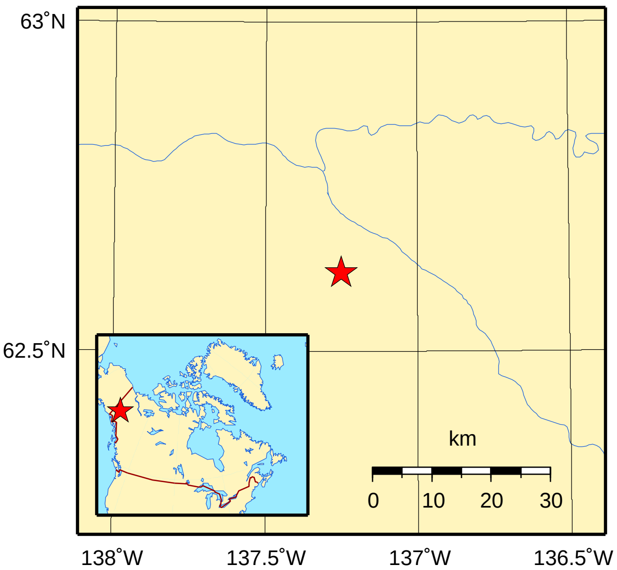
National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

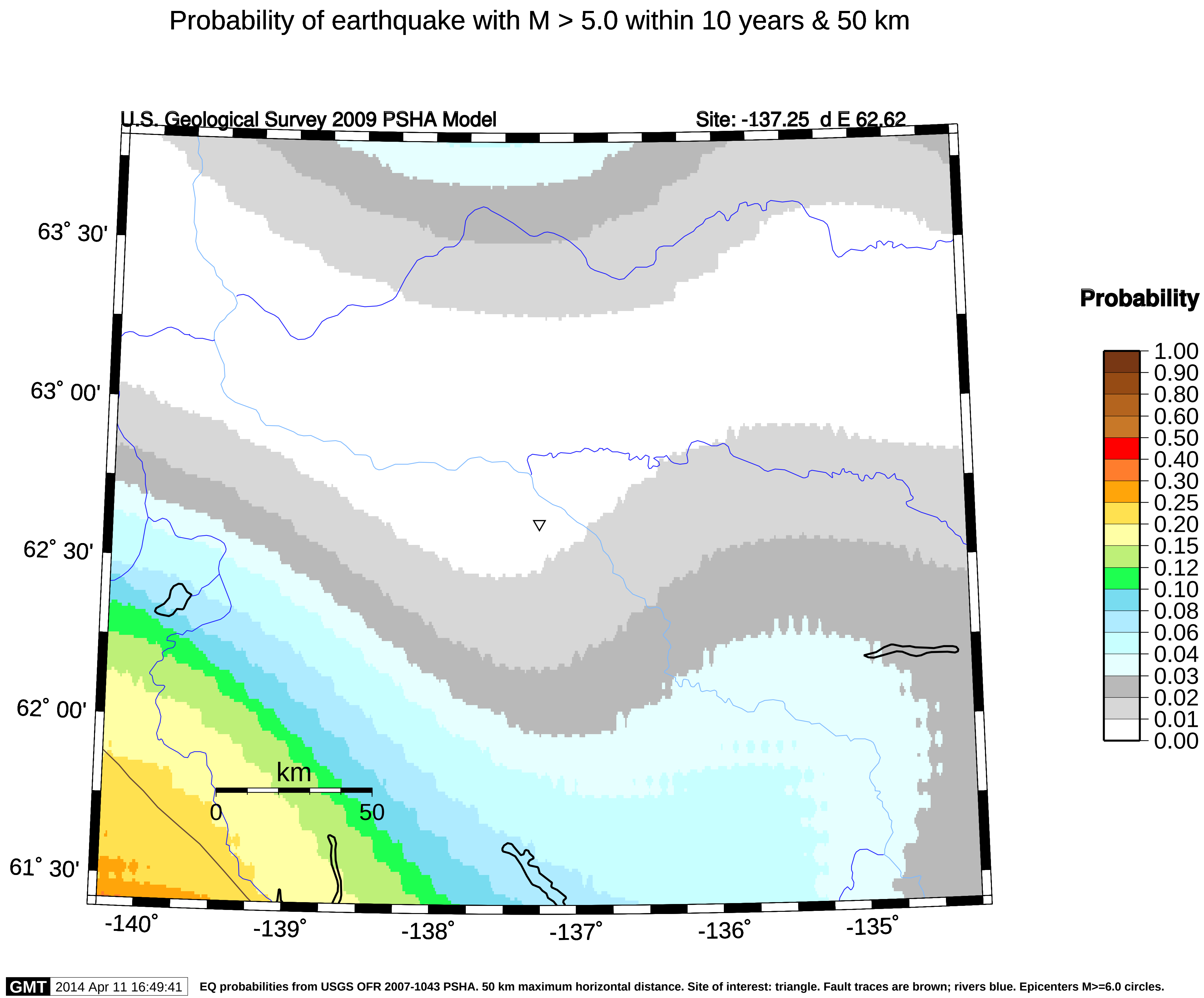
User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

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

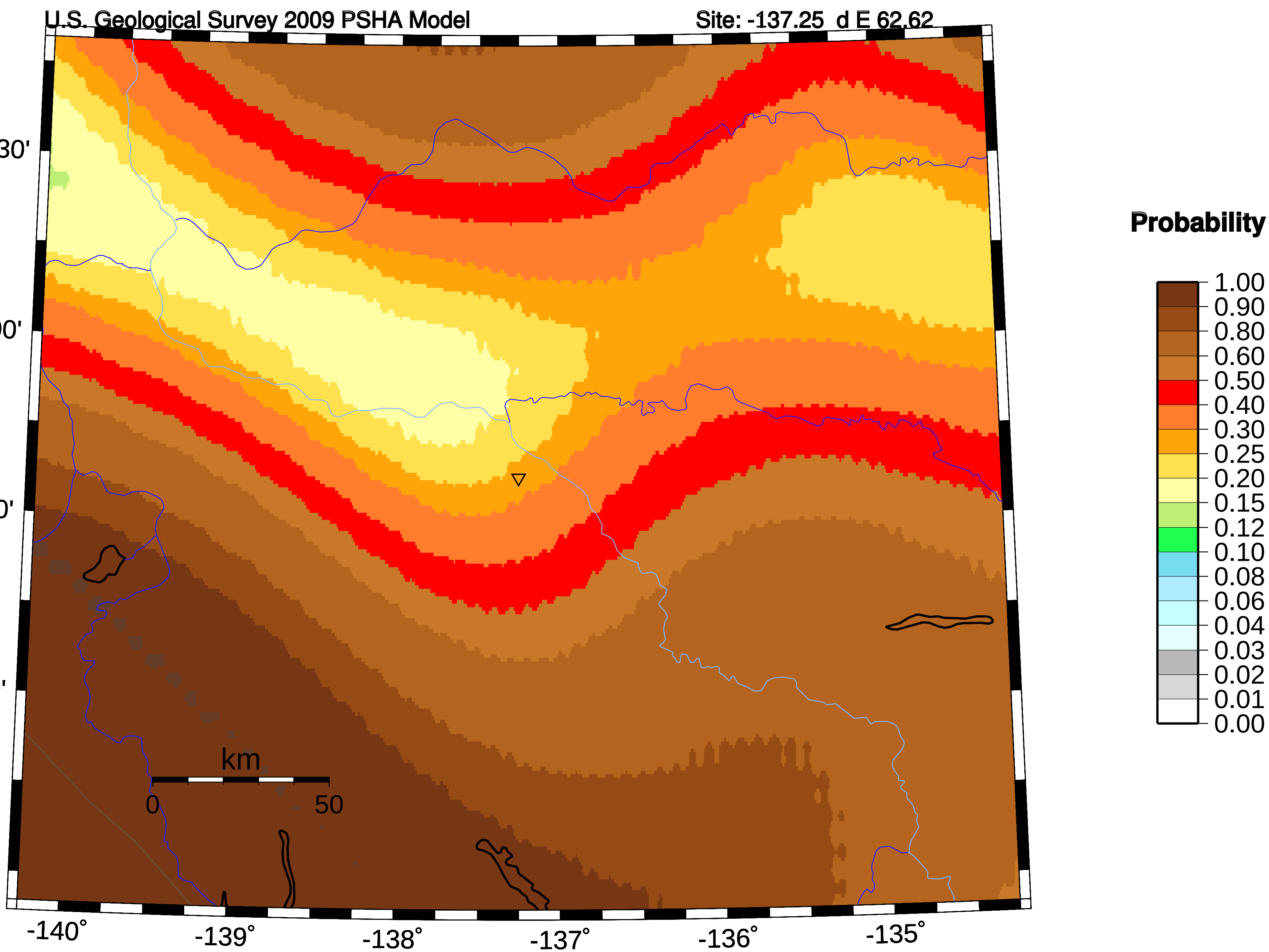
Aussi disponible en français





EQ probabilities from USGS OFR 2007-1043 PSHA. 50 km maximum horizontal distance. Site of interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

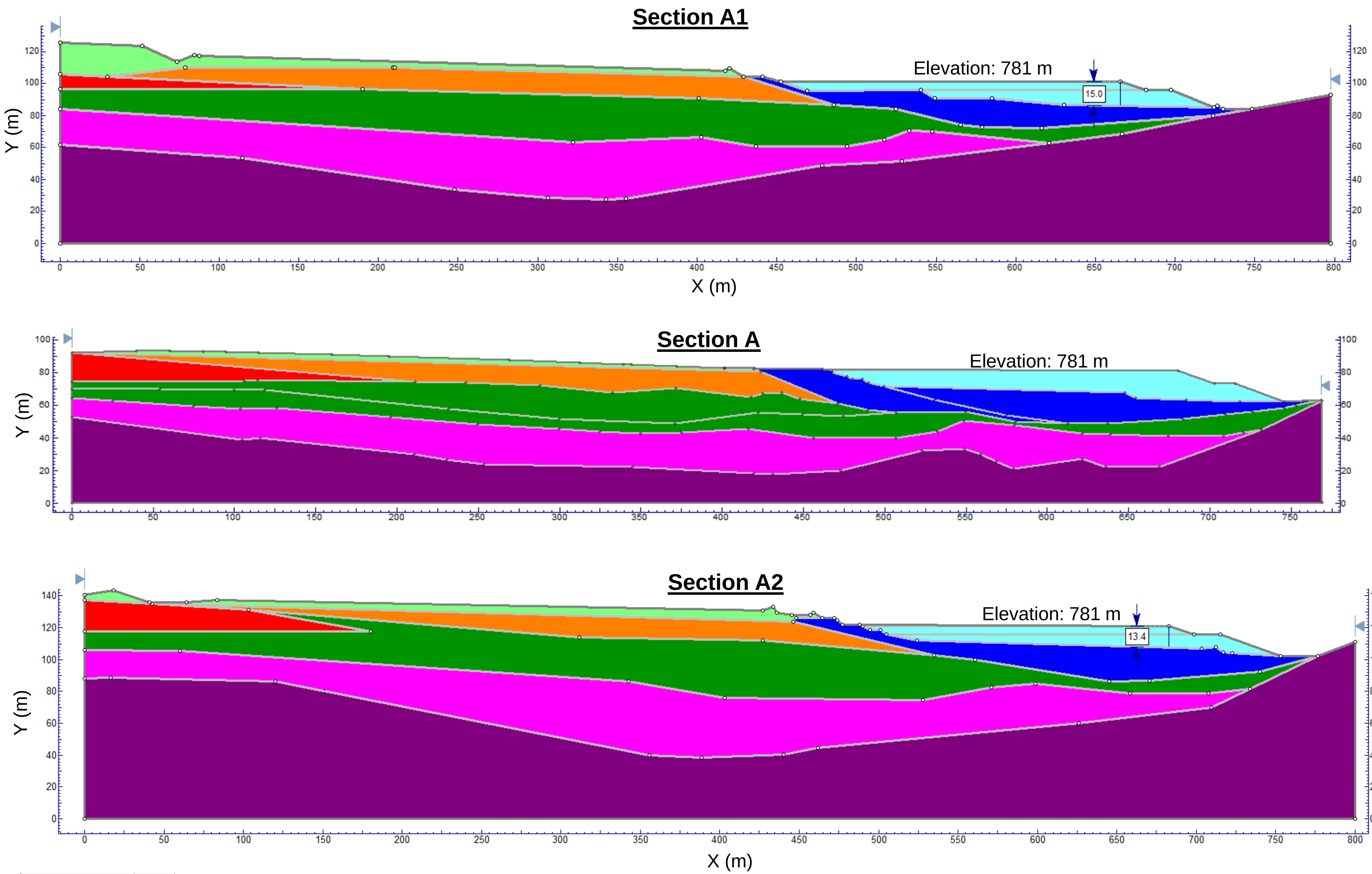
Probability of earthquake with M > 6.0 within 2475 years & 50 km



GMT 2014 Apr 11 16:53:50 EQ probabilities from USGS OFR 2007-1043 PSHA. 50 km maximum horizontal distance. Site of interest: triangle. Fault traces are brown; rivers blue. Epicenters M>=6.0 circles.

Appendix B: Stability Analysis Results

Appendix B-1: Section Geometries



Tailings		Clay (Undrained)	
Mixed OVB		Stage 2 Fill	
Top OVB		Bedrock	
Clay (Drained)		Existing Fill	

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MVFE Stage 2 Stability Analysis

Sections A1, A, and A2

Job No: 1CM002.040

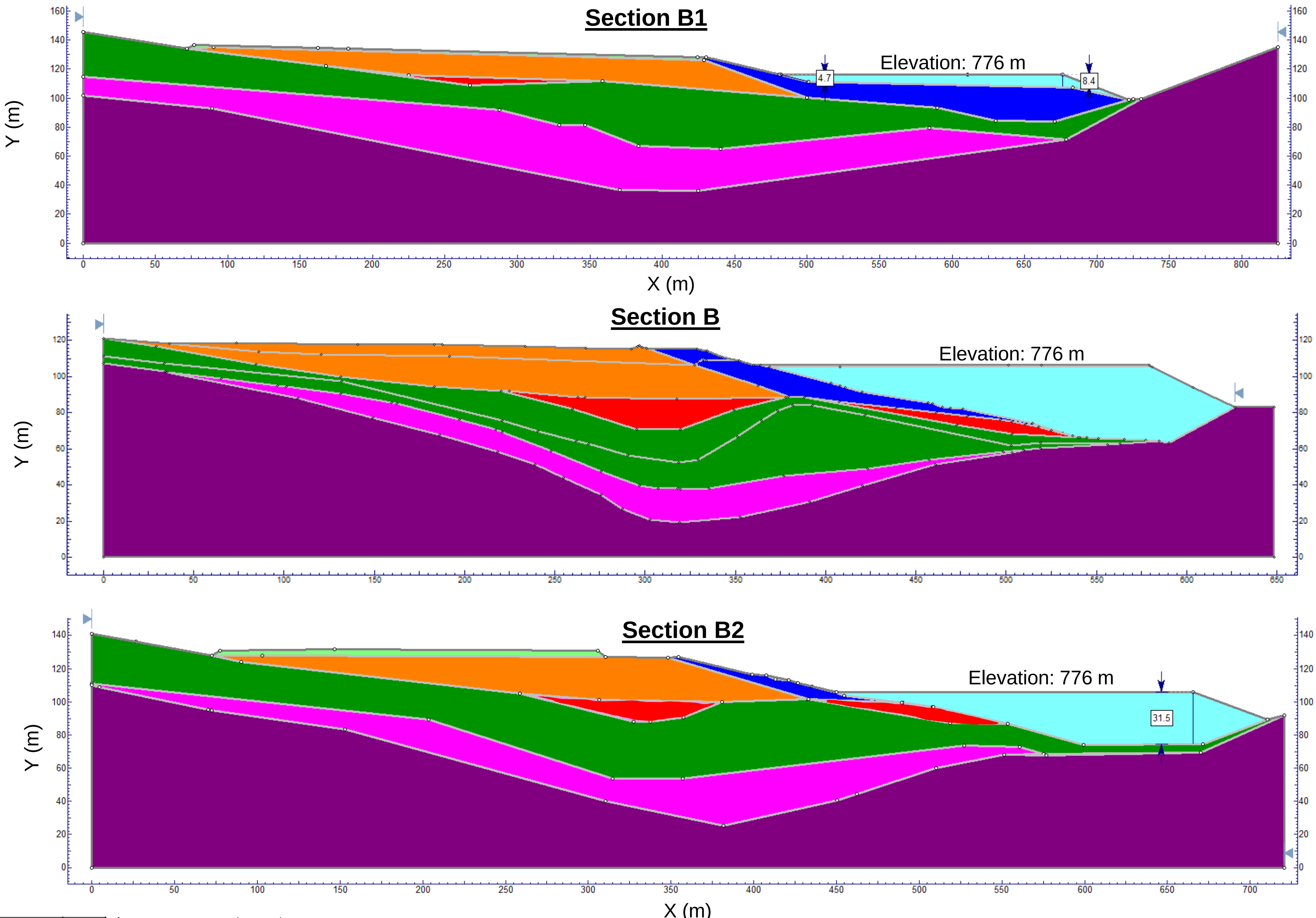
Filename: AppB1_GeometrySections.pptx

Minto Mine

Date: Sept 2015

Prepared by: KK

Figure: 1



Tailings	Clay (Undrained)
Mixed OVB	Stage 2 Fill
Top OVB	Bedrock
Clay (Drained)	Existing Fill

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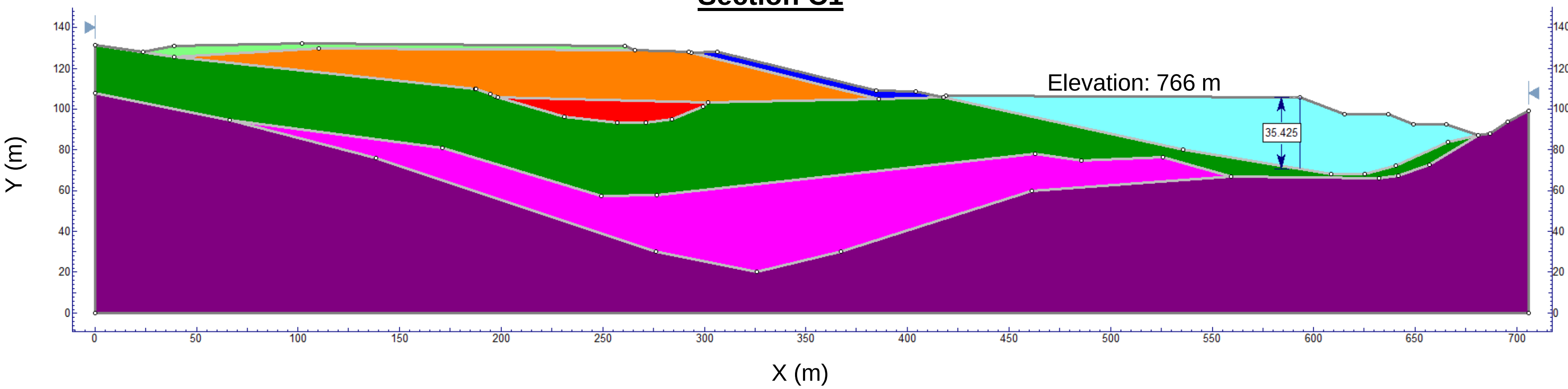
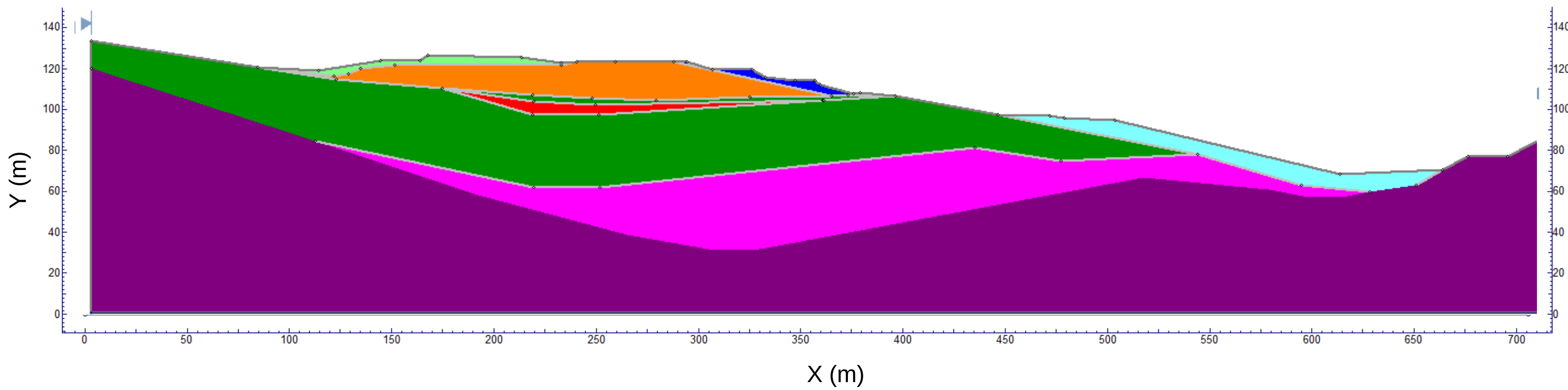
		Sections B1, B, and B2
Job No: 1CM002.040 Filename: AppB1_GeometrySections.pptx	Minto Mine	Date: Sept 2015 · Prepared by: KK · Figure: 2

Section C1



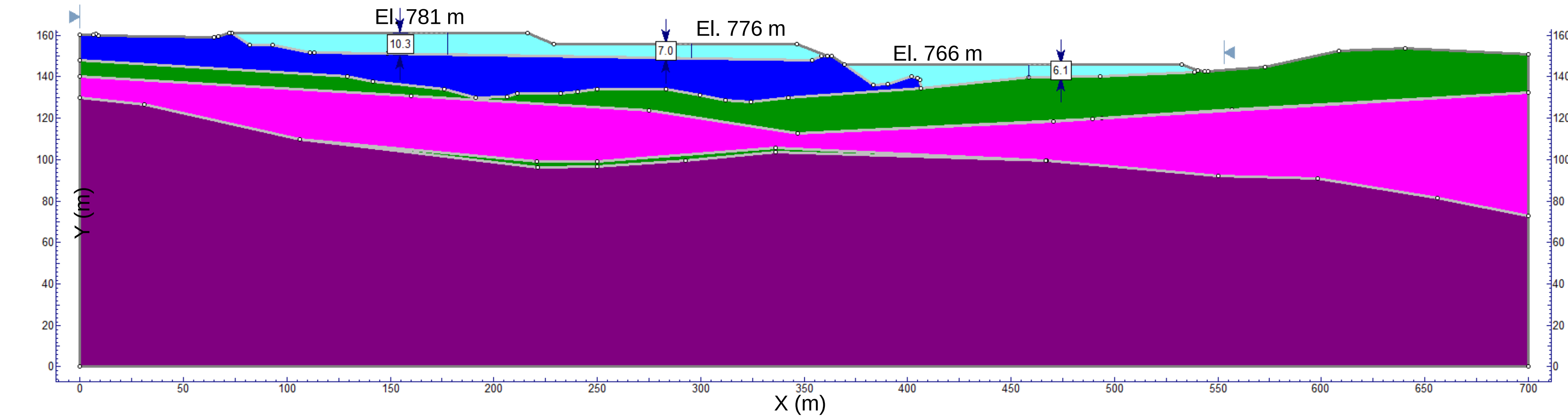
Section C



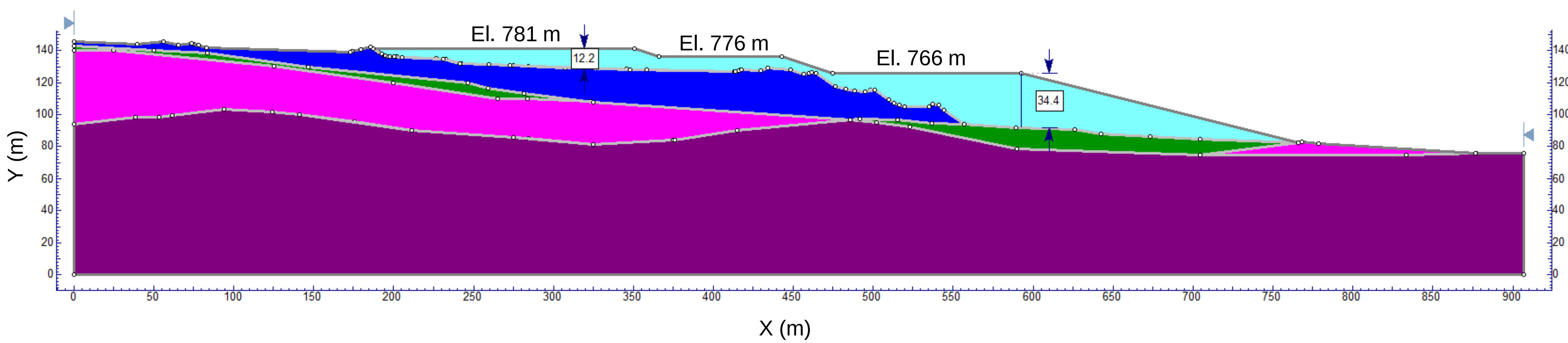
Tailings		Clay (Undrained)	
Mixed OVB		Stage 2 Fill	
Top OVB		Bedrock	
Clay (Drained)		Existing Fill	

srk consulting	capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.	MVFE Stage 2 Stability Analysis
		Sections C1 and C
Job No: 1CM002.040 Filename: AppB1_GeometrySections.pptx	Minto Mine	Date: Sept 2015 / Prepared by: KK / Figure: 3

Section D



Section E



Tailings		Clay (Undrained)	
Mixed OVB		Stage 2 Fill	
Top OVB		Bedrock	
Clay (Drained)		Existing Fill	

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MVFE Stage 2 Stability Analysis

Sections D and E

Job No: 1CM002.040

Filename: AppB1_GeometrySections.pptx

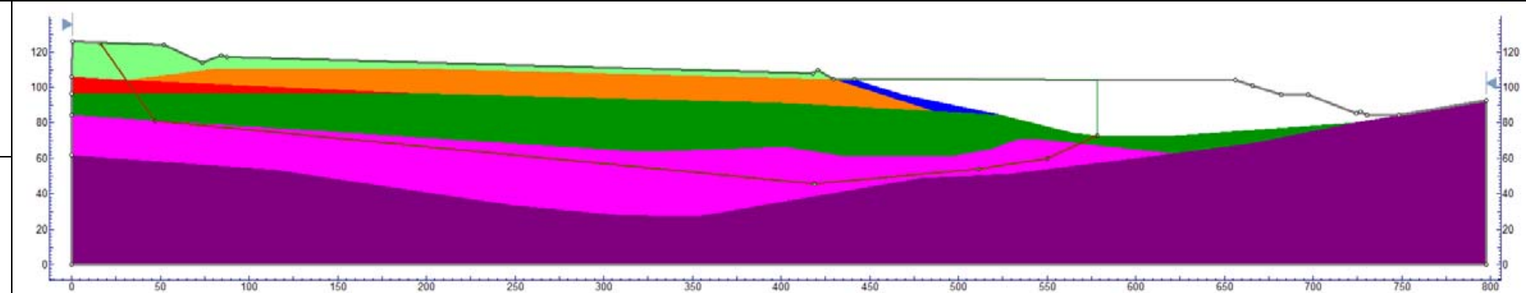
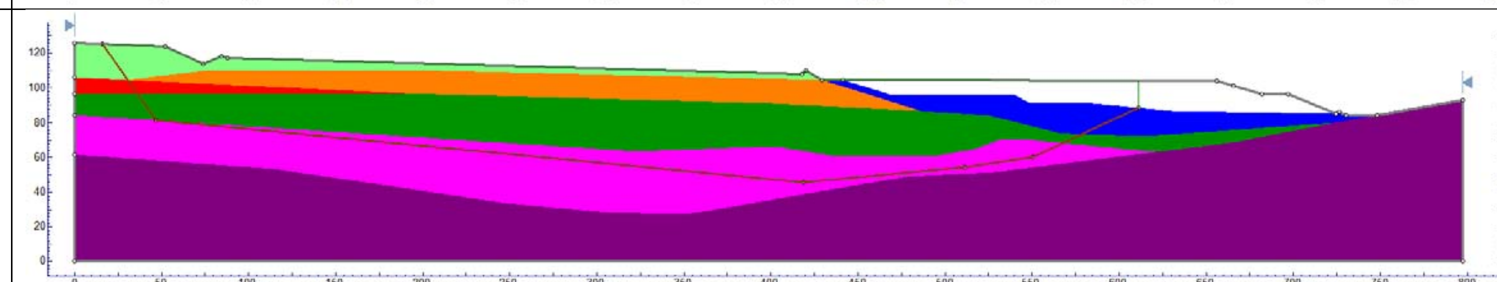
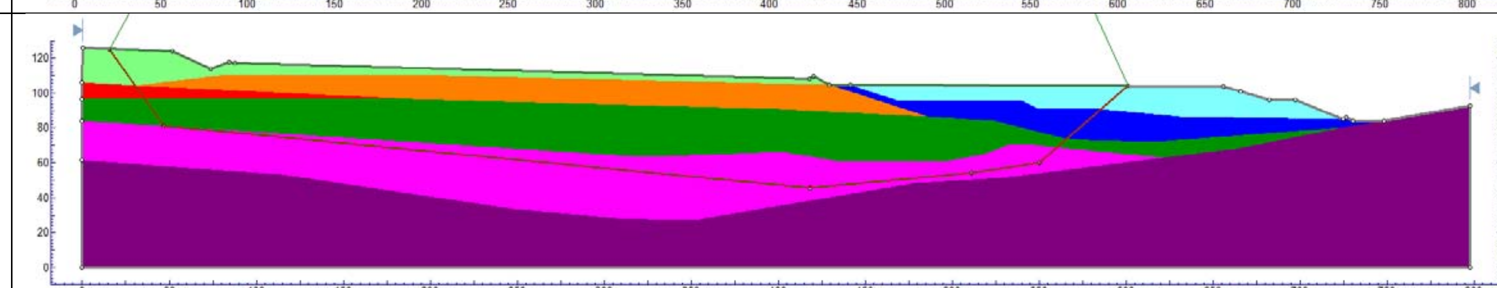
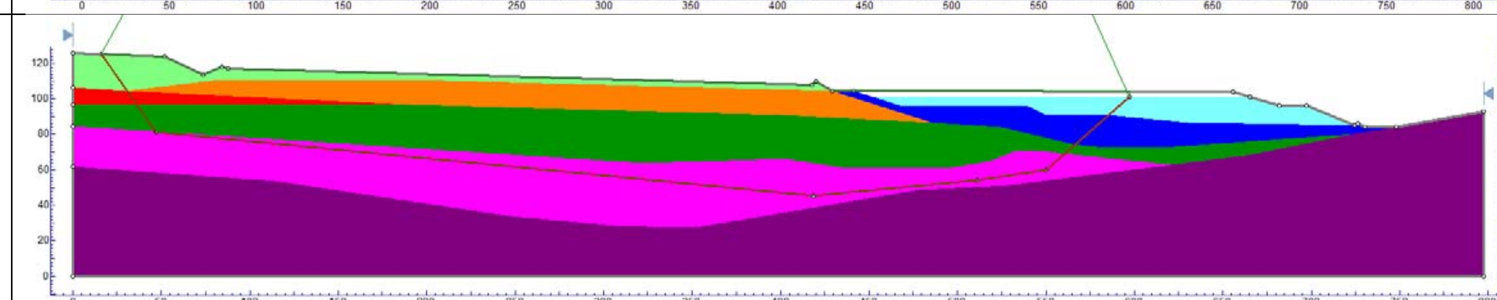
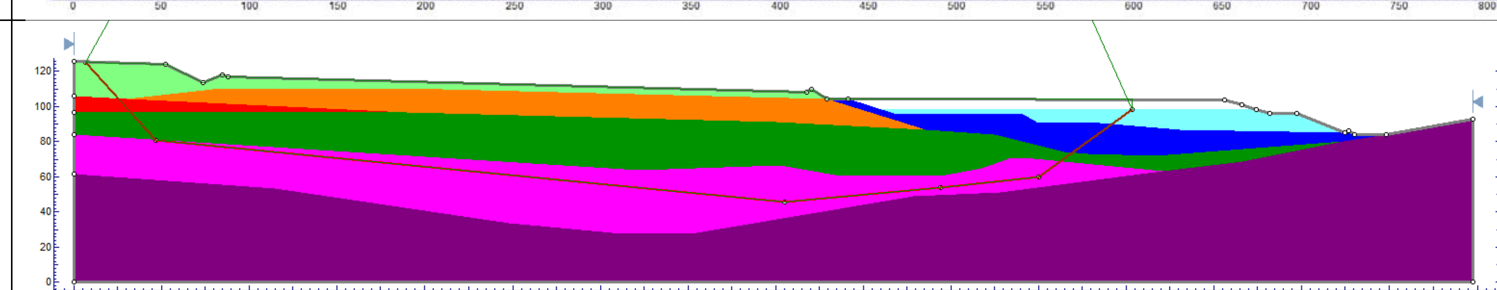
Minto Mine

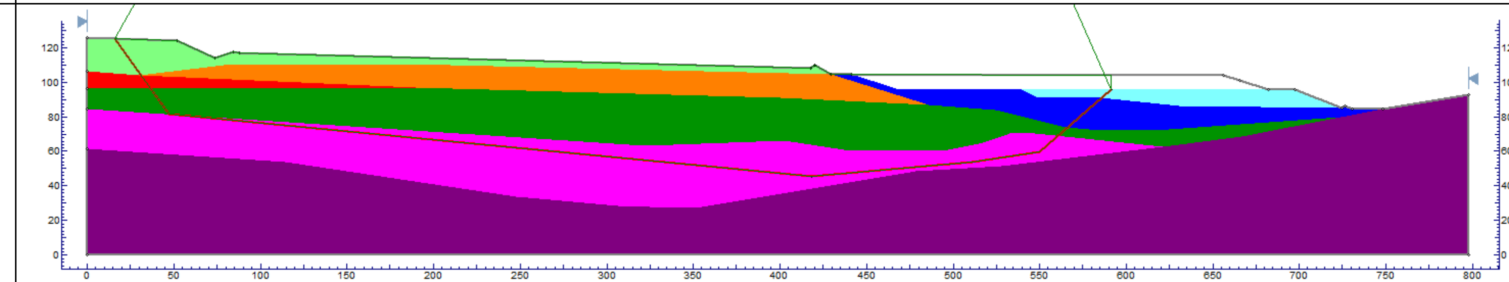
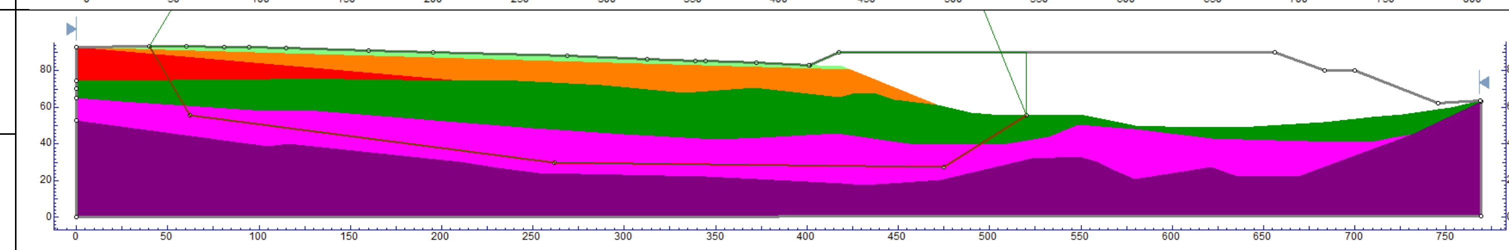
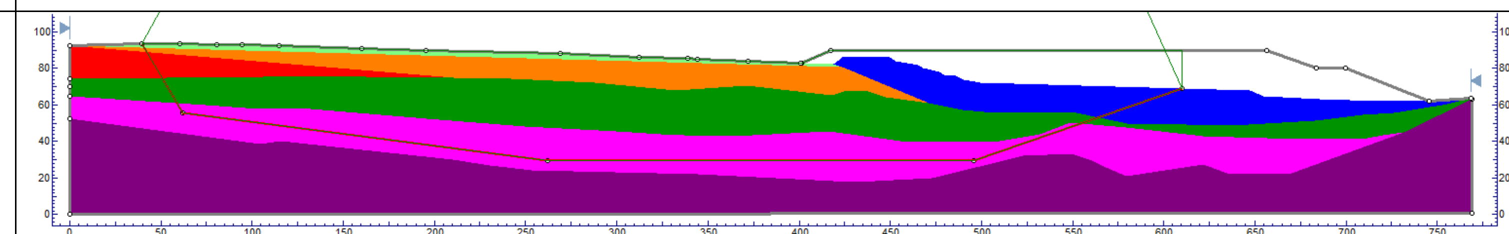
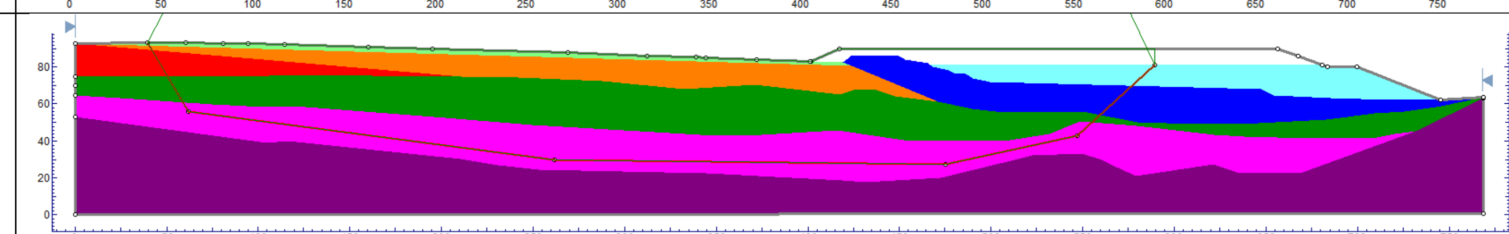
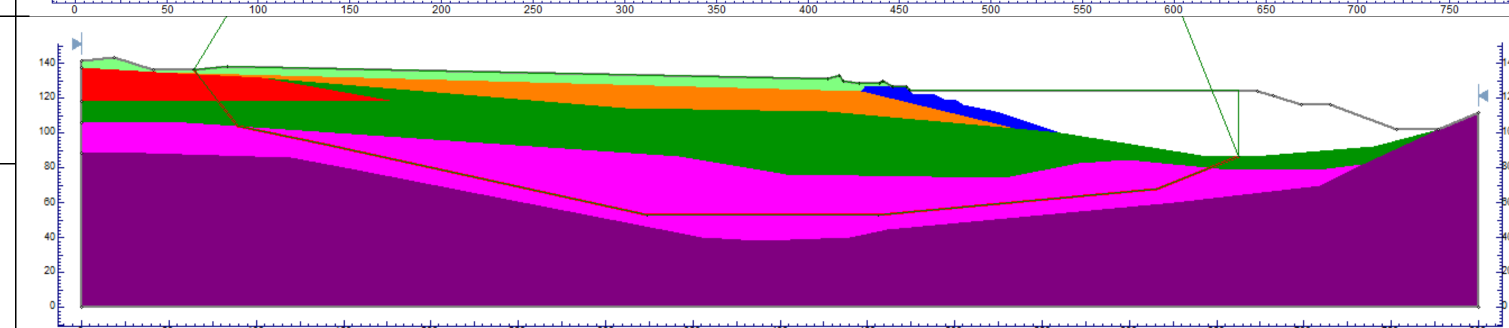
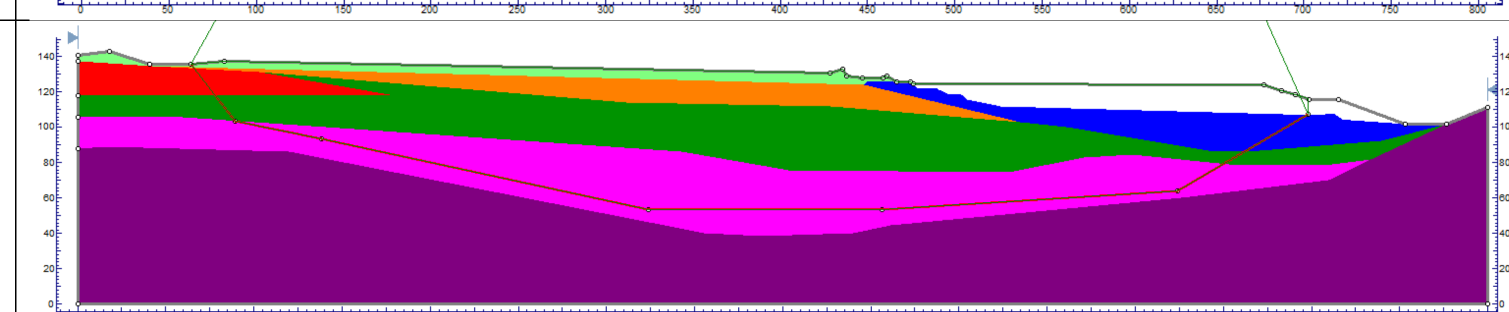
Date:	Prepared by:	Figure:
Sept 2015	KK	4

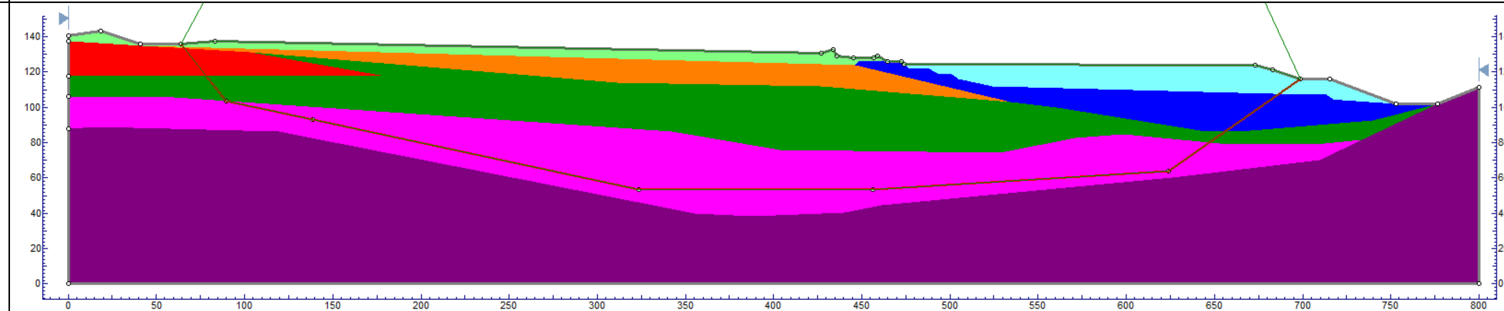
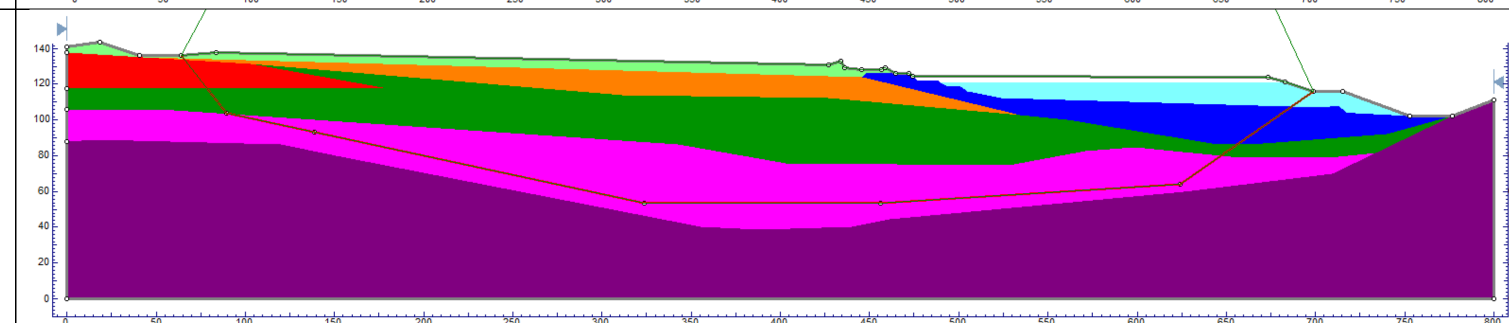
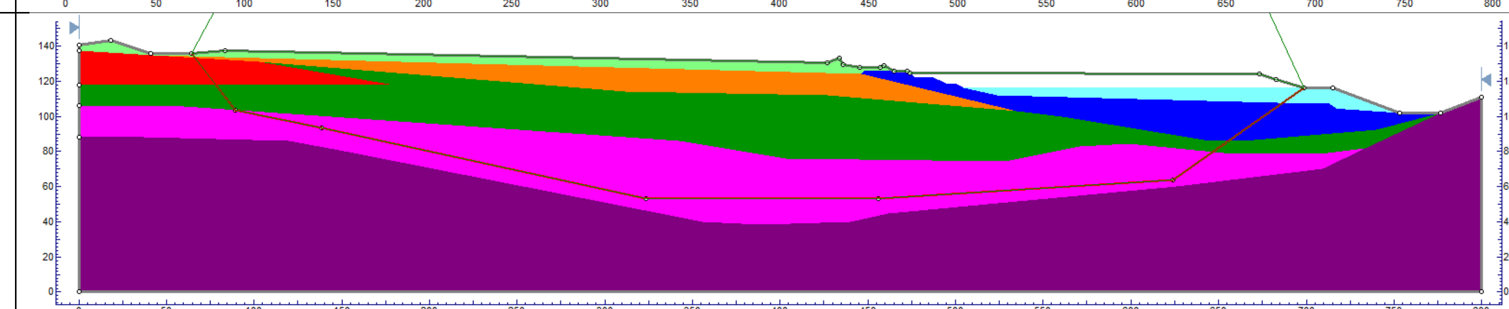
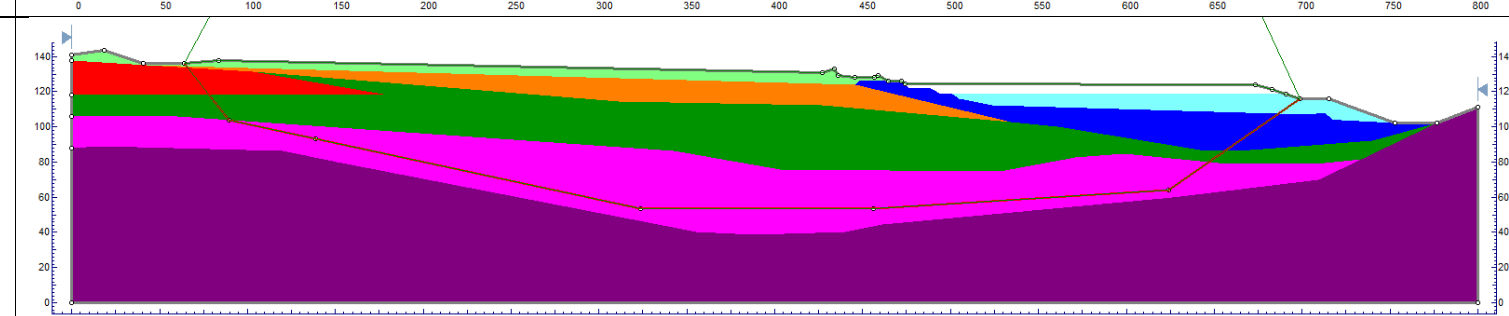
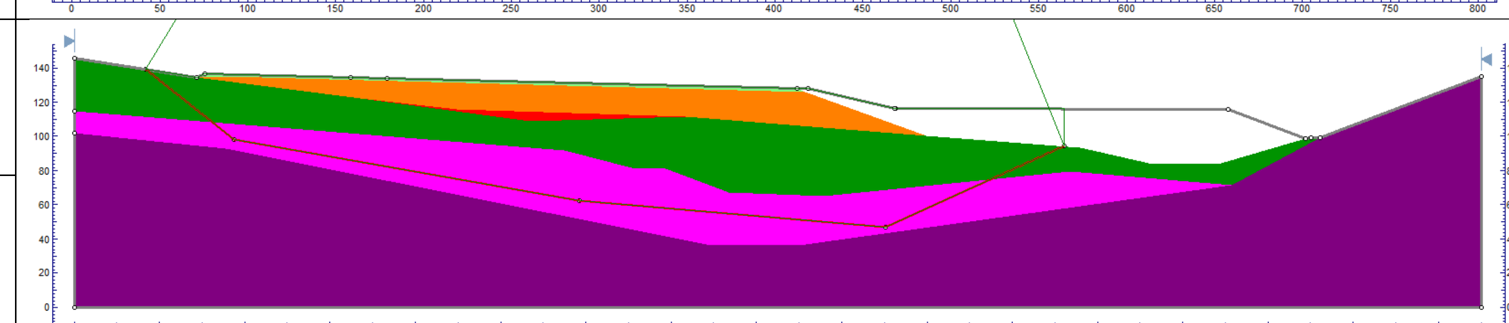
Appendix B-2: Limit Equilibrium Analysis Results

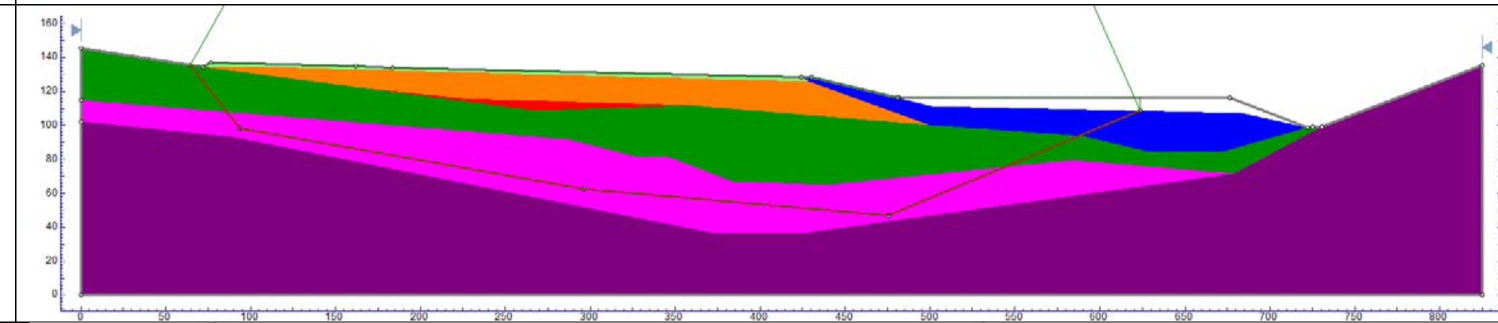
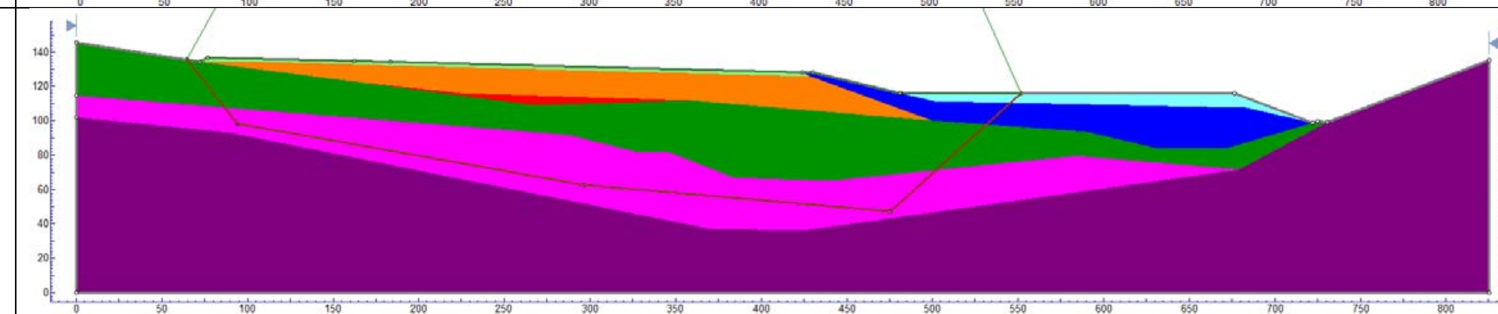
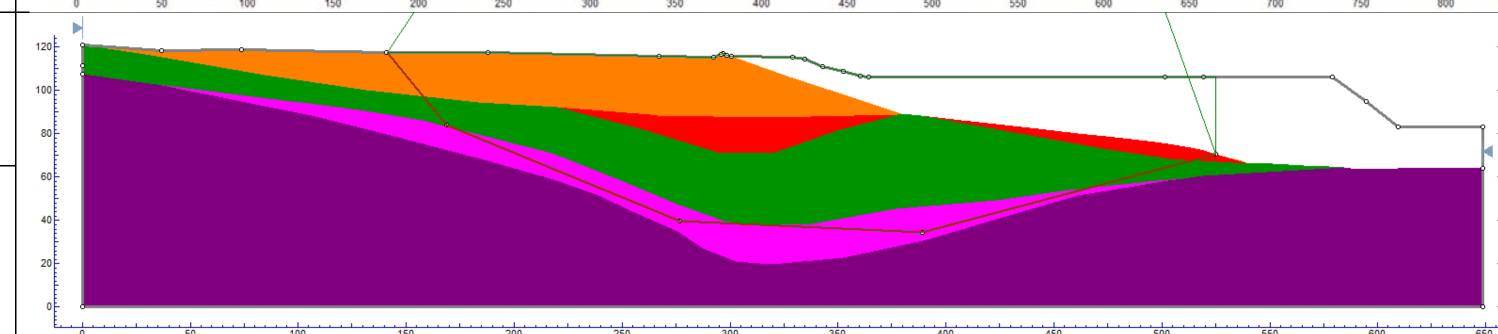
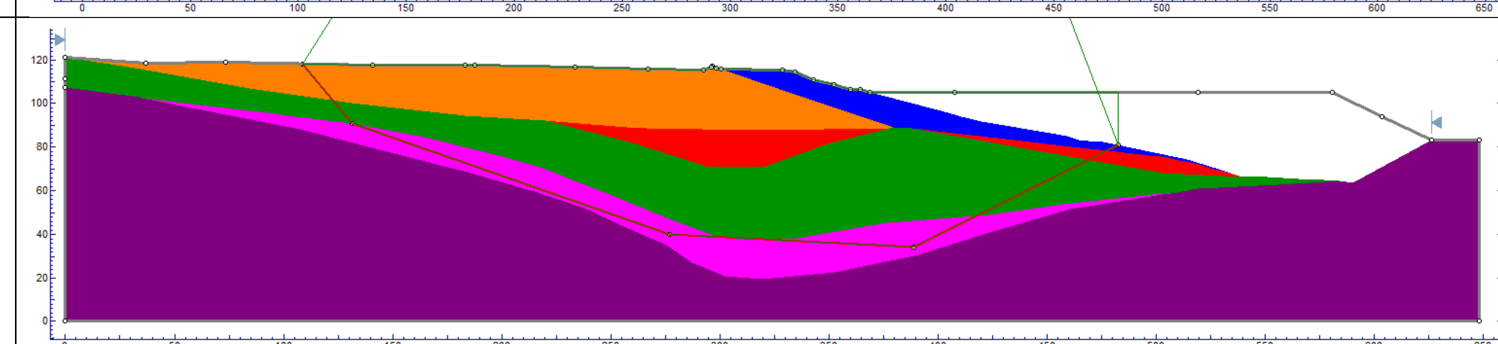
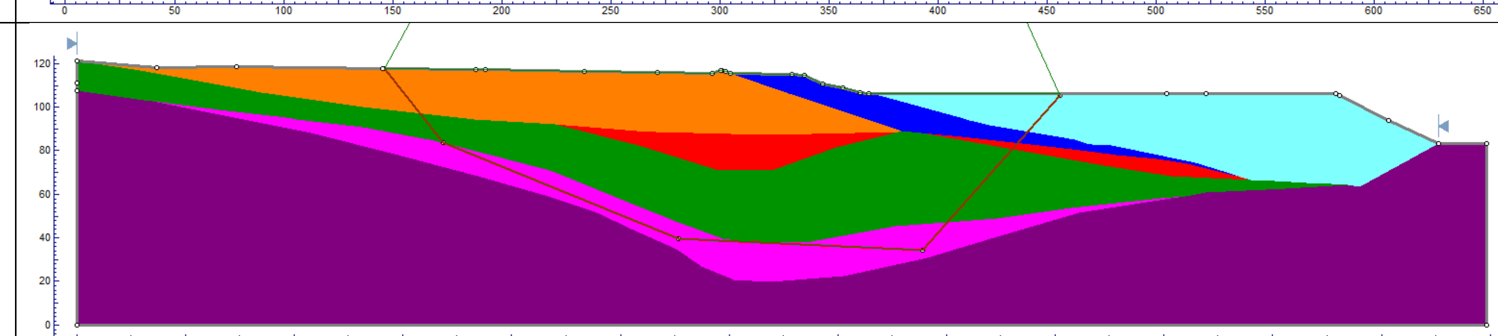
NOTE: Section ID's have been updated since the Preliminary Design Report completed in June 2014. The stability results are unchanged.

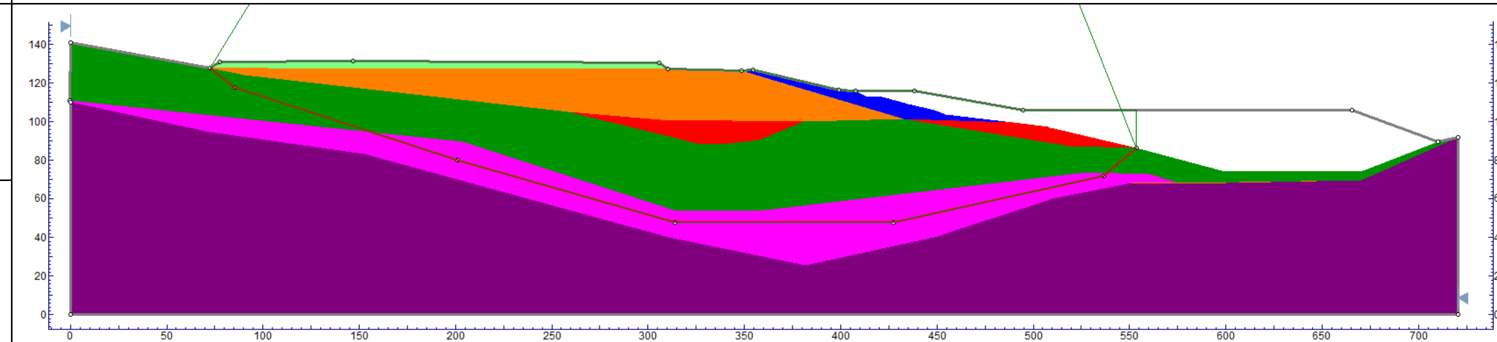
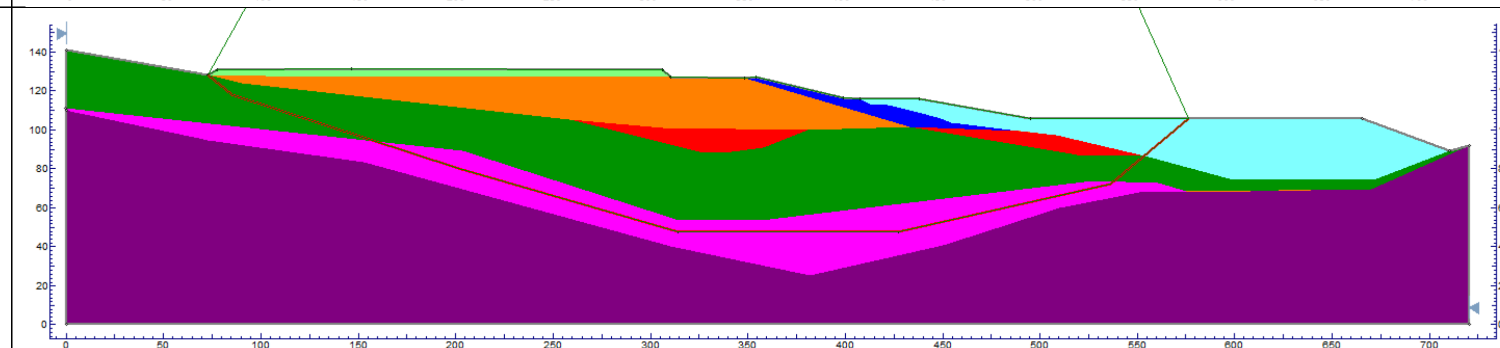
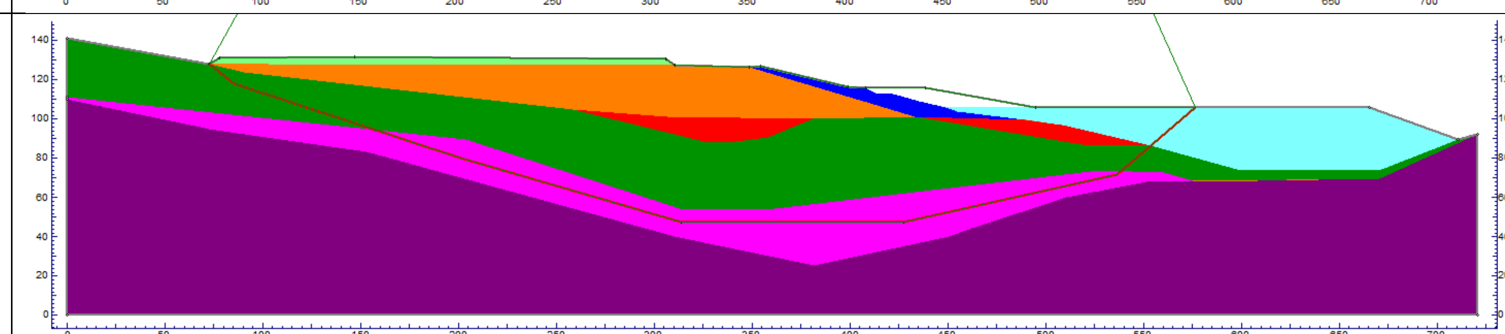
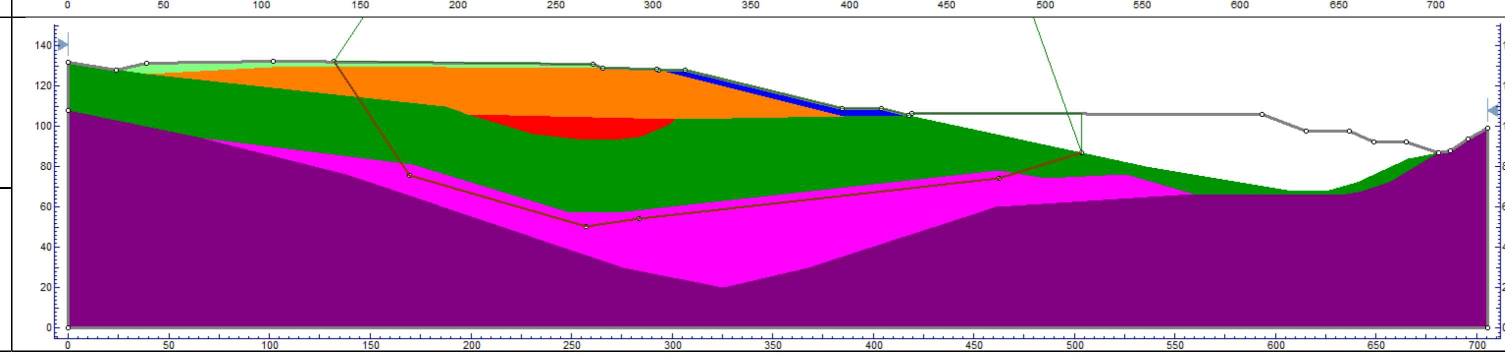
Part 1 – Cross Valley Stability Results

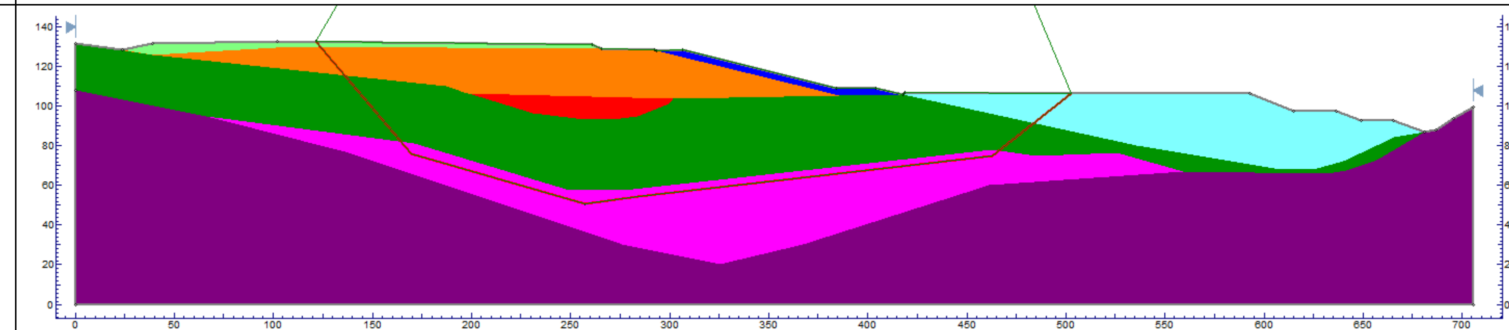
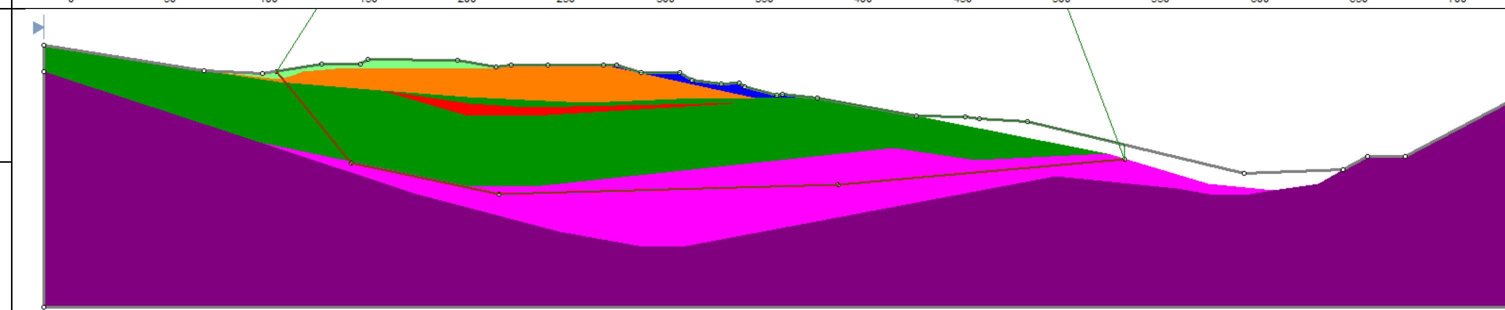
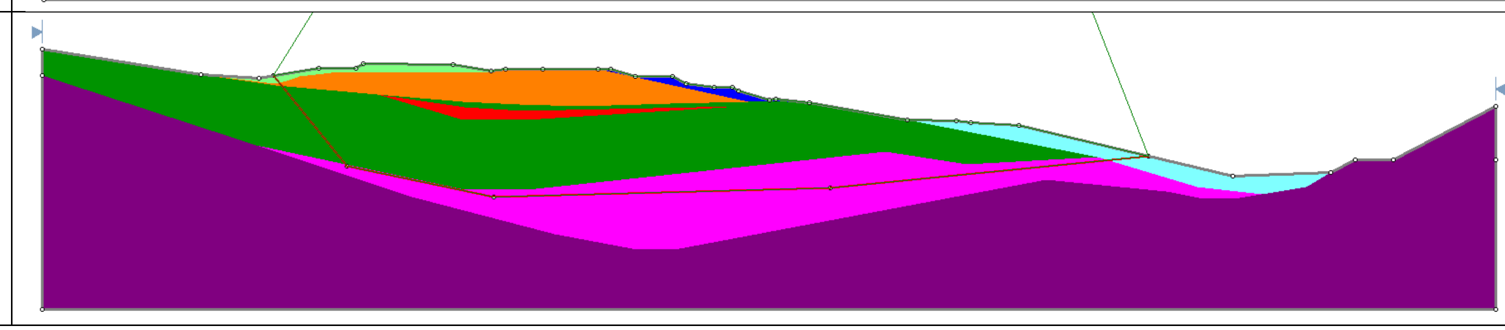
Run	Section	Model ID	Conditions	MVFES2 El. (m)	c_u (kPa)	FOS	Increase in FOS	Failure Mechanism
1	A1	A1_NoButtress	With No Buttress	-	15	0.44	-	
2	A1	A1_NoButtress_Cu50	With No Buttress	-	50	1.04	-	
3	A1	A1_Existing_Cu50	Current (With MVFE Stage 1)	-	50	1.88	-	
4	A1	A1_Stage2_Cu50	MVFE Stage 2	784	50	4.43	136%	
5	A1	A1_Stage2_03_Cu50	MVFE Stage 2	781	50	3.36	79%	
6	A1	A1_Stage2_07_Cu50	MVFE Stage 2	778.5	50	2.79	48%	

Run	Section	Model ID	Conditions	MVFES2 El. (m)	c_u (kPa)	FOS	Increase in FOS	Failure Mechanism
7	A1	A1_Stage2_05_Cu50	MVFE Stage 2	776	50	2.39	27%	
8	A	SectionA_NoButtress	With No Buttress	-	15	0.66	-	
9	A	SectionA_NoButtress_Cu40	With No Buttress	-	40	1.02	-	
10	A	SectionA_Existing_Cu40	Current (With MVFE Stage 1)	781	40	2.10	-	
11	A	SectionA_Stage2_Cu40	MVFE Stage 2	781	40	5.22	149%	
12	A2	A2_NoButtress	With No Buttress	-	15	0.33	-	
13	A2	A2_NoButtress_Cu65	With No Buttress	-	65	1.00	-	
14	A2	A2_Existing_Cu65	Current (With MVFE Stage 1)	-	65	2.07	-	

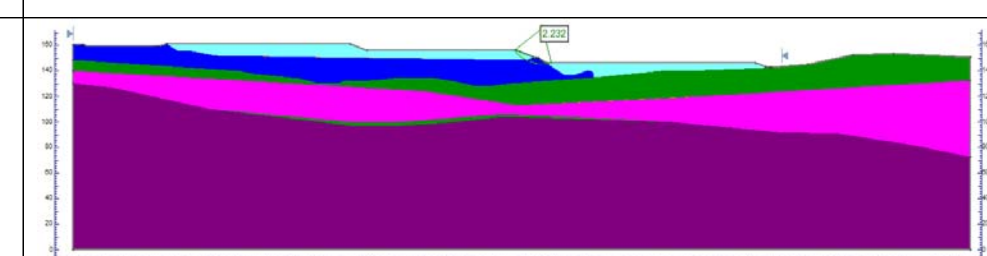
Run	Section	Model ID	Conditions	MVFES2 El. (m)	c_u (kPa)	FOS	Increase in FOS	Failure Mechanism
15	A2	A2_Stage2_Cu65	MVFE Stage 2	784	65	5.77	179%	
16	A2	A2_Stage2_02_Cu65	MVFE Stage 2	781	65	4.60	122%	
17	A2	A2_Stage2_04_Cu65	MVFE Stage 2	776	65	3.23	56%	
18	A2	A2_Stage2_06_Cu65	MVFE Stage 2	778.5	65	3.86	86%	
19	B1	B1_NoButtress	With No Buttress	-	15	0.59	-	
20	B1	B1_NoButtress_Cu60	With No Buttress	-	60	1.05	-	

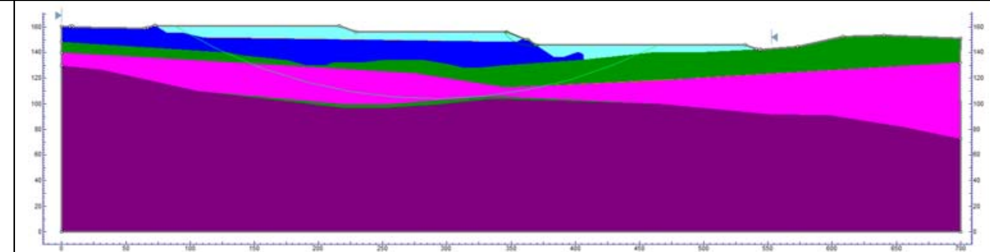
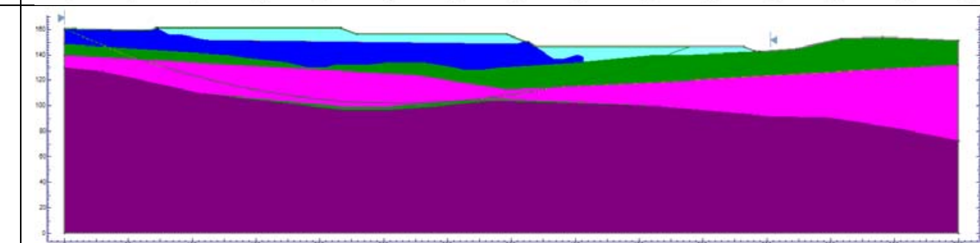
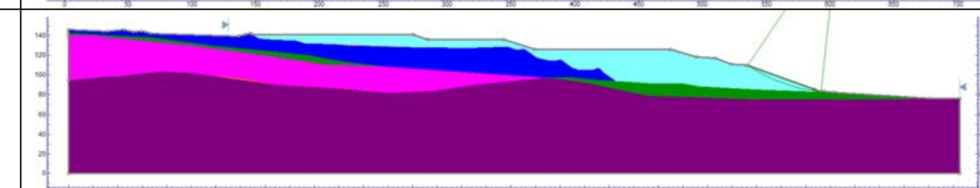
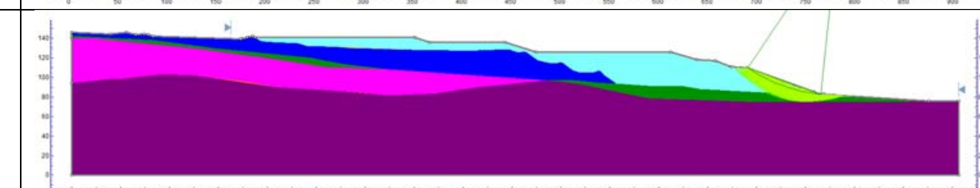
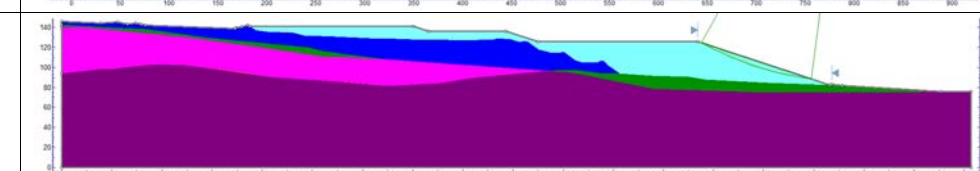
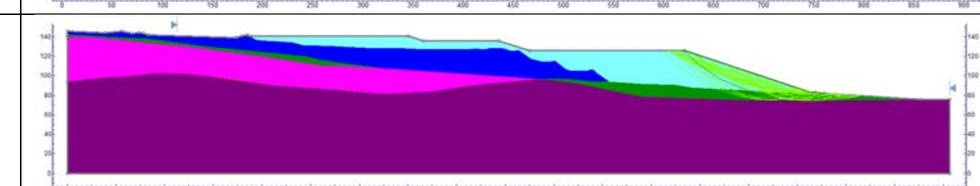
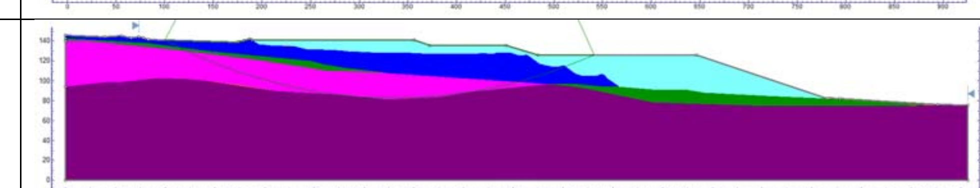
Run	Section	Model ID	Conditions	MVFES2 El. (m)	c_u (kPa)	FOS	Increase in FOS	Failure Mechanism
21	B1	B1_Existing_Cu60	Current (With MVFE Stage 1)	-	60	2.11	-	
22	B1	B1_Stage2_Cu60	MVFE Stage 2	776	60	3.08	46%	
23	B	SectionB_NoButtress	With No Buttress	-	15	0.58	-	
24	B	SectionB_NoButtress_Cu70	With No Buttress	-	70	1.02	-	
25	B	SectionB_Existing_Cu70	Current (With MVFE Stage 1)	-	70	1.63	-	
26	B	SectionB_Stage2_Cu70	MVFE Stage 2	776	70	6.22	282%	

Run	Section	Model ID	Conditions	MVFES2 El. (m)	c_u (kPa)	FOS	Increase in FOS	Failure Mechanism
27	B2	B2_Existing	Current (with No Buttress)	-	15	0.75	-	
28	B2	B2_Existing_Cu45	Current (with No Buttress)	-	45	1.02	-	
29	B2	B2_Stage2_02_Cu45	MVFE Stage 2	776	45	2.07	103%	
30	B2	B2_Stage2_Cu45	MVFE Stage 2	766	45	1.96	92%	
31	C1	C1_Existing	Current (with No Buttress)	-	15	0.70	-	
32	C1	C1_Existing_Cu40	Current (with No Buttress)	-	40	1.00	-	

Run	Section	Model ID	Conditions	MVFES2 El. (m)	c_u (kPa)	FOS	Increase in FOS	Failure Mechanism
33	C1	C1_Stage2_Cu40	MVFE Stage 2	766	40	1.91	91%	
34	C	SectionC_Existing	Current (with No Buttress)	-	15	0.00	-	
35	C	SectionC_Existing_Cu35	Current (with No Buttress)	-	35	1.00	-	
36	C	SectionC_Stage2_Cu35	MVFE Stage 2	N/A	35	1.14	14%	

Part 2 – Down Valley Stability Results

Run	Slide Model ID	Section	Stability Condition (Dump Surface/Deep Seated Stability)	Stage (Construction/ Reclamation)	c_u (kPa)	Seismic	Suggested Min. Design FOS	FOS	Comment	Failure Mechanism
37	E_Stage2_Cu35	D	Dump Surface	Construction & Reclamation	35	No	1.1	2.23	Shallow slope failure on second tier	

Run	Slide Model ID	Section	Stability Condition (Dump Surface/Deep Seated Stability)	Stage (Construction/ Reclamation)	c_u (kPa)	Seismic	Suggested Min. Design FOS	FOS	Comment	Failure Mechanism
38	E_Stage2_Cu35	D	Deep Seated Stability	Construction & Reclamation	35	No	1.3	3.55	Deep seated failure within undrained clay	
39	E_Stage2_Cu35	D	Deep Seated Stability	Construction & Reclamation	35	0.057g	1.0	1.54	Deep seated failure within undrained clay	
40	F_Stage2_Construction_Cu35	E	Dump Surface	Construction	35	No	1.0	2.13	Shallow slope failure on dump surface towards the toe	
41	F_Stage2_Construction_Cu35	E	Deep Seated Stability	Construction	35	No	1.1 - 1.3	2.30	Toe failure within mixed overburden layer near the toe	
42	F_Stage2_01_3H1V_Cu35_DumpFace	E	Dump Surface	Reclamation	35	No	1.1	2.31	Shallow slope failure	
43	F_Stage2_01_3H1V_Cu35	E	Deep Seated Stability	Reclamation	35	No	1.3	2.5	Deep seated failure within mixed overburden layer near the toe	
44	F_Stage2_01_3H1V_Cu35	E	Deep Seated Stability	Reclamation	35	0.057g	1.0	1.23	Deep seated failure within undrained clay at depth	

Appendix C: Three-dimensional Bedrock Surface Interpretation

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Memo

To:	File	**Client:**	Minto Exploration Ltd.
From:	Kerry Ko	**Project No:**	1CM002.027
Cc:	Iozsef Miskolczi, SRK	**Date:**	May 22, 2014
Subject:	Interpreted Bedrock Surface at the Minto Site		

1 Introduction

A bedrock surface for the Minto Mine, Yukon Territory 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 provide 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 (GEMCOM, 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 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 has been disposed in the pit.

Several areas exhibit bedrock visible from surface. These areas may not 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. Specific areas where the data was manually changed are highlighted in Figure 5.

4 References

Golder Associates, 1976. Geotechnical Investigation Minto Project Feasibility Study. Report submitted to Write Engineers LTD. Report number 1001.04.11.76.001.

EBA Engineering Consultants LTD, 1994. Geotechnical Evaluation Mill and Camp Site. Report submitted to Minto Explorations LTD. Report number 0201-11509.

EBA Engineering Consultants LTD, 1995. Geotechnical Design Tailings/Water Dam. Report submitted to Minto Explorations LTD. Report number 0201-95-11509.

EBA Engineering Consultants LTD, 1996. Geotechnical Drilling Program. . Letter submitted to Minto Explorations LTD. Report number 0201-96-11509.

EBA Engineering Consultants LTD, 1997. Geotechnical Program and Construction Inspection Reports. Report submitted to Minto Explorations LTD. Report number 0201-97-11509.

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ConeTec Investigations LTD., 2010. Field Data Report. Report prepared for EBA Engineering Consultants LTD. Job number 10-207.

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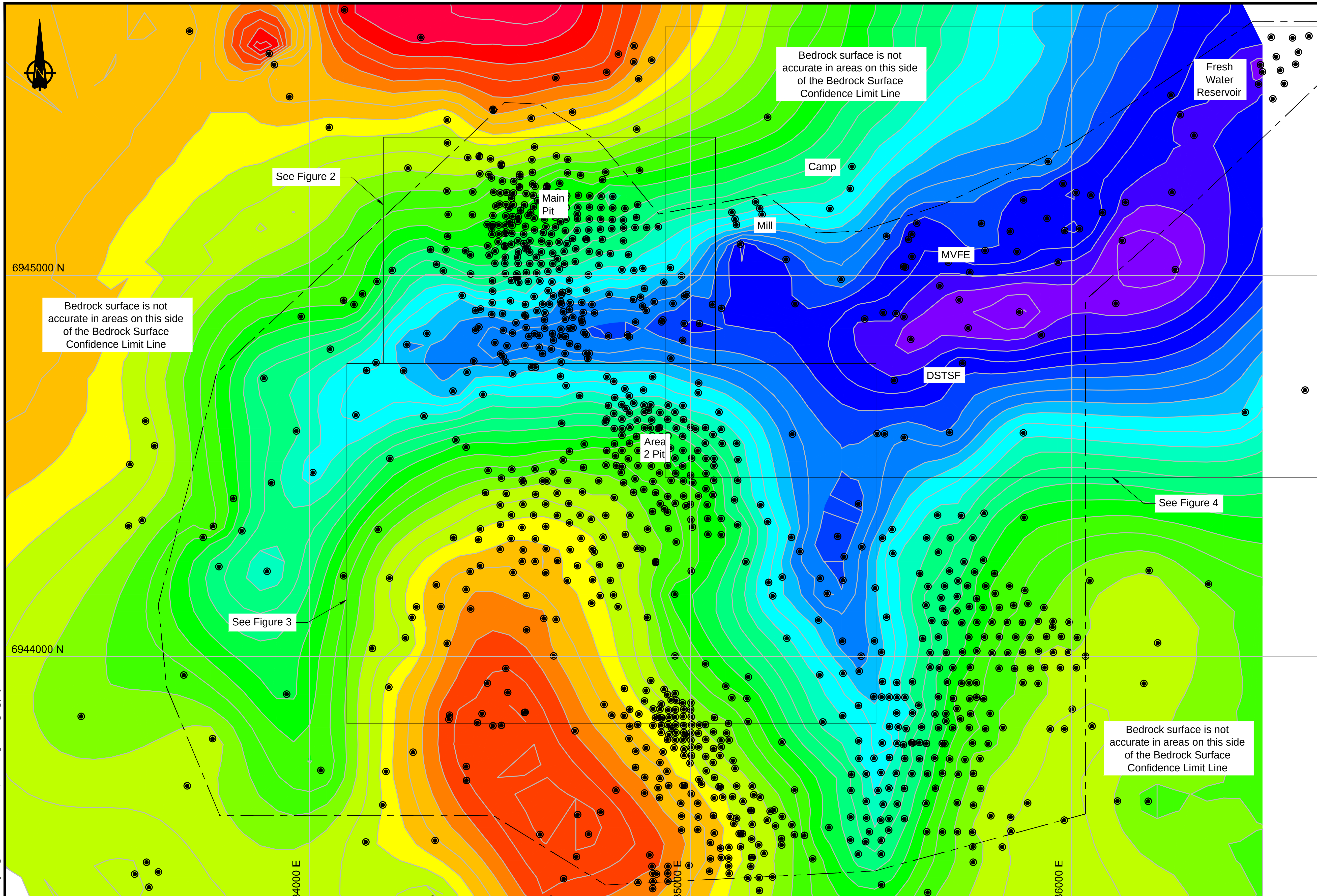
SRK Consulting (Canada) Inc., 2014. Minto Main Dam Phase 1 Field Investigation. Report submitted to Minto Explorations LTD. SRK project number 1CM002.013.

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

capstone MINTO MINE

OPERATED BY MINTO EXPLORATIONS LTD.

Bedrock Surface Review

Bedrock Contour Map

SRK JOB NO.: 1CM002.027

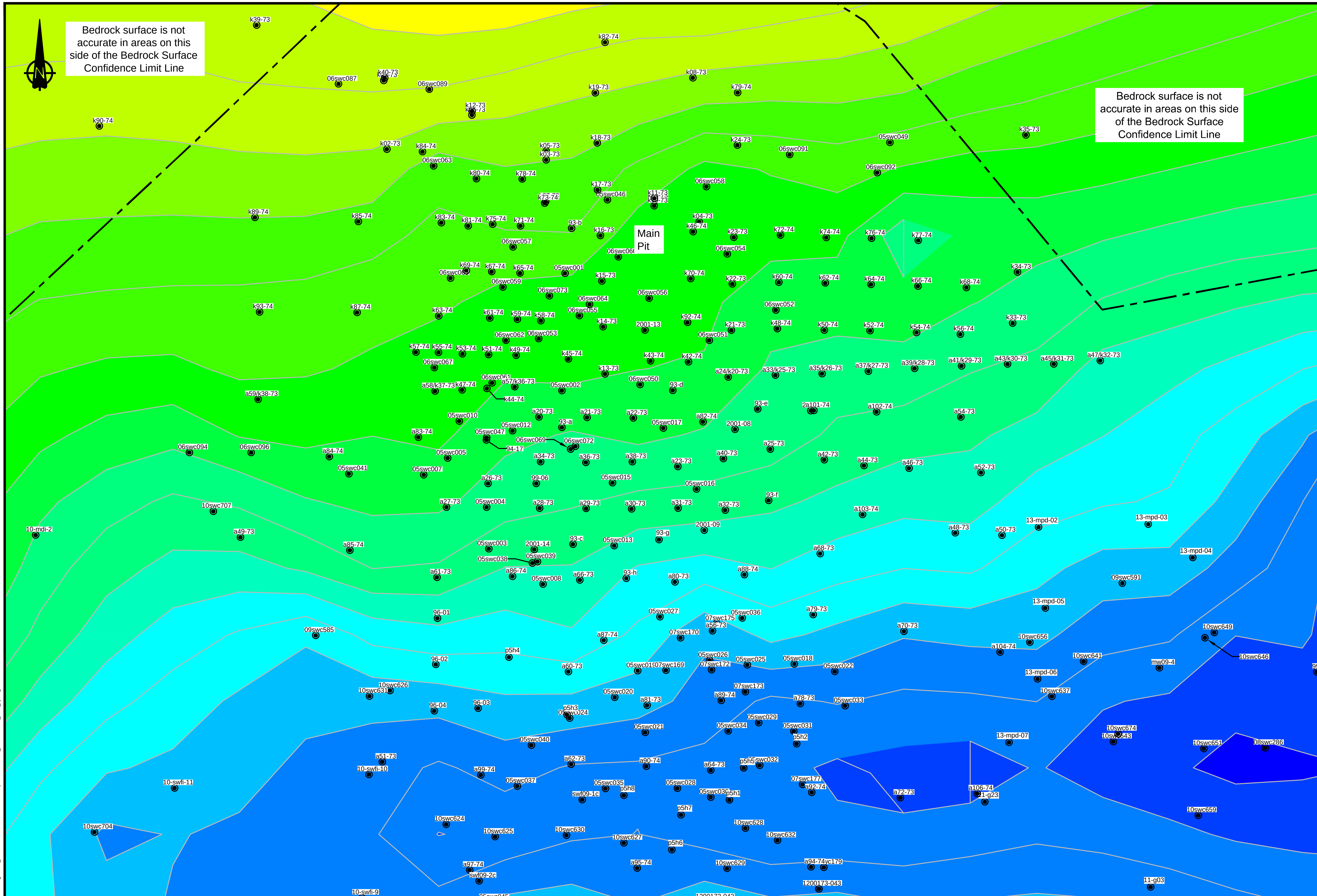
FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

Minto Mine

DATE: May 2014

APPROVED:

FIGURE: 1



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	
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 20 40 60 80 100
Scale in Metres

srk consulting

SRK JOB NO.: 1CM002.027

FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

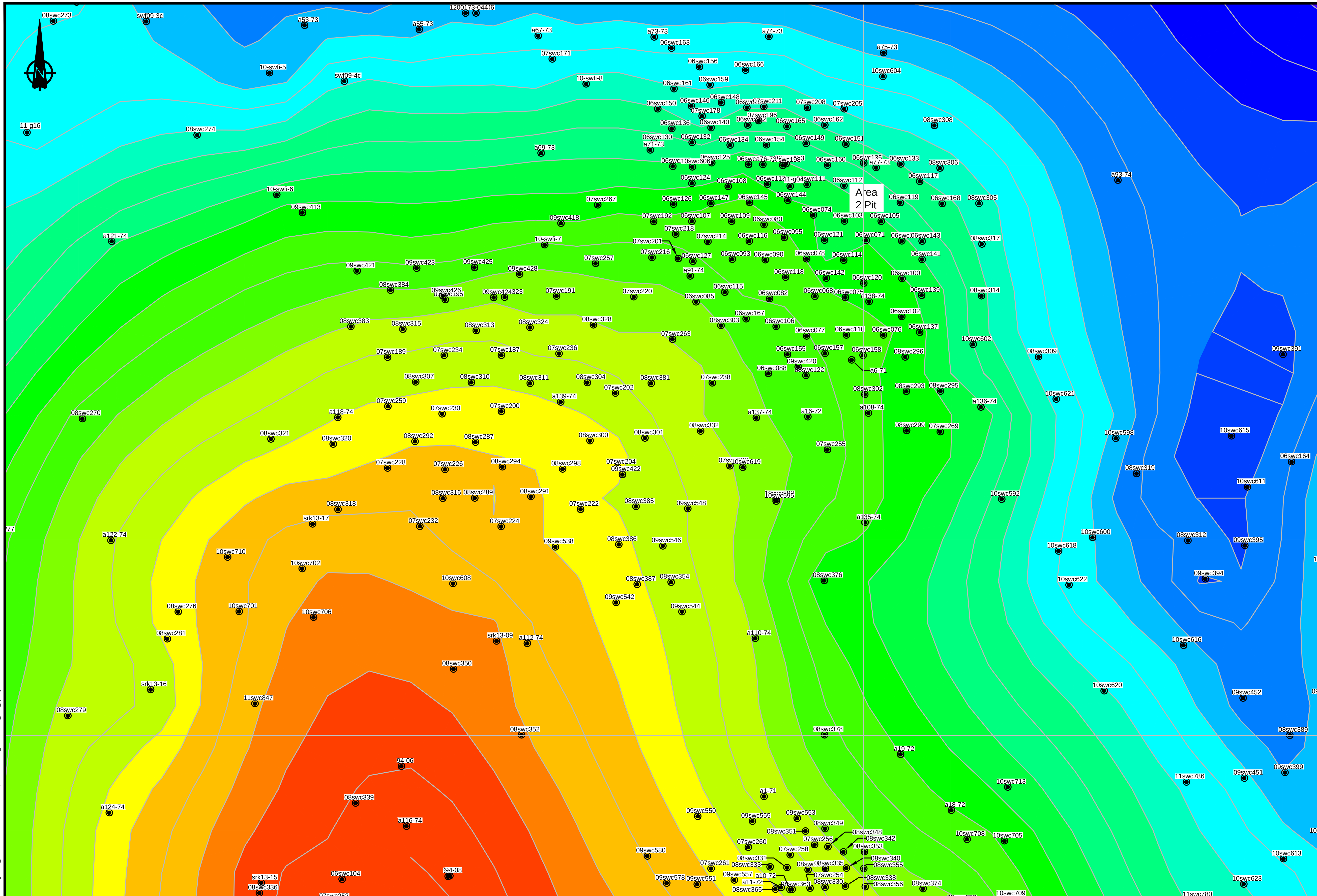
capstone MINTO MINE
OPERATED BY MINTO EXPLORATIONS LTD.

Minto Mine

Bedrock Surface Review

Bedrock Contour Map
Main Pit Area

DATE: May 2014 | APPROVED: | FIGURE: 2



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	
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 40 80 120 160 200

Scale in Metres

srk consulting

SRK JOB NO.: 1CM002.027

FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

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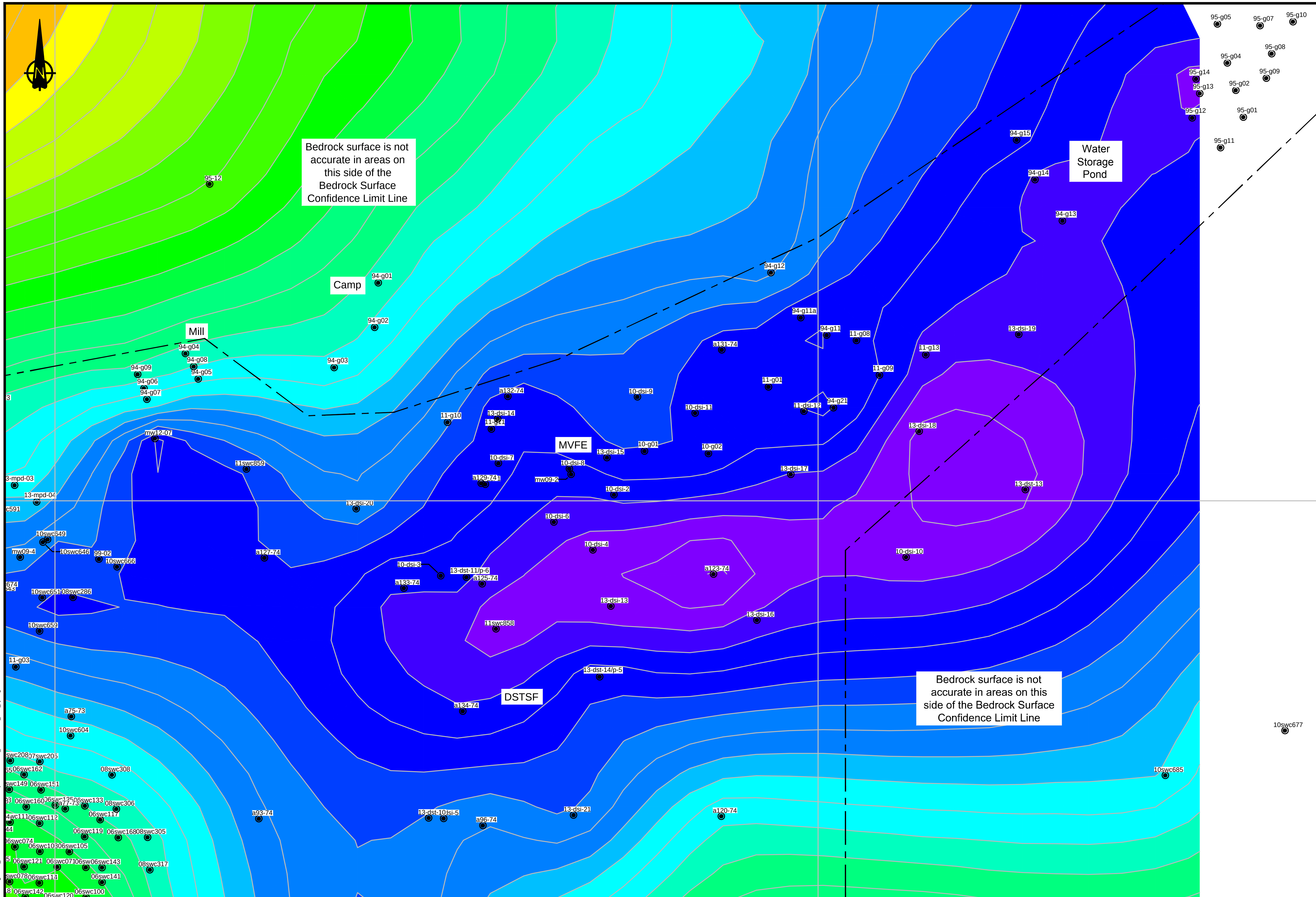
Minto Mine

Bedrock Surface Review

Bedrock Contour Map
Area 2 Pit Area

DATE:	APPROVED:	FIGURE:
May 2014		3

J:\01_SITES\Minto\1CM002.018_MainDam_PrelimDesign\040_AutoCAD\MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg



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	
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.

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SRK JOB NO.: 1CM002.027

FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

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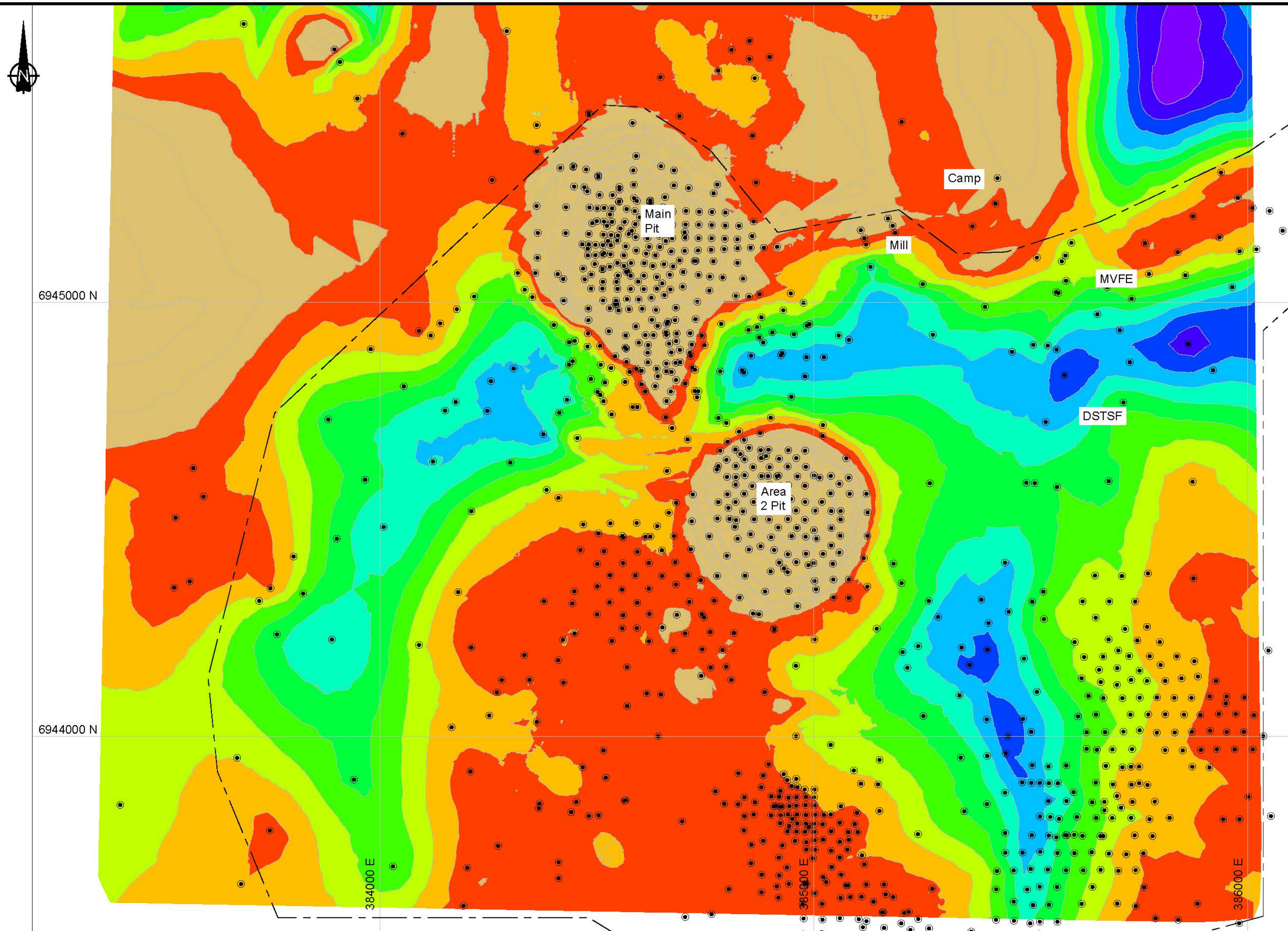
Minto Mine

Bedrock Surface Review

Bedrock Contour Map
DSTSF Area

DATE: May 2014 | APPROVED: | FIGURE: 4



LEGEND

Drillhole data used for bedrock surface

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

Boundary of Higher Confidence

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capstone MINTO MINE OPERATED BY MINTO EXPLORATIONS LTD.

SRK JOB NO.: 1CM002.027

FILE NAME: MintoBedrockSufaceUpdate20140227_1CM002.018_rev1_dj.dwg

Minto Mine

Bedrock Surface Review

Overburden Thickness Map (Excavated Pit and As-built)

DATE:	APPROVED:	FIGURE:
May 2014	PHM	5

Appendix D: 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, SRK	**Date:**	May 13, 2014
Subject:	Review of geotechnical strength properties at Minto Mine		

1 Introduction

This report 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 report 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 report 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 included 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: ϕ = sin-1(tan α) 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 provides 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 (2011b) and 60 kPa (EBA 2011c) 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 (2013b) Dry Stack Tailings Storage Facility (DSTSF) and at the South Wall Failure area have been excluded from this table but is 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) report “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 2011a), 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 (2014b)	South Wall Main Pit	Norwest (2014b) 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 (2014a)	DSTSF	Norwest (2014a) 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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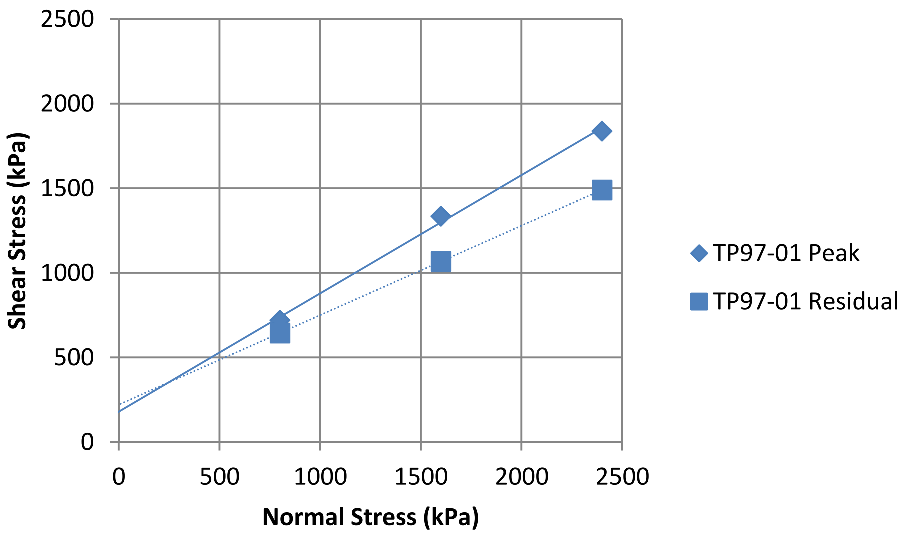
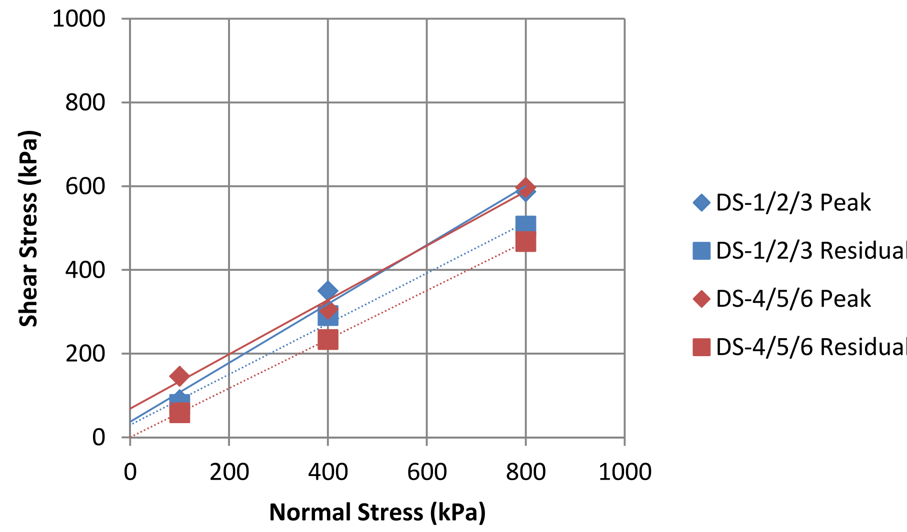
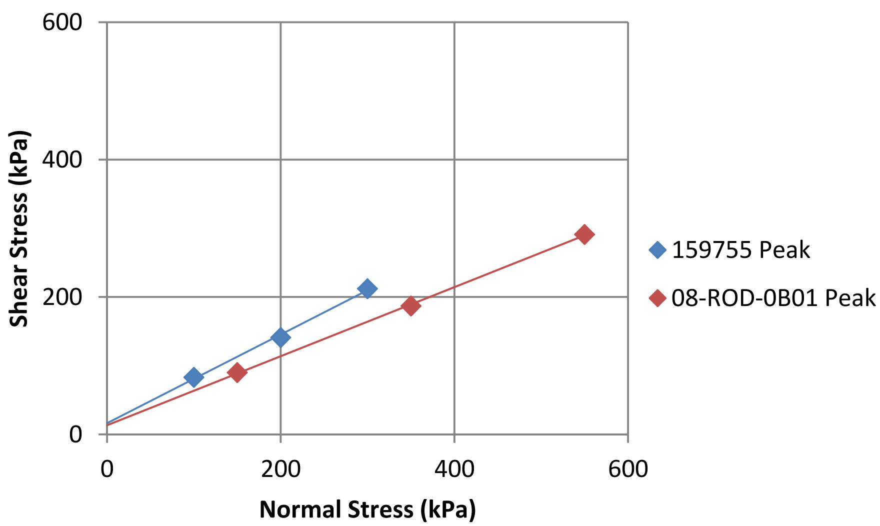
Figures



a) EBA (1998) Results



b) EBA (2009) Results



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.

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Strength Test Parameter Review

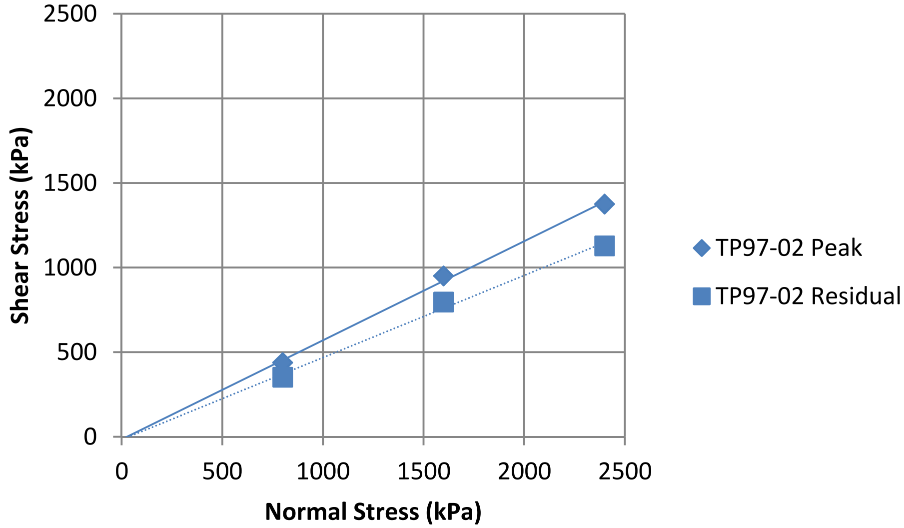
Direct Shear Test Results - Coarse Grained Soils

Job No: 1CM002.018.110

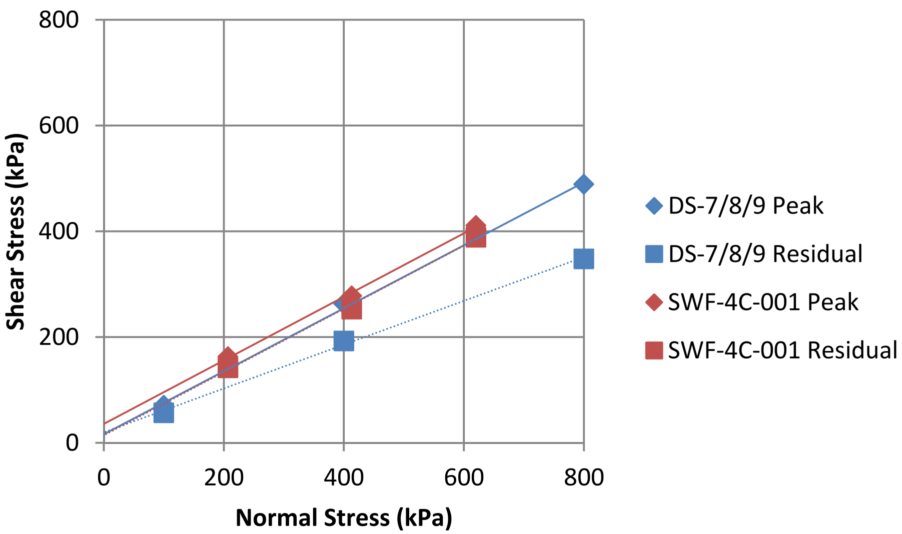
Filename: MintoMaterialProperties_Figures.pptx

Minto Mine

Date:	Approved:	Figure:
May 2014	PM	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.

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Strength Test Parameter Review

Direct Shear Test Results - Silts

Job No: 1CM002.018.110

Filename: MintoMaterialProperties_Figures.pptx

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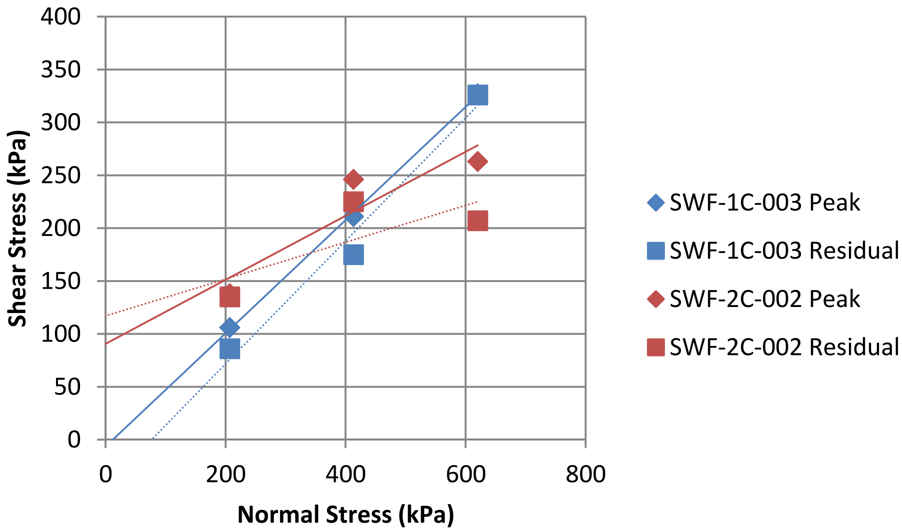
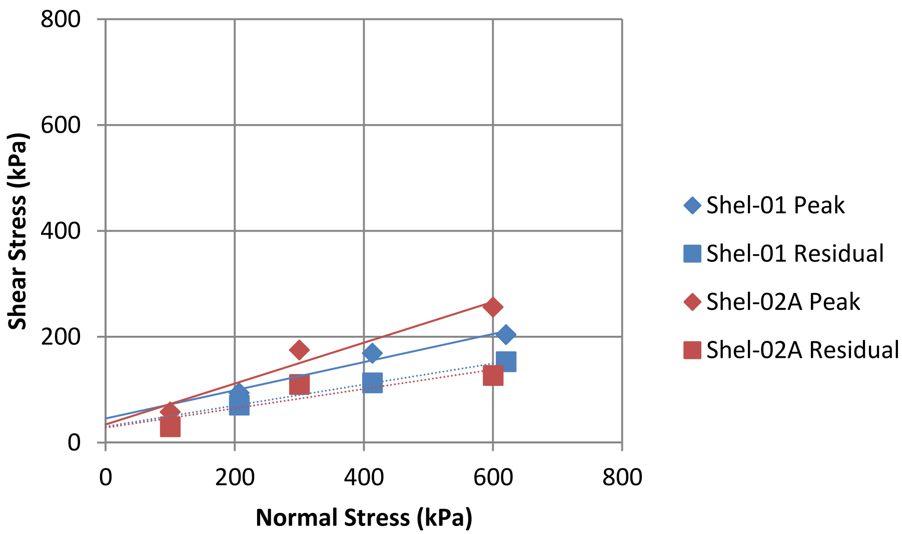
Date: May 2014 | Approved: PM | Figure: **2**



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.

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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– Strength Test Result Compilation

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													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)**	**Average peak 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

USCS	Description	Test ID	Plasticity Index	Density (prior to testing) Bulk (Mg/m^3)	Density (prior to testing) Dry (Mg/m^3)	Peak Cohesion (kPa)	Peak Friction Angle (°)	Residual Cohesion (kPa)	Residual 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