BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

Suite 800 - 1045 Howe Street, Vancouver, BC Canada V6Z 2A9
Telephone (604) 684-5900 Fax (604) 684-5909

BGC Project Memorandum

To:	**Mount Polley Mining Corporation**	**Doc. no:**	
Attention:	**MPMC – Mr. Luke Moger**	**cc:**	
From:	**Daryl Dufault, Todd Martin**	**Date:**	**October 3, 2013**
Subject:	**Mount Polley Tailings Storage Facility Stability Modeling – DRAFT**		
Project no:	**1197001.04**		

1.0   INTRODUCTION

Mount Polley Mining Corporation (MPMC) is in the process of evaluating means of reducing water inflows into the Mount Polley tailings storage facility (TSF), which currently operates under a significant net annual water balance surplus. MPMC is also developing a predictive monthly water balance model to identify and project the inflows and outflows from the tailings impoundment. Resolution and elimination of the net annual water surplus within the TSF is a necessary precursor to raising of the dam to accommodate a proposed mine expansion. The mine expansion will require raising of the dam crest in the order of 30 m above the currently permitted crest elevation. Resolution of the water balance issue, such that the volume of water in the TSF can be reduced and wide above-water tailings beaches can be established and maintained against the dam, is required before the TSF expansion design can proceed. As such, interim dam raising over the next few years, to above the currently permitted dam crest elevation of 970 m, is required to accommodate ongoing tailings production and the projected continuation of surplus water accumulation within the TSF.

Until a comprehensive and calibrated water balance is developed and vetted, a specific interim crest elevation for the next construction phase cannot be provided. MPMC requires some range of estimates for the extent of downstream shell extensions for a range of potential interim crest elevations so that foundation stripping and preparation for a downstream extension of the rockfill shell of the dam can commence in the fall of 2013, prior to the contractor demobilizing from site. Thus, MPMC has requested BGC Engineering Inc (BGC) to determine the downstream footprint for a range of crest targets to allow stripping in anticipation of the interim crest design.

This memorandum documents limit equilibrium stability analyses completed by BGC in support of downstream shell footprint area determination for the Mount Polley tailings impoundment.

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The analyses considered static loading conditions only. Six cross sections, located at representative sections of the Main, South, and Perimeter embankments, were analysed for four different crest elevations in order to provide a range of foundation stripping areas.

The crest elevations selected for evaluation are: 970 m (currently permitted crest elevation for the dam), 975 m, 980 m and 985 m. The crest elevations are not based on water balance predictions but have been selected to provide an indication of the potential footprint areas that could be required to be cleared in the future. They are also intermediate between the permitted crest El. 970 m, and the originally targeted crest elevation to accommodate the mine expansion (El. 1,000 m).

The assumptions, methodology, and results of stability analyses are provided in Section 3. The area of stripping required for each dam crest elevation is provided in Section 4 as both a table and a figure.

2.0   BACKGROUND INFORMATION

The Mount Polley Mine has been operating since 1996 and in the past two years has been operating with a significant excess of mine contact water stored in the TSF. Earlier in 2013, MPMC retained BGC to undertake a design for raising of the TSF embankment (made up of the Perimeter, Main and South dams) to a crest elevation of about 1,000 m, which would accommodate a planned expansion of the mine. The currently permitted TSF embankment crest elevation is El. 970 m. The embankment is planned to be raised above this permitted elevation in 2014 to accommodate ongoing tailings and water storage requirements.

BGC personnel visited the site June 10 and 11, 2013, to initiate the design assignment for the dam crest raise to El. 1,000 m. Subsequent to that site visit, and the on-site discussions with MPMC personnel, BGC issued the following memorandum:

> *#1197001.13.001 Mount Polley Mine Site Visit – Trip summary and path forward (18 June 2013)* outlining the findings of the site visit, the importance of reducing the pond volume and developing a comprehensive, reliable water balance.

A key conclusion provided within that memorandum was that there is currently a surplus of water in the TSF and the water balance is such that surplus water will continue to accumulate, as the TSF is in effect the site water management pond. From May 2012 to May 2013, for example, the water volume in the TSF increased by about 3.2 million m^3. If not dealt with, this ongoing accumulation will preclude raising the embankment as planned to accommodate the mine expansion, as the accumulating surplus will:

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- displace tailings storage capacity; and
- prevent the formation and development of wide, above-water beaches, a necessary element of the dam design, to separate the dam from the reclaim water pond.

BGC recommended in the above-referenced memorandum that priority be placed on developing a working water balance, understanding the sources contributing to the surplus, and devising strategies and timelines to eliminate that surplus. It is important to note that elimination of the surplus means achieving both of the following objectives:

- prevention of further water accumulation on an annual basis and, of equal importance;
- gradual reduction of the volume of water in the pond to increase tailings storage capacity and facilitate the development of wide above water tailings beaches against the dam.

The establishment and maintenance of wide, above-water tailings beaches represents a fundamental design component of the dam. Given the current water balance circumstances, BGC recommended that an interim raise design be evaluated which would provide sufficient tailings, water (including the accumulating surplus), and flood storage/freeboard capacity for another few years until the water surplus is eliminated, the volume of water in the TSF decreased, and above-water beaches are established and maintained.

A key question unanswered when the above-referenced memorandum was issued was how long it would take to resolve the water surplus situation, and thus the extent of the "interim" embankment raising that would be required to provide for mine production and tailings storage through to that time.

In a telephone conference between Mr. Luke Moger of MPMC, and Messrs. D. Dufault and T. Martin of BGC held on July 2, 2013, this question was discussed. BGC pointed out that, in the absence of wide above-water tailings beaches separating the till core of the embankment from the reclaim water pond, from a geotechnical perspective the dam was being operated more as a water-retaining dam than a tailings dam. As such, the question of how long the dam could continue to be operated in this manner could be assessed on the basis of generally-accepted design practice relating the core width of water-retaining earthfill dams to the hydraulic head acting across the core. MPMC authorized BGC to proceed with this assessment, which was documented in the following BGC memorandum:

> *#1197001.13.002 Mount Polley Mine – Revised Target Crest Elevation Assessment (25 July 2013)*

The assessment indicated that, under the current water surplus conditions, once the embankment crest reaches the currently permitted crest El. 970 m, the core width to hydraulic head ratio will already be at the generally accepted lower limit of design practice for water-retaining dams. As the water level increases, the core width to hydraulic head ratio will be less

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than the accepted limit in 2014 demonstrating the urgency with which the water balance surplus needs to be eliminated.

Still left unanswered at the current time is when the water balance surplus will be eliminated. As a consequence, also left unanswered is the scope of the interim raise required. Dam crest raising above crest El. 970 m will require an extension of the downstream rockfill shell of the dam. An extension of the downstream shell may also be required to achieve minimum required factor of safety criteria. As discussed in the aforementioned August 7 teleconference, prior to the contractor working on the 2013 embankment raise leaving site in the fall, MPMC would like to have an indication of the potential downstream shell extensions that would be required for a range of interim crest raise elevations. It was therefore agreed that BGC would undertake stability analyses to determine the downstream shell configurations for the various interim crest elevations evaluated, with the elevations intermediate between the permitted crest elevation of 970 m, and the originally targeted crest elevation of 1,000 m to accommodate the proposed mine expansion. The results of this analysis are provided in Section 3.

3.0   STABILITY MODELLING

3.1. Geometry and Cross Sections

Six representative cross sections were selected for stability modeling of the embankment. Cross section locations were chosen to provide a range of foundation conditions, spatial distribution around the dam, and with focus on locations where stripping will impact downstream infrastructure. The modelled sections are:

- South Embankment

    - Station 0+720 (cross section F): This location was selected as extends through the south embankment seepage recovery pond. Analysis of this section will therefore inform the interaction of stripping activities with the pond operations and location.

- Main Embankment

    - Station 1+900: Selected as it represents similar geometry and foundation conditions to section A, but without the constraint of the seepage recovery pond.

    - Station 2+060 (cross section A): This section has been previously modelled as a critical section for design, due both to the presence of glaciolacustrine soils in the foundation and the geometric constraint of the existing downstream seepage recovery pond.

    - Station 2+430: Selected to provide an eastern cross-section along the main embankment alignment, and account for the different foundation stratigraphy below that portion of the dam.

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- Perimeter Embankment
    - Stations 3+500: Selected due to the excavated till borrow pit to the downstream, which could affect stability.
    - Station 3+990 (cross section D): This location was selected as it is through the perimeter embankment seepage recovery pond in order to inform the interaction of stripping activities with the pond operations and location.

Figure 1 shows the location of the cross sections on the Stage 9 Crest Elevations Plan, issued by AMEC (2013).

The internal dam geometry for cross sections A, D, and F are based on the geometry presented in the 2013 AMEC as-built drawings. Where no as-built cross section was available (cross sections at Stations 1+900, 2+430 and 3+500) internal dam geometry was interpreted based on the nearest available section. At these locations, the surface representing the base of the dam was inferred based on local topography.

The Mount Polley tailings dam has been previously constructed using the modified centerline method. Recent and future raises are being constructed using the centerline method. The dam is comprised of a central core of compacted glacial till (Zone S), supported by a downstream rock fill shell (Zone C). Filters separate the core and rock fill units. Upstream support is provided by the deposited tailings. Foundation materials typically comprise glaciolacustrine soils, glacial till, or a combination thereof overlying bedrock.

Each cross section was analysed at the following crest elevations: 970 m (currently permitted crest elevation for the dam), 975 m, 980 m and 985 m. The dam geometry for crest elevations above 970 m followed the design of the 2013 raise with a vertical core alignment. Figures 2 through 7 show the existing geometry inferred for each cross section. The foundation stratigraphy was developed based on stratigraphic sections prepared by Knight Piesold and by AMEC (2012). Further discussion of the strength parameters applied to each unit is provided below.
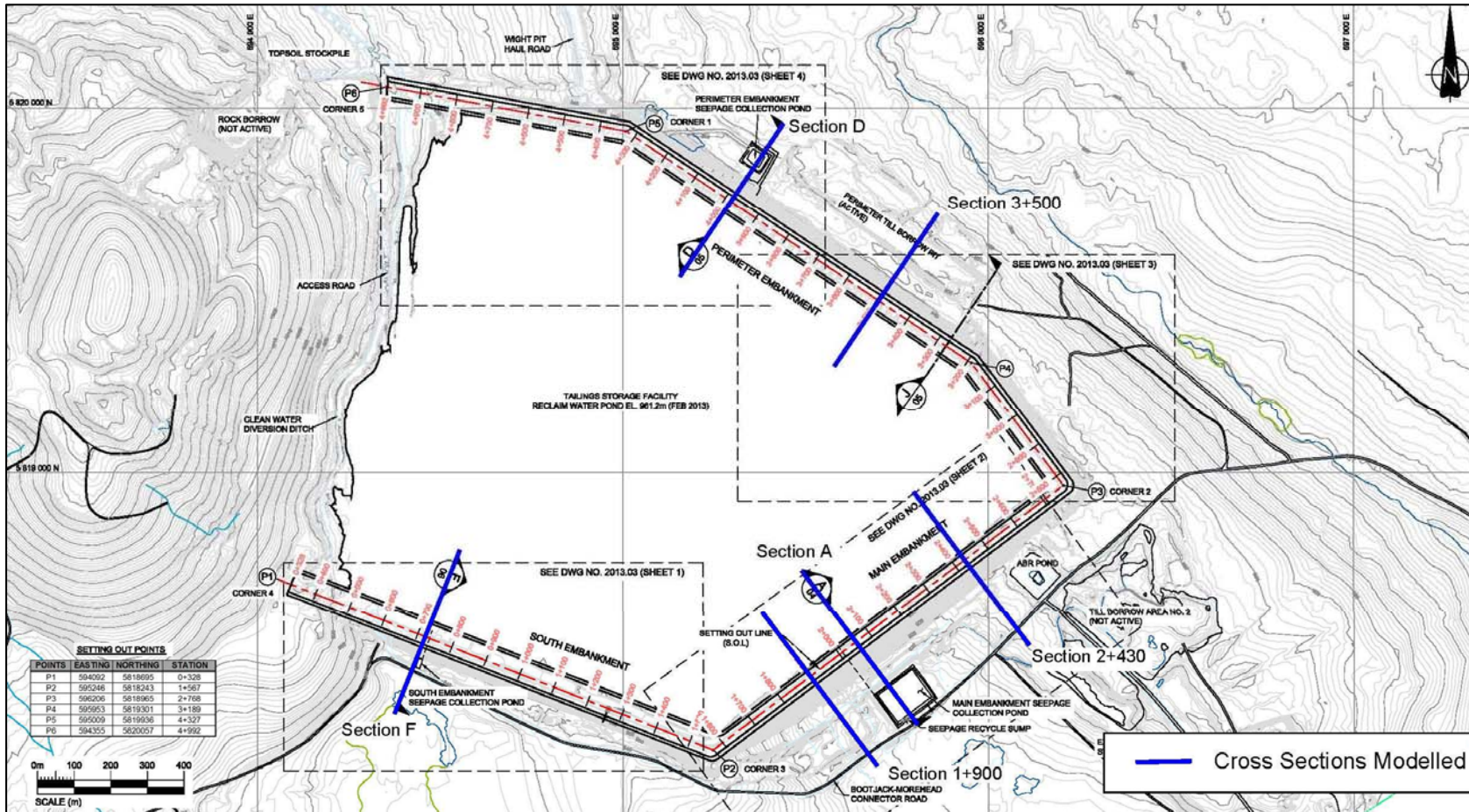
Figure 1   Modelled Cross Section Locations

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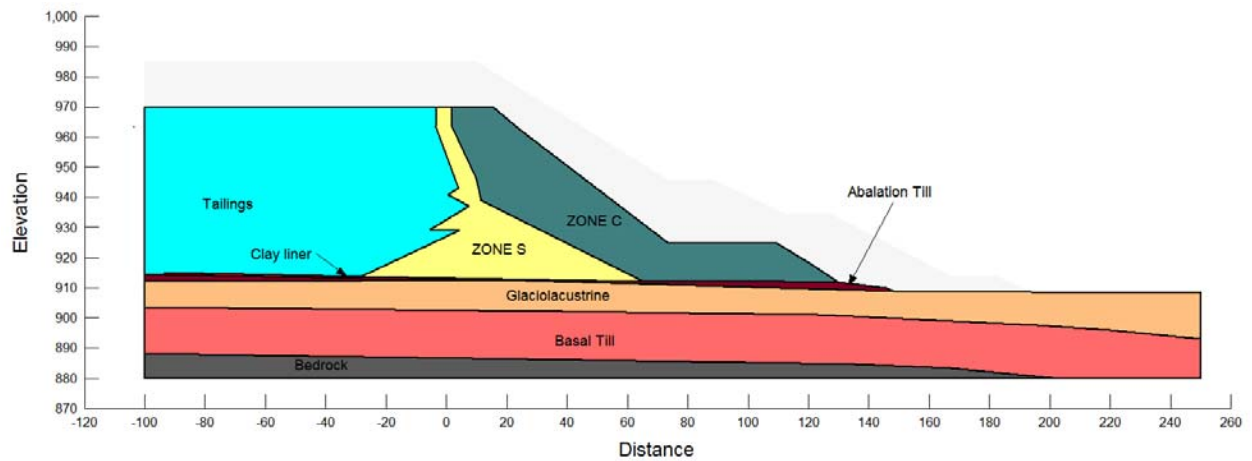**Figure 2   Cross Section A, Main Embankment**
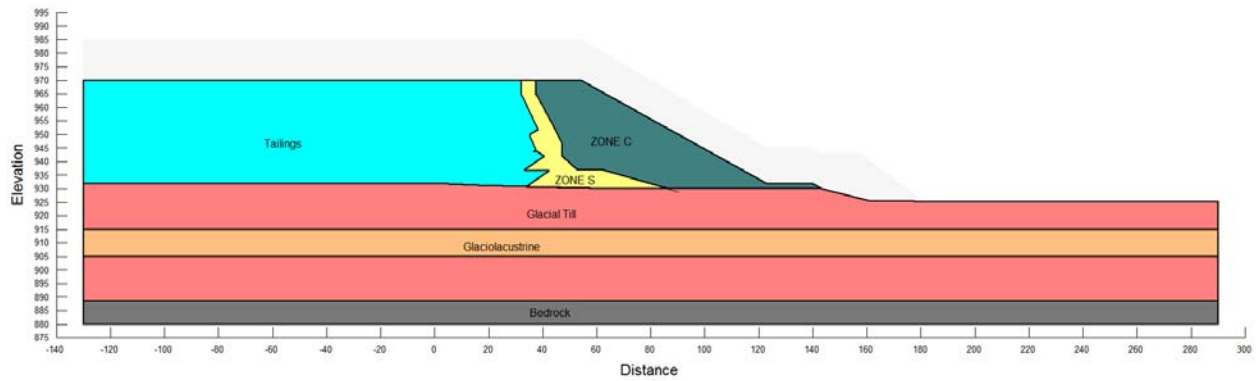
Figure 3   Cross Section D, Perimeter Embankment
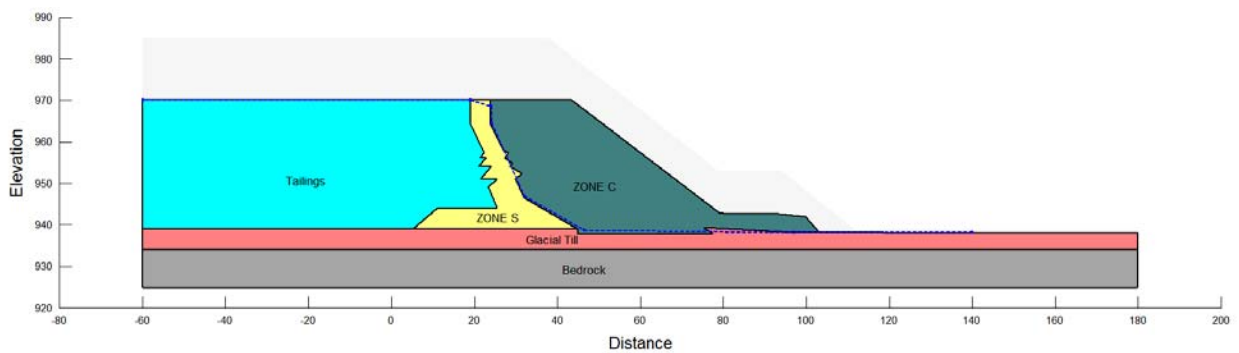
Figure 4   Cross Section F, South Embankment

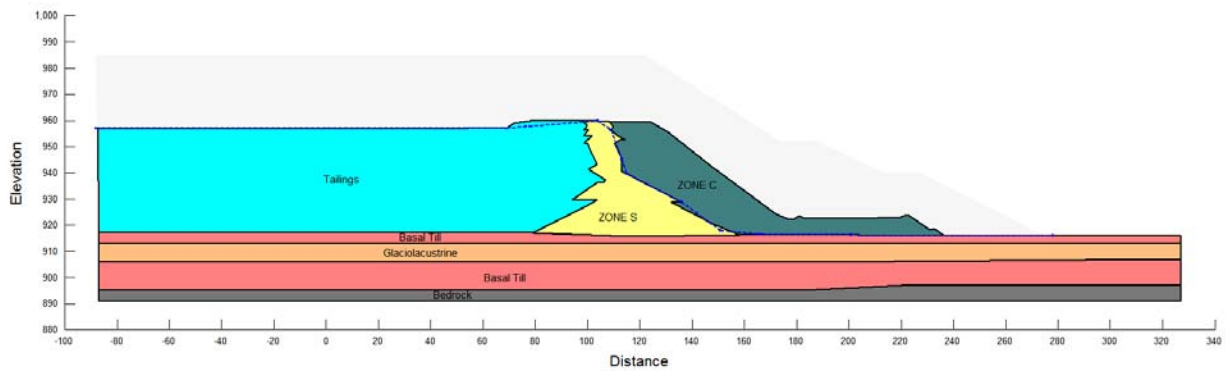**BGC ENGINEERING INC.**

Figure 5   Cross Section 1+900, Main Embankment
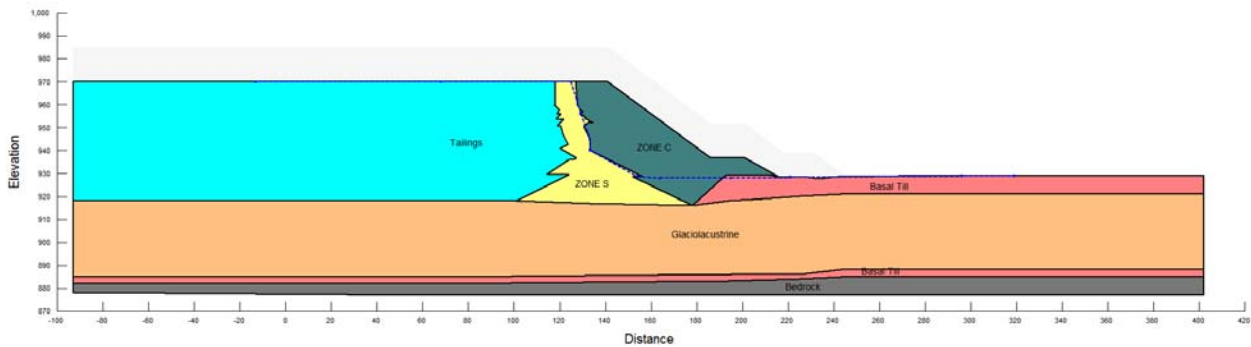
Figure 6   Cross Section 2+430, Main Embankment
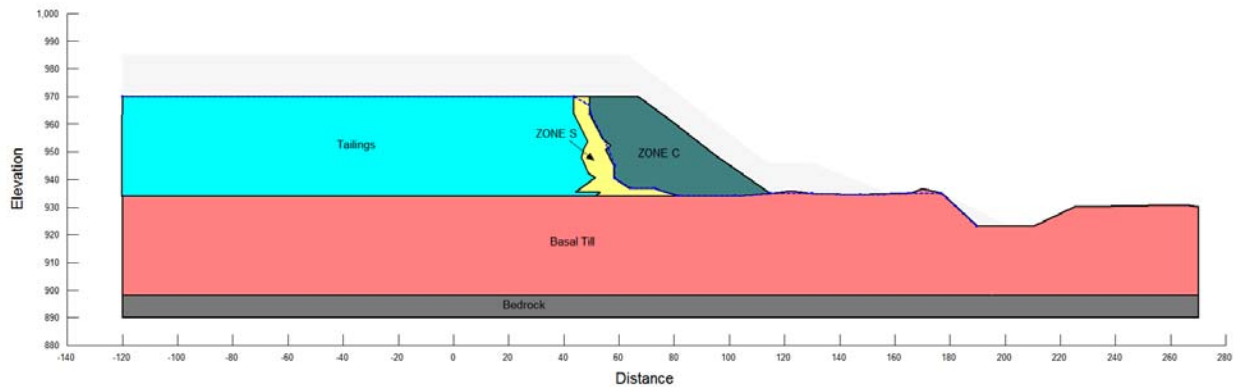
Figure 7   Cross Section 3+500, Perimeter Embankment

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Stability analyses focused on the downstream shell geometry. The following assumptions were used for the downstream shell configuration:

- The crest downstream of the core (i.e. including the rockfill and filter units) was maintained at a minimum width of 14 m in order to allow a sufficient construction width at the crest.
- Where inclinometers are present on the Main and Perimeter embankments, a 15 m wide bench was centered around the instruments to maintain access. On the South embankment, where inclinometers are not installed (owing to a lack of glaciolacustrine soils in the foundation and bedrock at shallow depth), benches were incorporated as per the design of the Perimeter embankment.
- Downstream rockfill slopes were limited to 1.3H: 1V (per existing slopes on the dam) or flatter.

For the purposes of analysis the following assumptions were made:

- Details of zone contacts within the dam (i.e. between core and filters) were locally simplified, as these are non-relevant to limit equilibrium analyses of the overall dam.
- Upstream fill was modeled as tailings for the purposes of stability analysis (assigning the upstream fill the same properties as the tailings simplifies the model design and is a slightly conservative assumption).
- Filters were not discretely modelled, but rather were incorporated into the downstream shell unit labeled Zone C.

3.2. Foundation Conditions

Subsurface conditions for each modelled cross section were developed based on interpretation of results from nearby boreholes (AMEC, 2012) and stratigraphic sections developed previously by Knight Piesold. Details of the assumed foundation conditions for analysis are provided below for each cross section:

- Cross Section A, Ch 2+060 (Figure 2): Subsurface conditions are based on the interpretation from Boreholes VW11-05 and VW11-06. The dam foundation was interpreted to consist of a 10 m thick glaciolacustrine layer increasing in thickness and depth beyond the toe of the downstream shell. Below the glaciolacustrine layer is a 15 m thick layer of till, underlain by bedrock. A seepage recovery pond is currently located at the toe of cross section A. It was assumed that this pond would be removed and not limit the placement of downstream shell for crest elevations greater than 970 m, as outlined in BGC, 2013.
- Cross Section D, Ch 3+990 (Figure 3): Subsurface conditions are based on SI11-04 and VW11-10. For the purposes of modeling, the foundation stratigraphy was simplified into three layers: glacial till (El. 932 m – 915 m), glaciolacustrine (El. 915 m – 905 m) and glacial till (El. 905 m – 888.6 m). Bedrock was found below an El. of 888.6 m.
- Cross Section F, Ch 0+720 (Figure 4): Subsurface conditions are based on VW11-01, which encountered a 5 m deposit of glacial till atop bedrock.

- Cross Section Ch 3+500 (Figure 7): Subsurface conditions are based on the closest borehole to the cross section, VW11-09, which shows a 37 m thick glacial till deposit over bedrock.
- Cross Section Ch 2+430 (Figure 6): The subsurface conditions are based on VW11-08 and SI11-02. These boreholes show 7 m of glacial till overlying approximately 35 m of glaciolacustrine/glaciofluvial deposits, underlain by a 2 m layer of glacial till on bedrock.
- Cross Section Ch 1+900 (Figure 5): The subsurface conditions are based on VW11-04 and SI11-01. VW11-04 encountered a 7 m layer of glaciolacustrine between layers of glacial till. The near surface layer of till has a thickness of 3.5 m. The bottom layer of till has a thickness of 11 m and is underlain by bedrock. SI11-01 encountered a second 1 m layer of glaciolacustrine at an elevation 900 m within a 10 m thick unit of glacial till. This layer was not discretely modeled as it is thin (and apparently discontinuous given its absence in VW11-04) compared to the overlying 7 m thick glaciolacustrine unit which will control stability.

3.3. Shear Strength Parameters

3.3.1. General

Shear strength parameters for stability analyses have been maintained from those used in previous AMEC analyses. The exception to this is the drained residual strength assigned to the glaciolacustrine unit. Some of the boreholes that penetrated this unit encountered zones with a laminated/varved structure, with some clayey layers. The lower bound drained residual shear strength condition considered for this unit in the is based on the assumption that clay varves within this unit, assumed to be laterally continuous, can undergo significant reduction in shear strength in response to shear strains. The result would be a decrease in shear strength fromm peak to residual conditions. Determination of this parameter is discussed in detail below in Section 3.4.2. A summary of the parameters used for all materials in the stability analyses is provided in Section 3.4.3.

3.3.2. Residual Shear Strength for Glaciolacustrine Foundation Unit

In the presence of glaciolacustrine soils, common concerns in terms of embankment stability are:

- potential pre-shearing in clayey varves (due to glacial drag, or post-glacial land sliding) that could lead to a very low (residual) operative shear strength parallel to bedding; or
- sufficient foundation straining induced by embankment loading that reduces shear strength in such materials from peak to or near residual.

The tailings dam instrumentation includes inclinometers, extending through the foundation overburden soils and seated into bedrock. Monitoring of the inclinometers to date has indicated no significant movements that would be consistent with either of the concerns listed above. AMEC (2012) undertook a sonic drilling program in the foundation of the dam, for installation of additional

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instrumentation, and to obtain an improved characterization of the glaciolacustrine soils in the foundation. That program similarly did not yield any evidence of the two concerns listed above.

Despite these findings, the application of the observational approach to ongoing dam raising requires a conservative approach including:

- the possibility of lower operative shear strengths in the glaciolacustrine foundation unit than currently assumed, and
- a contingency for a stabilizing buttress berm, triggered by established threshold criteria (amount/rate of inclinometer movement) be provided for.

Accordingly, the analyses for the interim dam raising accounted for residual shear strength conditions within the glaciolacustrine foundation units. The residual strength was estimated on the basis of index property data (derived from the AMEC 2011 site investigation program) and the empirical approach described by Stark and Eid (1994).

Atterberg Limits results for the glaciolacustrine samples obtained during site investigation programs (including sonic drilling) are presented in Figure 8 (AMEC, 2012)

All results, with the exception of one (a sample from drill hole SI11-02) have a liquid limit less than 50%, and classify as clay of low plasticity (CL) to clay of intermediate plasticity (CI). Gradation analyses of glaciolacustrine samples obtained from that same site investigation indicated clay fractions (see Figure 9) in the range of 20% to 35%.

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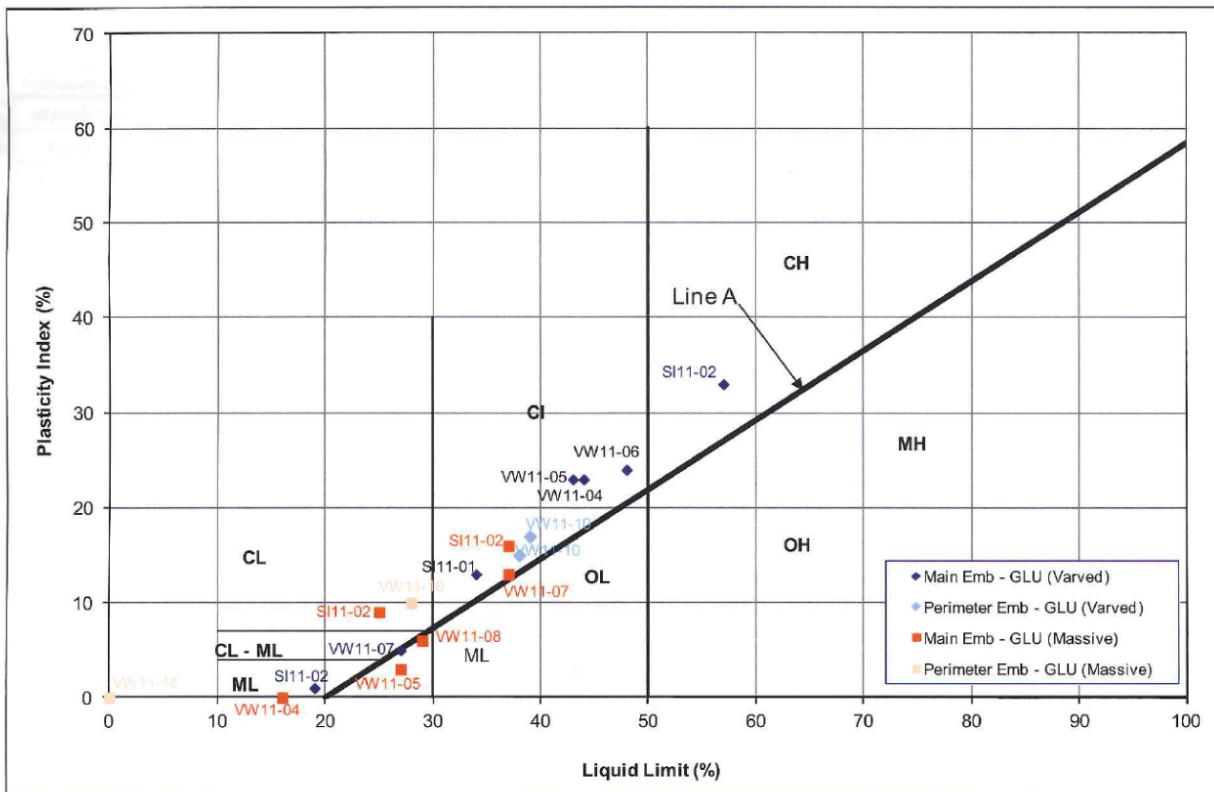
Figure 8   Atterberg limits test results: glaciolacustrine unit samples from AMEC 2011 sonic drilling program

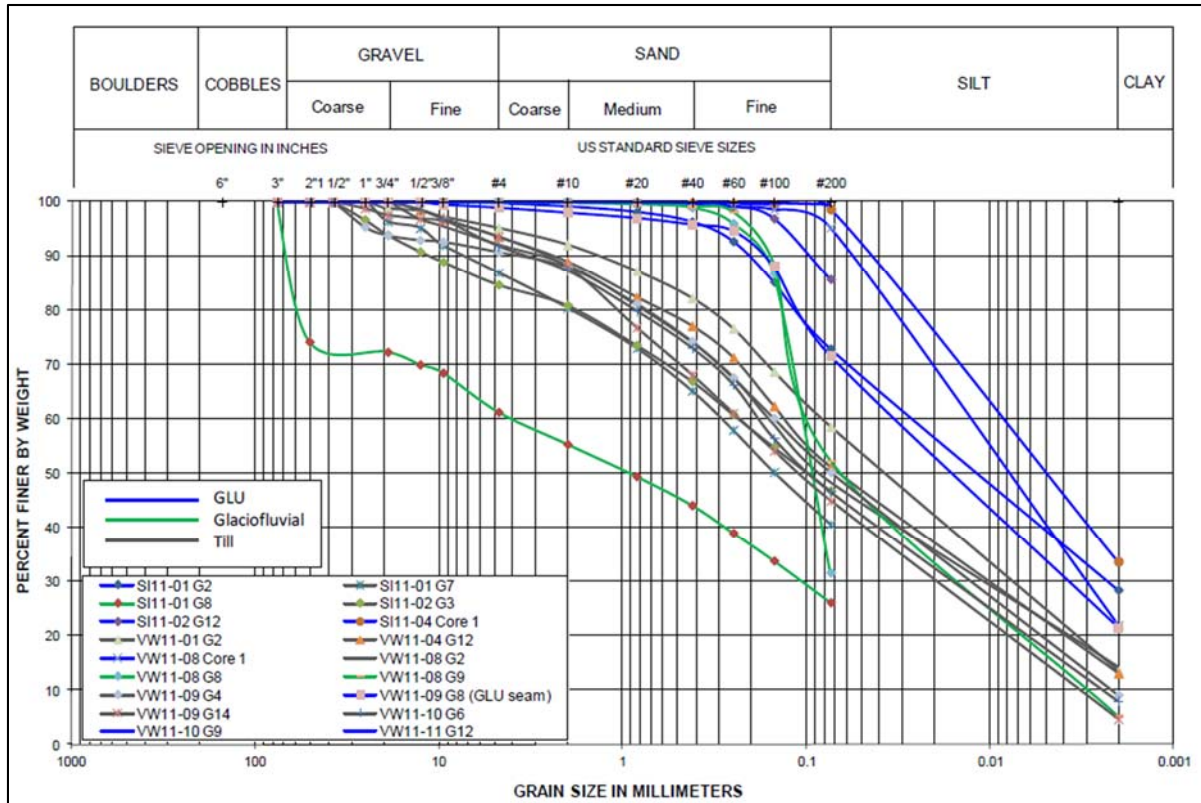**BGC ENGINEERING INC.**

Figure 9   Gradation test results from AMEC 2011 sonic drilling program

The approach of Stark and Eid (1994), illustrated in Figure 10, relates the drained residual strength of clays to liquid limit (derived from Atterberg limits tests) and the clay size fraction (% by dry weight finer than 0.002 mm). On the basis of the data in Figures 8 through 10, a drained residual strength of $\phi' = 18°$ was judged to be reasonable, and likely somewhat conservative.

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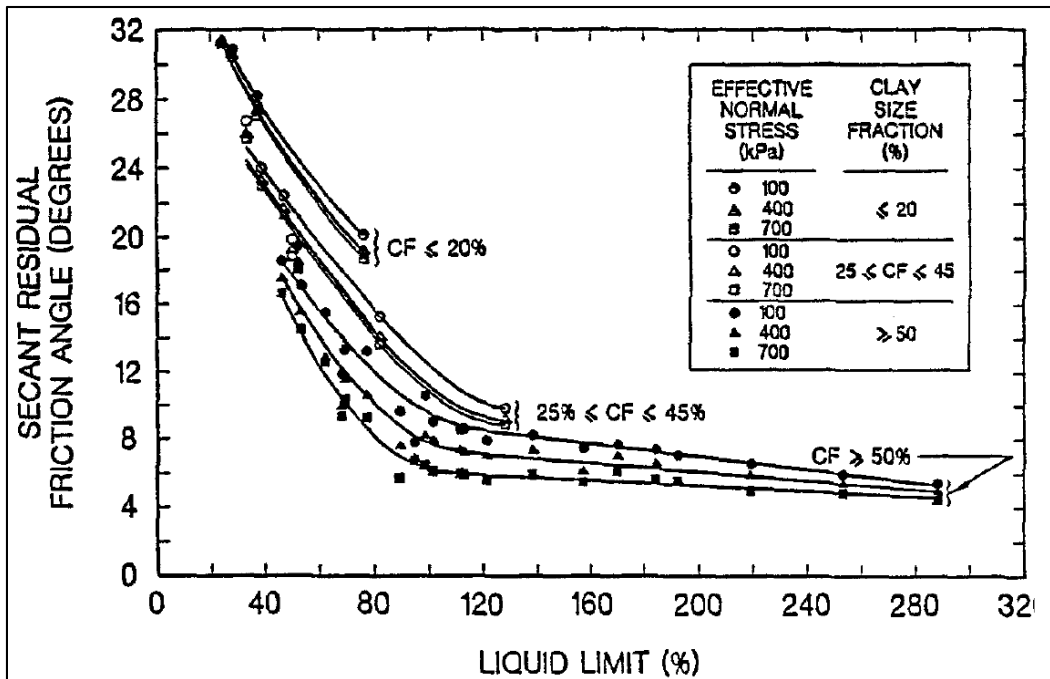
Figure 10   Relationship between Drained Residual Friction Angle and Liquid Limit (Stark and Eid, 1994)

3.3.3. Summary of Shear Strength Parameters Used for Analyses

Table 1 provides a summary of the shear strength parameters used for the stability analyses presented in Section 3.9.

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Table 1   Shear Strength Parameters

	Wet Unit Weight (kN/m³)	Effective Stress Shear Strength	
		ϕ' (deg)	c' (kPa)
Zone C (Rock Fill)	22	Average Leps [1]	
Zone S (Core)	20.5	35	0
Glaciolacustrine (Peak)	20	28	0
Glaciolacustrine (Residual)	20	18[2]	0
Basal Till	21	33	0
Tailings	18	30	0
Liquefied Tailings	18	$\tau/\sigma = 0.1$ Minimum strength = 0 kPa	
Bedrock	Impenetrable		

Notes:

    (1)   Leps (1970) published shear-strength-normal stress functions for rock fill.

    (2)   According to Stark and Eid (1994) and considering maximum Liquid limit of 50% for the glaciolacustrine unit.

3.4.   Pore Pressure Conditions

For the purposes of the stability analyses, the tailings and upstream phreatic surface were assumed level with the dam crest. Downstream of the core, the phreatic surface is assumed to follow the core, with drained conditions in the rockfill shell. Downstream of the core, the foundations are assumed to be fully saturated, with foundation pore pressures reflecting piezometer data. To date, foundation piezometers have indicated no discernible response to fill placement.

3.5.   Target Factors of Safety

The stability analyses considered both peak shear strength conditions and residual glaciolacustrine strength, combined with liquefied shear strength within the tailings. The local toe stability of the downstream shell was also checked. Details of these loading conditions, and the corresponding factor of safety criteria, are given below.

3.5.1.   Peak Shear Strength Conditions

This model assesses the long term stability of the dam, using peak shear strengths of materials. The minimum required factor of safety under these loading conditions is 1.5. For cross section A the seepage recovery pond at the toe will limit the extent to which the downstream shell can be extended to fulfill this criterion. For 970 m crest elevation along cross section A only, a factor of safety of 1.3 will be applied for designs which constrain the downstream shell extension by maintaining the current seepage pond position. Relocation of the seepage recovery pond would be required to achieve a factor of safety of 1.5, as indicated in the stability analysis results for

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Section A at the permitted design crest El. 970 m (Table 2). Relocation of the pond is therefore required for any crest raise above El. 970 m.

3.5.2. Residual Strength Case

For residual shear strength conditions in the glaciolacustrine unit, and considering liquefaction of the impounded tailings, the minimum required factor of safety is 1.1. A liquefied shear strength was used for the impounded tailings in this case simply for purposes of conservatism. Given the dam section geometry, the shear strength of the tailings does not have a significant bearing on the factor of safety in any case.

3.5.3. Bench and Toe Stability

The stability of each raise of the downstream shell was also analyzed to check that the proposed local benches, and the rockfill shell toe, satisfy the factor of safety criteria given above.

3.6. Potential Slip Surfaces Considered

For both shear strength cases, two potential slip surface geometries were examined based on the subsurface conditions present in each cross section.

For sections containing glaciolacustrine deposits - cross sections 2+060 (A), 3+990 (D), 1+900 and 2+430- a wedge slip surface geometry was used to model a horizontal shear plane within the glaciolacustrine deposit which would be the most likely situation given that any horizontal and continuous clayey varves would represent the critical residual strength stability condition. Because of its structure, the shear strength of the glaciolacustrine unit is anisotropic, with a higher shear strength across the laminated structure than along it. For the sections not containing glaciolacustrine deposits, cross sections 0+720 (F) and 3+500, circular slip surfaces were assumed, as the shear strength of the till foundation would be isotropic.

3.7. Model Analysis Software

The limit equilibrium stability software, Slope-W computer (GeoSlope, 2007), was used for analysis utilizing the Morgenstern-Price solution method, consistent with previous stability analyses of the dam.

3.8. Model Results

Stability analysis was carried out for each of the six cross sections at the four dam crest elevations. For each section, the safety factors obtained from the two loading conditions were compared to the factor of safety criteria given in Section 3.6. The downstream shell design was then adjusted to satisfy the safety factor requirement associated with the critical loading case for each section.

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The governing slip surface for the 985 m elevation of each cross section is shown in Figures 11 to 22 for both steady state and residual strength loading conditions. In some instances (e.g. Figure 11), the requirement to maintain benches for instrumentation access resulted in geometries that yielded factors of safety somewhat higher than the minimum required values.

Table 2 summarizes the results of the modeling. Also provided is the location of the downstream toe for each crest elevation, expressed as an offset from the dam setting-out line.
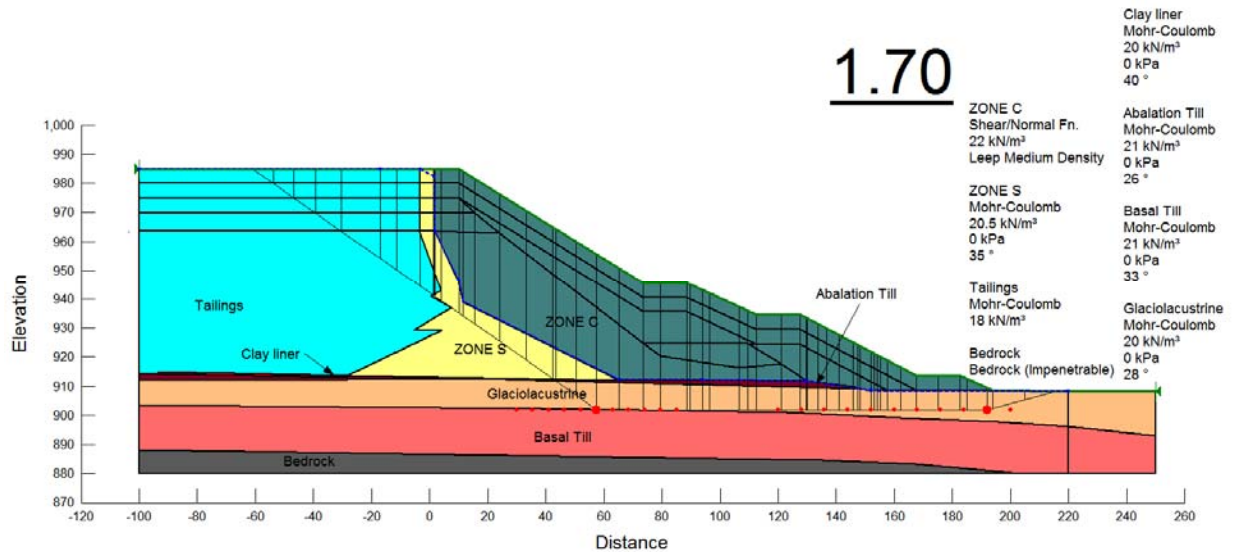
Figure 11   Cross Section 2+060 (A), El. 985 m, Peak Strength
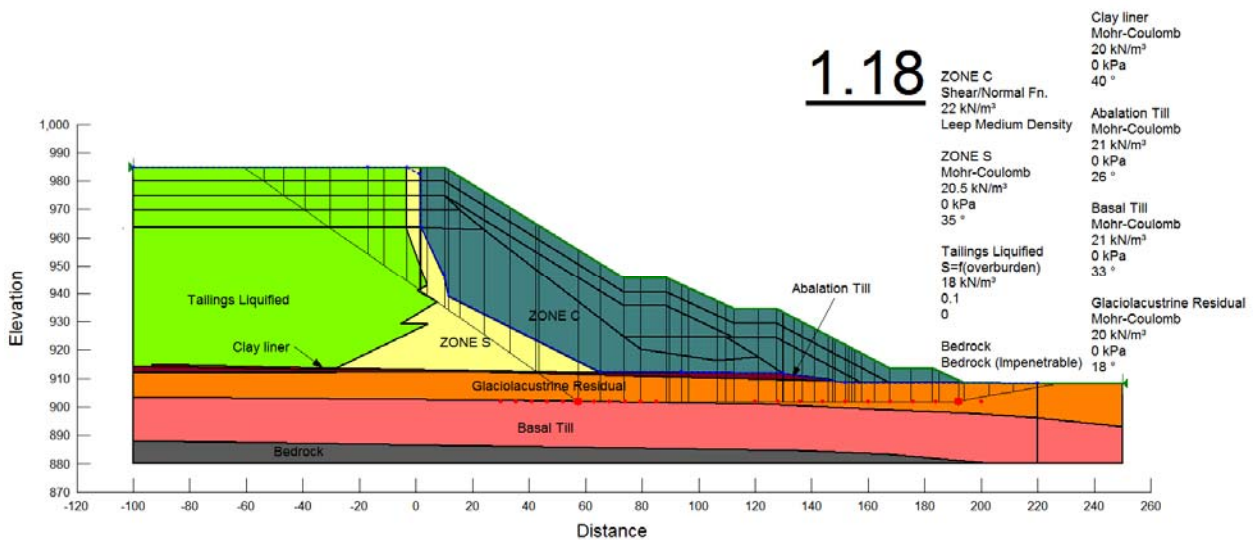
Figure 12   Cross Section 2+060 (A), El. 985 m, Residual Strength

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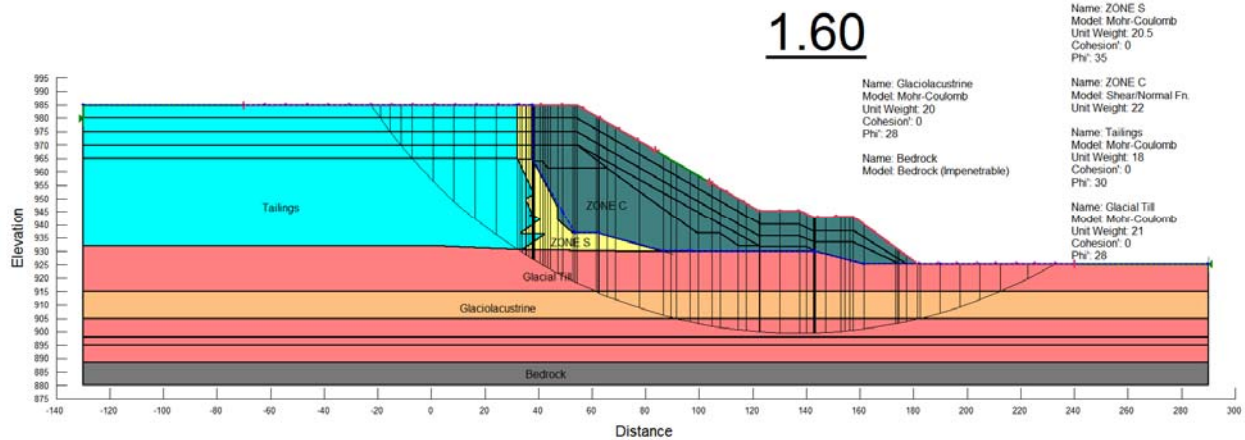
Figure 13   Cross Section 3+990 (D), El. 985 m, Peak Strength
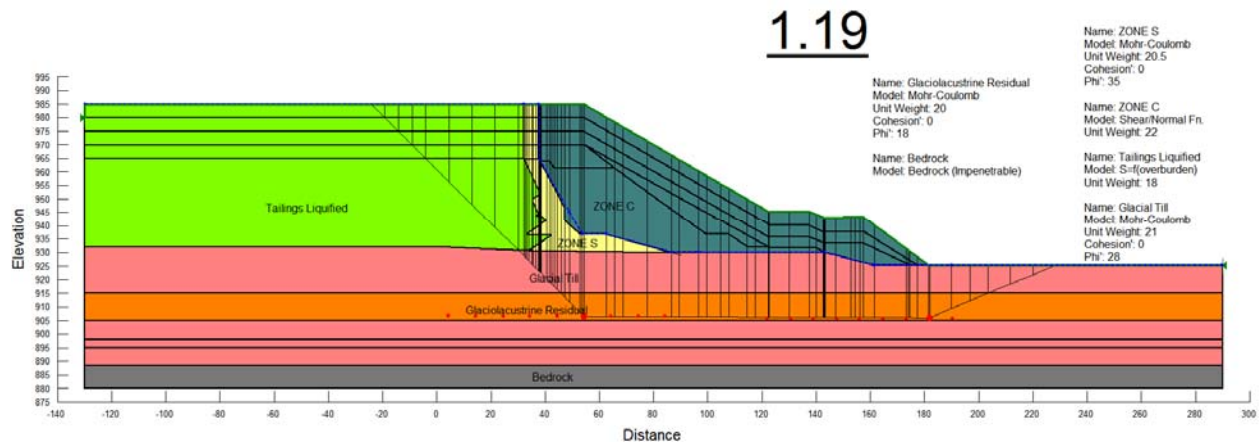
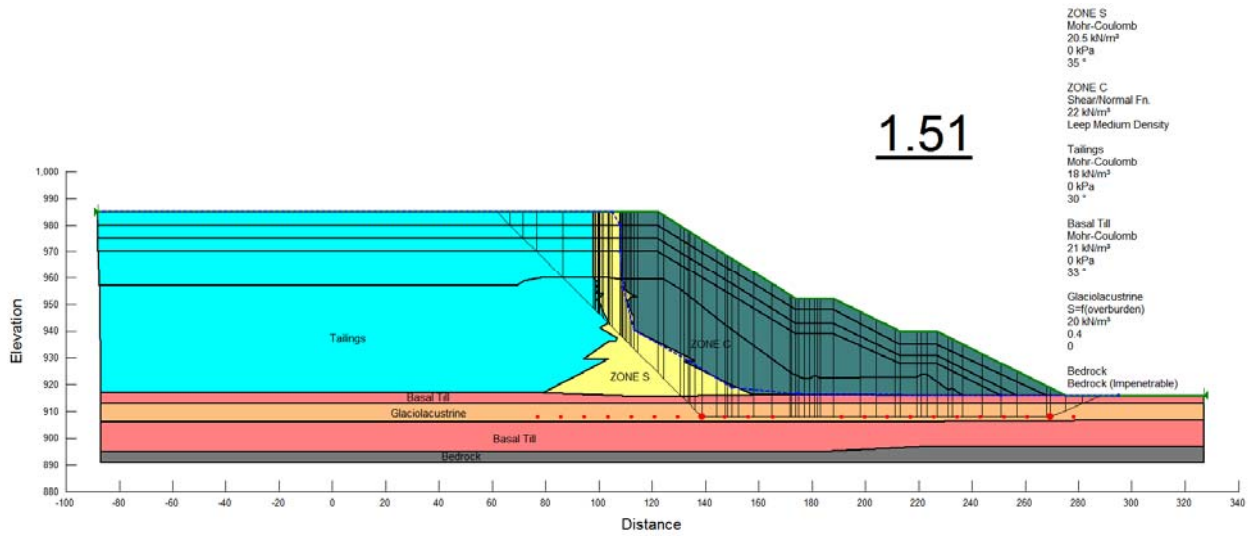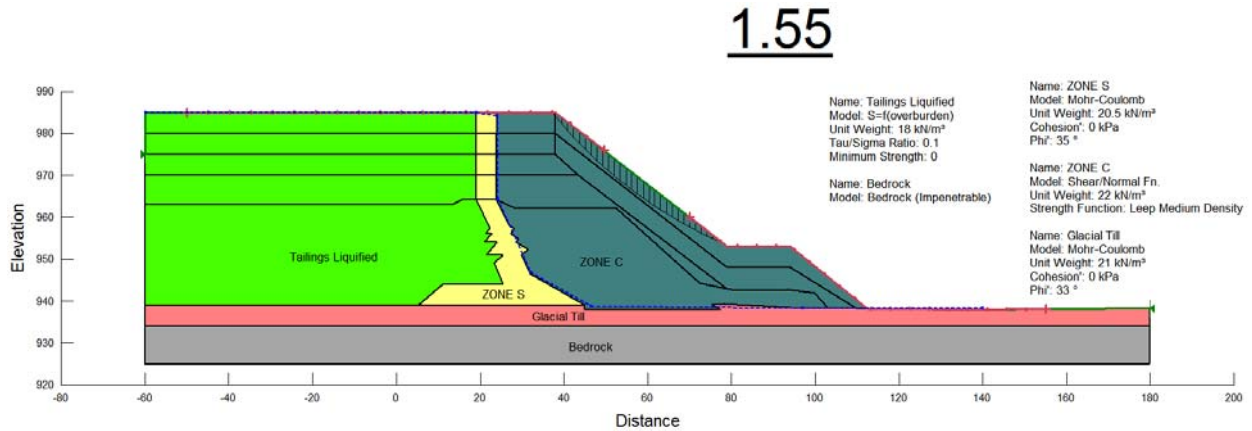**Figure 14   Cross Section 3+990 (D), El. 985 m, Residual Strength**
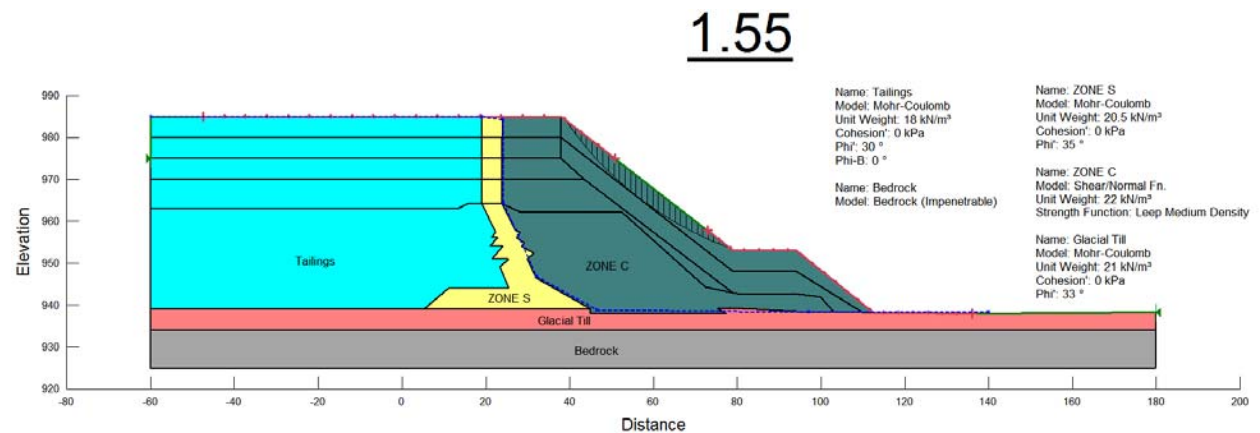
Figure 15   Cross Section 0+720 (F), El. 985 m Peak Strength

Figure 16   Cross Section 0+720 (F), El. 985 m, Residual Strength

Figure 17   Cross Section 1+900, El. 985m, Peak Strength
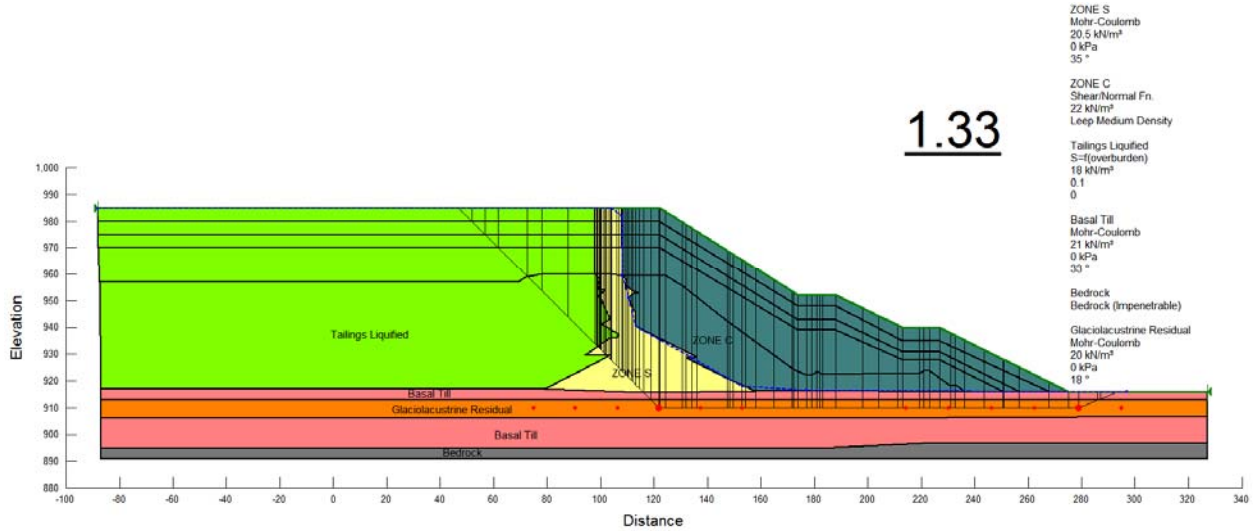
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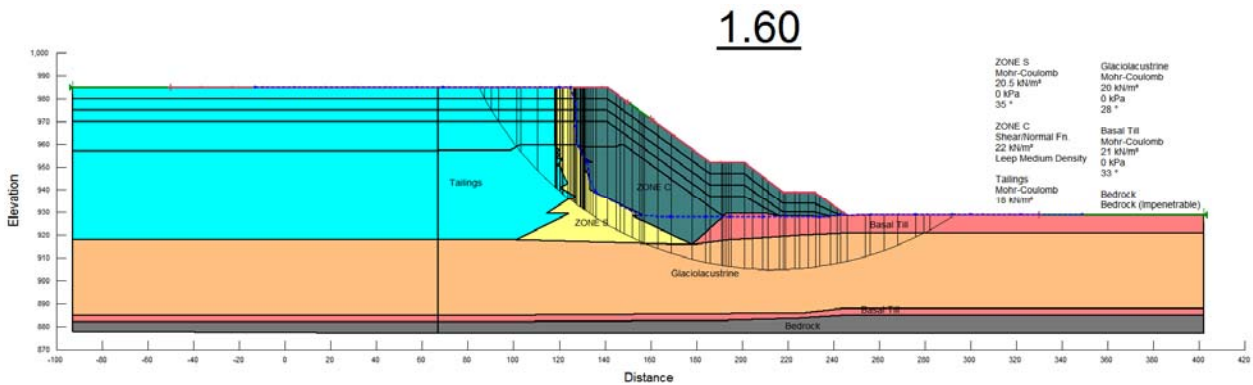**Figure 18   Cross Section 1+900, El. 985 Residual Strength**

Figure 19   Cross Section 2+430, El. 985 m, Peak Strength

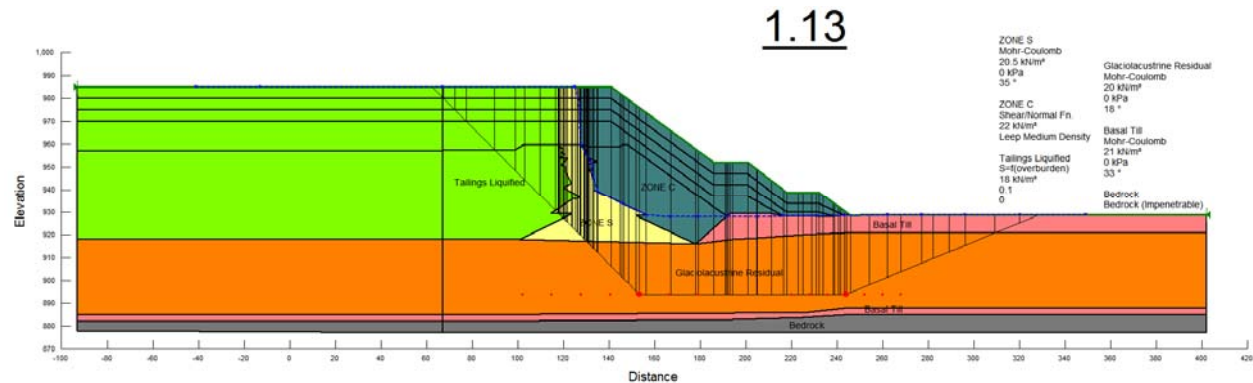**Figure 20   Cross Section 2+430, El. 985 m Residual Strength**

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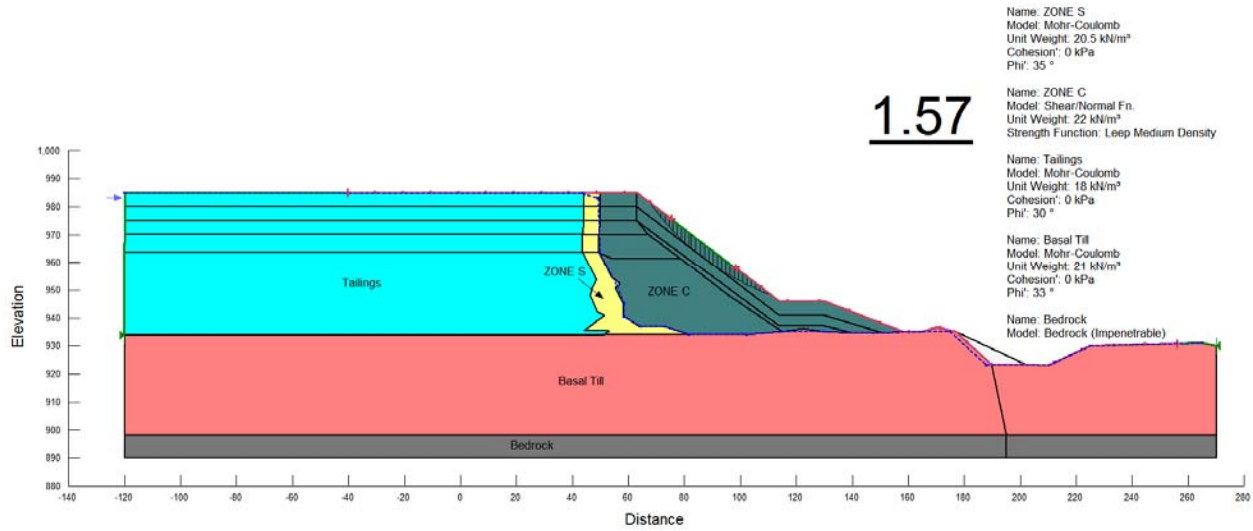
Figure 21　　Cross Section 3+500, El. 985 m, Peak Strength
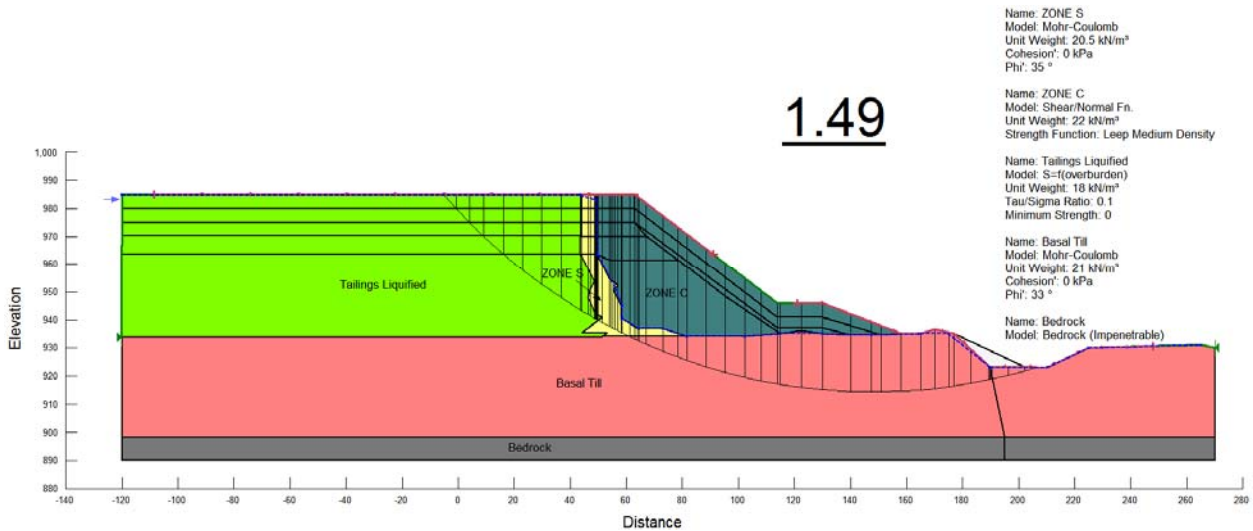
Figure 22　　Cross Section 3+500, El. 985 m, Peak Strength

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Table 2   Results of Stability Modeling

Cross Section	Crest Elevation	Factor of Safety (FOS)		Downstream Toe Location	
		Peak Shear Strength Target FOS = 1.5	Residual Shear Strength Target FOS = 1.1	Offset from setting out line (m)	Offset from 2012 dam toe (m)
A (2+060)	970 m, with Seepage Recovery Pond	1.42[2]	1.04[3]	130	0
	970 m, Seepage Recovery Pond Relocated	1.54	1.10	157	27
	975 m[1]	1.60	1.13	157	27
	980 m[1]	1.64	1.15	167	37
	985 m[1]	1.70	1.17	194	64
D (3+990)	970 m	1.50	1.23	105	0
	975 m[1]	1.61	1.22	137	32
	980 m[1]	1.58	1.21	140	35
	985 m[1]	1.60	1.19	143	38
F (0+720)	970 m[1]	1.61	1.61	82	0
	975 m[1]	1.57	1.57	82	0
	980 m[1]	1.55	1.55	88	6
	985 m[1]	1.55	1.55	91	9
1+900	970 m	1.53	1.32	151	15
	975 m	1.51	1.36	156	20
	980 m	1.51	1.33	167	31
	985 m	1.52	1.52	175	39
2+430	970 m[1]	1.61	1.22	96	0
	975 m[1]	1.64	1.20	119	23
	980 m[1]	1.65	1.19	122	26
	985 m	1.60	1.13	126	30
3+500	970 m[1]	1.61	1.61	71	0
	975 m[1]	1.60	1.57	96	25
	980 m[1]	1.55	1.50	106	35
	985 m[1]	1.56	1.49	114	43

Notes:
   (1)   Design governed by local stability of bench or maximum slope of 1.3H:1V and spacing of instrument benches
   (2)   Target factor of safety for peak strength analysis without relocation of existing seepage collection pond is 1.3.
   (3)   Target factor of safety cannot be met due to limits of shell geometry imposed by seepage collection pond at toe

BGC Project Memorandum
To: MPMC – Mr. Luke Moger   From: Daryl Dufault, Todd Martin   October 3, 2013
Mount Polley Tailings Storage Facility Stability Modeling – DRAFT   Project no. 1197001.04

4.0   Foundation Stripping Requirements

Based on the findings of stability analysese discussed in Section 3.0, a series of preliminary downstream stripping footprints were determined, one for each modeled crest elevation. The final stripping footprints were developed by interpolating between the required buttress locations (as determined by the slope stability analyses). The stripping areas were estimated in autocad based on the interpolated stripping extents. Footprints and areas are 'neat' and do not included any contingency. Final limits at each stripping stage should be extended a minimum of 5 m beyond the projected toe. Stripping footprints are shown in Figure 23 and the required downstream shell stripping areas are also provided in Table 3. The footprints as shown in Figure 23 will also be provided in dxf format.

Table 3   Downstream Shell Stripping Areas for a Range of Crest Elevations

TSF Dam Crest Elevation (m)	Area to be Stripped (m^2) [1]
970	60,300
975	127,600
980	166,000
985	207,000

Note: (1) Areas provided are beyond the 2012 design toe

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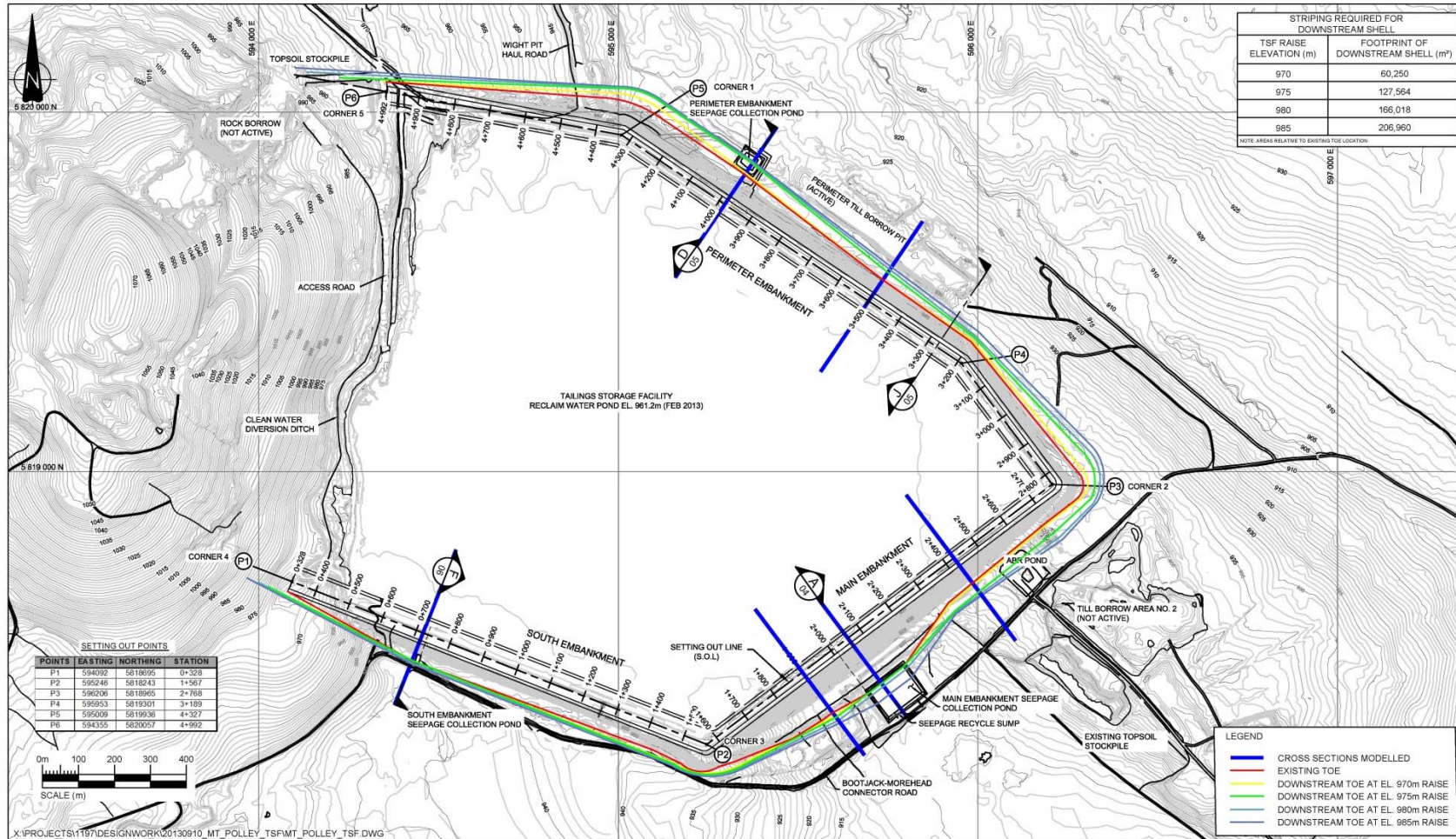
Figure 23   Foundation Stripping Areas for a Range of Crest Elevations

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5.0   CLOSURE

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Yours sincerely,

BGC ENGINEERING INC.
per:

Daryl Dufault, P.Eng.   Todd E. Martin, P.Eng., P.Geo.
Senior Geotechnical Engineer   Senior Geotechnical Engineer

Reviewed by:
Thomas G. Harper, P.E. (Washington)
Senior Civil Engineer

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