Klohn Crippen Berger

April 14, 2008

Greens Creek Mining Company
P.O. Box 32199
Juneau, Alaska
99803-2199

Mr. Tom Zimmer
Surface Operation Manager

Dear Mr. Zimmer:

Stage 2 Tailings Storage Facility Expansion 2008 Design Overview
Pond 6 Expansion

Please find enclosed ten (10) copies of our Stage 2 Tailings Facility Expansion 2008 Design Overview for the Pond 6 Expansion.

Please feel free to contact us if you have any questions.

Yours truly,

KLOHN CRIPPEN BERGER LTD.

Robert W. Chambers, P.Eng. (BC)
Project Manager

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TABLE OF CONTENTS

PAGE

1. INTRODUCTION 1
2. POND 6 EXCAVATION 3
3. FOUNDATION DRAINAGE 4
4. PERIMETER ROAD AND DITCH 5
5. DEGRIT BASIN 7
6. SOIL-BENTONITE WALLS 8
7. UTILITIES 9
   7.1 Wet Well A 11
8. SLOPE STABILITY 12
   8.1 Sludge Placement 13
   8.2 Pond 6 Expansion Area 15
      8.2.1 Hynes-Griffin Deformations 17
9. INSTRUMENTATION 18
10. RELOCATION OF SERVICES 19
11. CONSTRUCTION SEQUENCE 20
12. CLOSING REMARKS 21
REFERENCES 22

TABLES

Table 8.1 Material Properties 13
Table 8.2 Sludge Placement Stability Analysis Results 14
Table 8.3 Pond 6 Expansion Area Stability Analysis Results 16
Table 8.4 Pond 6 Excavation Temporary Condition Stability Analysis Results 16
Table 8.5 Summary of Seismic Deformation Estimates 17

FIGURES

Figure 3.1 Gradation of Foundation Gravelly Sand 4

TABLE OF CONTENTS
(continued)

APPENDIX

Appendix I Slope Stability Plan and Sections

1. INTRODUCTION

Greens Creek Mine is an underground polymetallic (zinc, silver, gold and lead) mine on northern Admiralty Island, Alaska (see Drawing D-54001[1]) that is owned and operated by Greens Creek Mining Company (GCMC).

Mine tailings are dewatered at the mill site; about one-half of the tailings are utilized as backfill in the mine, and the remainder are transported to the Tailings Facility for surface storage. An incremental expansion of the Tailings Facility storage capacity, hereafter referred to as the Stage 2 Expansion, included extension of the pile into 6 main areas known as the Southeast, Northeast, Northwest/Pit 5, Pond 6, East Ridge, and the Southwest expansion areas. Work began in 2004 and design and construction continues through 2008. As each area is developed, detailed designs are prepared taking into account overall requirements for seepage control and drainage, integration with existing construction, local ground conditions, temporary construction constraints and incorporation of new performance data. Because the expansion is occurring over a number of years, GCMC is able use an observational and adaptive approach to adjust and improve the design.

Regulatory approval for the expansion was granted after a tailings site review by the U.S. Department of Agriculture Forest Service (USFS) and other Federal, State and Local Agencies. With the USFS as the lead agency, a Final Environmental Impact Statement (FEIS) was issued on October 24, 2003 with a Record of Decision (ROD) supporting the tailings disposal expansion plan. The tailings area is operated under a Waste Management Permit (WMP) issued by the Alaska Department of Environmental Conservation (ADEC) on November 7, 2003 (ADEC, 2003), and a General Plan of Operations (GPO)

1 All drawings referenced in this document are found in Stage 2 Expansion of Tailings Facility - Pond 6 Expansion Construction (KCBL, 2008).

(KGCMC, 2004) submitted to the USFS. The expansion plan was presented in concept in the Design Overview for Forest Service Submission (Klohn Crippen, 2004). Since the 2004 report, detailed drawings and specifications have been submitted for the construction of the Southeast, Pond 7 (Klohn Crippen, 2005), Northwest/Pit 5 and Northeast Expansion areas (Klohn Crippen, 2007).

Expansion activities planned for 2008 and 2009 include the excavation of sludge from the existing Pond 6, along with the peat and sand below Pond 6, construction of a foundation for a tailings footprint expansion into this area, and construction of a degrit basin upstream of Pond 7. The perimeter road will be extended around the new tailings pile toe. Raises and extensions of existing slurry walls will be done as required for containment of tailings contact water. This design report provides a general outline of the design for these activities, including a stability summary, and details of the planned 2008 expansion construction sequencing. Drawing D-54002 shows the general arrangement of the Stage 2 Expansion, including the updated toe line for the expansion.

This report was prepared by Klohn Crippen Berger Ltd. for the account of Greens Creek Mining Company. The material in it reflects Klohn Crippen Berger's best judgment in light of the information available to it at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Klohn Crippen Berger Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

2. POND 6 EXCAVATION

Pond 6 is to be dewatered to facilitate removal of the pond sludge, which will be hauled and placed in a constructed containment on the tailings pile, located within defined limits such that overall geotechnical stability is not compromised. The recommended limits of the sludge placement reservoir are shown in Appendix I, Figure I-1.

Based on historical drill logs from the area the entire Pond 6 footprint is assumed to be founded on relatively low permeability glacial till underlaying a less than 4 ft layer of sand which in turn is underlaying peat that varies in thickness between 2 ft and 12 ft. All peat and sand will be excavated to expose the glacial till, which is expected to act as a low hydraulic conductivity aquitard along the foundation of the tailings pile (Environmental Design Engineering, 2007). The peat will be segregated for storage within containment at an owner-designated location, such that overall pile geotechnical stability is not compromised.

3. FOUNDATION DRAINAGE

A 1 ft thick blanket of granular material will be placed on the natural till foundation in the Pond 6 footprint to facilitate foundation drainage to Wet Well A (see Section 7 for discussion regarding Wet Well A). The grain size distribution for the granular material is shown on Figure 3.1 below.

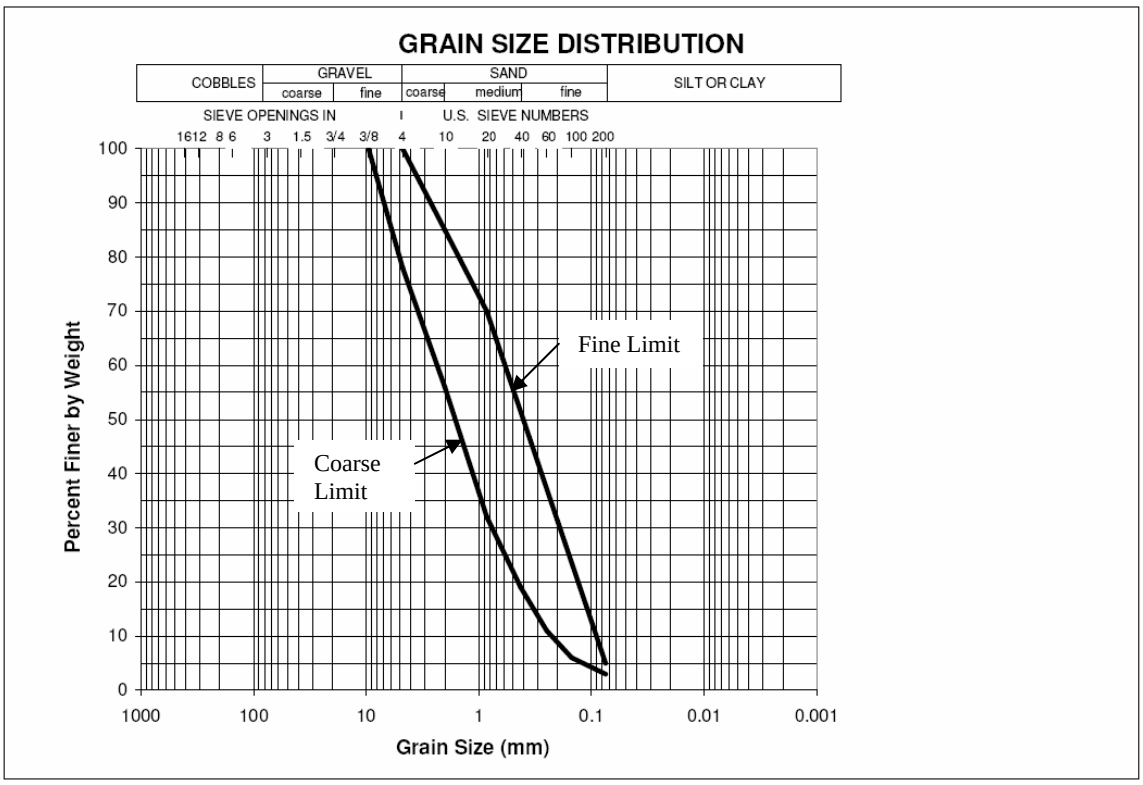

Figure 3.1 Gradation of Foundation Gravelly Sand

The drainage of the foundation will be supplemented with a system of 'burrito' style drains, consisting of a filter sock-covered perforated HDPE pipe within a channel of clean drain gravel and wrapped in a non-woven geotextile. The plan layout and details of the 'burrito' style drains are shown on Drawings D-54004 and D-54005, respectively.

4. PERIMETER ROAD AND DITCH

Currently the Tailings Facility is accessed by a series of 'dirty' roads, defined as roads on which a vehicle is likely to come into contact with tailings or other mine derived materials. Use of these roads requires that a vehicle be cleaned in the truck wash prior to returning to the road system outside of containment. In addition, the entire facility is surrounded by a perimeter 'clean' road, which may be used without requiring the vehicle to be cleaned in the truck wash prior to leaving the facility. The existing 'clean' roads and 'dirty' roads are separated from each other by ditches which discharge into Pond 6.

The existing Embankment Access Road, south of Pond 6, will remain in service as a clean perimeter road, and will be raised to an elevation of 153 ft, to accommodate the raising of the existing soil-bentonite wall (see Section 6 for further discussion on the soil-bentonite walls). The plan location of the Embankment Access Road is shown on Drawing D-54004, and details of the raise are shown on Drawing D-54008.

A 'dirty' road will be constructed around the perimeter of the Pond 6 Expansion, on the alignment shown on Drawing D-54006. The dirty road will tie into the existing 'dirty' road. The maximum road grade will be 6%. Details for the road construction including dimensions and fill material are shown on Drawing D-54007. Non-woven geofabric will act a permeable filter barrier between the tailings and the compacted rockfill of the new road.

The reinforced polypropylene-lined south ditch will be extended around the outside of the new 'dirty' road. The extended south ditch, carrying surface contact water flows from the southeast corner of the pile and sediment-laden flows from the truck wash, will report to the degrit basin that is to be constructed upstream of Pond 7 (see Section 5 for further discussion of the degrit basin).

A small lined ditch will collect surface water from between the new dirty road and the clean Embankment Road as required by the respective road grades. This small ditch will report to the main perimeter ditch at the location shown on Drawing D-54006.

Flows from the west buttress ditch, containing sediment-laden contact water from the north of the pile and flows from Pond 9, will also be extended along the new 'dirty' perimeter road and report to the Pond 7 degrit basin along a separate channel.

The new perimeter ditch alignments are shown in plan view on Drawing D-54006, and in profile on Drawing D-54007.

5. DEGRIT BASIN

A degrit basin will be constructed upstream of Pond 7 to collect sediment from surface ditches prior to discharge into Pond 7. Design and details for the degrit basin and appurtenant structures are to be generated by Greens Creek Mining Company. In 2008 a temporary lined degrit basin is planned on the footprint as shown on Drawing D-54006, to provide degritting until the completion of the Pond 6 expansion, currently scheduled to occur in 2009. Design and details of the temporary degrit basin are to be generated by Greens Creek Mining Company.

6. SOIL-BENTONITE WALLS

In selected areas, groundwater that has come in contact with the tailings is contained within the tailings pile footprint by a series of soil-bentonite walls that have been constructed around the tailings pile.

As part of the planned expansion into the Pond 6 expansion area, the following soil-bentonite walls will be constructed:

- The existing soil-bentonite wall in the Embankment Access Road is to be raised to the elevations as shown on Drawing D-54008. Foundation drains installed below the degrit basin and from the northeast corner of the Pond 6 excavation will reduce the risk of the foundation water level rising above the maximum height of the soil-bentonite wall. These foundation drains will discharge into Wet Well A (see Section 7.1 for discussion of Wet Well A).
- A new soil-bentonite wall will be constructed between the existing cut-off wall housed in the Embankment Access Road and the existing cut-off wall housed in the Saddle Road, northeast of Pond 7. The wall will be constructed to the elevations as shown of Drawing D-54008. The alignment of this wall will be confirmed during the excavation of Pond 6 such that the wall is founded on a minimum 10 ft thickness of till and does not interfere with existing utilities corridors.

These walls are expected to contain seepage from the Pond 6 expansion, and help prevent it entering the natural environment.

7. UTILITIES

Since Pond 6 functions as a sediment storage pond for surface and foundation flows, numerous active pipes currently report to the Pond 6 area, and will need to be extended and rerouted to allow removal of Pond 6. In addition there are also several pipes that are no longer in service, or were originally installed as temporary conduits, which will be decommissioned and capped or removed.

The utility layouts and pipe descriptions are shown on Drawing D-54004, Sheets 1 and 2.

The existing pipes to be extended to the perimeter surface ditch south of the pile include:

- 10" ø Solid DB-04 Feed to Pond 6 (to be capped at the edge of the dirty road for possible future use);
- 6" ø Solid French Drain from Pond 6;
- 6" ø Solid French Drain from 2002 Southeast Liner;
- 16" ø Solid French Drain from Pond 9; and
- 8" ø Solid French Drain from Pond 9 Underdrain.

The existing pipes to be extended to Wet Well A include:

- 8" ø Solid Above Liner French Drains from Southeast 2;
- 8" ø Solid Below Liner French Drains from Southeast 2;
- 12" ø Solid Spare from Northwest expansion; and
- 6" ø Solid Pipe from Wet Well 2 to Wet Well A.

The existing pipes to be decommissioned and capped or removed include:

- 12" ø Solid Backwash from Pit 5 to Pond 6;
- 8" ø Solid Stormwater Bypass to Wet Well 1;
- 8" ø Solid Stormwater Bypass to Wet Well 1;
- 6" ø Solid Wet Well 3 to Tank 6 Bypass to Pond 6;
- 12" ø Solid Water Treatment Feed; and
- 8" ø Solid Pipe from Southeast 2.

New pipes from Wet Well A are:

- 6" ø Solid Pipe from Wet Well A to Pond 7 will be a pumped line that will operate until closure of the site; and
- 12" ø Solid Pipe from Wet Well A to standpipe (standpipe will be located at WP16 on D-54004). This 12" ø pipe will act as a gravity drain from Wet Well A after closure. As a part of this expansion it will be extended to no closer than 50 ft to the existing Saddle Road soil-bentonite wall, and capped. A standpipe will be installed at the location of the capped pipe to mark its location. The 12" ø pipe will be accessed and extended at closure to an outfall west of the tailings storage facility, provided water in Wet Well A meets discharge quality standards.

Miscellaneous pipes not listed above are:

- 6" ø Solid Wet Well 3 to Tank 6 Feed is a pumped line that will be brought to the surface in the southwest corner of the pile and discharged into the west surface ditch; and
- Three 8" ø Perforated Pond 6 foundation drains will report to Wet Well A (see Section 3 for further discussion).

Pipe grades for gravity lines will be a minimum of 1%, but final pipe alignments may be field fit to reduce fills and reduce the need for cutting through the natural till foundation. All pipes will be SDR 9 HDPE.

Buried pipes not embedded in 'burrito' style drains (see Section 3) will be bedded in compacted gravelly sand screened to the gradation shown in Figure 3.1, and as shown on Drawing D-54005.

7.1 Wet Well A

In addition to the pipe installations, a new wet well, designated 'Wet Well A' will be installed at the natural low point in the till surface under the Pond 6 footprint, to collect foundation drainage flows and flows from other sources as summarized above. Flows from Wet Well A will report, via a pumped line, to Pond 7 during operations. Post-closure, flows from Wet Well A will report to an outfall point west of the tailings facility, provided water from Wet Well A meets discharge quality standards. Environmental Design Engineering, hydrological consultants for Greens Creek, expects the outflow pipes from Wet Well A to transmit flows from the Wet Well with no significant head buildup in the Well. The outflow pipes, as listed in Section 7 above, are oversized, to mitigate the possibility of water building up in the well if additional flows are introduced to the Wet Well in the future, based on recommendations by Environmental Design Engineering. Detailed design of Wet Well A will be provided by Greens Creek Mining Company. The approximate location of Wet Well A is shown in plan on Drawing D-54006.

8. SLOPE STABILITY

A series of cross sections through the tailings pile were analyzed to assess the stability of the proposed design. In addition to analyses of the new Pond 6 Expansion area, models of the entire tailings pile were analyzed to determine the limits of a possible containment for Pond 6 sludge and peat materials on top of the pile. The modeled sections are shown in plan and section views in Appendix I.

Material properties were taken directly from the Static-Peak, Static-Residual and Post-Earthquake design conditions described in Stage 2 Tailings Expansion Overall Stability Update (Klohn Crippen, 2006). Table 8.1 is a summary of those material properties, with the exception of the properties for the geosynthetic liner system, which were adopted from more recent studies as summarized in the Stage 2 Tailings Expansion Northwest/Pit 5 and Northeast Expansion Area Design Overview (KCBL 2007b). Geosynthetic liner properties will be checked by testing the specific materials used for construction.

The phreatic surface in the tailings was assumed to be a maximum height of 35 ft above the liner at the centre of the expansion area and decreasing linearly to 3 ft above the toe (KCBL 2006). The phreatic surface for the bedrock and the till was taken from Stage II Tailings Expansion Hydrologic Analysis Update (EDE, 2007).

Table 8.1 Material Properties

SOIL TYPE	TOTAL UNIT WEIGHT (pcf)	STATIC (EFFECTIVE STRENGTHS)			POST-EARTHQUAKE (Appendix VIII)	
		Peak Friction Angle (degrees)	Residual Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)
New Tailings	128	39	32	0	28	0
Old Tailings	120	33	32	0	Function [1]	
Geosynthetic Liner System	125	23	20	0	N/A[3]	N/A
Peat	67	27	N/A	0	N/A	N/A
Sand and Gravel	120	33	N/A	0	Function [2]	
Sand	120	33	N/A	0	N/A	N/A
Silty Clay	120	30	N/A	0	N/A	N/A
Silty Sandy Till	120	33	N/A	0	N/A	N/A
Compacted Rockfill/Roadfill	120	40	N/A	0	N/A	N/A

Notes: 1. The strength, below the water table, is specified as a function where S_u (post-liquefaction residual strength) is a function of depth and varies from 324 psf at surface to 2297 psf at a vertical effective stress of 9 tsf (approx. 140 ft depth).
2. Where required, post-liquefaction strength of the sand and gravel layer, based on SPT data and using Seed and Harder, 1990, gives an undrained residual strength of about Su = 1640 psf.
3. N/A indicates that the soil does not liquefy during the MDE, therefore static properties were maintained in the post-liquefaction analysis.

8.1 Sludge Placement

Approximately 11,700 yd^3 of sludge from the Pond 6 excavation is to be placed on the existing tailings pile. This low strength material was conservatively assumed to have a friction angle of 0° and a unit weight of 120 pcf, although over time the material is expected to drain and consolidate, resulting in increased long-term strength.

Two sections of the entire tailings pile were modeled, one running north-south, and one running east-west, to analyze its stability with a 10 ft thick pocket of low strength sludge included.

The analysis indicated that an area on the pile is suitable as a location within which a layer of low-strength material has little or no effect on the overall geotechnical stability of the finished pile. This limit is shown on Figure I-1 of Appendix I.

Results from the stability analysis are summarized in Table 8.2. The minimum target Factors of Safety (FOS) are defined in the Design Overview for Forest Service Submission (Klohn Crippen, 2004).

Table 8.2 Sludge Placement Stability Analysis Results

SECTION	CALCULATED FACTOR OF SAFETY			FIGURE NO.
	Static-Peak	Static-Residual	Post-Earthquake	
Minimum Target FOS	FOS ≥ 1.5	FOS ≥ 1.3	FOS ≥ 1.1	
1 (North to South)	2.1	1.8	1.7	I-2
1 (South to North)[1]	2.0	1.7	1.6	I-3
2 (East to West)	1.9	1.6	1.5	I-4
2 (West to East)	1.8	1.4	1.4	I-5, I-6

Note: 1. Shallow sloughs through the peat stockpiled in the Northeast of the Tailings Facility with lower FOS were identified during modeling. These slip surfaces are deemed to be of low significance and would be handled by maintenance. The slip surfaces represented by the FOS quoted in the above table are ones where the slip surfaces travel through the tailings. The slip surfaces corresponding to the tabulated FOS are shown in Appendix I.

The pile configuration modeled for analysis of the north-south section (Section 1) was the ultimate pile build-out. The pile configuration modeled for analysis of the west-east section (Section 2) was a pile built to the maximum crest elevation of 330 ft, without the East Ridge Expansion. The East Ridge Expansion is an approved tailings storage facility expansion to the east of the existing facility, not yet designed, which, when constructed, may result in higher factors of safety on slip surfaces running west to east due to buttressing effects generated by extending the tailings pile to the steep bedrock slopes east of the current B-road.

Based on the material properties presented in Table 8.1, the assumed phreatic levels, and the proposed sludge placement limits shown on Figure I-1, factors of safety meet the target design criteria for geotechnical stability of the pile. Based on our assessment the configuration of the pile raised to the ultimate crest elevation, without the proposed East Ridge build-out, has lower safety factors than for the intermediate cases which may occur during construction. Hence, only the ultimate case was analyzed.

Containment berms for the sludge will consist of compacted tailings built on the top of the existing pile, with 3H:1V upstream and downstream side slopes, raised downstream as the sludge level rises. The crest of the containment berm will be 20 ft wide. No analysis has been done of the temporary containment berm, as other similar tailings slopes have been shown to be stable.

8.2 Pond 6 Expansion Area

Two sections, Section 3 and Section 4, were analyzed to assess the geotechnical stability of the Pond 6 expansion area. Two scenarios were modeled on Section 3; Section 3a represents the ultimate build-out through the Pond 6 expansion, and section 3b the temporary condition after complete excavation of all sludge and peat from below Pond 6. It is expected that intermediate conditions between these two extremes will not result in lower factors of safety, and therefore was not analyzed. Sections 3a, 3b, and 4 are shown on Figures I-7, I-8 and I-9 respectively.

The ‘dirty’ road that runs around the ultimate toe of the tailings pile in the Pond 6 expansion area creates a buttressing effect and thus improves the pile’s overall geotechnical stability. Table 8.3 shows the results from the stability analysis on Sections 3a and 4.

As discussed above, the phreatic surface in the tailings was established at 3 ft above the toe of the pile, increasing linearly to 35 ft above the foundation below the crest of the finished pile. This corresponds to the most conservative scenario wherein Wet Well A is not transmitting foundation drainage flows, and the water table has built up to the level of the perimeter surface ditch.

Table 8.3 Pond 6 Expansion Area Stability Analysis Results

SECTION	CALCULATED FACTOR OF SAFETY			FIGURE NO.
	Static-Peak	Static-Residual	Post-Earthquake	
Minimum Target FOS	**FOS ≥ 1.5**	**FOS ≥ 1.3**	**FOS ≥ 1.1**	
3a (Ultimate Build-out)	2.0	1.8	1.7	I-7
4	2.0	1.9	1.8	I-9

Table 8.4 shows the results from the stability analysis on Sections 3b. The target factor of safety for the temporary construction scenario is 1.3 for a peak static condition (Klohn Crippen, 2006).

Table 8.4 Pond 6 Excavation Temporary Condition Stability Analysis Results

SECTION	CALCULATED FACTOR OF SAFETY	FIGURE NO.
	Static-Peak	
Minimum Target FOS	**FOS ≥ 1.5**	
3b (Temporary Excavation)[1]	1.3	I-8

Note: 1. The peak strength static condition was assessed for the temporary condition since the geometry of this condition will only exist for a short period. A 10 ft wide rockfill berm, or alternative approved shoring, along the toe of the excavation slope will be required to stabilize the exposed peat face, which has a factor of safety of 1.0 when unsupported.

The calculated factors of safety presented in Table 8.3 and Table 8.4 meet or exceed minimum target factors of safety, and conform to the results found in previous stability analyses through the Pond 6 area, as given in Klohn Crippen (2006). It is noted that relatively flat till foundation layer and the buttressing effect provided by the road

embankment are favorable for stability. Based on these analyses the Pond 6 Expansion meet the target design criteria for geotechnical stability.

8.2.1 Hynes-Griffin Deformations

Seismic deformations of the ultimate tailings pile in the Pond 6 expansion area were assessed using pseudo-static methods (Hynes-Griffin, et al. 1984). Hynes-Griffin provide families of curves which can be used to estimate deformation based on the ratio of the yield acceleration of a slope stability section (the pseudostatic acceleration at which the calculated factor of safety is 1.0) to the peak ground acceleration under the Maximum Design Earthquake (MDE) and the Design Basis Earthquake (DBE). Curves are provided to calculate mean, mean plus 1 standard deviation and an upper bound. Generally the Hynes-Griffin method is considered to overestimate deformations, as is appropriate for a screening tool. Upper bound results are considered statistically unlikely and are not reported in Table 8.5, however the upper bound deformations under the MDE based on these analyses do not exceed 2 ft. Sections 3 and 4 (Figures I-7 and I-8, respectively) were used for the deformation analyses using static residual parameters for calculation of the yield acceleration.

The results of the Hynes-Griffen deformation analysis is given in Table 8.5 below.

Table 8.5 Summary of Seismic Deformation Estimates

YIELD ACCELERATION	DEFORMATION (inches)			
	MDE (PGA=0.3g)		DBE (PGA=0.15g)	
	Mean	Mean + 1 Std. Dev.	Mean	Mean + 1 Std. Dev.
0.21g	negligible[1]	7	negligible	negligible

Note: 1. Results are less than the lower limits of the Hynes-Griffen curves (< 4 inches).

These results indicate that predicted deformations in the Pond 6 expansion area are small and unlikely to affect utilities in the area.

9. INSTRUMENTATION

As part of the Pond 6 expansion, environmental and geotechnical monitoring will be expanded to include the instrumentation shown in Drawing D-54009.

To monitor pore water pressure, and provide data regarding phreatic surface assumptions used in geotechnical stability modeling, three geotechnical instrumentation locations will be established along a line from within the pile to the toe. Vibrating wire piezometers will be placed in the foundation sand, and in the tailings below and above the predicted water table.

Suction lysimeters and environmental sampling tubes will be established at all three new instrumentation locations, as directed by the Greens Creek Environmental Department. GCMC will instrument Wet Well A in a similar fashion to the Pond 7 Wet Well.

All monitoring schedules for the instruments will be consistent with the GPO.

10. RELOCATION OF SERVICES

Due to time constraints it is expected that only the degrit pad portion of Pond 6 will be excavated and filled and the degrit basin constructed in 2008. The remainder of the work is scheduled to be done in 2009. To facilitate this scheduling decision, temporary diversions of surface flows and foundation flows currently reporting to Pond 6 will be required during the 2008 construction season.

Pipes currently reporting to Pond 6 will be diverted to the west or south surface ditches as directed, and the ditches will be diverted along temporary channels to the degrit basin. Pond 6 will be dewatered by pumping as required until the new designed drainage facilities are installed in 2009. Greens Creek will direct the construction of these temporary diversions in the field as required.

11. CONSTRUCTION SEQUENCE

2008 Construction will include part of the Pond 6 work plus additional work on the Northwest/Pit 5 Expansion, which is discussed in KCBL (2007b). The preliminary construction sequencing is summarized below:

- March-April: Drill and Blast in the Northwest Expansion area, that began in 2006 and 2007, will continue prior to main contractor mobilization.
- May: Dewatering of Pond 6 and excavation of degrit pad footprint to occur. Degrit pad fill to be placed along with required permanent utilities below the degrit pad. Temporary diversions of utilities to be installed.
- June: Temporary lined degrit basin to be constructed. Water treatment plant in Pit 5 to be decommissioned and demolished.
- July: Northwest/Pit 5 remaining liner foundation to be brought to grade. Bedding sand for liner to be placed and compacted.
- August: Northwest/Pit 5 liner to be installed, along with sand interlayer and geocomposite. Sand service layer to be installed only as required for tailings storage from 2008 through the 2009 construction season.

12. CLOSING REMARKS

This report presents the design for the Stage 2 Tailings Facility Pond 6 Expansion Construction. The designs presented in this report are consistent with the existing safety criteria for geotechnical stability and water management in the tailings facility. Design assumptions, particularly assumed material strata elevations, will be confirmed during construction.

The Stage 2 Expansion design incorporates successful techniques used in past construction projects for the tailings area development.

If you have any questions or wish to discuss the contents of this report, please do not hesitate to call.

Yours truly,

KLOHN CRIPPEN BERGER LTD.

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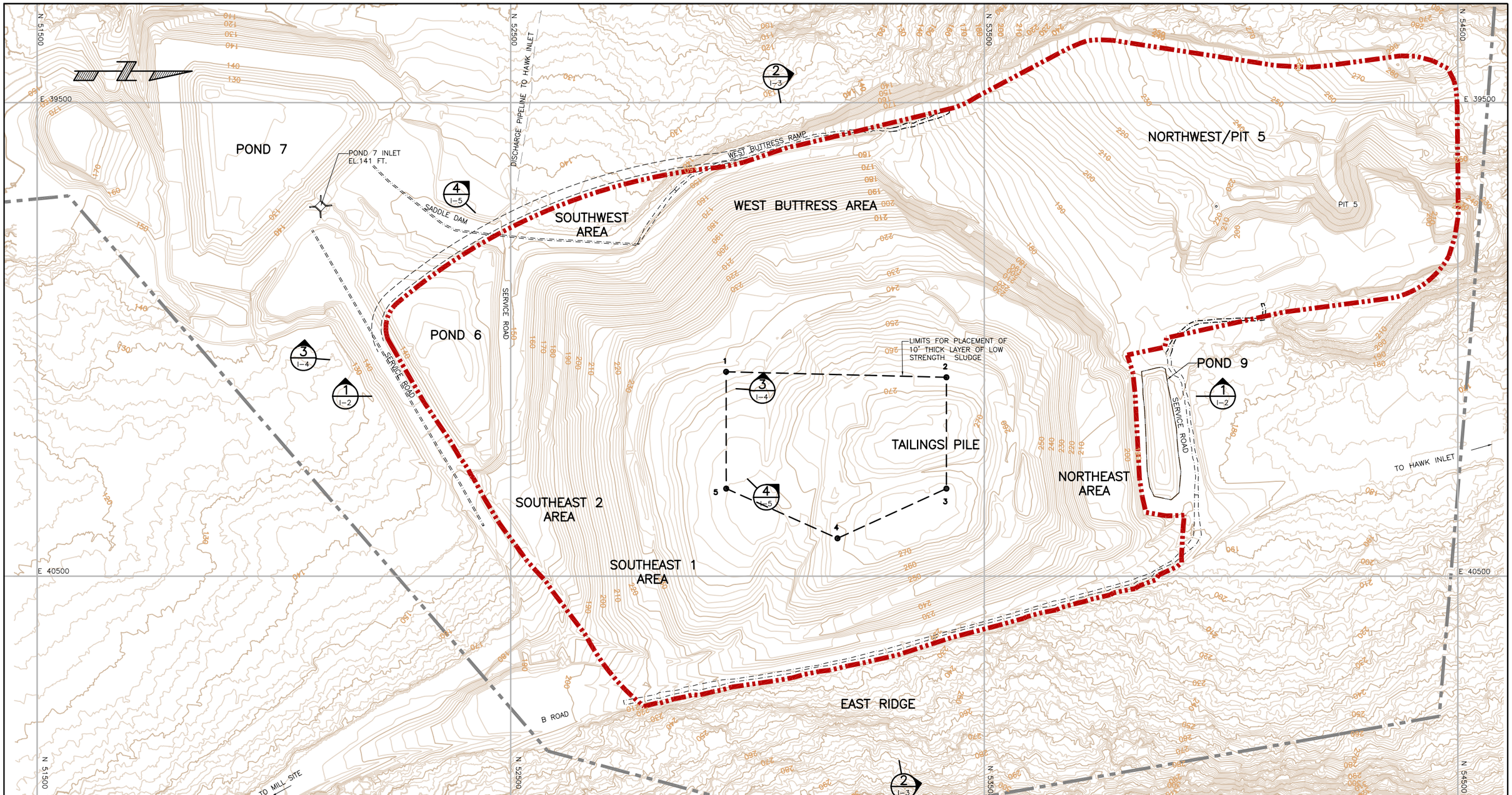
APPENDIX I

Slope Stability Plan and Sections

Figure I-1 Stability Plan
Figure I-2 Stability Section 1 (N to S)
Figure I-3 Stability Section 1 (S to N)
Figure I-4 Stability Section 2- With East Ridge (E to W)
Figure I-5 Stability Section 2- With East Ridge (W to E)
Figure I-6 Stability Section 2- Without East Ridge (W to E)
Figure I-7 Stability Section 3a
Figure I-8 Stability Section 3b
Figure I-9 Stability Section 4



LEGEND

- PROPOSED TAILINGS EXPANSION
- EXISTING SOIL–BENTONITE WALL
- LEASE BOUNDARY
- PROPOSED ROAD

GENERAL NOTES:

1. BASE TOPOGRAPHY PROVIDED BY GCMC (FEBRUARY, 2008).
2. LOCATIONS AND ELEVATIONS ARE IN FEET UNLESS OTHERWISE NOTED.

POINT	NORTHING (FT.)	EASTING (FT.)
1	52 955.0	40 068.0
2	53 420.0	40 080.0
3	53 420.0	40 315.0
4	53 190.0	40 420.0
5	52 955.0	40 315.0

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED APRIL 14, 2008

SCALE: 0 200 FT.

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CLIENT: GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT: STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE: STABILITY PLAN

PROJECT No. M07802A54

FIG. No. I–1

KCB–R–ILD

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Date: 4/14/2008
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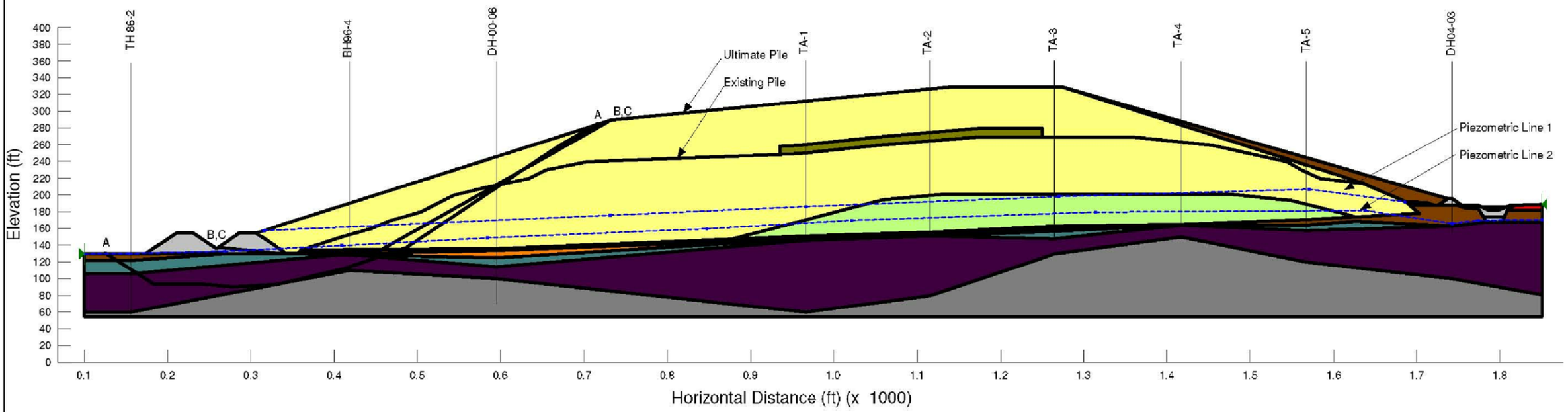
Xrefs: BM_Greens CK_FEB2008



SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		RESIDUAL SHEAR STRENGTH PARAMETERS		POST EARTHQUAKE SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		φ'	c' (psf)	φ'	c' (psf)	φ'	c' (psf)		
	Tailings	39	0	32	0	28[1]	0	128	1
	Old Tailings	33	0	32	0	Function [2]		120	1
	Peat	27	0	27	0	27	0	67	2
	Sand and Gravel	33	0	33	0	Function[3]		120	2
	Sand	33	0	33	0	33	0	120	2
	Silty Clay	30	0	30	0	30	0	120	2
	Silty Sandy Till	33	0	33	0	33	0	120	2
	Compacted Rockfill	40	0	40	0	40	0	120	1
	Uncompacted Rockfill	38	0	38	0	38	0	120	1
	Water	0	0	0	0	0	0	62.4	1
	Sludge	0	0	0	0	0	0	120	2
	Bedrock	Impenetrable							1

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	2.17	PEAK SHEAR STRENGTH PARAMETERS
B-B	1.85	RESIDUAL SHEAR STRENGTH PARAMETERS
C-C	1.74	POST LIQUIFIED SHEAR STRENGTH PARAMETERS

NOTES:

1. POST–EARTHQUAKE TAILINGS ABOVE THE WATER TABLE HAVE RESIDUAL STRENGTH VALUES.
2. THE STRENGTH, BELOW THE WATER TABLE, IS SPECIFIED AS A FUNCTION WHERE POST–LIQUEFACTION RESIDUAL STRENGTH IS A FUNCTION OF DEPTH AND VARIES FROM 324 PSF TO 2297 PSF AT 140 FT DEPTH.
3. THE MAXIMUM SHEAR STRENGTH IS 1640 psf.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED APRIL 14, 2008

SCALE NTS

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CLIENT: GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT: STAGE 2 TAILINGS EXPANSION 2008 STAGE 2 DESIGN

TITLE: STABILITY SECTION 1 (NORTH TO SOUTH)

PROJECT No. M07802A54

FIG. No. I–2

KCB–R–ILD

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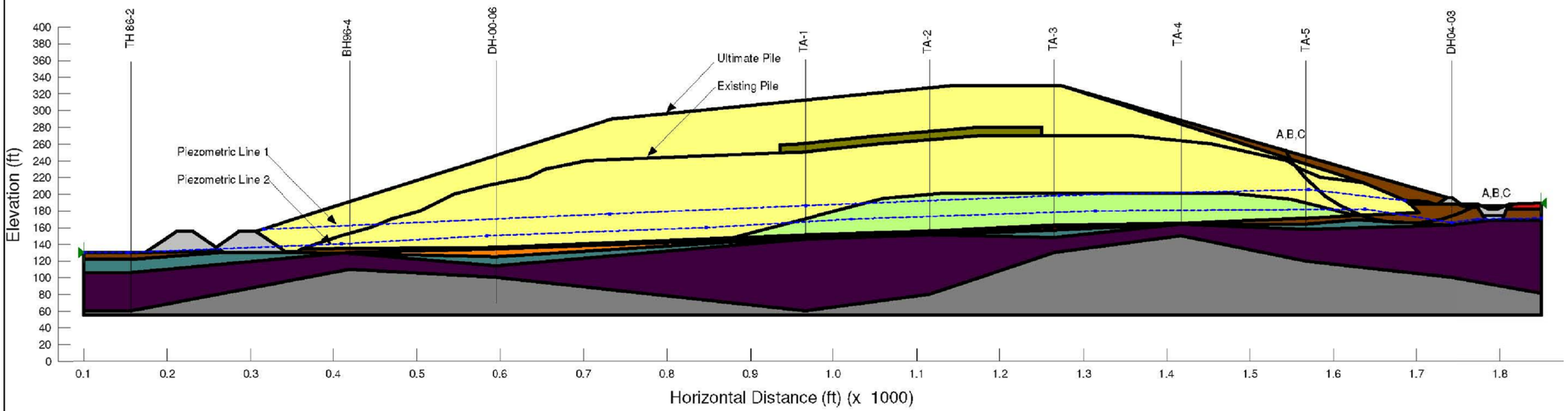

SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		RESIDUAL SHEAR STRENGTH PARAMETERS		POST EARTHQUAKE SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		φ'	c' (psf)	φ'	c' (psf)	φ'	c' (psf)		
	Tailings	39	0	32	0	28[1]	0	128	1
	Old Tailings	33	0	32	0	Function [2]		120	1
	Peat	27	0	27	0	27	0	67	2
	Sand and Gravel	33	0	33	0	Function[3]		120	2
	Sand	33	0	33	0	33	0	120	2
	Silty Clay	30	0	30	0	30	0	120	2
	Silty Sandy Till	33	0	33	0	33	0	120	2
	Compacted Rockfill	40	0	40	0	40	0	120	1
	Uncompacted Rockfill	38	0	38	0	38	0	120	1
	Water	0	0	0	0	0	0	62.4	1
	Sludge	0	0	0	0	0	0	120	2
	Bedrock	Impenetrable							1

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	2.00	PEAK SHEAR STRENGTH PARAMETERS
B-B	1.77	RESIDUAL SHEAR STRENGTH PARAMETERS
C-C	1.65	POST LIQUIFIED SHEAR STRENGTH PARAMETERS

NOTES:

1. POST–EARTHQUAKE TAILINGS ABOVE THE WATER TABLE HAVE RESIDUAL STRENGTH VALUES.
2. THE STRENGTH, BELOW THE WATER TABLE, IS SPECIFIED AS A FUNCTION WHERE POST–LIQUEFACTION RESIDUAL STRENGTH IS A FUNCTION OF DEPTH AND VARIES FROM 324 PSF TO 2297 PSF AT 140 FT DEPTH.
3. THE MAXIMUM SHEAR STRENGTH IS 1640 psf.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED APRIL 14, 2008

SCALE NTS

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CLIENT
GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT
STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE
STABILITY SECTION 1
(SOUTH TO NORTH)

PROJECT No. M07802A54

FIG. No. 1–3

KCB–R–LD

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Date: 4/14/2008
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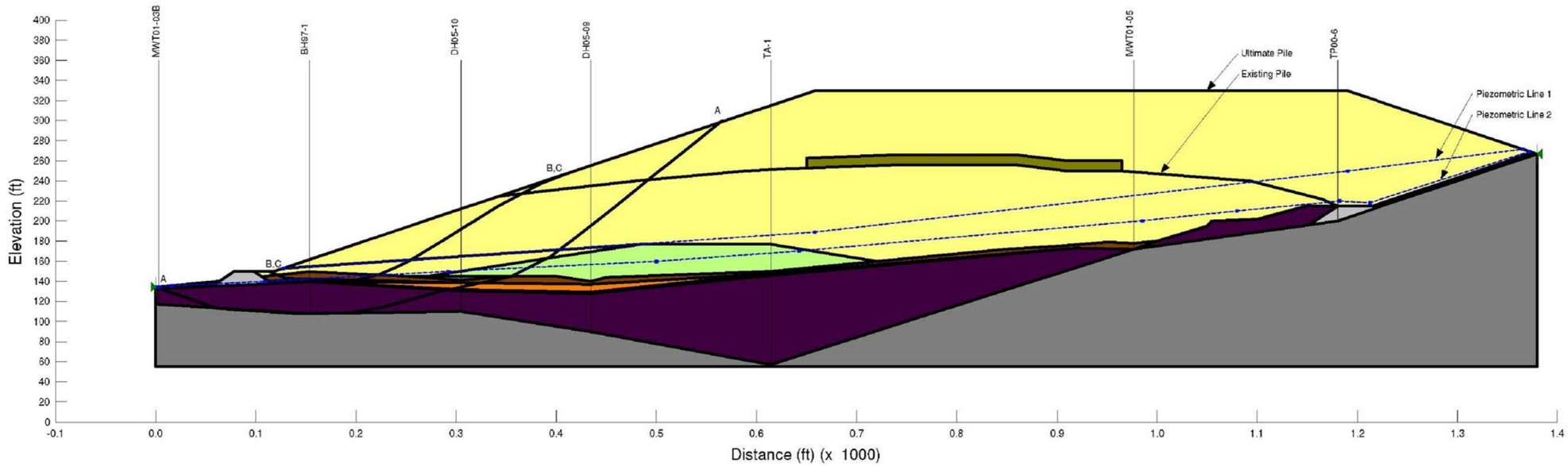

SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		RESIDUAL SHEAR STRENGTH PARAMETERS		POST EARTHQUAKE SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		φ'	c' (psf)	φ'	c' (psf)	φ'	c' (psf)		
	Tailings	39	0	32	0	28[1]	0	128	1
	Old Tailings	33	0	32	0	Function [2]		120	1
	Peat	27	0	27	0	27	0	67	2
	Sand and Gravel	33	0	33	0	Function[3]		120	2
	Sand	33	0	33	0	33	0	120	2
	Silty Clay	30	0	30	0	30	0	120	2
	Silty Sandy Till	33	0	33	0	33	0	120	2
	Compacted Rockfill	40	0	40	0	40	0	120	1
	Sludge	0	0	0	0	0	0	120	1
	Geosynthetic Liner	23.8	0	20	0	20	0	125	2
	Bedrock	Impenetrable							2

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	1.97	PEAK SHEAR STRENGTH PARAMETERS
B-B	1.60	RESIDUAL SHEAR STRENGTH PARAMETERS
C-C	1.57	POST LIQUIFIED SHEAR STRENGTH PARAMETERS

NOTES:

1. POST-EARTHQUAKE TAILINGS ABOVE THE WATER TABLE HAVE RESIDUAL STRENGTH VALUES.
2. THE STRENGTH, BELOW THE WATER TABLE, IS SPECIFIED AS A FUNCTION WHERE POST-LIQUEFACTION RESIDUAL STRENGTH IS A FUNCTION OF DEPTH AND VARIES FROM 324 PSF TO 2297 PSF AT 140 FT DEPTH.
3. THE MAXIMUM SHEAR STRENGTH IS 1640 psf.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED APRIL 14, 2008

SCALE NTS

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CLIENT

GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT

STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE

STABILITY SECTION 2
WITH EAST RIDGE
(EAST TO WEST)

PROJECT No. M07802A54

FIG. No. I-4

KCB-R-ILD

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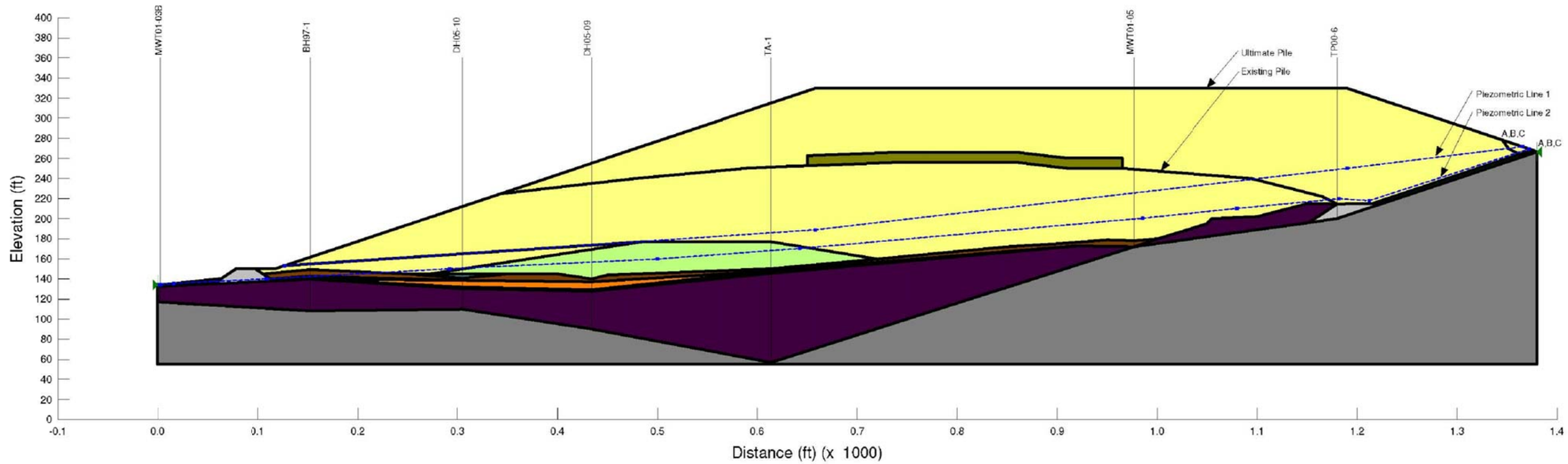

SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		RESIDUAL SHEAR STRENGTH PARAMETERS		POST EARTHQUAKE SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		ϕ'	c' (psf)	ϕ'	c' (psf)	ϕ'	c' (psf)		
	Tailings	39	0	32	0	28[1]	0	128	1
	Old Tailings	33	0	32	0	Function [2]		120	1
	Peat	27	0	27	0	27	0	67	2
	Sand and Gravel	33	0	33	0	Function[3]		120	2
	Sand	33	0	33	0	33	0	120	2
	Silty Clay	30	0	30	0	30	0	120	2
	Silty Sandy Till	33	0	33	0	33	0	120	2
	Compacted Rockfill	40	0	40	0	40	0	120	1
	Sludge	0	0	0	0	0	0	120	1
	Geosynthetic Liner	23.8	0	20	0	20	0	125	2
	Bedrock	Impenetrable							2

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	1.86	PEAK SHEAR STRENGTH PARAMETERS
B-B	1.49	RESIDUAL SHEAR STRENGTH PARAMETERS
C-C	1.41	POST LIQUIFIED SHEAR STRENGTH PARAMETERS

NOTES:

1. POST-EARTHQUAKE TAILINGS ABOVE THE WATER TABLE HAVE RESIDUAL STRENGTH VALUES.
2. THE STRENGTH, BELOW THE WATER TABLE, IS SPECIFIED AS A FUNCTION WHERE POST-LIQUEFACTION RESIDUAL STRENGTH IS A FUNCTION OF DEPTH AND VARIES FROM 324 PSF TO 2297 PSF AT 140 FT DEPTH.
3. THE MAXIMUM SHEAR STRENGTH IS 1640 psf.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED APRIL 14, 2008

SCALE NTS

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CLIENT
GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT
STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE
STABILITY SECTION 2
WITH EAST RIDGE
(WEST TO EAST)

PROJECT No. M07802A54

FIG. No. I-5

KCB-R-ILD

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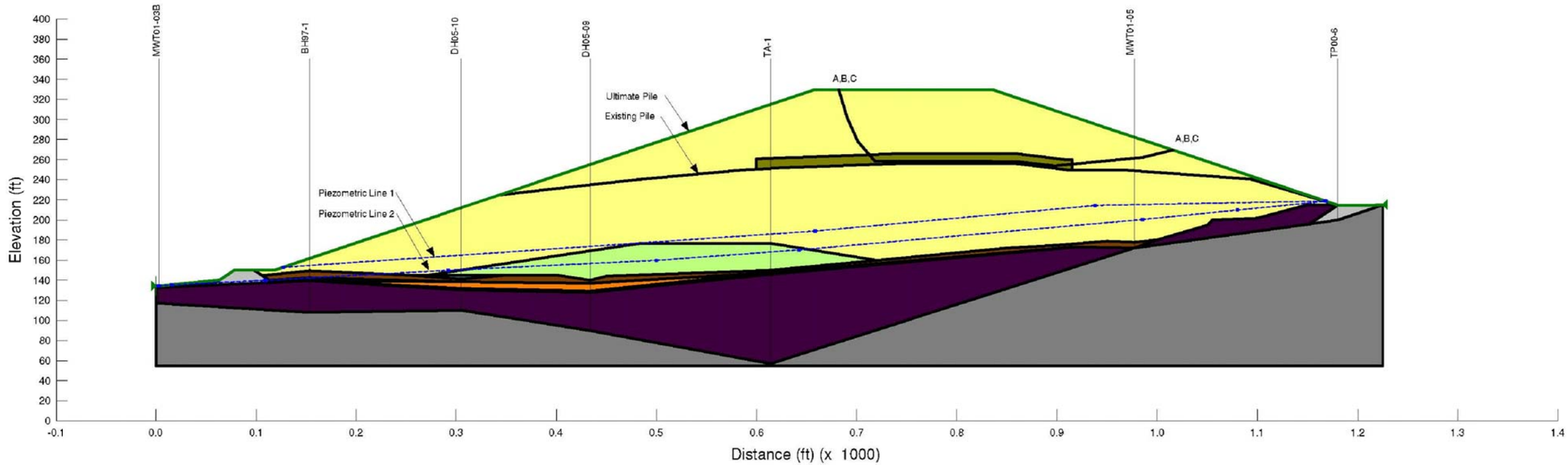

SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		RESIDUAL SHEAR STRENGTH PARAMETERS		POST EARTHQUAKE SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		ϕ'	c' (psf)	ϕ'	c' (psf)	ϕ'	c' (psf)		
	Tailings	39	0	32	0	28[1]	0	128	1
	Old Tailings	33	0	32	0	Function [2]		120	1
	Peat	27	0	27	0	27	0	67	2
	Sand and Gravel	33	0	33	0	Function[3]		120	2
	Sand	33	0	33	0	33	0	120	2
	Silty Clay	30	0	30	0	30	0	120	2
	Silty Sandy Till	33	0	33	0	33	0	120	2
	Compacted Rockfill	40	0	40	0	40	0	120	1
	Sludge	0	0	0	0	0	0	120	1
	Geosynthetic Liner	23.8	0	20	0	20	0	125	2
	Bedrock	Impenetrable							2

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	1.77	PEAK SHEAR STRENGTH PARAMETERS
B-B	1.37	RESIDUAL SHEAR STRENGTH PARAMETERS
C-C	1.37	POST LIQUIFIED SHEAR STRENGTH PARAMETERS

NOTES:

1. POST-EARTHQUAKE TAILINGS ABOVE THE WATER TABLE HAVE RESIDUAL STRENGTH VALUES.
2. THE STRENGTH, BELOW THE WATER TABLE, IS SPECIFIED AS A FUNCTION WHERE POST-LIQUEFACTION RESIDUAL STRENGTH IS A FUNCTION OF DEPTH AND VARIES FROM 324 PSF TO 2297 PSF AT 140 FT DEPTH.
3. THE MAXIMUM SHEAR STRENGTH IS 1640 psf.

NOT FOR CONSTRUCTION

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SCALE NTS

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CLIENT
GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT
STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE
STABILITY SECTION 2
WITHOUT EAST RIDGE
(WEST TO EAST)

PROJECT No. M07802A54

FIG. No. I-6

KCB-R-ILD

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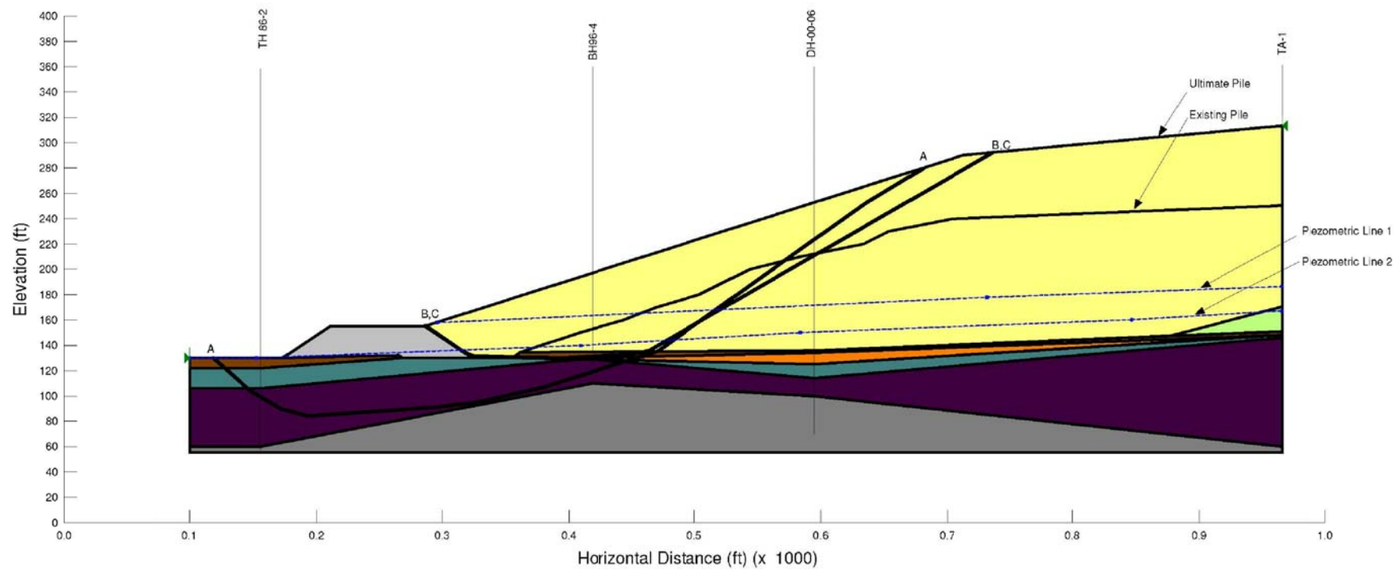

SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		RESIDUAL SHEAR STRENGTH PARAMETERS		POST EARTHQUAKE SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		φ'	c' (psf)	φ'	c' (psf)	φ'	c' (psf)		
	Tailings	39	0	32	0	28[1]	0	128	1
	Old Tailings	33	0	32	0	Function [2]		120	1
	Peat	27	0	27	0	27	0	67	2
	Sand	33	0	33	0	33	0	120	2
	Silty Clay	30	0	30	0	30	0	120	2
	Silty Sandy Till	33	0	33	0	33	0	120	2
	Compacted Rockfill	40	0	40	0	40	0	120	1
	Sludge	0	0	0	0	0	0	120	1
	Bedrock	Impenetrable							2

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	2.06	PEAK SHEAR STRENGTH PARAMETERS
B-B	1.81	RESIDUAL SHEAR STRENGTH PARAMETERS
C-C	1.70	POST LIQUIFIED SHEAR STRENGTH PARAMETERS

NOTES:

1. POST-EARTHQUAKE TAILINGS ABOVE THE WATER TABLE HAVE RESIDUAL STRENGTH VALUES.
2. THE STRENGTH, BELOW THE WATER TABLE, IS SPECIFIED AS A FUNCTION WHERE POST-LIQUEFACTION RESIDUAL STRENGTH IS A FUNCTION OF DEPTH AND VARIES FROM 324 PSF TO 2297 PSF AT 140 FT DEPTH.
3. THE MAXIMUM SHEAR STRENGTH IS 1640 psf.

NOT FOR CONSTRUCTION

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SCALE NTS

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CLIENT
GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT
STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE
STABILITY SECTION 3A
ULTIMATE BUILDOUT

PROJECT No. M07802A54

FIG. No. I-7

KCB-R-LD

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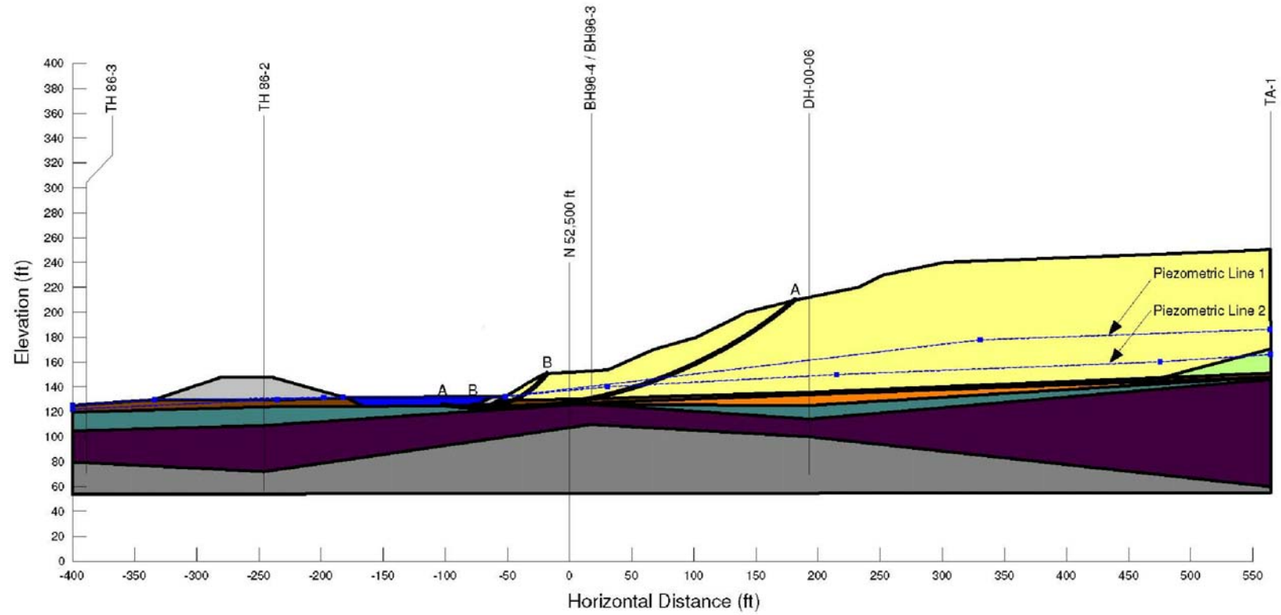

SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		ϕ'	c' (psf)		
	Tailings	39	0	128	1
	Old Tailings	33	0	120	1
	Peat	27	0	67	2
	Sand	33	0	120	2
	Silty Clay	30	0	120	2
	Silty Sandy Till	33	0	120	2
	Compacted Rockfill	40	0	120	1
	Sludge	0	0	120	1
	Bedrock	Impenetrable			2

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	1.91	PEAK STATIC STRENGTH PARAMETERS
B-B	1.31	PEAK STATIC STRENGTH PARAMETERS

NOT FOR CONSTRUCTION

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CLIENT

GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT

STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE

STABILITY SECTION 3B
TEMPORARY EXCAVATION

PROJECT No. M07802A54

FIG. No. I-8

KCB-R-ILD

Time: 11:22:13
Date: 4/14/2008
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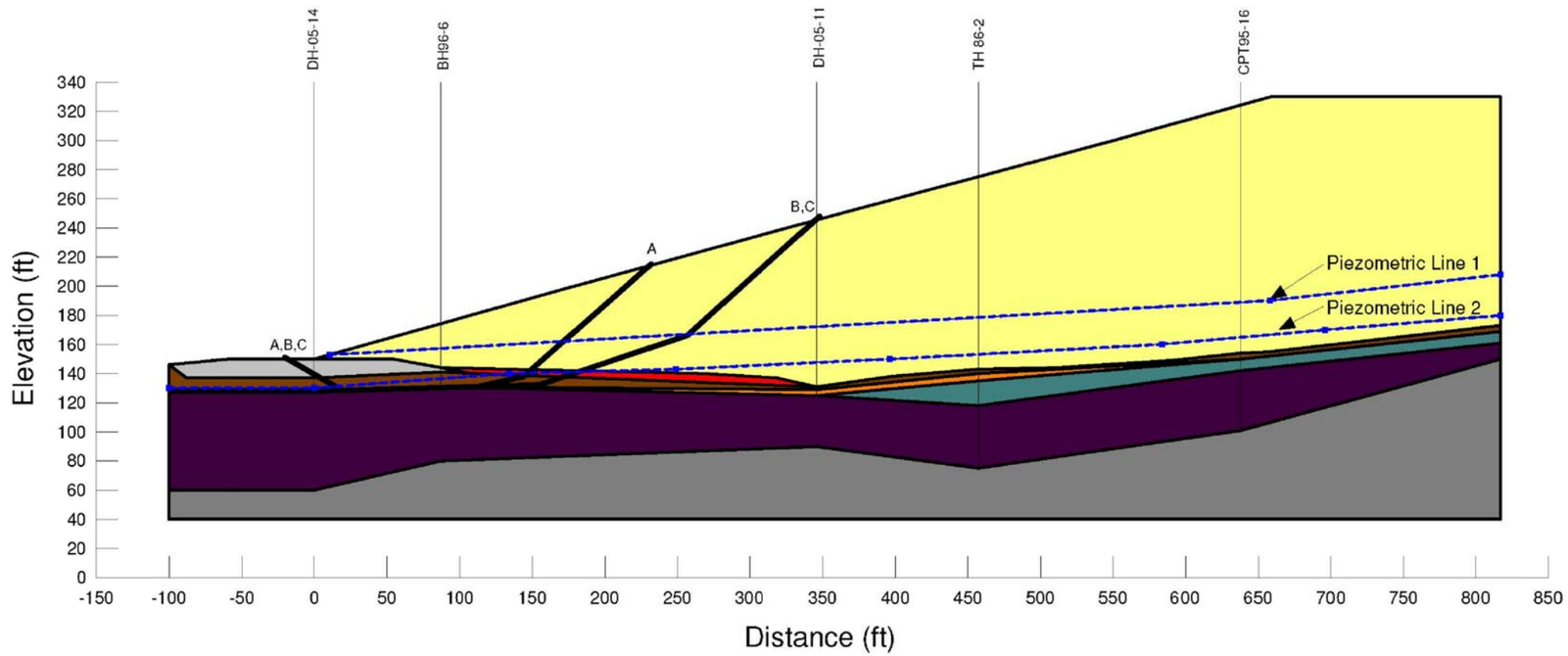

SOIL TYPE AND DESCRIPTION

SECTION COLOR	SOIL TYPE	PEAK SHEAR STRENGTH PARAMETERS		RESIDUAL SHEAR STRENGTH PARAMETERS		POST EARTHQUAKE SHEAR STRENGTH PARAMETERS		UNIT WEIGHT (pcf)	PIEZOMETRIC SURFACE
		φ'	c' (psf)	φ'	c' (psf)	φ'	c' (psf)		
	Tailings	39	0	32	0	28[1]	0	128	1
	Old Tailings	33	0	32	0	Function [2]		120	1
	Peat	27	0	27	0	27	0	67	2
	Sand and Gravel	33	0	33	0	Function[3]		120	2
	Sand	33	0	33	0	33	0	120	2
	Silty Clay	30	0	30	0	30	0	120	2
	Silty Sandy Till	33	0	33	0	33	0	120	2
	Compacted Rockfill	40	0	40	0	40	0	120	1
	Sludge	0	0	0	0	0	0	120	1
	Bedrock	Impenetrable							2

CALCULATED FACTOR OF SAFETY

SECTION	FACTOR OF SAFETY	STRENGTH PARAMETER
A-A	2.03	PEAK SHEAR STRENGTH PARAMETERS
B-B	1.90	RESIDUAL SHEAR STRENGTH PARAMETERS
C-C	1.81	POST LIQUIFIED SHEAR STRENGTH PARAMETERS

NOTES:

1. POST-EARTHQUAKE TAILINGS ABOVE THE WATER TABLE HAVE RESIDUAL STRENGTH VALUES.
2. THE STRENGTH, BELOW THE WATER TABLE, IS SPECIFIED AS A FUNCTION WHERE POST-LIQUEFACTION RESIDUAL STRENGTH IS A FUNCTION OF DEPTH AND VARIES FROM 324 PSF TO 2297 PSF AT 140 FT DEPTH.
3. THE MAXIMUM SHEAR STRENGTH IS 1640 psf.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED APRIL 14, 2008

SCALE NTS

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CLIENT
GREENS CREEK MINING COMPANY

Klohn Crippen Berger

PROJECT
STAGE 2 TAILINGS EXPANSION
2008 STAGE 2 DESIGN

TITLE
STABILITY SECTION 4

PROJECT No. M07802A54

FIG. No. I-9

KCB-R-LD

Time: 11:22:13
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