Klohn Crippen Berger

February 12, 2007

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

Mr. Tom Zimmer
Surface Operation Manager

Dear Mr. Zimmer:

Stage 2 Tailings Pile Expansion
Northwest/Pit 5 and Northeast Expansion Area Design Overview

1. INTRODUCTION

Greens Creek Mine is an underground polymetallic (zinc, silver, gold and lead) mine on northern Admiralty Island, Alaska (Drawing D-47001) that is owned and operated by Kennecott Greens Creek Mining Company (KGCMC). Mine tailings are dewatered at the mill site; about one-half of the tailings are utilized as backfill in the mine, and the remainder is 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 5 main areas known as the Southeast, Northeast, Northwest/Pit 5, Pond 6, and the Southwest expansion areas. Work began in 2004 and design and construction continues through 2007. As each area is developed, detailed designs are prepared taking into account overall requirements regarding 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, KGCMC 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 USDA 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) (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 and Pond 7 Expansion areas in 2006. (Klohn Crippen, 2005)

Expansion activities planned for 2007 include development of a geosynthetic liner foundation for tailings storage in the Northwest/Pit 5 area and construction of a lined retention pond in the Northeast area. The perimeter road will be extended around the new tailings pile toe. Raises and extensions of existing slurry walls will be done as required to maintain containment of contact water. This design letter report provides a general outline of the design for these activities, including a stability summary, and details of the planned 2007 expansion construction. Drawing D-47002 shows the general arrangement of the Stage 2 Expansion, including the updated Expansion footprint.

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2. GEOSYNTHETIC LINER

2.1 Geosynthetic Liner Testing Program

A lab testing program to assess the shear strength between various geosynthetic and earth fill materials was completed in 2006. Tests that included a geosynthetic surface were completed in accordance with ASTM D5321.

In the 2005 geosynthetic liner design that was constructed in the Southeast expansion area, the liner interface with the lowest frictional strength was the interface between the textured geomembrane and the geocomposite. To increase the frictional strength along the geomembrane and geocomposite interface for the Northwest/Pit 5 expansion, an interlayer of granular material will be placed between the two liner surfaces in areas where the grade of the foundation is less than 15%. A series of laboratory tests were completed to assess the strength benefit that can be expected by the inclusion of the granular interlayer. A gradation of the granular material is presented in Figure 2.1.

Results of the 2006 liner strength testing program are summarized in Table 2.1.

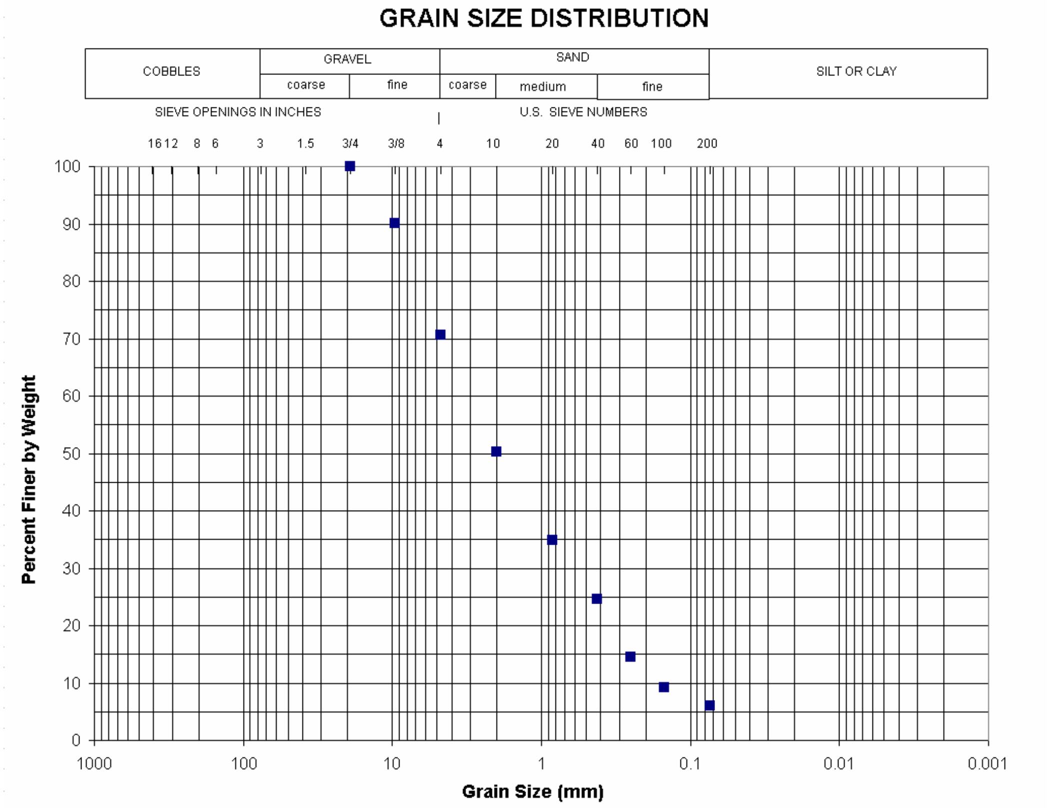

Figure 2.1 Gradation of Granular Interlayer and Bedding/Service Layer

Table 2.1 Summary of 2006 Geosynthetic Liner Testing Program

Material in Top of Shear Box	Material in Bottom of Shear Box	Peak Strength		"Residual" Strength	
		ϕ'	c' (psf)	ϕ'	c' (psf)
Poly Flex **Geocomposite** - 200mil mesh 8oz/8oz	Poly Flex HDPE Geomembrane - 80mil Textured	25.5°	0	11.8°	0
Poly Flex **Geocomposite** - 200mil mesh 8oz/8oz	Poly Flex HDPE **Geomembrane** - 80mil Textured w/ **Granular Interlayer**	23.8°(1,2)	0	See Note 2	See Note 2
Bedding/Service Layer	Poly Flex HDPE **Geomembrane** - 80mil Textured	28.8°(2)	383	See Note 2	See Note 2
Poly Flex **Geocomposite** - 200mil mesh 8oz/8oz	Bedding/Service Layer	32.4°(2)	0	See Note 2	See Note 2

Notes:
1. Sample sheared at interface between geomembrane and granular interlayer.
2. Test did not show any apparent strength reduction to a residual value during shearing. Large displacement test results are reported as peak strength.

A direct shear testing program was undertaken, during which geomembrane samples were subjected to normal loading similar to the maximum loading expected beneath the completed tailings pile. After the direct shear testing program was completed, samples of the geomembrane material that were sheared against the granular interlayer were collected from the lab and inspected for damage. None of the samples showed any indication that the granular material had punctured the geomembrane or damaged it sufficiently to harm its effectiveness as barrier to fluid flow. Scour caused by the displacement of soil particles along the geomembrane surface was observed to occur mainly in the texturing and did not penetrate severely (>10% of thickness) into the geomembrane. Degree of scour was observed to increase with applied normal stress.

2.2 Geosynthetic Liner Design

The following liner designs were selected for areas where the slope of the foundation is less than 15% (Figure 2.2) and greater than 15% (Figure 2.3). The extent of the liner placement areas are shown on Drawing D-47003.

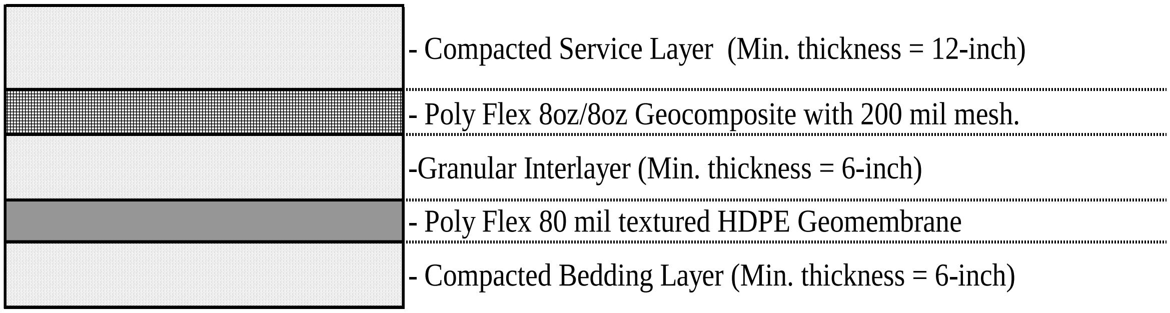

Figure 2.2 Schematic of Liner Design where Foundation Slope < 15%

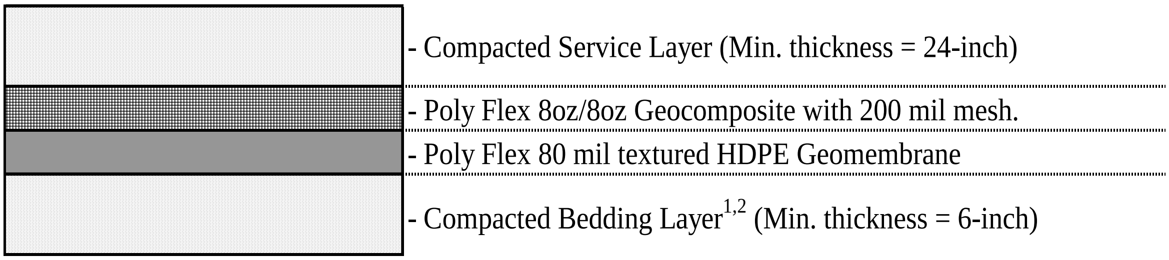

Figure 2.3 Schematic of Liner Design where Foundation Slope > 15%

The revisions to the liner design from the 2005 design include the following:

- Granular interlayer was added between geocomposite and geomembrane interface in areas where the foundation slope is less than 15%;
- Geotextile between geomembrane and bedding sand was omitted.

Peak and residual friction angles for the geosynthetic liner systems presented in Figure 2.2 and 2.3 were selected based on the 2006 testing program and are summarized in Table 2.2. Design friction angles from the 2005 design are also included in Table 2.2.

Table 2.2 Summary of Geosynthetic Liner Strength Parameters

LINER INTERFACE	PEAK STRENGTH		RESIDUAL STRENGTH	
	ϕ'	c' (psf)	ϕ'	c' (psf)
2006 Liner (no granular interlayer)	25.5°	0	11.8°	0
2006 Liner w/ granular interlayer	23.8°	0	20.0°(1)	0
2005 Liner Design	24.2°	0	12.5°	0

Notes:
1. Assuming coverage of granular interlayer over 83% of the liner area the design friction angle was reduced from tested value of 23.8° to 20° (0.83*tan(23.8°)=tan(20°)).

3. NORTH TO WEST CORRIDOR

A series of pipes will convey contact surface water through the Northwest Excavation area from the Northeast Retention Pond to the West Buttress ditch along an alignment called the North to West Corridor, as shown on Drawings D-47004 and D-47011. The pipes included in the corridor are:

- Solid 18-inch pipe from Northeast Retention Pond;
- Solid 8-inch pipe for DB04;
- Perforated 8-inch pipe from Pit 5 below liner drain;
- Solid 8-inch NE below liner drain (NOTE: this pipe transitions from being perforated through the NE Expansion to being solid in the North to West Corridor); and
- Solid 8-inch spare pipe from Northeast Retention Pond.

Pipe grades were designed to reduce the length and depth of trenching required through bedrock and till, while still keeping the pipe below the Hydraulic Grade Line established between the inlet and outlet of the 18-inch pipe, as directed by Environmental Design Engineering (EDE).

A lean mix cement backfill will provide load support throughout the corridor, while minimizing the width requirements for pipe trenches. Capacity and sizing of the pipes was based on hydrological analyses by EDE (Environmental Design Engineering (2002 and 2003).

4. NORTHWEST - PIT 5 EXPANSION AREA DESIGN

4.1 Excavation and Fill Plan

Excavation and fill grades in the NorthWest/Pit 5 expansion area were established to facilitate placement of the geosynthetic liner system on slopes no steeper that 3H:1V, while minimizing the volume of rock cuts required. The base of Pit 5 is designed with a minimum slope of 1.3% for drainage, as described in Section 5.1. Slope stability was the key parameter in the development of the foundation contour. Slope stability modelling is summarized in Section 4.4. The foundation grading plan is shown on Drawing D-47003.

Construction of the liner foundation is planned to be undertaken in two stages. In 2007 drilling and blasting will occur south of the existing water treatment plant (WTP), with enough of a distance buffer to protect the WTP during drill and blast operations and to keep it operational for one more season. Liner will be placed in areas where the foundation at grade. The removal of the WTP and completion of the NW/Pit 5 lined foundation will occur during the 2008 construction season.

In 2006, excavation of the tailings in the northwest corner of the existing tailings pile began in order to investigate two areas where tailings may be in direct contact with bedrock and to provide space to install a geosynthetic liner in portions of the tailings pile foundation in the northwest corner. Excavation will continue in 2007 to find the extents of these unlined bedrock areas. The liner will be tied in to the Northwest/Pit 5 liner system to the north, and to natural till to the south. Grading for the liner foundation in this area will be constrained by maximum (3H:1V) and minimum (2%) slope criteria, as directed in Section 4.6 of the Technical Specifications, however the final contour will reflect the field conditions exposed by the tailings excavation in order to reduce the need for bedrock excavation as much as possible.

4.2 Roads

A perimeter road around the NW/Pit 5 expansion area is shown on Drawing D-47003. Details of the road construction including dimensions and fill materials are shown on Drawing D-47010. To collect contaminated seepage that may flow through the road fill, the liner has been extended under the road and sloped back into the tailings pile.

The maximum grade of the perimeter roads was set at 10% to allow for safe travel of mine vehicles. The grade of the natural slopes along the western edge is greater than 10%. To facilitate a road grade of 10% a rockfill berm along the road alignment in the southwest corner of the NW/Pit 5 area is required to raise the road elevation. Details of the berm are shown on Drawing D-47013.

4.3 Drainage

Drainage for the NW/Pit 5 expansion area can be separated into three components: surface drainage; above liner drainage; and below liner drainage.

Surface drainage will be controlled by perimeter ditches that run parallel to the perimeter road (refer to Drawing D-47010). Ditches along the north and east edges will route the flow into the proposed retention pond in the NE area. Ditches along the west edge will connect to the existing West Buttress ditch and flow directly to Pond 7. The new ditches are within the footprint of the liner.

Above liner drainage within the tailings pile will be collected and flow within the service layer which includes 12-inches of granular material and a layer of geocomposite. This flow will include seepage from the tailings but the primary source of water is expected to be at the edges of the tailings pile from infiltration of surface water and runoff that is not collected by the surface ditches. Flow within the service layer will be collected in a series of finger drain pipes along the base of the tailing placement area (refer to Drawings D-47004 and D-47012). The pipes discharge into the West Buttress ditch that flows to Pond 7. The drain pipes specified are 8-inch HDPE DR9 heavy walled (0.96-inch) pipes to meet manufacturer recommended deformation criteria under the ultimate load from the tailings.

Below liner drainage within the tailings pile will be collected and flow within the bedding layer which includes 6-inches of granular material. This flow will include seepage from the tailings but the primary source of water is expected to be at the edges of the tailings pile from infiltration of surface water and runoff that is not collected by the surface ditches. Similar to the above liner drainage, the flow within the bedding layer will be collected in a series of finger drain pipes along the base of the tailing placement area (refer to Drawing D-47012) that discharge into the West Buttress ditch that flows to Pond 7. The drain pipes specified are 8-inch HDPE DR9 heavy walled (0.96-inch) pipes to meet manufacturer recommended deformation criteria under the ultimate tailings load.

4.4 Slope Stability

A series of cross sections through the NW/Pit 5 expansion area were analyzed to assess the stability of the proposed design against slope failure. These sections are shown in plan and section views in Appendix II. The cross sections presented in Klohn Crippen, 2006 for the NW expansion area were updated for this analysis.

For the following reasons the foundation below the bedding layer was considered a hard layer in the stability analysis sections, and potential failure planes were not considered to extend into the foundation.

- Critical failure surfaces for the area will, most likely, occur within the geosynthetic liner system along the geomembrane-granular interlayer interface because the design friction angle (20°) is substantially less than any of the other earth materials.
- The foundation of the expansion is expected to consist mainly of competent bedrock.

The material properties for the liner interfaces were taken from testing as described in Section 3, parameters for the other materials were taken directly from the Static-Peak, Static-Residual and Post-Earthquake design conditions in Klohn Crippen, 2006. Table 4.1 is a summary of those material properties. The phreatic surface within the geosynthetic liner was assumed to be a 3ft above the liner along the entire length. The phreatic surface in the tailings was assumed to be a maximum height of 35ft above the liner at the centre of the expansion area and decreasing linearly to 3ft above the toe.

Table 4.1 Material Properties (Klohn Crippen, 2006)

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	Function [1]	
Old Tailings	120	33	32	0	Function [2]	
Geosynthetic Liner System	125	See Table 3.2		0	N/A[4]	N/A
Sand Drainage Blanket	120	40	N/A[4]	0	N/A	N/A
Peat (Unit 6)	67	27	N/A	0	N/A	N/A
Sand and Gravel (Unit 5)	120	33	N/A	0	Function [3]	
Sand (Unit 4)	120	33	N/A	0	N/A	N/A
Silty Clay (Unit 3)	120	30	N/A	0	N/A	N/A
Silty Sandy Till (Unit 2)	120	33	N/A	0	N/A	N/A
Main Embankment Till	120	33	N/A	0	N/A	N/A
Compacted Rockfill/Road Fill	120	40	N/A	0	N/A	N/A
Roadfill/Native	130	36	N/A	0	N/A	N/A

Notes:
1. The best estimate of post earthquake MDE strength for new tailings is that pore pressure would rise 33% over static conditions for material below the water table.
2. 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).
3. The maximum shear strength is 1640 psf.
4. N/A indicates that the soil does not liquefy during the MDE, therefore static properties were maintained in the post-liquefaction analysis.

Results from the stability analysis are summarized in Table 4.2. The minimum target Factors of Safety (FOS) are defined in the Design Overview for Forest Service Submission (Klohn Crippen, 2004).

Table 4.2 Northwest/Pit 5 Expansion Area Stability Analysis Results

SECTION	CALCULATED FACTOR OF SAFETY		
	Static-Peak	Static-Residual	Post-Earthquake
	FOS $\geq$ 1.5	FOS $\geq$ 1.3	FOS $\geq$ 1.1
H (East to West)[1]	1.5	1.3	1.1
H (West to East)[1]	1.5	1.3	1.1
X1	1.9	1.6	1.6
X2	2.0	1.7	1.6
Y[1]	1.5	1.3	1.1

Note:
1. Shallow toe sloughs with lower FOS were generated during modeling. The slip surfaces represented by these FOS are ones where all slip surfaces with a lower FOS are smaller. The slip surfaces represented by the given FOS are shown in Appendix II.

Based on the material properties presented in Table 4.1 and the assumed phreatic levels, the proposed NW/Pit 5 expansion area development plan meets stability design criteria.

5. NORTHEAST EXPANSION AREA DESIGN

5.1 Retention Pond

To replace the existing North Retention Pond, a lined surface water retention pond will be located inside of containment south of the Northeast slurry wall. To maximize tailings storage the pond will be rectangular in shape and built as close to the existing slurry wall as possible without compromising the stability of the wall. The base of the pond will be a minimum offset of 24 ft. from the slurry wall as shown on Drawing D-47014. Peat below the pond base will be entirely excavated and replaced with gravel to control settlement. The pond will be located such that the base of the pond is a minimum of 24 feet from the existing slurry wall, so temporary side slopes will not exceed 2H:1V, as shown in Drawing D-47014.

The pond will be fed by the perimeter ditch from Pit 5 and the perimeter ditch running along the east toe of the tailings pile. It will consist of a 180 ft. by 50 ft. settling pond lined with reinforced polypropylene, requiring an excavation to El. 180 ft, allowing for 0.25 ac-ft of storage as required for the 25 yr-24 hr storm event as per EDE (2002). A 6-inch layer of bedding sand underlying an 18-inch layer of rock armoring will protect the liner.

A pond outlet set at El. 185 ft. leading to a 6-ft diameter sump will decant the water from the settling pond to the inlet to an 18-inch HDPE gravity pipe from the sump. The pipe inlet will be set at el. 177 ft. The 18-inch pipe will transport the water from the sump to the west buttress ditch. A second outlet from the settling pond will be a 16-inch riser leading to the solid existing 16-inch pipe running south and to Pond 7.

5.2 Drainage

Drainage for the NE expansion area can be separated into two components: surface drainage; and foundation drainage.

Surface drainage will be controlled by perimeter ditches that run along the toe of the final tailings pile footprint (refer to D-47014). The ditches will route flow into the proposed retention pond in the NE area.

Foundation drainage will be controlled by a perforated drain pipe at El. 174ft (invert) that runs parallel to the perimeter road as shown on D-47014. The proposed drain pipe (8-inch HDPE DR9) will be founded in a large zone of rockfill to collect groundwater from the foundation and drain it in two directions.

- The perforated pipe will be connected to the existing French drain that flows to the south, and
- The perforated pipe will transition into a solid pipe that will continue through the proposed N to W corridor.

5.3 Liquefaction Assessment of Shallow Sand

The original design concept for the NE area presented in Klohn Crippen, 2004 included the removal of shallow sands that were expected to be liquefiable within the expansion footprint. This recommendation was made prior to any geotechnical drilling in the area. A drilling program in late 2004 included 4 holes within the expansion area footprint. The shallow sand was identified in 3 of the 4 holes.

In Klohn Crippen, 2006 these sands were identified as potentially liquefiable under MDE. The 2006 analyses were influenced by the presence of a peat layer which complicates the simplified Seed analysis, since the peat is buoyant with a unit weight of about 67 pcf, which is outside the range of normal soils for which Seed’s approach was intended (usually about 100 pcf to 120 pcf). A re-analysis of the liquefaction susceptibility under MDE, detailed in Appendix I, indicated the following:

- The shallow medium dense sand identified in the NE expansion area is not liquefiable under MDE in zones with 5 ft or more of tailings overlying the sand, assuming a maximum foundation water table elevation of 180ft. Including tailings in the analysis reduces the influence of the peat and makes the Seed simplified method more applicable.
- The shallow medium dense sand identified in the NE expansion area with less than 5ft of tailings thickness is potentially liquefiable, assuming a maximum foundation water table elevation of 180ft. This assessment is still influenced by the peat, but is a conservative approach.

The post-liquefaction strength of the shallow sand has been revised to a τ/σ_v' ratio of 0.12. This revision is based on the method proposed by Idriss (2004), where:

$$\frac{\tau}{\sigma'_{vo}} = 0.02 + 0.025\sqrt{(N_1)_{60-CS}} \quad \text{(Idriss, 2004)}$$

This approach was previously proposed for sand beneath the West Buttress (Klohn Crippen, 2006) but was not used since the West Buttress sands were removed or were not considered liquefiable.

As specified in Klohn Crippen (2006), the sand is expected to soften if the factor of safety against liquefaction is less that 1.4. As described in Appendix 1, the friction angle for the softened sand is set at 26 degrees.

These changes regarding the shallow sand in the NE expansion area are discussed further in a memo summarizing the revised analyses which has been attached to this letter as Appendix I. It should be noted that the liquefaction assessment is made for the MDE which is a condition that applies to closure. Further assessment will be made on closure once the final stockpile geometry and water table conditions are established.

5.4 Slope Stability

The slope stability section for the NE expansion area (Section 1a) that was included in the Klohn Crippen (2006) report was revised to include the currently proposed layout and is shown in Appendix I, Figure 4. The current layout includes a new 50ft wide retention pond that runs parallel to the roadway along the toe of the tailings. This new pond offsets the toe of the tailings approximately 70ft south from the south edge of the road. The foundation conditions along Section 1a were checked by comparing the section stratigraphy to nearby drill hole logs. Similar failure surfaces to those presented in Klohn Crippen, 2006 were confirmed for use in the stability assessment.

The ultimate tailings level along Section 1a is less than the maximum crest elevation of the tailings pile (330ft) because the section is close to the eastern edge of the tailings pile. The maximum tailings elevation rises to the west of Section 1a, however the elevation of the top of the sand layer falls off to the west. Section 1a was assumed to be the critical stability section for the NE expansion area because it runs through the area where the sands are the shallowest. A stability section (Section 1b, shown in Appendix I, Figure 3) was run to the west of Section 1a in order to confirm this assumption, and the critical failure surface was found to have a Factor of Safety greater than 1.3 for the post-earthquake condition.

The material properties summarized in Table 4.1 were used in the stability analyses, with the exception of the post-liquefied strength ratio, $\tau/\sigma_v' = 0.12$, for the sands north and south of the NE expansion area. The phreatic surface in the tailings is similar to the surface used in the Klohn Crippen, 2006 analysis. The phreatic surface that was used in the Klohn Crippen, 2006 analysis for the foundation materials was modified to reflect the revised foundation water level (El. 180ft) through the NE expansion area.

Results from the stability analysis are summarized in Table 5.1.

Table 5.1 Northeast Expansion Area Stability Analysis Results

SECTION	**CALCULATED FACTOR OF SAFETY**		
	Static-Peak	**Static-Residual**	**Post-Earthquake**
	FOS $\geq$ 1.5	**FOS $\geq$ 1.3**	**FOS $\geq$ 1.1**
1a$^{(1)}$	1.8	1.6	1.2$^{(2)}$

Notes:
1. Section 1a as presented in Klohn Crippen, 2006.
2. This assumes the area of medium dense sand, as shown on Figure 3 of Appendix I, does not liquefy but does soften.

6. SLURRY WALLS

In areas where tailings are not placed above a geosynthetic liner system, groundwater that has come in contact with the tailings is contained within the tailings pile footprint by a series of slurry walls that have been constructed around the tailings pile.

As part of the planned expansion into the NW/Pit 5 and NE expansion areas, two of the existing slurry walls are scheduled to be raised and/or extended:

- The slurry wall along the existing west buttress road (Drawing D-47003) will be extended along the alignment and raised to the elevations shown on Drawing D-47013. A drain will be installed on the upstream side of the wall to reduce the risk of the foundation water level from rising above the maximum height of the slurry wall. This raise and extension of the slurry wall is expected to contain seepage from the southwest corner of the NW/Pit 5 expansion within the footprint of the tailings pile.
- The slurry wall north of the NE expansion area will be extended along the alignments and raised to the elevations shown on Drawing D-47015. A drain will be installed on the upstream side of the wall to prevent the foundation water level from rising above the maximum height of the slurry wall. This raise and extension of the slurry wall should contain seepage from the NE expansion area within the footprint of the tailings pile and restrict flow into the NW/Pit 5 foundation.

7. INSTRUMENTATION

As part of the Northwest/ Pit 5 and Northeast expansions, environmental and geotechnical monitoring will be expanded, to include the instrumentation shown in Drawing D-47016.

To monitor water pressure six geotechnical instrumentation locations will be established in the Northwest/Pit 5 area, and two in the Northeast area. Vibrating wire piezometers will be placed above and below the liner to monitor pore pressures in the underdrains, in the service layer, and in the tailings.

Suction lysimeters and environmental sampling tubes will be established at all existing monitoring wells, with two additional wells to be installed in the Northwest.

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

8. RELOCATION OF SERVICES

The Water Treatment Plant (WTP) currently located in Pit 5 will remain operational during 2007, and will be decommissioned and demolished during the 2008 construction season, to be replaced by a new WTP. The location and design of the new facility will be confirmed in 2007.

In order to maintain operation of the WTP during 2007, and to protect the integrity of the pipes, four existing utility pipelines along two alignments will be temporarily relocated as needed in the field under the direction of KGCMC.

9. CONSTRUCTION SEQUENCE

The preliminary construction sequencing is summarized below:

- February-March: Drill and Blast in the Northwest Expansion area, that began in 2006, will continue prior to main contractor mobilization and continue throughout the construction season.
- April: Completion of the Northwest Excavation and removal of the co-disposal test plots in the Northeast expansion area.

- May: Construction of the Northeast Retention Pond and North to West corridor to begin. Construction of the new Water Treatment Plant to begin.
- June: Completion of the Northeast retention Pond and foundation preparation in the Northwest/Pit 5 expansion area.
- July: Liner installation in the Northwest/Pit 5 expansion area.
- August - September: Completion of construction of the new Water Treatment Plant (WTP), in preparation for decommissioning of the WTP in Pit 5.

Some contaminated material may be placed in Northwest expansion areas after liner construction is complete, and temporary stability requirements will be addressed during the year as required.

10. CLOSING REMARKS

This report presents the design for the Stage 2 Tailings Facility Expansion 2007 Construction. The designs presented in this report are consistent with the existing safety criteria for seismic 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.

Lowell Constable, E.I.T.
Project Engineer

Robert W. Chambers, P. Eng.
Project Manager

Len Murray, P. Eng.
Senior Reviewer

REFERENCES

Alaska Department of Environmental Conservation (ADEC) (2003). "*Waste Management Permit 0211-BA0001 6*", Kennecott Greens Creek Mining Company. November 7.

ASTM D 5321, "Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method." ASTM International.

Environmental Design Engineering (EDE) (2002). "*Kennecott Greens Creek Mining Company, Stage II Tailings Expansion Hydrologic Analysis*". Engineering report submitted to KGCMC. February, 2002. 59 pages; attachments.

Environmental Design Engineering (2003). *"Kennecott Greens Creek Mine - Site Water Balance"*. Engineering report submitted to KGCMC. 48 pages; attachments.

Kennecott Greens Creek Mining Company (KGCMC) (2004). "General Plan of Operation-Appendix 3 – Tailings Impoundment, April 2004".

Klohn Crippen Consultants Ltd. (2004). Kennecott Greens Creek Mining Company, "Stage 2 Tailings Facility Expansion, Design Overview for Forest Service Submission." April 8, 2004.

Klohn Crippen (2005). "Stage 2 Expansion of Tailings Facility Contract No. P05001-CW1-Civil Works." April 2005.

Klohn Crippen (2006). "Stage 2 Tailings Expansion Overall Stability Update-Final Report." March 2006.

Subcommittee D35.01. "Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method" (ASTM D-5321)

APPENDIX I

Northeast Expansion Area- Liquefaction Assessment of Medium Dense Sand

Klohn Crippen Berger

MEMORANDUM

TO: Len Murray **DATE:** February 5, 2007

FROM: Rick Friedel **FILE NO:** M07802A47.210
LOG NO:

SUBJECT: Northeast Expansion Area - Liquefaction Assessment of Medium Dense Sand

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Len,

This memo is a review of the liquefaction potential of the sands that have been identified in the foundation of the Northeast (NE) expansion area of the Greens Creek tailings pile. A stability analysis of the area is also included.

As part of the Stage 2 expansion of the Greens Creek tailings pile, Kennecott Greens Creek Mining Company (KGCMC) is preparing to begin development of the NE expansion area in 2007. The proposed expansion is rectangular in shape and covers approximately 60,000 sq. ft (400ft x 150ft) and is located at the junction of the Pit 5 access road and the B-Road.

A preliminary design for the expansion was completed as part of the “Stage 2 Tailings Facility Expansion – Design Overview for Forest Service Submission,” report dated April 8, 2004. The design was completed prior to geotechnical drilling in the area and called for the removal of any loose liquefiable sands in the foundation prior to tailings placement.

Four geotechnical drill holes (DH04-01 to -04) were completed in the area in late 2004. SPT results and sample descriptions from each location have been used in this assessment of the liquefaction potential of the sand in the area.

REVIEW OF SPT RESULTS AND SAMPLE DESCRIPTIONS

SPT results and sample descriptions for the sand (N blows < 25) identified in each drillhole directly below the Peat unit are summarized below. Drill hole locations are shown on Figure 1.

DH04-01

Medium Dense Sand: Identified
Thickness: 4ft (El. 177ft to 173ft)
SPT Tests: 1
SPT3 (4,7,6) ~El.175.5ft
Recovery: 2 inch
$(N_1)_{60\text{-}CS} = 23$

Description: (GP) Gravel, poorly graded, dark grey and black, angular, some quartz pebbles, 0.1 inch to 0.3 inch diameter, single angular 1 inch pebble plugged sampler tip.

DH04-02

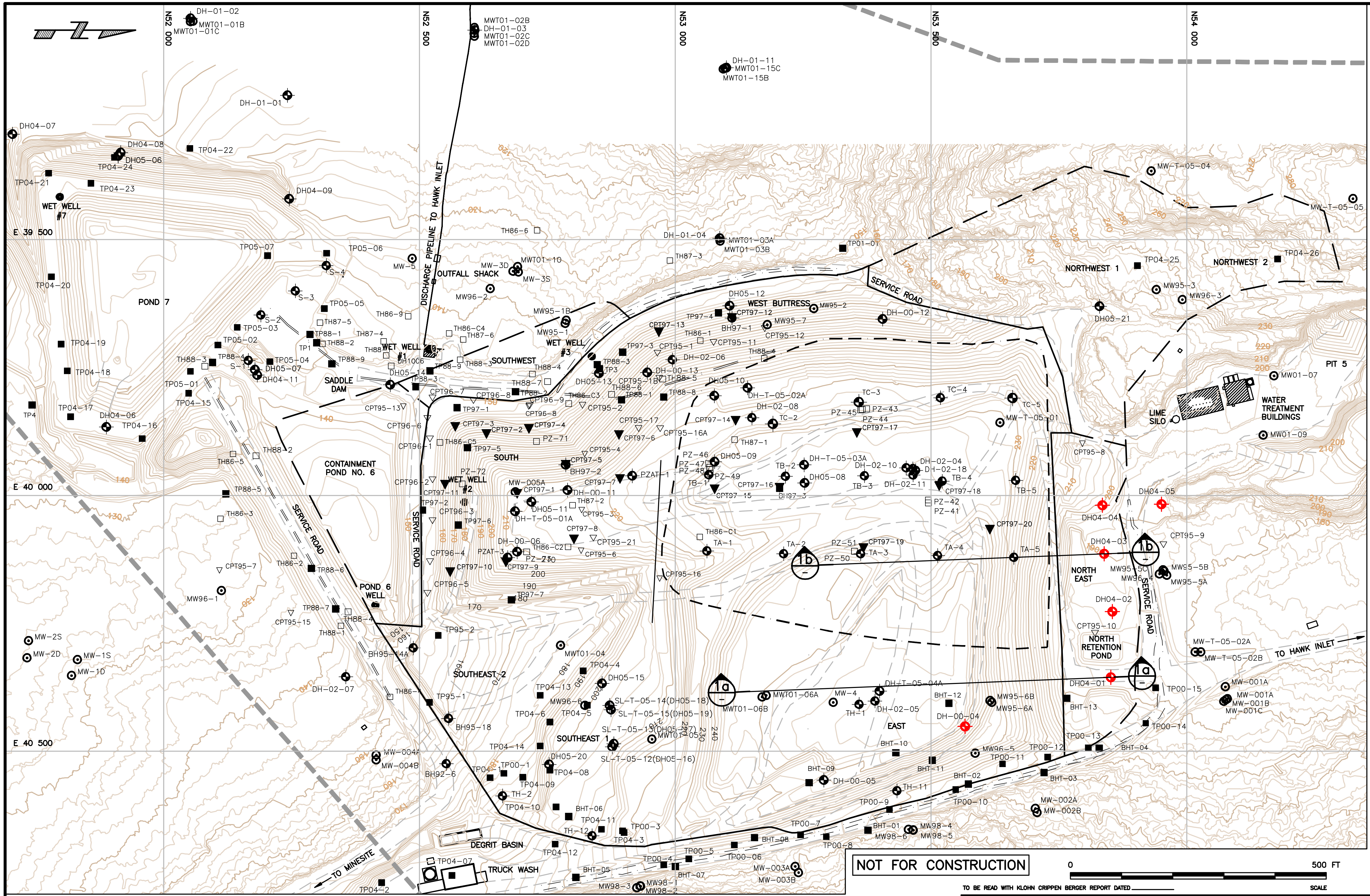
Medium Dense Sand: Identified
Thickness: 4.1ft (El. 168.1ft to 164ft)
SPT Tests: 2
SPT3 (9, 9, 13) ~El.168.1ft
Recovery: 6 inch
(N1)60-CS = 39
Description: (GP) Gravel, sandy with trace silt, dark grey and black, loose to firm, wet, poorly graded, gravel rounded to sub-angular.
SPT4 (8,7,6) ~El.166ft
Recovery: 5 inch
(N1)60-CS = 25
Description: (GP) Gravel, some sand with trace silt, dark grey and black, loose, wet, poorly graded, no bedding, no odour, gravel rounded to angular, some quartz clasts, angular.

DH04-03

Medium Dense Sand: None Identified
Thickness: n/a
SPT Tests: 0

DH04-04

Medium Dense Sand: Identified
Thickness: 6.3ft (El. 172.3ft to 166ft)
SPT Tests: 2
SPT5 **(8,8,6) ~El.172.3ft**
Recovery: 11 inch
(N1)60-CS = 24
Description: (0) Peat, soft, wet, amorphous and sl. fibrous, reddish brown, greenish stone in top of sampler.
SPT6 (14/4/6) ~El.168.7ft
Recovery: 10 inch
(N1)60-CS = 15
Description: Upper 6 inches: (0) Peat, soft, wet, amorphous and sl. fibrous, reddish brown, greenish stone in top of sampler.
Lower 4 inches: (SW) Sand, some silt, some gravel, grey, wet, loose, rapid dilatancy, interbedded with, (GP) Gravel, sandy, loose, angular to sub-rounded, black.



NOT FOR CONSTRUCTION

0 500 FT
SCALE

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED ________

LEGEND

— · —	OLD TAILINGS FACILITY BOUNDARY
——	CURRENT TOE LINE OF TAILINGS FACILITY
— —	STAGE 2 TAILINGS TOE LINE
= = =	EXISTING ROAD
= : =	SLURRY WALL
▬ ▬	LEASE BOUNDARY
◎	WET WELL
☐ DH1006	DRILL HOLE KLOHN CRIPPEN 1980
■ TP1	TEST PIT, KLOHN CRIPPEN 1980
☐ TH86-2	DRILL HOLE SRK 1986
☐ TH87-2	DRILL HOLE SRK 1987
☐ TH88-2	DRILL HOLE SRK 1988
⊙ MW88-5	DRILL HOLE SRK 1988
■ TP88-9	TEST PIT SRK 1988
⊙ MW-5	MONITORING WELL SRK 1988
⊙ MW-1S	MONITORING WELL SRK 1988
⊕ S-1	DRILL HOLE SRK 1988
■ TP95-1	TEST PIT SRK 1995
⊕ BH92-6	DRILL HOLE SRK 1992
⊕ TA-2	DRILL HOLE KGCMC 1994
⊕ TB-2	DRILL HOLE KGCMC 1994
⊕ TC-2	DRILL HOLE KGCMC 1994
⊙ MW95-3	MONITORING WELL SRK 1995
☐ PZ-43	PIEZOMETER KGCMC 1995
⊕ BH95-18	DRILL HOLE SRK 1995
▽ CPT95-8	CONE PENETRATION TEST, SRK 1995
⊙ MW96-3	ENVIRONMENTAL DESIGN ENG.1996
▽ CPT96-1	CONE PENETRATION TEST, CONETEC 1996
▼ CPT97-1	CONE PENETRATION TEST, CONETEC 1997
⊕ BH97-1	DRILL HOLE, KLOHN CRIPPEN 1997
■ TP97-1	TEST PIT, KLOHN CRIPPEN 1997
⊙ MW98-1	DRILL HOLE SRK 1998
⊕ TH-1	MONITORING WELL R & M ENGINEERING
■ BHT-01	TEST PIT ENVIRONMENTAL DESIGN ENGINEERING 1999
■ TP00-11	TEST PIT, KLOHN CRIPPEN 2000
⊕ DH-00-04	DRILL HOLE, KLOHN CRIPPEN 2000
⊕ PZAT-1	MONITORING WELL, ENVIRONMENTAL DESIGN ENGINEERING 2000
⊙ MWT01-03B	MONITORING WELL, ENVIRONMENTAL DESIGN ENGINEERING 2001
⊙ MW-001A	MONITORING WELL, ENVIRONMENTAL DESIGN ENGINEERING 2001
⊕ DH-01-04	DRILL HOLE, KLOHN CRIPPEN 2001
■ TP01-01	TEST PIT, KLOHN CRIPPEN 2001
⊕ DH-02-04	DRILL HOLE, KLOHN CRIPPEN 2002
■ TP04-15	TEST PIT, KLOHN CRIPPEN 2004
⊕ DH-04-01	DRILL HOLE, KLOHN CRIPPEN 2004
■ TP05-04	TEST PIT, KLOHN CRIPPEN 2005
⊕ DH05-07	DRILL HOLE, KLOHN CRIPPEN 2005
⊕ DH-T-05-03A	DRILL HOLE, ENVIRONMENTAL DESIGN ENGINEERING 2005
⊙ MW-001A	MONITORING WELL, ENVIRONMENTAL DESIGN ENGINEERING, 2005

NOTES

1. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.
2. TAILINGS PILE CONTOURS AND BASE TOPOGRAPHY PROVIDED BY KGCMC (OCTOBER 2005).

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA,STATEMENTS,CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

CLIENT
KENNECOTT GREENS CREEK
MINING COMPANY

Klohn Crippen Berger

PROJECT
STAGE 2 TAILINGS EXPANSION
OVERALL STABILITY UPDATE

TITLE
NORTHEAST EXPANSION
STABILITY PLAN

PROJECT No.	FIG. No.	REV
M07802 A47	1	–

Date: 2/6/2007
Scale: 1=1(PS)
Drawing File: M\M07802\A47 - 2006 Stage 2 Design\400 Design\410 Drawings\2007Design_Letter\Fig1.dwg (plee)
Xrefs: Pond 7contours-GA R-1A

Based on the above logs, loose sand is not present in the NE area however a layer of medium dense sand is present but is not continuous. DH04-01 and DH04-02 indicate that there is a 4 foot thick medium dense sand layer that slopes to the west. No sand was identified in DH04-03 and the samples collected during SPT5 and SPT6 (previously identified as sand) in DH04-04 were predominantly peat. It is not conclusive whether the medium dense sand layer extends further west than DH04-02.

SIMPLIFIED LIQUEFACTION ASSESSMENT METHODS

As mentioned above, samples from SPT5 and SPT6 from DH04-04 were mainly peat. Based on that observation, the SPT results from DH04-04 were excluded from the analysis.

To assess the liquefaction susceptibility of the medium dense sands identified in DH04-01 and DH04-02 SPT results were analyzed using the simplified empirical methods specified in Youd et al. (2001) and Boulanger (2004). It was assumed in the analysis that peak ground acceleration (PGA) is 0.15g for the Design Base Earthquake (DBE), and 0.3g for the Maximum Design Earthquake(MDE), and is not modified by the soil column (KCBL 2007). The results of a previous similar analysis were originally reported in the “Stage 2 Tailings Expansion Overall Stability Update” report dated March 1, 2006. The results presented in that report showed that the sand was stable against liquefaction under the DBE but could possibly liquefy under the MDE. These analyses were completed based on the foundation conditions prior to any tailings placement. The analyses for DH04-01 and DH04-02 have been revised to reflect the 2007 design foundation water level and the added weight of future tailings placement. Results of the revised analysis are summarized in Table 1.

The revised analysis shows that, under existing conditions, the SPT data indicate that liquefaction or softening will not occur under DBE. This agrees with the March, 2006 Analysis (Klohn Crippen 2006).

Table 1 also shows that under existing conditions the sand is predicted to be susceptible to liquefaction under MDE. This conclusion is surprising since the $(N_1)_{60\text{-}CS}$ ranges from 23 blows per foot to 39 blows per foot. However, the simplified analysis, on which the assessment is based, is greatly influenced by the presence of the peat layer, which has a buoyant unit weight of almost zero. It is expected that if a detailed liquefaction triggering analysis were done using more sophisticated techniques, the sand at shallow depth with such high blow counts would be found to be not susceptible to liquefaction.

The proposed design for the Northeast expansion includes lowering the water table and placement of tailings in part of the area. Both of these conditions help overcome the anomalous influence of the low density peat in the simplified analysis. Table 1 shows that lowering the water table to El. 180 ft and placement of 5 ft thickness of tailings is sufficient to raise the calculated factor of safety against liquefaction to 1.2. As the tailings thickness increases to 10 ft, the safety factor further increases.

In summary, this revised analysis predicts that lowering the design foundation water level to 180ft is sufficient to raise the FOS under MDE against liquefaction above 1.1 for two of the three sand samples and placement of 5ft of tailings in addition to lowering the water table raise the FOS against liquefaction above 1.1 for all three samples.

These results indicate that the following assumptions are reasonable when assessing the stability of the NE expansion area under the post-earthquake design condition:

- The medium dense sand identified in the NE expansion area is not liquefiable under MDE in areas where 5ft of tailings thickness or greater overlie the area, assuming a foundation water table elevation of 180ft or below.

- The shallow medium dense sand identified in the NE expansion area with less than 5ft of tailings thickness is potentially liquefiable under MDE, assuming a maximum foundation water table elevation of 180ft. This is considered to be a conservative assumption since more sophisticated analyses would likely show that sand with $(N_1)_{60\text{-}CS} \geq 23$ would not be liquefiable.

Table 1 – Summary of Revised Factor of Safety Against Liquefaction

Drillhole	**SPT Test**	**Elev. (ft)**	**$(N_1)_{60\text{-}CS}$**	**Design Scenario**	**FOS Against Liquefaction**			
					DBE (Youd et al.)	**MDE (Youd et al.)**	**DBE (Boulanger)**	**MDE (Boulanger)**
DH04-01	SPT3	174.8	23	Existing Conditions Water Table at El. 185ft.	2.0	**0.8**	1.9	**0.8**
				Water Table at El. 180ft.	2.9	1.2	2.7	1.1
				5ft Tailings Thickness, Water Table at El. 180ft	3.2	1.3	3.0	1.2
				10ft Tailings Thickness, Water Table at El. 180ft	3.3	1.4	3.0	1.3
DH04-02	SPT3	167.4	39	Existing Conditions Water Table at El. 185ft.	2.2	**0.9**	2.2	**0.9**
				Water Table at El. 180ft.	3.8	1.6	3.8	1.6
				5ft Tailings Thickness, Water Table at El. 180ft	4.9	2.0	4.9	2.0
				10ft Tailings Thickness, Water Table at El. 180ft	5.6	2.3	5.6	2.3
	SPT4	165.3	25	Existing Conditions Water Table at El. 185ft.	1.5	**0.6**	1.4	**0.6**
				Water Table at El. 180ft.	2.2	**0.9**	2.2	**0.9**
				5ft Tailings Thickness, Water Table at El. 180ft	2.8	1.2	2.8	1.2
				10ft Tailings Thickness, Water Table at El. 180ft	3.2	1.3	3.1	1.3

Softening is expected in areas where the factor of safety against liquefaction is less than 1.4, as discussed in Klohn Crippen (2006).. Since only a small percentage of the non-liquefiable sand is under less than 10 feet thickness of tailings, tan(ø') is reduced linearly as the factor of safety against liquefaction reduces from 1.4 to 1.3, based on ø'=33° at FOS against liquefaction=1.4, and ø'=0° at FOS against liquefaction=1.0. As such, ø' is assumed to reduce to 26° for the non-liquefiable sands for the post-earthquake condition for modelling.

POST-LIQUEFIED STRENGTH OF MEDIUM DENSE SAND OUTSIDE OF NE AREA

Sands identified beneath the existing tailings pile to the south (DH00-04) of the NE expansion area have been reported as liquefiable in Klohn Crippen (2006). A review of the previous assessment confirms this conclusion. The average $(N_1)_{60\text{-CS}}$ value for the sand in DH00-04 is 15 blows per foot which is lower than the average $(N_1)_{60\text{-CS}}$ value of about 25 blows per foot for the medium dense sand beneath the NE expansion footprint.

An average SPT $(N_1)_{60\text{-CS}}$ of 15 was used to estimate the liquefied strength of the sand beneath the existing tailings pile and in the NE expansion footprint where ultimate tailings thickness is less than 5 ft, based on a relationship between τ/σ_{vo}' vs. $(N_1)_{60\text{-CS}}$.

$$\frac{\tau}{\sigma_{vo}'} = 0.02 + 0.025\sqrt{(N_1)_{60-CS}} \quad \text{(Idriss, 2004)}$$

Assuming an average $(N_1)_{60\text{-CS}}$ value of 15, the post liquefied strength of the tailings was estimated as $\tau/\sigma_v' = 0.120$.

NORTHEAST EXPANSION AREA STABILITY ANALYSIS

The stability section for the NE expansion area (Section 1a) that was included in the Klohn Crippen (2006) report was revised to include on the currently proposed layout. The current layout includes a new 50ft wide retention pond that runs parallel to the roadway along the toe of the tailings. This new pond offsets the toe of the tailings approximately 70ft south from the south edge of the road. The foundation conditions along Section 1a were checked by comparing the section stratigraphy to nearby drill hole logs. Similar failure surfaces to those presented in Klohn Crippen, 2006 were used to assess stability.

The ultimate tailings level along Section 1a is less than the maximum crest of the tailings pile (330ft) because the section is close to the eastern edge of the tailings pile. The maximum tailings elevation rises to the west of Section 1a, however the elevation of the top of the sand layer falls off to the west. Section 1a was assumed to be the critical stability section for the NE expansion area because it runs through the area where the medium dense sand is the shallowest. This assumption was confirmed by a preliminary stability analysis performed on Section 1b, taken approximately 100 ft west of Section 1a, where the sand layer is deeper. The critical slip surface passes through the deeper sand layer generating more frictional resistance and increasing the factor of safety to over 1.3. Section 1b is shown in Figure 4.

Based on the assumptions discussed previously regarding the liquefaction behaviour and extent of the shallow sands beneath the NE expansion area, a post-liquefied strength ratio of τ/σ_v’ = 0.120 was selected for the sands north and south of the NE expansion area. The strength parameters for the other materials were taken directly from the Post-Earthquake design conditions in Klohn Crippen, 2006. The calculated minimum post earthquake factor of safety against slope failure for Section 1a is 1.2 which is greater than the design criteria factor of safety of 1.1. This assumes the area of medium dense sand, as shown on Figure 3, does not liquefy.

Seismic deformations were assessed using pseudo-static methods (Hynes-Griffin, et al. 1984), assuming post liquefaction material properties, on Section 1a. The results are summarized in Table 2. The Hynes-Griffin method assumes a relatively large base amplification factor based on case histories from many sites and is primarily intended to be used as a screening tool. As such, it tends to overestimate the deformation, and actual deformations are expected to be at the low end of the predicted deformation range. Since the predicted deformations for the mean and mean plus 1 standard deviation are less than 6 ft for the MDE, these deformations are not expected to significantly affect the pile stability. These deformation values are similar to values predicted in other parts of the pile, as reported in Klohn Crippen (2006).

Table 2 – Summary of Seismic Deformation Estimates

				Deformation (feet)			
				DBE (PGA=0.15g)		**MDE (PGA=0.30g)**	
		Static FOS	**Yield Acceleration**	**Mean**	**Mean + 1 Std. Dev.**	**Mean**	**Mean + 1 Std. Dev.**
Section 1a	Post Liquifaction	1.2	0.035g	0.75	2.7	1.6	5.6

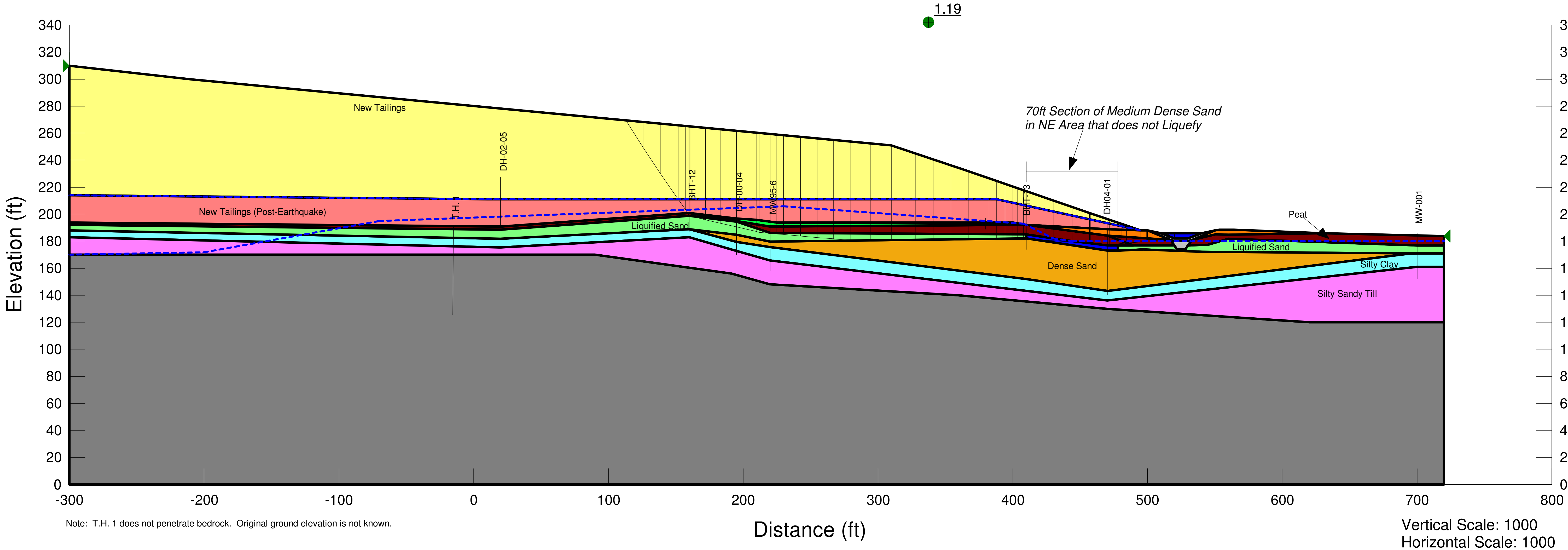
Figure 3 - Section 1a: Northeast Expansion Area Stability

Description: Greens Creek Stage II - Option 5
Comments: Northeast Section 1
File Name: NE Section1a-Post Earthquake-fspec3 - No Liq (tau-sig = 0.120) Opt softened2.gsz
Analysis Method: Morgenstern-Price
Slip Surface Option: FullySpecified
Seismic Coefficient: horz: 0, vert: 0

Sand in NE Area Not Liquefied

Liquified Sand Strength:
tau/sigma ratio = 0.120

Material #: 1 Description: New Tailings Wt: 128 Cohesion: 0 Phi: 32
Material #: 2 Description: Softened New Tailings Wt: 128 Cohesion: 0 Phi: 28.4
Material #: 3 Description: Compacted Rock Fill Wt: 120 Cohesion: 0 Phi: 40
Material #: 4 Description: Old Tailings Wt: 120 Cohesion: 0 Phi: 32
Material #: 5 Description: Peat Wt: 67 Cohesion: 0 Phi: 27
Material #: 6 Description: Gravelly Sand Wt: 120 Cohesion: 0 Phi: 33
Material #: 7 Description: Liquified Sand Wt: 120 Tau/Sigma Ratio: 0.12
Material #: 8 Description: Sand Wt: 120 Cohesion: 0 Phi: 33
Material #: 9 Description: Silty Clay Wt: 120 Cohesion: 0 Phi: 30
Material #: 10 Description: Silty Sand Till Wt: 120 Cohesion: 0 Phi: 33
Material #: 11 Description: Bedrock
Material #: 12 Description: Water Wt: 62.4
Material #: 13 Description: Softened Sand Wt: 120 Cohesion: 0 Phi: 26



Note: T.H. 1 does not penetrate bedrock. Original ground elevation is not known.

Vertical Scale: 1000
Horizontal Scale: 1000

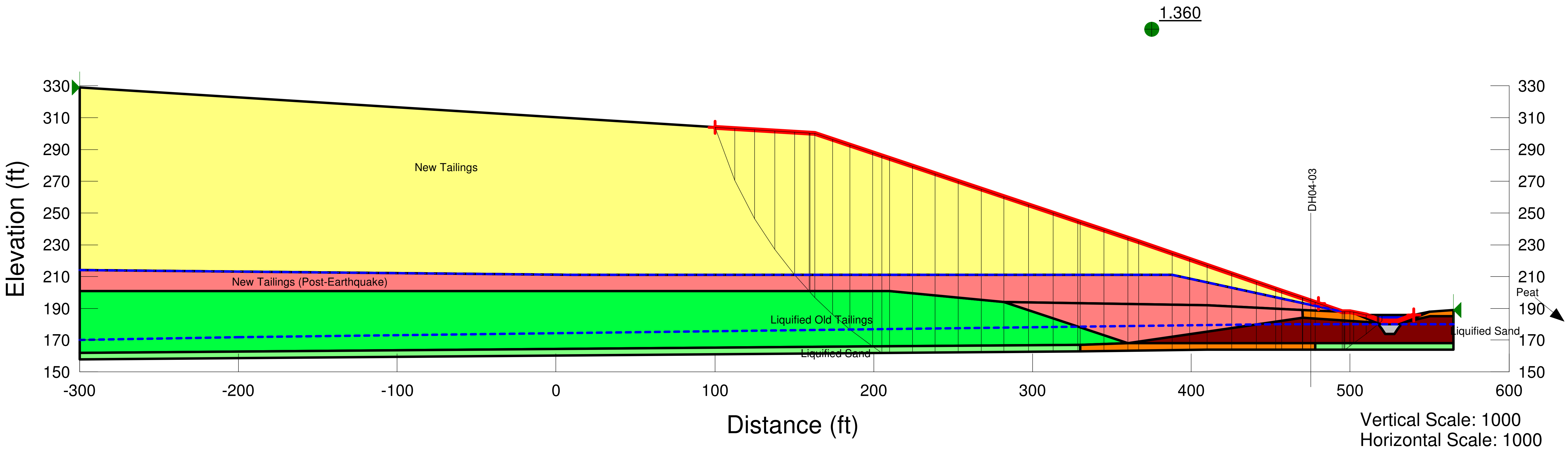
Figure 4- Section 1b- Northeast Expansion Area Stability

Description: Greens Creek Stage II - Option 5
Comments: Northeast Section 1b
File Name: NE True Section1b-Post Earthquake-entryexit - No Liq (tau-sig = 0.120) rev1.gsz
Analysis Method: Morgenstern-Price
Slip Surface Option: EntryAndExit
Seismic Coefficient: horz: 0, vert: 0

Sand in NE Area Not Liquefied

Liquified Sand Strength:
tau/sigma ratio = 0.120

Material #: 1 Description: New Tailings Wt: 128 Cohesion: 0 Phi: 32
Material #: 2 Description: Softened New Tailings Wt: 128 Cohesion: 0 Phi: 28.4
Material #: 3 Description: Compacted Rock Fill Wt: 120 Cohesion: 0 Phi: 40
Material #: 4 Description: Liq Old Tailings Wt: 120 Strength Fn: 1
Material #: 5 Description: Peat Wt: 67 Cohesion: 0 Phi: 27
Material #: 6 Description: Gravelly Sand Wt: 120 Cohesion: 0 Phi: 33
Material #: 7 Description: Liquified Sand Wt: 120 Tau/Sigma Ratio: 0.12
Material #: 8 Description: Sand Wt: 120 Cohesion: 0 Phi: 33
Material #: 9 Description: Silty Clay Wt: 120 Cohesion: 0 Phi: 30
Material #: 10 Description: Silty Sand Till Wt: 120 Cohesion: 0 Phi: 33
Material #: 11 Description: Bedrock
Material #: 12 Description: Water Wt: 62.4



ESTIMATED DISPLACEMENT OF LOOSE SAND DURING DYNAMIC LOADING

Liquefaction analyses presented above indicate that some foundation sands near the toe of the tailings pile may liquefy during dynamic loading. Stability analysis indicate that the tailings pile is stable if the sands liquefy but some settlement and shear strain would occur. Empirical methods were used to estimate the amount of settlement and shear strain that may occur.

Tokimatsu and Seed (1987) developed an empirical relationship based on $(N_1)_{60}$ and CSR values that estimates the amount of volumetric strain that will occur after initial liquefaction, refer to Figure 5. CSR and $(N_1)_{60}$ values for each of the three SPT tests (Table 2) are plotted on Figure 5. Based on Figure 5 the maximum amount of volumetric strain likely to occur in the loose sand after initial liquefaction is less than 1.5%. This amount of strain corresponds to less than 1-inch of settlement in the 4ft sand layer. In sands that do not reach a fully liquefied state the settlements would be even less (approximately 0.2% strain). Neither of these settlement magnitudes are considered to be significant.

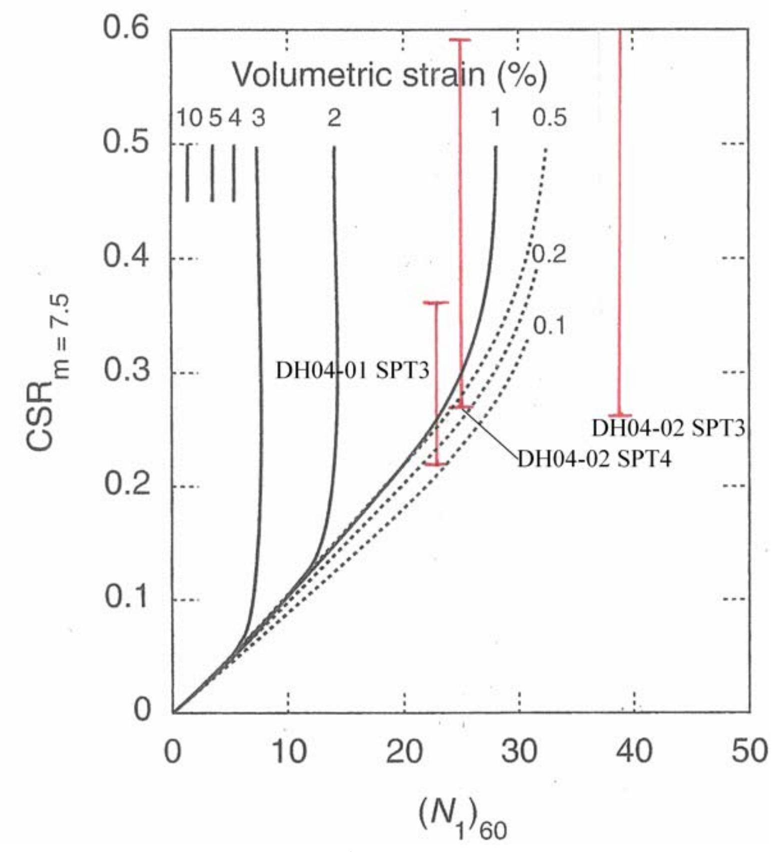


Figure 5 – Estimation of Volumetric Strain in Saturated Sands Based on CSR and $(N_1)_{60}$ values. (Tokimatsu and Seed, 1987)

STRAINS INDUCED UNDER MDE

Even if factor of safety against slope instability exceeds 1.1, shear strains can still occur if the predicted zones of the foundation liquify. Simplified empirical relationships for predicting shear strain (lateral displacement) have been developed by Tokimatsu and Seed (1987) (Figure 6) and Shamoto et al. (1998) (Figure 7). CSR and $(N_1)_{60}$ or N_a[1] values for each of the three SPT tests (Table 2) are plotted on Figures 6 and 7. Based on the Figures the maximum amount of shear strain likely to occur in the sand after initial liquefaction is less than 10%. This amount of strain is equivalent to less than 5-inch of lateral displacement in the 4ft thick layer of sand.

The magnitude the predicted settlements and shear strains are not considered to be significant.

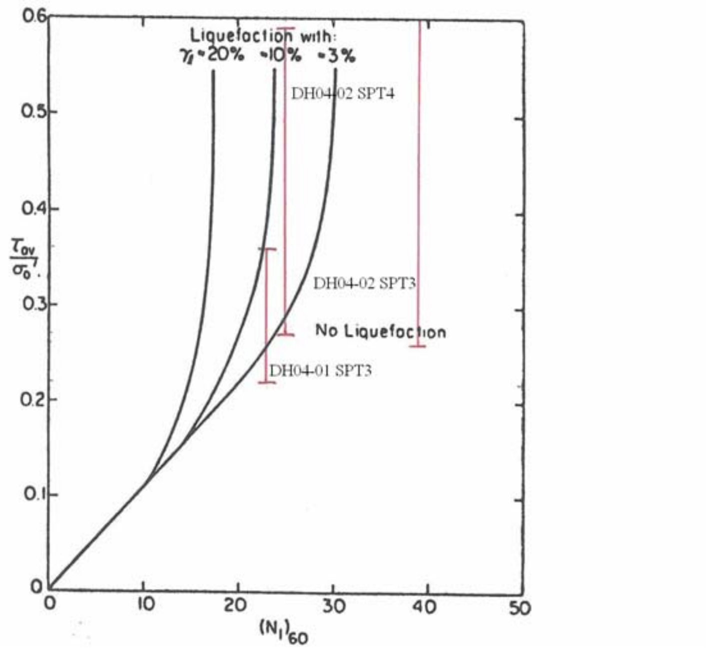


Figure 6 – Tentative Relationship between Cyclic Stress Ratio Causing Liquefaction, SPT N-Value and Limiting Shear Strain. (Tokimatsu and Seed, 1987)

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[1] N_a is the standardized blow count used in Japanese practice. $(N_1)_{60\text{-}CS} = 1.1*(N_a)$

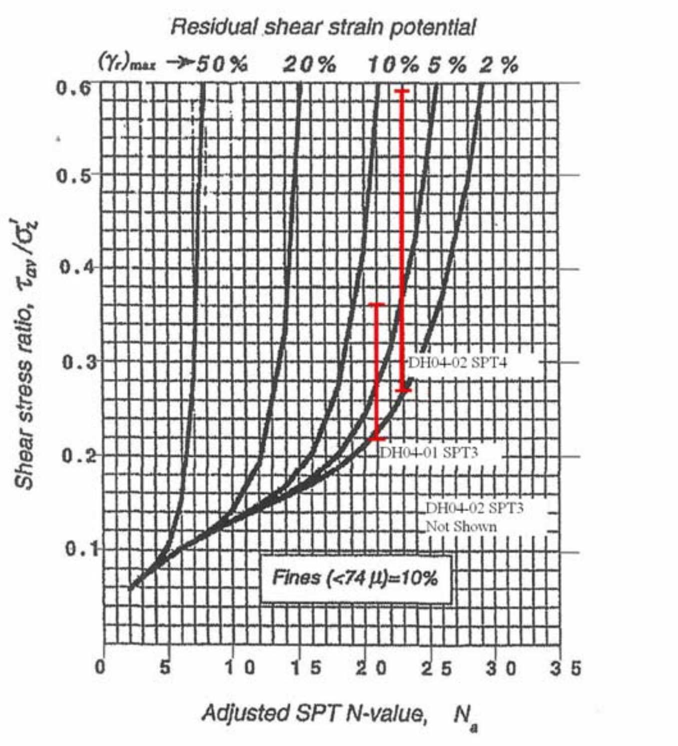


Figure 7 – Recommended Estimates of Limiting Shear Strains for Sandy Soils with ~10% Fines. (Shamoto et al., 1998)

CLOSING

This memo summarizes the liquefaction and stability assessment for the NE expansion area. Based on this analysis the medium dense sand beneath the NE area does not need to be removed prior to developing the area for placement of tailings.

Attachments: Drill hole Logs: DH04-01, DH04-02, DH04-03, DH04-04, DH04-05

REFERENCES

Boulanger, R.W., and I.M. Idriss (2004). State Normalization of Penetration Resistance and Effect of Overburden Stress on Liquefaction Resistance. 11th International Conf. on Soil Dynamics and Earthquake Engineering and 3rd International Conf. on Earthquake Geotechnical Engineering, Berkeley, CA, Jan. 7-9, 2004.

Hynes-Griffin, M.E. and Franklin, AG. (1984). "Rationalizing the Seismic Coefficient Method." US Army Corps of Engineers – Geotechnical Laboratory, Miscellaneous Paper gL-04-13. 37 pages.

Klohn Crippen (2006). Stage 2 Tailings Expansion Overall Stability Update – Final Report. March 1, 2006.

Klohn Crippen Berger Ltd. (KCBL 2007). Ground Response Analysis – Draft. January 17, 2007.

Kramer, Steven L. (1996). "Geotechnical Earthquake Engineering", Prentice Hall, First Edition.

Shamoto , Y., Zhang, J.-M. and Tokimatsu, K. (1998). "Methods for evaluating residual post-liquefaction ground settlement and horizontal displacement", Soils and Foundations, Special Issue, p. 69-83.

Seed, R.B., Cetin, K.O., Moss, R. and et al. (2001). "Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation", Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor W.D. Liam Finn, San Diego, California, March 21-31.

Seed, H.B., K. Tokimatsu, L.F., Harder, and R.M., Chung (1985). "Influence of SPT procedures in soil liquefaction resistance evaluations", ASCE Journal of Geotechnical Engineering, Vol. 111, p. 1425-1445

Tokimatsu, K. and Seed, H.B. (1987). "Evaluation of Settlements in Sands Due to Earthquake Shaking", ASCE Journal of Geotechnical Engineering, Vol 118, p. 874.

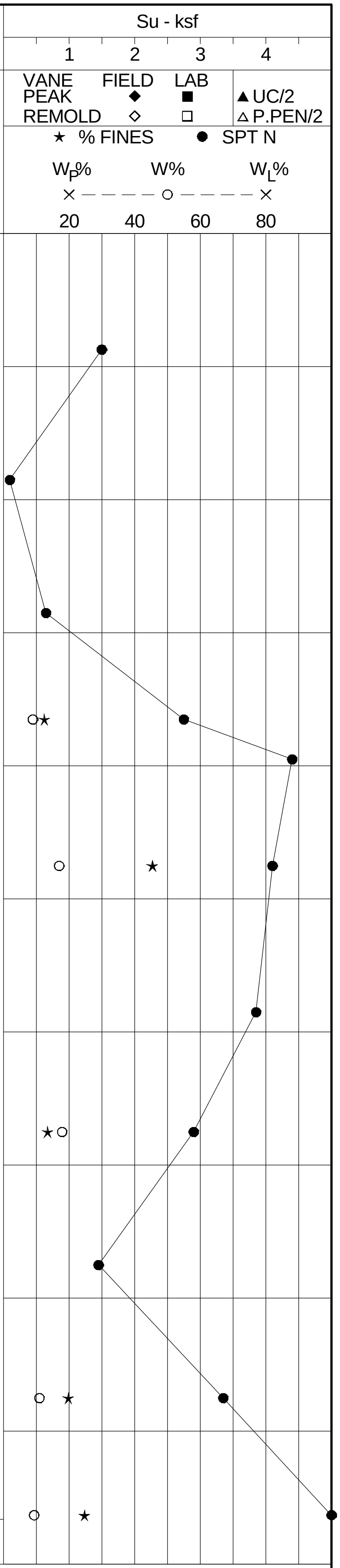
Youd, T.L., I.M., Idriss, R.D. Andrus, I. Arango, G. Castro, J.T. Christian, R. Dobry, W.D.L. Finn, L.F. Harder, M.E. Hynes, K. Ishihara, J.P. Koester, S.S.C. Liao, W.F Marcuson III, G.R. Martin, J.K. Mitchell, Y. Moriwaki, M.S. Power, P.K. Robertson, R.B. Seed, K.H. Stoke II (2002). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 27, No. 10, pp. 817-833.

TEST HOLE LOG

STARTED: 11/30/2004 **FINISHED:** 12/1/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 189.0
COORDINATES (ft): N 53851 E 40355

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Depth / Elev.	DESCRIPTION OF MATERIALS
	12,21,9	6	SPT-1		GRAVEL (GP) Sandy, some to trace silt, poorly graded, firm to dense, angular, grey and black, moist to wet.
10	1,<1,<1	7	SPT-2	4.9 184.1	PEAT (PT) Dark brown, wet, amorphous, soft, odour of decayed vegetal, trace brown grass.
	4,7,6	2	SPT-3	12.0 177.0	GRAVEL (GP) Poorly graded, loose, 0.1 inch to 0.3 inch diameter, angular, dark grey and black, wet, some quartz clasts. Single angular pebble 1 inch diameter stuck in spoon tip. Gravel appears to be argillite road fill slough. Interpretation of interval not conclusive.
	22,27,28	12	SPT-4	16.0 173.0	SAND (SP) Gravelly, some to trace silt, poorly graded, dense, gravel angular with some quartz clasts, shell fragments.
20	26,45,43	12	SPT-5		Sand coarsening in lower part of interval, more uniform gradation, less gravel, particles subrounded to rounded, grey, moist, (MARINE). Weathered phyllite stone at 20.25 ft; boulder weathered, talcy.
	21,34,48	14	SPT-6	22.0 167.0	SAND (SP) Silty, uniform gradation, medium to fine grained, firm to dense, subrounded to subangular, occasional irregular silty nodules to 0.8-in. diameter, grey, moist.
30	24,36,41	16	SPT-7		Sand, uniformly fine grained, silty, possible trace clay.
	16,28,30	13	SPT-8		Sand, uniformly graded, medium grain size, very dense.
40	3,11,18		SPT-9		Sand, silty, uniformly graded, medium grain size, firm to loose, rounded to subrounded, moist to wet.
	26,31,36		SPT-10		Sand, silty, trace clay, uniformly graded, firm to dense, rounded to subrounded, moist to wet. Two chloritic angular rock fragments to 1 inch diameter.
	50/3"		SPT-11	45.7 143.3	CLAY (CL) Silty, sandy, trace gravel, hard, poorly graded, gravel subangular, dark grey with pale green mottling, dry.
50				48.3 140.7	

INSTRUMENT DETAILS



Continued Next Page

PROJECT NO.: PM7802 A38
PROJECT: 2004 Geotechnical Investigation
LOCATION: Tailings Facilty Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** DH04-01

KLOHN CRIPPEN

TEST HOLE LOG

Su - ksf

1 2 3 4

STARTED: 11/30/2004 **FINISHED:** 12/1/2004

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 189.0

COORDINATES (ft): N 53851 E 40355

VANE FIELD LAB
PEAK ◆ ■ ▲ UC/2
REMOLD ◇ □ △ P.PEN/2

★ % FINES ● SPT N

W_P% W% W_L%
× — — — — ○ — — — — ×
20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
60					End of Hole at 48.3 ft Drill Notes: 1) Drill hole terminated at 48 ft. 2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 in long split-spoon, 1.5 inch inside diameter; sand catcher used; no sleeve. 3) Hole flushed with freshwater and backfilled with 0.375 inch bentonite chips (200 lbs, approx. 2.8 cubic feet).	
70						
80						
90						
100						

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2004 Geotechnical Investigation

LOCATION: Tailings Facilty Expansion

LOGGED BY: KE **CHECKED BY:**

SHEET 2 **OF** 2 **HOLE NO.:** DH04-01

TEST HOLE LOG

STARTED: 12/1/2004 **FINISHED:** 12/1/2004

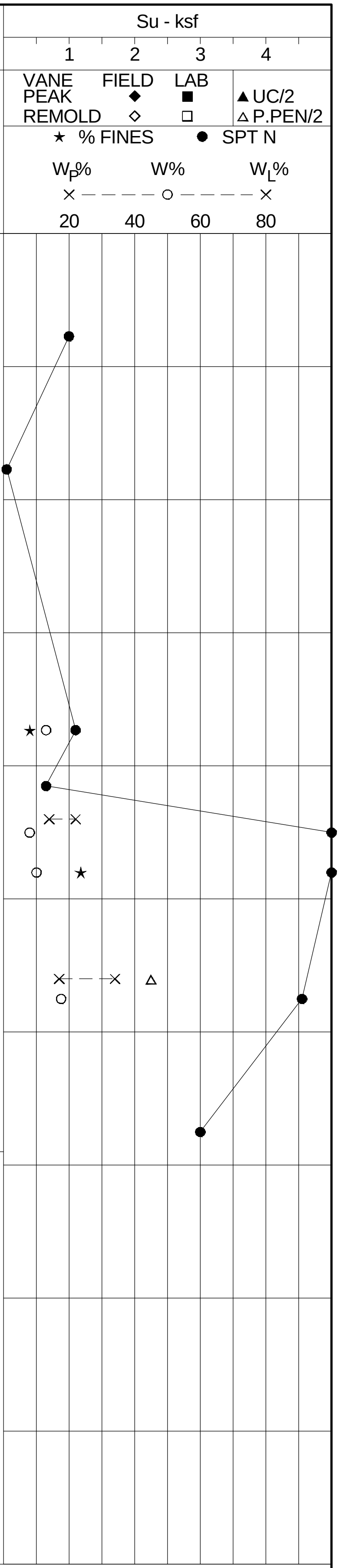
DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 186.0

COORDINATES (ft): N 53854 E 40227

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	9,8,12	5	SPT-1		GRAVEL (GP) Sandy, poorly graded, compact, angular, dark grey and black, no odour, wet, (ARGILLITE ROCK FILL). 5.0	
10	1		SPT-2		181.0 PEAT (PT) Orangey brown, wet, amorphous, odour of decayed vegetal, occasional twigs and decayed wood chunks, soft . 17.9	
	9,9,13	6	SPT-3		168.1 GRAVEL (GP) Sandy, trace silt, poorly graded, compact, clasts to 0.5 inch diameter, rounded to subangular, dark grey and black, no odour, wet, In lower part interval: decreasing sand content, some angular quartz clasts, loose.	
20	8,7,6	5	SPT-4		22.0	
	24,32		SPT-5		164.0 CLAY (CI-CL) Silty, some sand, some gravel, medium to low plasticity, very stiff, gravel angular to 0.4 inch diameter, blue-grey, moist.	
	40,50/5"		SPT-6		Gravelly from 25.4 ft to about 27 ft. 27.0	
30	41,41,50	3	SPT-7		159.0 CLAY (CI) Trace silt, medium plasticity, very stiff, medium grey, no odour, moist, no bedding.	
	13,24,36	12	SPT-8		Trace sand, indistinct contorted laminae to less than 0.1 inch thick. 34.5 151.5	
40					End of Hole at 34.5 ft	
50						

Drill Notes:
1) Drill hole terminated at 34.5 ft.
2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 in long split-spoon, 1.5 inch inside diameter; sand catcher used; no sleeve.
3) Hole flushed with freshwater and backfilled with 0.375 inch bentonite chips (200 lbs, approx. 2.8 cubic feet).



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2004 Geotechnical Investigation

LOCATION: Tailings Facilty Expansion

LOGGED BY: KE **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** DH04-02

TEST_KC TEST HOLE IMP 2004DH-050317GJL.GPJ KC_DATA.GDT 3/29/05

TEST HOLE LOG

STARTED: 12/2/2004 **FINISHED:** 12/2/2004

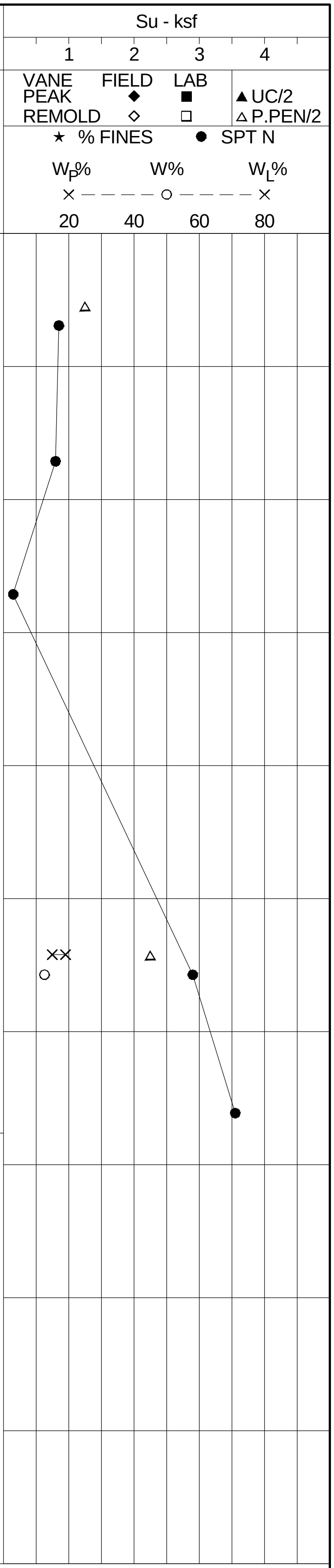
DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 190.0

COORDINATES (ft): N 53838 E 40114

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.		DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
				1.0 189.0	SAND (SP) Some silt, uniform gradation, loose, beige, moist, (BEDDING SAND FILL).	
	5,8,9	14	SPT-1		SILT (ML) Gravelly, some sand, some clay, poorly graded, firm, clasts to 0.2 inch diameter, gravel angular, black, silt grey-green, no odour, moist, (GENERAL FILL OR SPOIL WITH ARGILLITE).	
	6,8,8	12	SPT-2		Angular argillite clasts to 1 inch diameter.	
10				12.0 178.0		
	1,1,2	16	SPT-3		PEAT (PT) Orangey brown, wet, fibrous, odour of decayed vegetal, soft.	
20				27.1 162.9		
	17,27,31	10	SPT-4		SILT (ML-CL) Some clay, low plasticity, very stiff, blue-grey, no odour, moist, no bedding.	
30	22,33,38	16	SPT-5	33.8 156.2		
40					End of Hole at 33.8 ft	
50						

Drill Notes:
1) Drill hole terminated at 33.8 ft.
2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 in long split-spoon, 1.5 inch inside diameter; sand catcher used; no sleeve.
3) Hole flushed with freshwater and backfilled with 0.375 inch bentonite chips (200 lbs, approx. 2.8 cubic feet).



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2004 Geotechnical Investigation

LOCATION: Tailings Facilty Expansion

LOGGED BY: KE **CHECKED BY:**

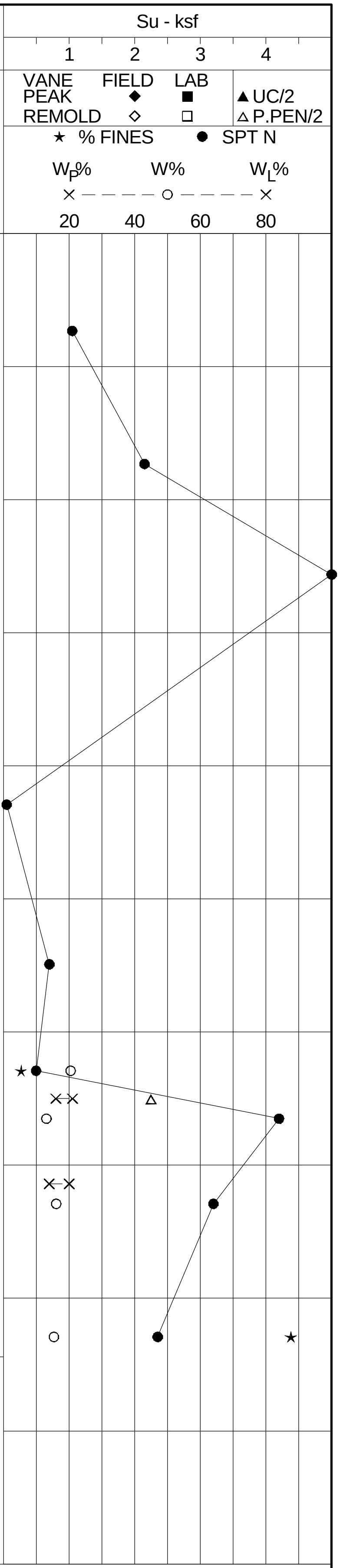
SHEET 1 **OF** 1 **HOLE NO.:** DH04-03

TEST HOLE LOG

STARTED: 12/2/2004 **FINISHED:** 12/4/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 199.0
COORDINATES (ft): N 53835 E 40019

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.		DESCRIPTION OF MATERIALS
	4,3,18	11	SPT-1		SAND (SP) Silty, some gravel, trace clay, poorly graded, soft to firm, gravel angular, mottled, brown and grey, moist to damp, (RANDOM FILL). Suspected stone on final interval.
				5.0 194.0	PEAT (PT) Dark brown, wet, fibrous, odour of decayed vegetal, soft.
	6,11,32	12	SPT-2	7.5 191.5	GRAVEL (GP) Sandy, poorly graded, dense, angular, black, (ARGILLITE FILL).
10					
	50/2"	3	SPT-3	12.7 186.3 12.9 186.1	CLAY (CI) Sandy, silty, some gravel, medium to low plasticity, angular clasts, blue-grey, no odour, moist, no bedding, (FILL). Argillite boulder – cored section. Siliceous veins at approx. 30° to core axis, occasional quartz blebs to 0.4 inch diameter, disseminated sulfide mineralization on parting fabric, dark grey to black, (FILL).
				16.3 182.7	PEAT (PT) Reddish brown, wet, amorphous, slight odour of decayed vegetal, soft.
20	0,0,1	2	SPT-4		
	8,8,6	11	SPT-5	26.7 172.3	SAND (SW) Some silt, some gravel, well graded, loose, clasts to 0.3 inch diameter, angular to subangular, grey, wet, rapid dilatancy, interbedded with, GRAVEL (GP) Sandy, loose, angular to subrounded, wet.
30	14,4,6	10	SPT-6		About half of sample is peat (likely slough).
	5,32,52	11	SPT-7	33.0 166.0	SILT (ML) Clayey, trace fine sand, low plasticity, hard, blue-grey, no odour, moist, laminae to 0.1 inch thick.
	14,23,41	16	SPT-8	35.7 163.3	SILT (ML) Some clay, low plasticity, hard, blue-grey, no odour, moist, laminae 0.04 inch to 0.08 inch thick. Fine sand laminae to 0.08 inch thick in lower portion of interval, interbedded with, SAND (SP) Some to trace silt, medium to fine grained, poorly graded (uniform), firm to dense, grey, moist to wet, silty lenses to 0.07 inch thick.
40	18,21,26	17	SPT-9	42.2 156.8	
50					End of Hole at 42.2 ft

INSTRUMENT DETAILS



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KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2004 Geotechnical Investigation
LOCATION: Tailings Facilty Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** DH04-04

TEST HOLE LOG

STARTED: 12/2/2004 **FINISHED:** 12/4/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 199.0
COORDINATES (ft): N 53835 E 40019

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS

Su - ksf
1 2 3 4

	FIELD	LAB	
VANE PEAK	◆	■	▲ UC/2
REMOLD	◇	□	△ P.PEN/2

★ % FINES ● SPT N

W_P% W% W_L%
× — — — — ○ — — — — ×
20 40 60 80

60

Drill Notes:
1) Drill hole terminated at 42.2 ft.
2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 inch long split-spoon, 1.5 inch inside diameter; sand catcher used; no sleeve.
3) Hole flushed with freshwater and backfilled with 0.375 inch bentonite chips (200 lbs, approx. 2.8 cubic feet).

70

80

90

100

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2004 Geotechnical Investigation
LOCATION: Tailings Facilty Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** DH04-04

TEST_KC TEST HOLE IMP 2004DH-050317GJL.GPJ KC_DATA.GDT 3/29/05

TEST HOLE LOG

STARTED: 12/4/2004 **FINISHED:** 12/5/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 192.0
COORDINATES (ft): N 53950 E 40016

SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.
13,11,11	4	SPT-1
32,9,5	7	SPT-2
7,8,6	6	SPT-3
2,2,2	4	SPT-4
1,1,2	5	SPT-5
28,59,60/4.5"	15	SPT-7
18,26,26	18	SPT-8

DEPTH (feet): 10, 20, 30, 40, 50

DESCRIPTION OF MATERIALS

GRAVEL (GP)
Sandy, trace silt, trace clay, poorly graded, compact, gravel angular to subangular, dark grey to black, wet, occasional white quartz clasts, (FILL).

8.3
183.7 8.8
183.2

PEAT (PT)
Reddish brown, wet, amorphous, odour of decayed vegetal, soft .

SAND (SP)
Gravelly, poorly graded, medium to coarse grained, compact, gravel angular, green and grey, wet, some quartz clasts.

12.0
180.0

SAND (SW)
Gravelly, trace to some silt, well graded, very loose to compact, gravel angular to subrounded, brownish grey, no odour, wet.

20.0
172.0

SAND (SP)
Silty, poorly graded (uniform), very dense, grey, moist, faint bedding 0.04 inch to 0.12 inch thick. Compact and wet in lower portion of interval.

29.2
162.8 29.8
162.2

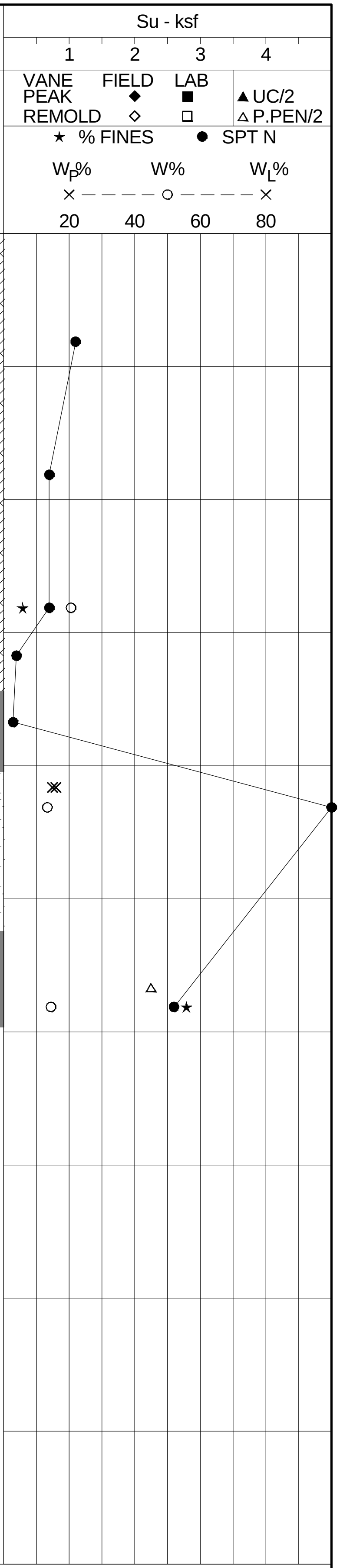
SILT (ML)
Some clay, some fine sand, very stiff, low plasticity, blue grey, moist, no bedding.

End of Hole at 29.8 ft

Drill Notes:
1) Drill hole terminated at 33 ft.
2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 in long split-spoon, 1.5 inch inside diameter; sand catcher used; no sleeve.
3) Lost circulation at about 2 ft. Placed 50 lb. bentonite chips in hole, hydrated them, and reamed the hole to seal open zone. 4)Lost circulation at about 20 ft. Circulated about 120 US gallons of bentonite mud until circulation returned, then re-bored the drill hole.

Well Installation Notes:
1) Hole flushed with freshwater.
2) Installed 2 in. diameter standpipe piezometer.

INSTRUMENT DETAILS



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KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2004 Geotechnical Investigation
LOCATION: Tailings Facilty Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** DH04-05-PZ

TEST HOLE LOG

Su - ksf
1 2 3 4

STARTED: 12/4/2004 **FINISHED:** 12/5/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 192.0
COORDINATES (ft): N 53950 E 40016

VANE	FIELD	LAB	
PEAK	◆	■	▲ UC/2
REMOLD	◇	□	△ P.PEN/2

★ % FINES ● SPT N

W_P% W% W_L%
× — — — — ○ — — — — ×
20 40 60 80

DEPTH (feet) | SPT BLOWS PER 6" | RECOVERY (in.) | SAMPLE No. | SYMBOL | DESCRIPTION OF MATERIALS | INSTRUMENT DETAILS

60
70
80
90
100

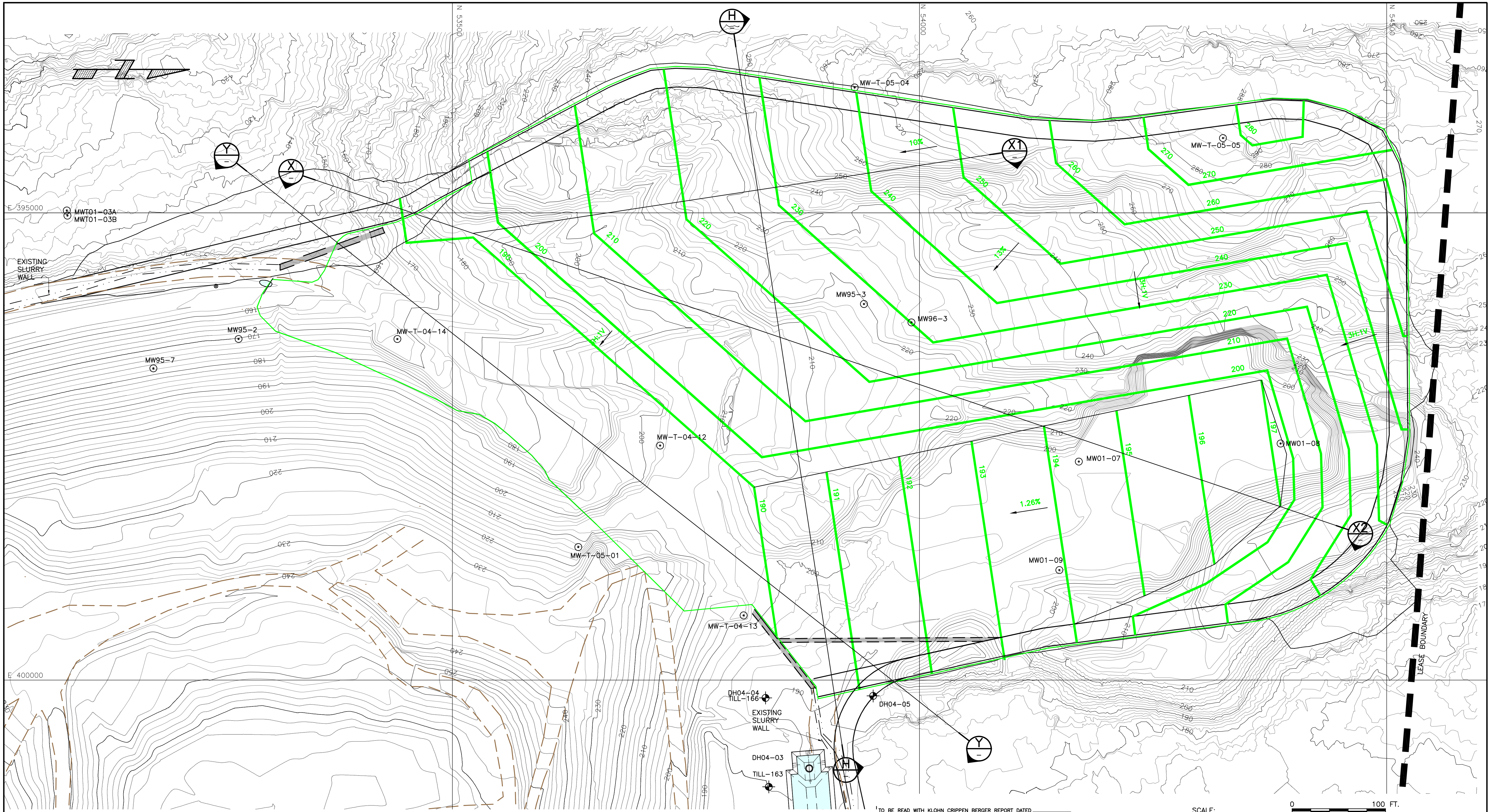
3) 5 ft long No. 20 slotted screen;
4) 25 ft threaded pvc pipe with o-ring seals.
5) Bottom of piezometer at 26.2 ft below collar; 3.8 ft stickup.
6) Static water level in drill hole during installation at 3.2 ft below collar.
7) Clean silica sand to approx. 20.2 ft below collar.
8) 25 lb bentonite chips poured into hole to seal screened interval.
9) Poured in about 15 US gallons of cement-bentonite grout. Grout did not come to surface. Added more grout 2 days later to surface.

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2004 Geotechnical Investigation
LOCATION: Tailings Facilty Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** DH04-05-PZ

APPENDIX II

Northwest/ Pit 5 Slope Stability Plan and Sections



TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED ______

SCALE: 0 100 FT.

LEGEND

▬ ▬	SLURRY WALL EXTENSION
—	EDGE OF LINER
191 —	LINER FOUNDATION CONTOUR

DRAFT

NOT FOR CONSTRUCTION

CLIENT
KENNECOTT GREENS CREEK
MINING COMPANY

Klohn Crippen Berger

PROJECT
STAGE 2 TAILINGS EXPANSION
2007 CONSTRUCTION

TITLE
NORTHWEST PIT 5 EXPANSION
STABILITY PLAN

PROJECT No.
M07802 A47

FIG. No.
FIGURE 4.1

REV.
–

KCB-R-ILD

Time: 10:26:27
Date: 1/25/2007
Scale: 1=1(PS)
Drawing File: M\M07802\A47 – 2006 Stage 2 Design\400 Design\410 Drawings\2007Design_Letter\Fig4.1.dwg (awong)
Xrefs: Tails1031cond, GEN ARR

Stability Section H (E to W)- Peak

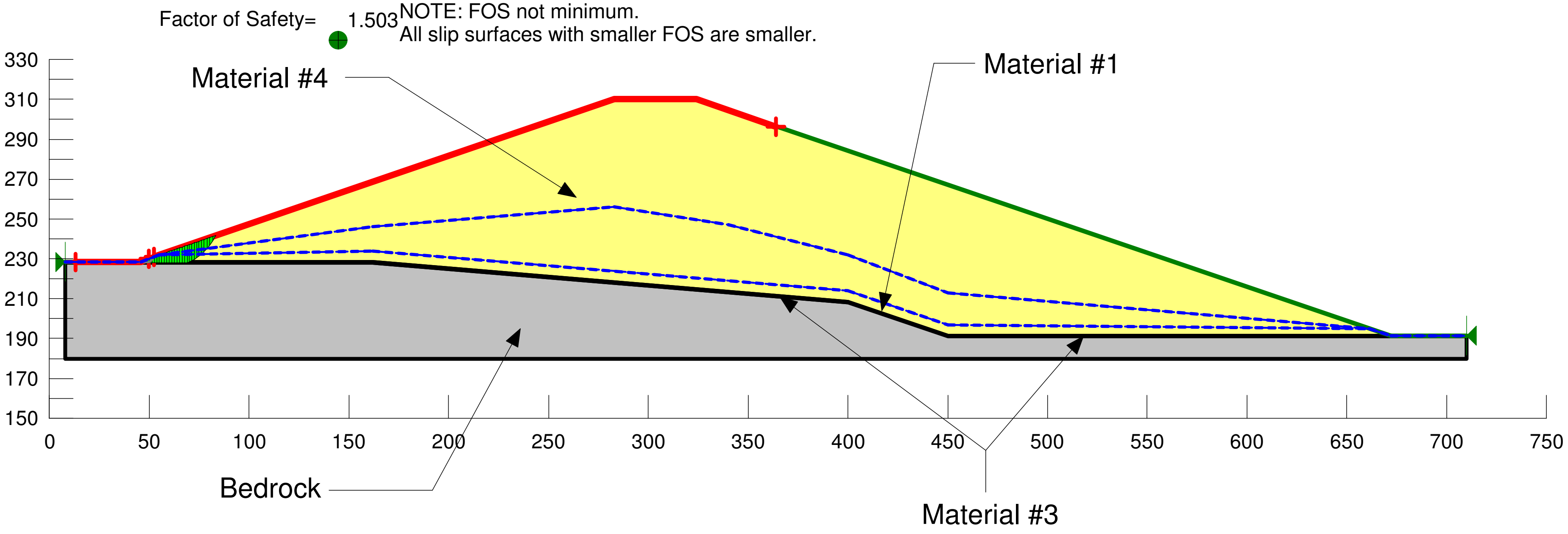
Comments: NW/Pit 5 Section H- Static
Description: 2006 Liner with sand interlayer
on horizontal benches
Name: Section H peak (rev1)-static.gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner under crest
3 ft above liner at toe

Material #: 3
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 23
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 24
Piezometric Line: 1

Material #: 4
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 39
Piezometric Line: 2



Factor of Safety= 1.503 NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller.

Stability Section H (E to W)- Residual

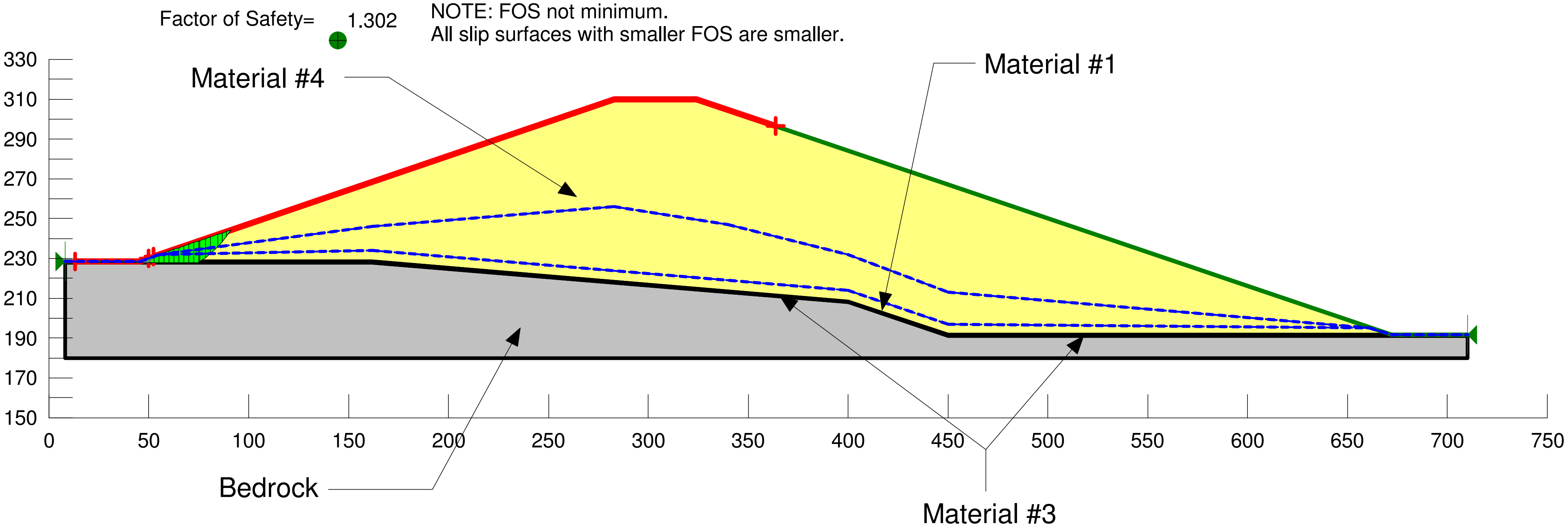
Comments: NW/Pit 5 Section H- Static
Description: 2006 Liner with sand interlayer
on horizontal benches
Name: Section H res (rev1)-static.gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner under crest
3 ft above liner at toe

Material #: 3
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 12.5
Piezometric Line: 1

Material #: 4
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 2



NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller.

Stability Section H (E to W)- Post- Liquifaction

Comments: NW/Pit 5 Section H- Static
Description: 2006 Liner with sand interlayer
on horizontal benches
Name: Section H Liq (rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner under crest
3 ft above liner at toe

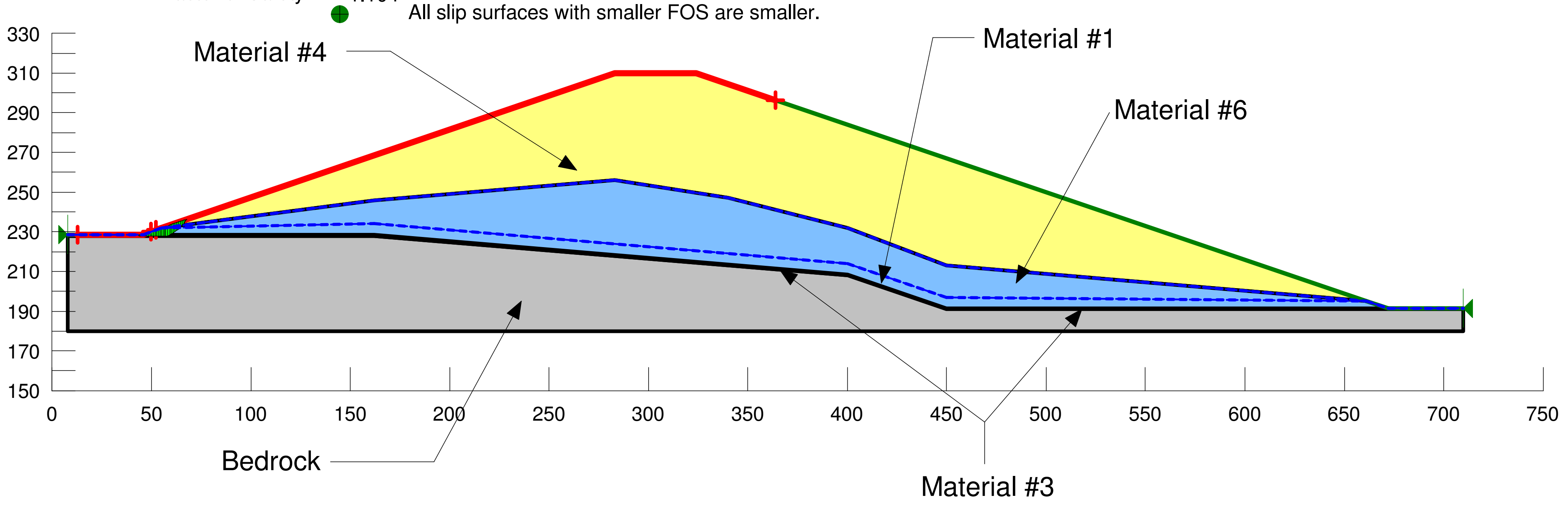
Material #: 3
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 23
Piezometric Line: 1

Material #: 6
Description: Post Liquifaction New Tails
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 28.4
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 24
Piezometric Line: 1

Material #: 4
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 39
Piezometric Line: 2

Factor of Safety= 1.104 NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller.



Stability Section H (W to E)- Peak

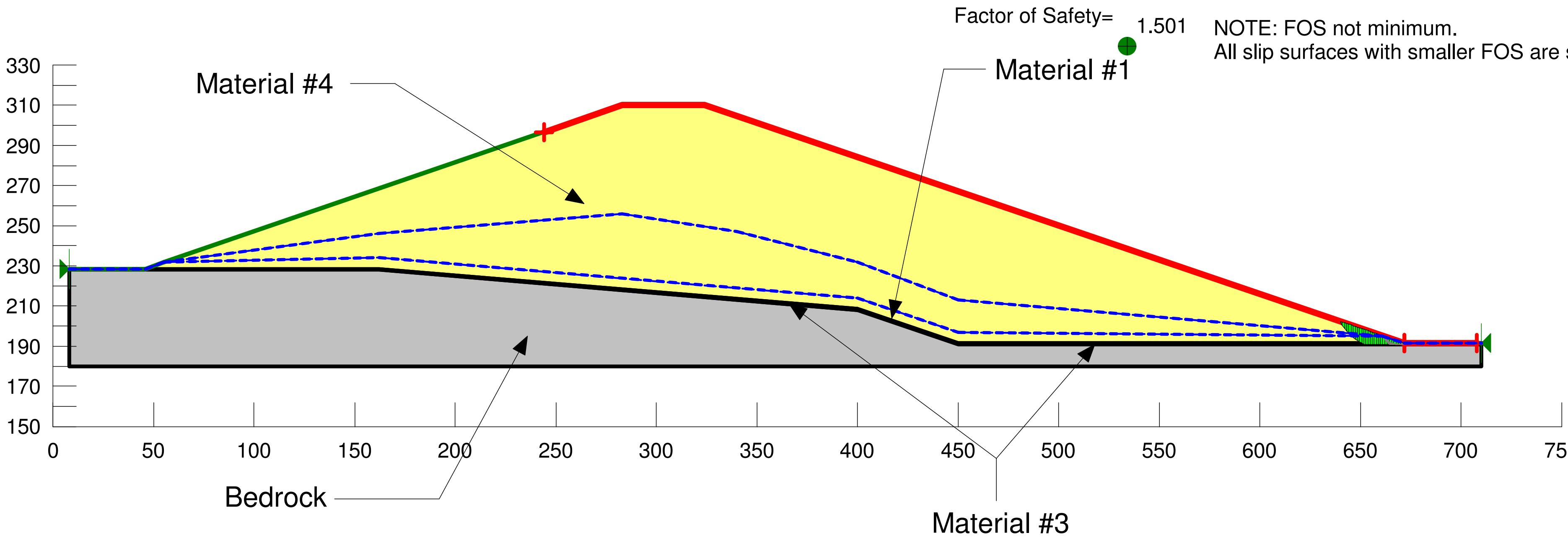
Comments: NW/Pit 5 Section H- Static
Description: 2006 Liner with sand interlayer
on horizontal benches
Name: Section H peak L-R(rev1)-static.gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner under crest
3 ft above liner at toe

Material #: 3
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 23
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 24
Piezometric Line: 1

Material #: 4
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 39
Piezometric Line: 2



NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller.

Stability Section H (W to E)- Residual

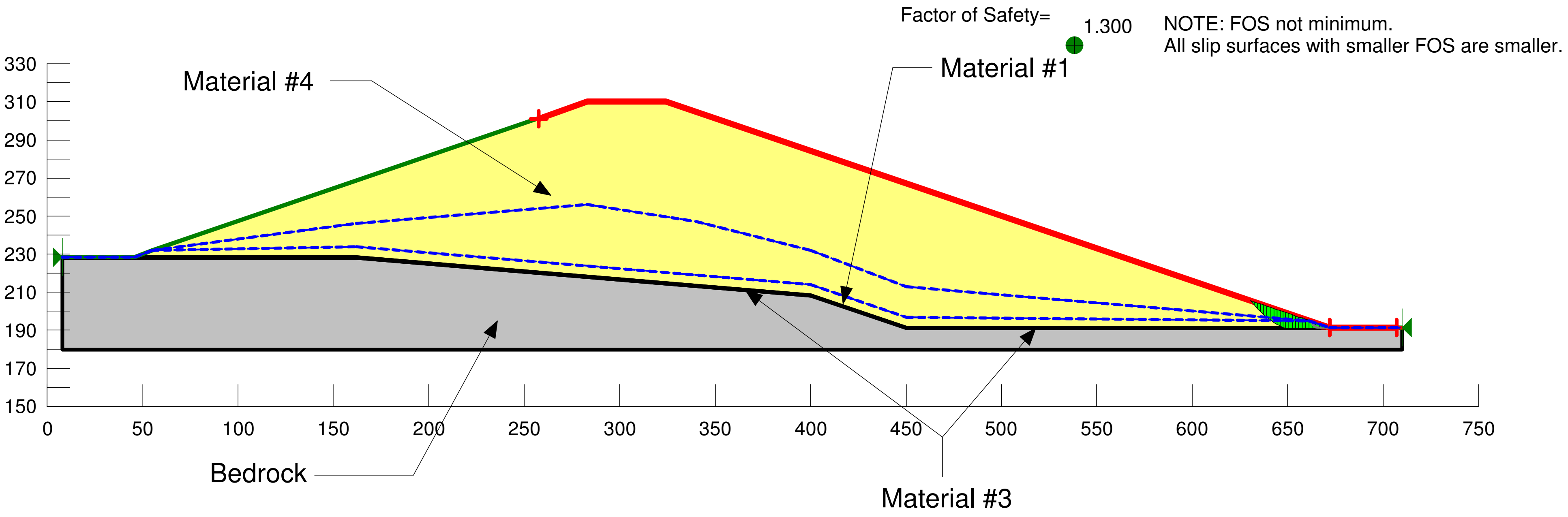
Comments: NW/Pit 5 Section H- Static
Description: 2006 Liner with sand interlayer
on horizontal benches
Name: Section H res L-R(rev1)-static.gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner under crest
3 ft above liner at toe

Material #: 3
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 12.5
Piezometric Line: 1

Material #: 4
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 2



Factor of Safety= 1.300

NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller.

Stability Section H (W to E)- Post-Liquifaction

Comments: NW/Pit 5 Section H- Static
Description: 2006 Liner with sand interlayer
on horizontal benches
Name: Section H Liq L-R(rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

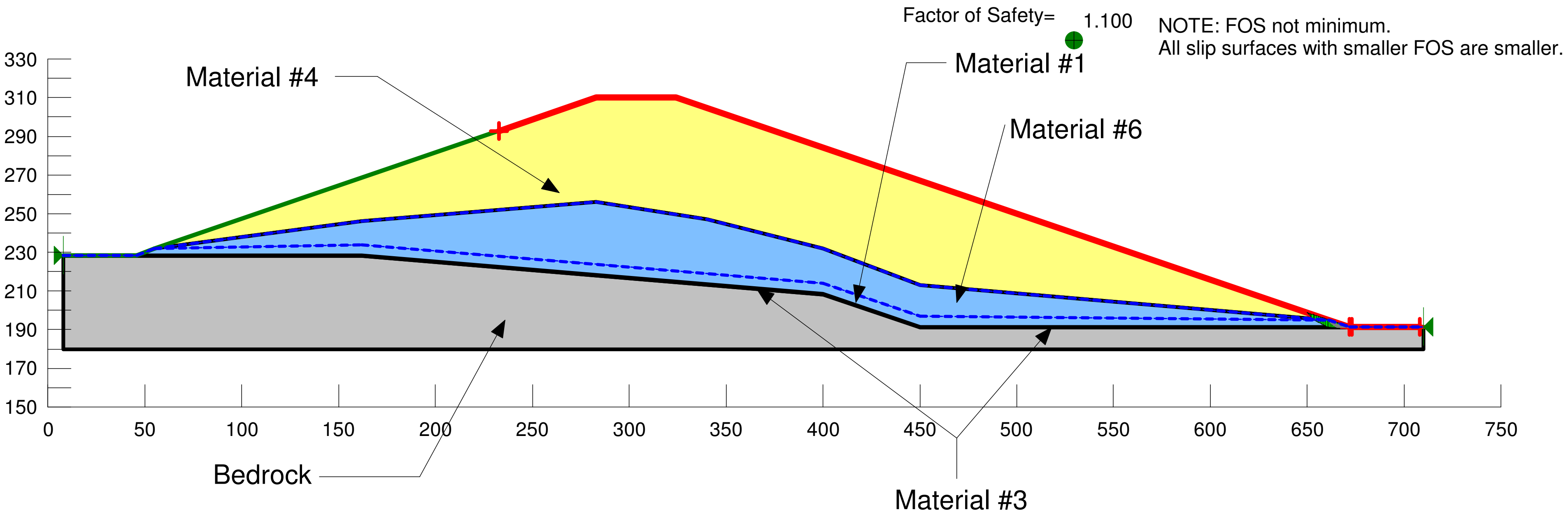
Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner under crest
3 ft above liner at toe

Material #: 3
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 23
Piezometric Line: 1

Material #: 6
Description: Post Liquifaction New Tails
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 28.4
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 24
Piezometric Line: 1

Material #: 4
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 39
Piezometric Line: 2



NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller.

Stability Section X1- Peak

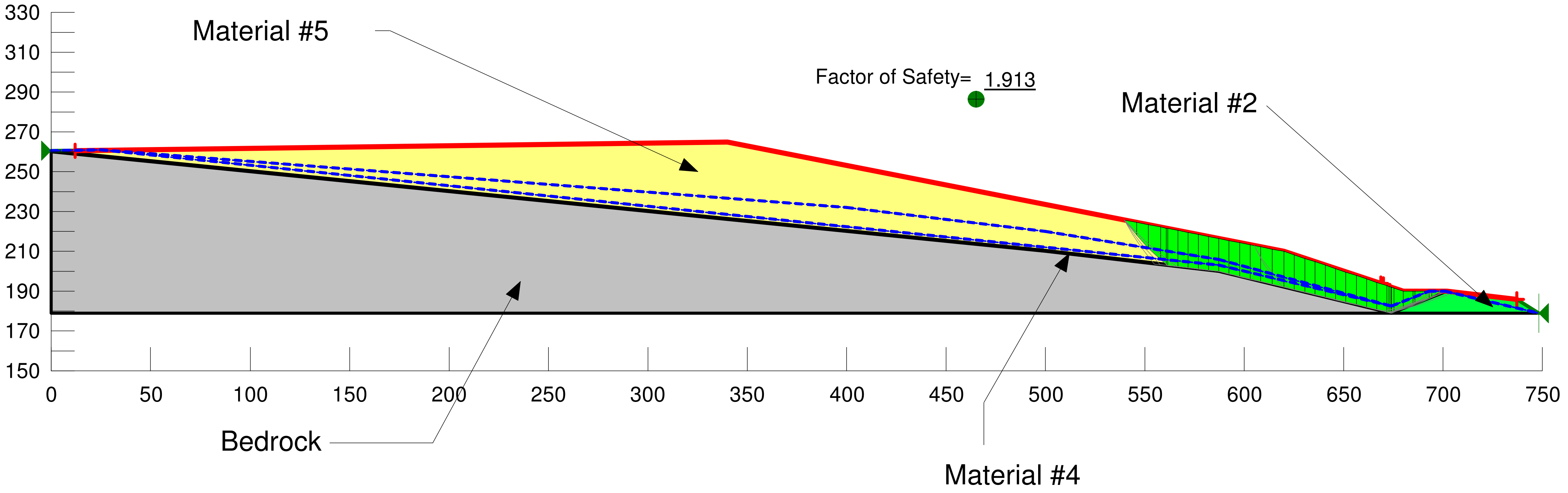
Comments: NW/Pit 5 Section X1- Static 061211
Description: 2006 Liner with sand interlayer
on all slopes
Name: Section X1 peak (rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 39
Piezometric Line: 1

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at crest
- 3 ft above liner at toe

Material #: 2
Description: Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 23
Piezometric Line: 2



Stability Section X1- Residual

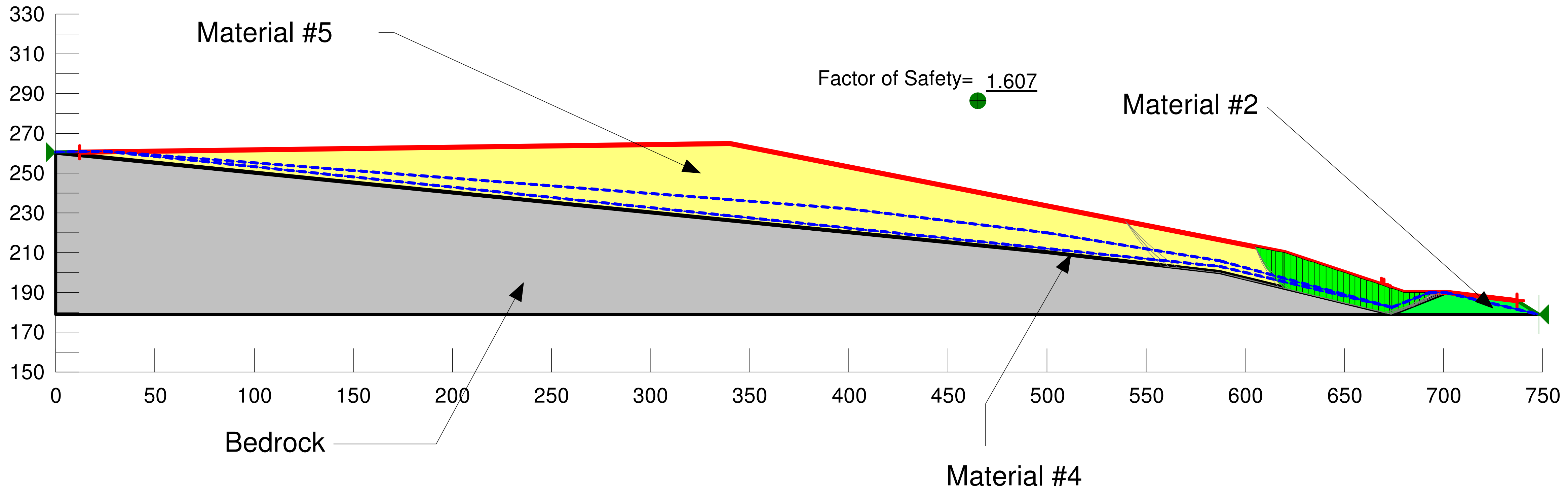
Comments: NW/Pit 5 Section X1- Static 061211
Description: 2006 Liner with sand interlayer
on all slopes
Name: Section X1 res (rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 1

Material #: 2
Description: Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 2

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at crest
- 3 ft above liner at toe



Stability Section X1- Post-Liquifaction

Comments: NW/Pit 5 Section X1- Static 061211
Description: 2006 Liner with sand interlayer
on all slopes
Name: Section X1 Liq (rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

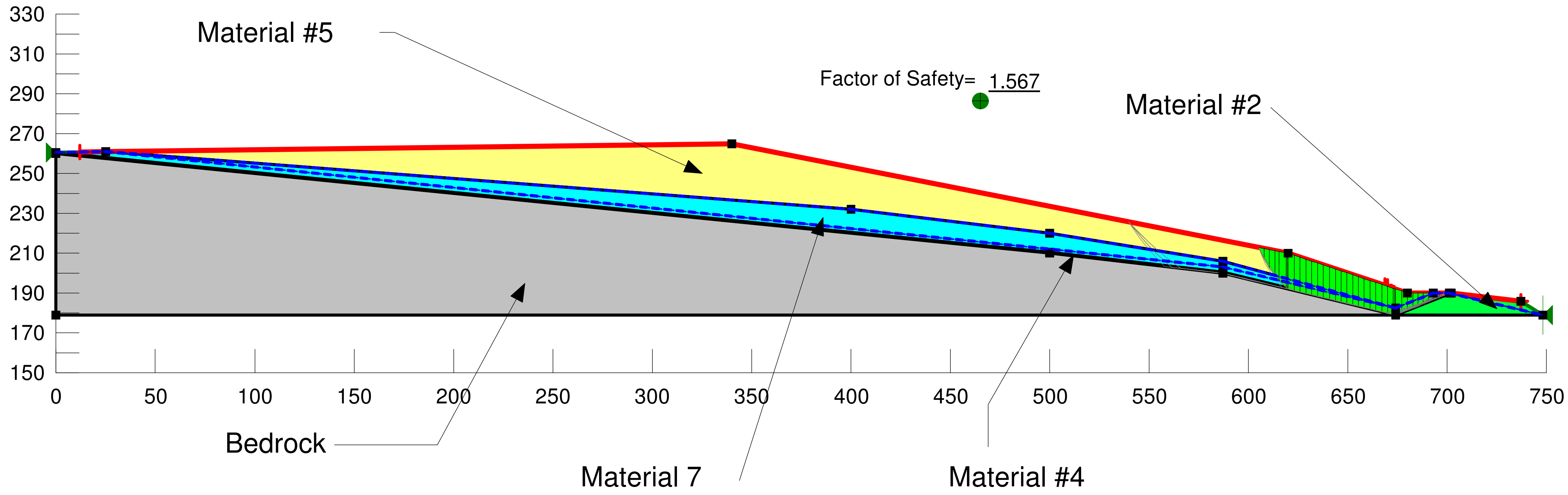
Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at crest
- 3 ft above liner at toe

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 1

Material #: 2
Description: Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 7
Description: Post Liquifaction New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 28.4
Piezometric Line: 1

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 2



Stability Section X2- Peak

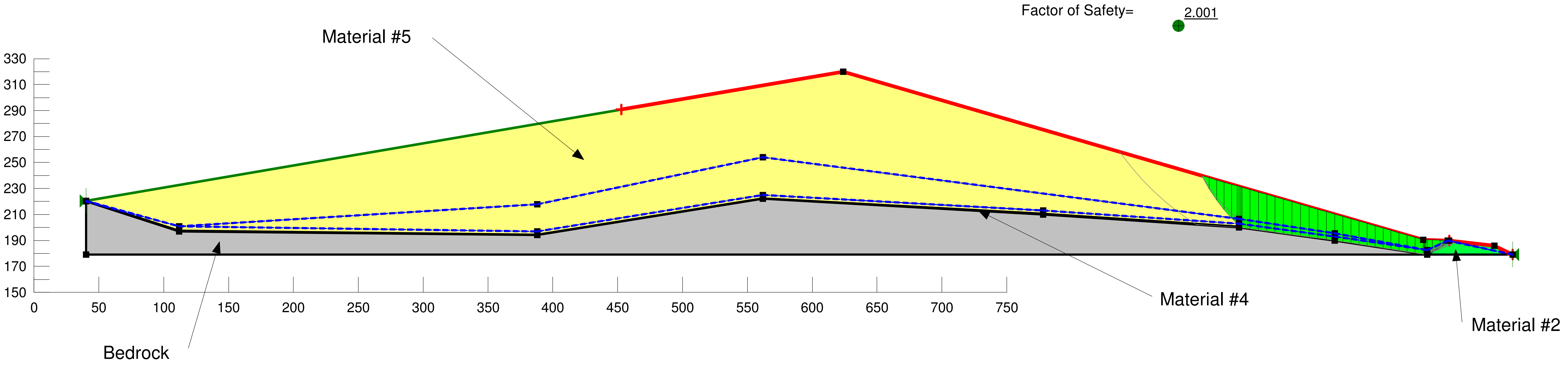
Comments: NW/Pit 5 Section X2
Description: 2006 Liner with sand interlayer
on all slopes
Name: Section X2 peak(rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at crest
3 ft above liner at toe

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 39
Piezometric Line: 1

Material #: 2
Description: Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 23
Piezometric Line: 2



Stability Section X2- Residual

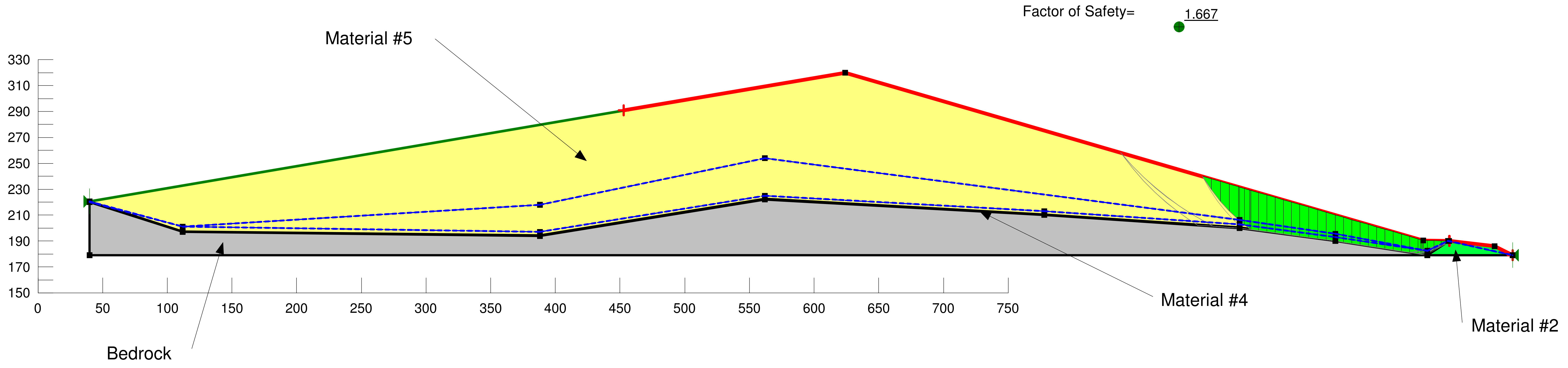
Comments: NW/Pit 5 Section X2
Description: 2006 Liner with sand interlayer
on all slopes
Name: Section X2 res(rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at crest
3 ft above liner at toe

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 1

Material #: 2
Description: Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 2



Stability Section X2- Post Liquifaction

Comments: NW/Pit 5 Section X2
Description: 2006 Liner with sand interlayer
on all slopes
Name: Section X2 Liq(rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

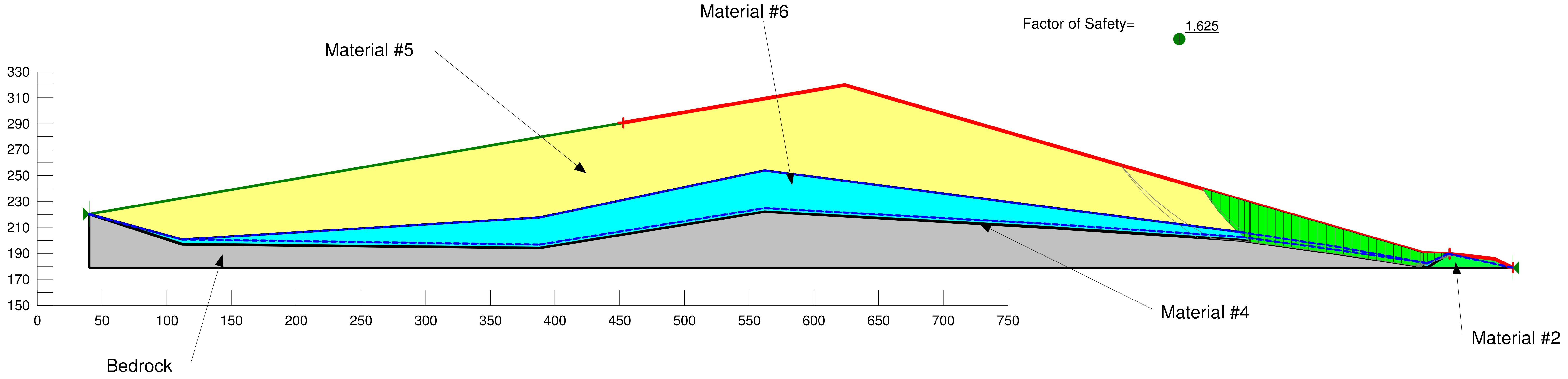
Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at crest
3 ft above liner at toe

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 1

Material #: 2
Description: Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 6
Description: Post Liquifaction New tailings
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 28.4
Piezometric Line: 1

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 2



Stability Section Y- Peak

Comments: NW Excavation
Description: 2006 Liner with sand interlayer
Name: NW Excavation peak(rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

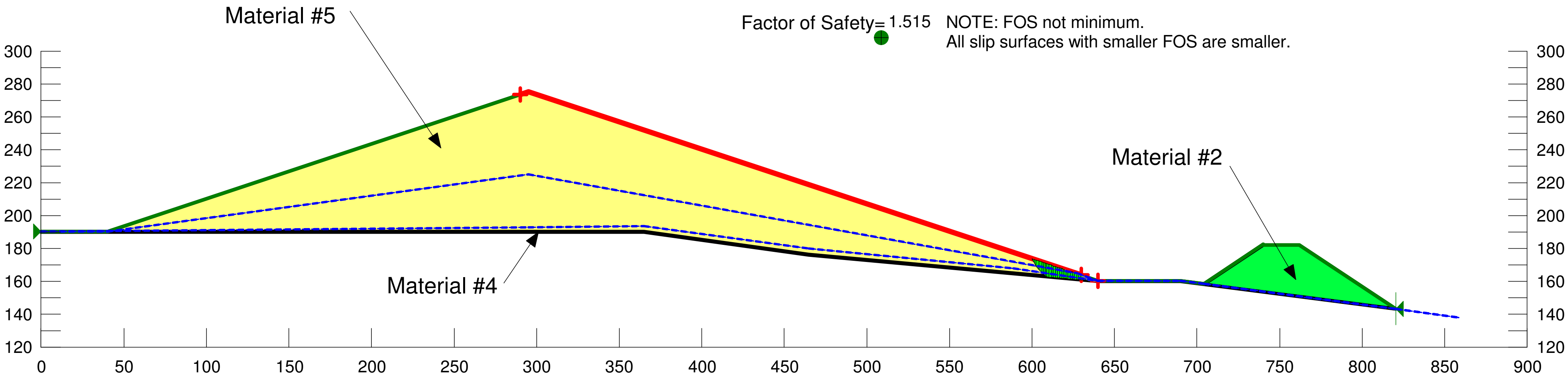
Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at Crest
3 ft Above Liner at toe

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 39
Piezometric Line: 1

Material #: 2
Description: Compacted Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 24
Piezometric Line: 0

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 23
Piezometric Line: 2



Stability Section Y- Residual

Comments: NW Excavation
Description: 2006 Liner with sand interlayer
Name: NW Excavation res (rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

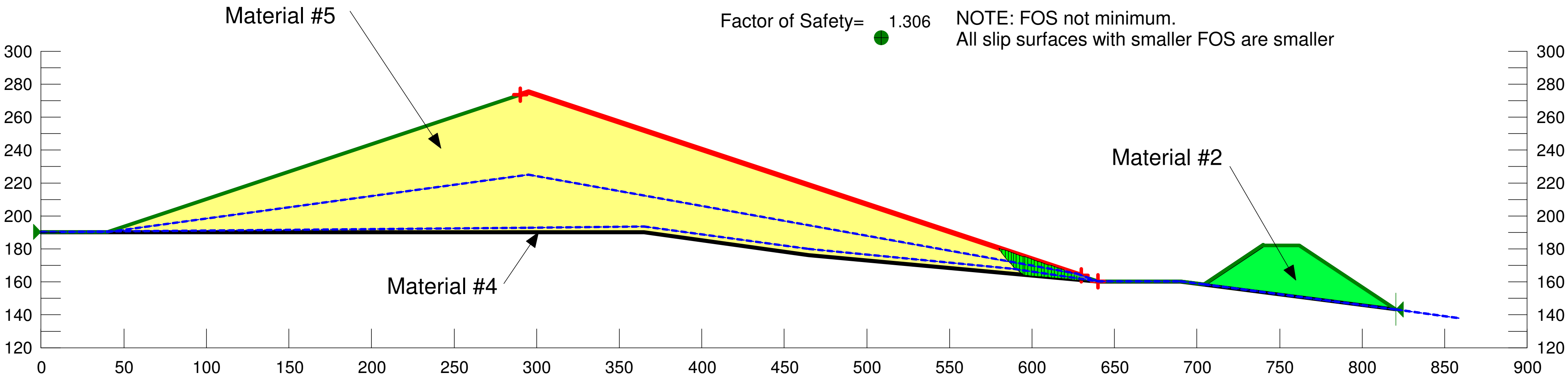
Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at Crest
3 ft Above Liner at toe

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 1

Material #: 2
Description: Compacted Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 1
Description: 2006 Liner (no sand)
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 12.5
Piezometric Line: 0

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 2



Factor of Safety= 1.306

NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller

Stability Section Y- Post-Liquifaction

Comments: NW Excavation Section Y
Description: 2006 Liner with sand interlayer
Name: NW Excavation Liq (rev1).gsz
Analysis Method: Morgenstern Price
P.W.P. Option: Piezometric Line
Seismic Coefficient: horz:0, vert:0

Phreatic Surfaces:
Geosynthetic Liner - 3 ft above Liner
New Tailings - 35 ft above liner at Crest
3 ft above liner at toe

Material #: 5
Description: New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 32
Piezometric Line: 1

Material #: 2
Description: Compacted Rock Fill
Model: MohrCoulomb
Wt: 120
Cohesion: 0
Phi: 40
Piezometric Line: 1

Material #: 6
Description: Post Earthquake New Tailings
Model: MohrCoulomb
Wt: 128
Cohesion: 0
Phi: 28.4
Piezometric Line: 1

Material #: 4
Description: 2006 Liner w/ Sand interlayer
Model: MohrCoulomb
Wt: 125
Cohesion: 0
Phi: 20
Piezometric Line: 2

Factor of Safety= 1.115

NOTE: FOS not minimum.
All slip surfaces with smaller FOS are smaller.