KLOHN CRIPPEN

March 1, 2006

Kennecott Greens Creek Mining Company
P.O. Box 32199
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Mr. Tom Zimmer
Surface Operation Manager

Dear Mr. Zimmer:

Greens Creek Mine
Stage 2 Tailings Expansion Overall Stability Update

We are pleased to submit ten (10) copies of our report titled "Stage 2 Tailings Expansion Overall Stability Update".

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

Yours truly,

KLOHN CRIPPEN CONSULTANTS LTD.

Robert W. Chambers, P.Eng. (B.C.)
Project Manager

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EXECUTIVE SUMMARY

The Kennecott Greens Creek Mine (KGCMC) tailings storage facility (TSF) has been undergoing an incremental expansion since 2004. The expansion is scheduled to take place over about 5 years and is designed to increase the capacity by about 4.7 million dry short tons. The expansion plan was presented in concept in the Design Overview for Forest Service Submission (Klohn Crippen, 2004). The expansion includes extension of the pile into 5 main areas known as the Southeast, Northeast, Northwest, Pond 6, and the Southwest expansion areas. As each area is developed, detailed designs are prepared taking into account overall design requirements regarding seepage control and drainage, constraints of existing construction, local ground conditions, temporary construction constraints and incorporation of new performance data. These performance data include new material strength values and updated compaction and piezometer data. Using this incremental approach rather than adopting one detailed design which is fixed for the 5-year scheduled expansion, KGCMC is able to respond to changing conditions and can build on experience.

As the TSF construction has unfolded over the years, proven construction techniques such as placement of under-drains, installation of piping, collection and distribution of leachate, and control of contact and non contact waters have been developed. These techniques have helped KGCMC construct a well organized containment for the tailings in a wet environment, having very limited surface area.

Development of the site is not without challenges and KGCMC expend much effort in managing day to day tailings placement to accommodate the wet climate and tight space restrictions. Construction is monitored through regular surveys of the TSF, frequent nuclear densometer (Troxler) testing, periodic balloon density readings, piezometer readings in both the tailings and natural ground, and observations by KGCMC staff.

Over the course of Klohn Crippen's involvement on the project, there have been a number of areas where there has been uncertainty in the geotechnical aspects of the pile. As data on the tailings strength and the piezometric surface have become available during development, the application of conventional industry practice analytical techniques demonstrates (Klohn Crippen, 2003c) that both the static stability and the seismic stability of the TSF under the Design Basis Earthquake (DBE), will be adequate. However, special measures are needed to achieve the design safety factors in lined areas.

Some uncertainty remains in the performance of the TSF under the more severe Maximum Design Earthquake (MDE) condition which is the design criteria event for closure. These factors for closure include reliable prediction of the groundwater table and distribution of saturation in the TSF over the very long term.

This report provides an update on the geotechnical design for the TSF with a special focus on stability. In preparing this report, a detailed assessment was made of geotechnical data previous to 2004 and in addition, new data were collected. The new data included drilling and SPT sampling, and laboratory testing conducted on critical materials. These materials include the tailings and a shallow, thin, intermittent sand and gravel layer, which is located beneath the surface peat.

In addition to a review of the geotechnical data, the report focuses on two main issues:

- Seismic behavior of the shallow sand and gravel layer; and
- Seismic behavior of the tailings (liquefaction and deformation potential).

These issues control the long term stability of the TSF. Liquefaction and the associated phenomena of cyclic mobility (lumped together in this report under the term liquefaction), require shaking of loose saturated or near saturated soil. Hence a critical part of the analysis is the ability to predict the distribution of saturation in the pile and the location of the phreatic surface. The phreatic surface was provided by EDE. For this evaluation, it is assumed that only material below the phreatic surface is saturated.

During preparation of the 2004 Design Overview for Forest Service Submission (KC 2004), it was suspected that SPT data previous to 2004 had underestimated the density of the shallow sand and gravel layer, partly because of the very thin and intermittent nature of the layer, which often lead to inclusion of peat within the blow count zone. In addition, drilling disturbance, especially where hollow stem augers were used, was suspected to have contributed to low SPT values. Consequently, in 2004/2005 very careful SPT testing was undertaken using mud rotary techniques with hammer energy and velocity measurements. Our interpretation of the 2004/2005 drilling and re-interpretation of previous data, excluding partial SPTs and SPTs impacted by disturbance due to drilling, leads to the conclusion that the shallow sand and gravel layer, is dense. Except for a local area beneath the Northeast expansion, the average SPT $(N_1)_{60cs}$ value is higher than previously estimated, averaging about 30 blows per foot. Consequently, the shallow sand and gravel layer is not liquefiable, except locally in the northeast corner, where it will be removed prior to expansion into that area.

The seismic behavior of the tailings has been more difficult to assess. In situ techniques such as SPT and CPT, which are widely used for liquefaction assessment of cohesionless soils, have proven to be not well suited to assessment of the KGCMC tailings. SPT is a technique designed for use in natural granular soils or soils with up to about 35% silt content. In these soils, pore pressure generated by SPT can be accounted for or is not a major factor. However, the KGCMC tailings contain over 80% silt and behave similar to a clayey non liquefiable soil under conventional CPT analysis. Conversely, SPT $(N_1)_{60cs}$ analysis suggests that parts of the saturated tailings could liquefy under the MDE (but not under DBE). To help resolve this inconsistency, a limited program of non-destructive shear wave testing was completed. In situ shear wave velocity measurement has been widely used for liquefaction assessments and the results from the TSF suggest that the tailings would not liquefy under the MDE.

To further assess tailings liquefaction potential, a series of laboratory tests were conducted on samples re-constituted in the laboratory. The tailings were tested in both cyclic triaxial and cyclic shear box apparatus, using material with as-placed moisture contents and at a starting density as low as 88% of standard Proctor density (SPD), which is below the specified minimum for placement of 90% SPD. The laboratory tests indicate that the tailings would not liquefy under the MDE loading and has a safety factor against liquefaction of between 1.1 and 1.5 depending on how the tests are interpreted.

Overall, KC concludes that the weight of evidence indicates that the new tailings will not liquefy under the MDE, although some softening could occur below the water table. The old tailings may be liquefiable under MDE. For the analyses in this report the old tailings below the water table were assumed to liquefy. This was done as a precautionary step to consider the potential consequences if, in the future, it is fully determined that the old tailings are liquefiable. We have recommended that sampling and testing of the tailings continue over the operating life of the mine to check this conclusion.

The performance of tailings under seismic loading is the subject of investigation in many research institutions and the understanding of behavior of silt under seismic loading will improve in the future. Consequently, while KC believes that the new tailings will not liquefy, we also believe that it is sensible to assess the consequence of such liquefaction, in case subsequent data or the evolving state of practice were to result in a different conclusion. Consequently this report includes, in Appendix VIII, a detailed analysis of the stability of the TSF under the assumption that all tailings below the water table liquefies. Our conclusion is that in this case a modest rock toe berm around the West Buttress would suffice to prevent flow failure of the pile and limit deformation to the order of several feet. On other sides of the pile berms are not needed.

The above conclusions, as they relate to closure, could change if the predicted water table is higher or if the tailings pile above the water table is almost fully saturated on closure. Consequently, the report recommends that KGCMC look closely at the closure water level and saturation level predictions. We have provided some sensitivity analysis to show how the TSF stability could be affected by variations in the water table level. KGCMC and their design consultants have some control over the closure water table since the final cover design can be used to control water levels and saturation.

In addition to looking at the pile stability under seismic loading, the report includes an assessment of stability of areas with HDPE composite under liners. The stability of these areas was checked using laboratory derived residual strength values of the liner materials used in the 2004 and 2005 construction of the Southeast expansion area. Some modifications to the Southeast area design were necessary to accommodate a low residual strength at the HDPE/geotextile contact within the composite liner. This experience will be used in the design of other lined areas.

Laboratory static shear box tests including peak and residual tests were conducted to assess the frictional strength of the tailings. This has resulted in adoption of higher than previously assumed peak strengths for both the old (pre 1996) and the new tailings.

The strength data to date indicate the following:

- Old tailings average peak friction angle = 33º
- Old tailings average residual friction angle = 32º

- New tailings lower bound peak friction angle = 39º
- New tailings lower bound residual friction angle = 32º

Previous stability analyses used friction angles of 28º and 32º for old and new tailings, respectively. These previous design values were based on 1997 CPT results.

The current analyses use a new tailings design peak friction angle of 39º with a sensitivity range from 32º to 42º. The design peak friction angle for the old tailings is 33º with a sensitivity range from 28º to 33º.

These revised design friction angles are shown, in Appendix VIII, to have little impact on the design, since the pile performance is governed by the seismic condition.

A stability assessment was also completed on temporary construction conditions looking at temporary slopes but also at the likelihood of generating pore pressure during construction. A reasonable operating criteria is that pore pressure in the pile should not be allowed to approach 70% of the height of tailings above the measuring point, without undertaking a stability assessment. Pore pressure analysis based on consolidation parameters derived from laboratory data shows that, provided the under drains perform as they have done to date, there will be no significant sustained pore pressure rise in the tailings or foundation due to construction at the average annual placement rate of 7.2 ft (0.6 ft/month). This conclusion is confirmed by observed piezometer performance.

An additional analysis was done for the Southeast corner assuming average TSF placement rates of rise of 4.5 ft/month and 9 ft/month. The Southeast corner is the currently active storage site. This analysis shows, in general, that placement rates should be limited to less than 4.5 ft per month for no more than 6 months in any one area. Areas with placement rates faster than this or maintained for a longer period should be assessed for stability. The report recommends that piezometers be installed in areas whose sustained rate of rise exceeds 4.5 ft/month for 6 months; the piezometer locations would be selected on a case by case basis to be placed in the lower third; middle third and upper third of the planned placement.

In conclusion, the report recommends a number of activities to be continued over time to provide back up for the design assumptions and for continued improvement of understanding of the TSF behavior. These recommendations include:

- Re-evaluate the TSF closure water table and saturation levels utilizing a saturated-unsaturated flow model with infiltration values appropriate to the final closure cover design;
- Install instruments and regularly monitor water pressure and saturation and compare to modeled predictions and calibrate the model as necessary. Include identification of perched water tables;
- Liaise with closure cover designers so that the long term water level is included as a criteria in final cover design;
- Monitor water levels during construction and adjust the rate of fill rise in a given area if construction pore pressures higher than the 70% of the tailings thickness are measured;
- Continue to strive to compact the tailings to as high a density as possible but no lower than 90% SPD.
- Undertake on-going index and engineering property tests on the tailings to help improve the understanding of the seismic behavior of the KGCMC tailings.

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DRAWINGS

1. INTRODUCTION

Greens Creek Mine is located on northern Admiralty Island, about 18 miles southwest of Juneau, Alaska (Drawing D-41001), and is jointly owned by HECLA and Kennecott Greens Creek Mining Company (KGCMC) and is operated by KGCMC. It is an underground polymetallic (zinc, silver, gold and lead) mine. 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 Storage Facility (TSF). To accommodate the projected mine tailings storage requirements, an incremental expansion of the Tailings Facility storage capacity, hereafter referred to as the Stage 2 Expansion, began in 2004 and is expected to proceed until 2007.

The Stage 2 Expansion represents approximately 80% of the total tailings capacity increase outlined in the Final Environmental Impact Statement (FEIS), and will increase the capacity of the TSF (over the previously permitted pile configuration) by about 2.7 million yd^3 (about 4.7 million dry short tons). Drawing D-41002 shows the general arrangement of the Stage 2 Expansion overlaid on the existing tailings facility, including the new truck wash facility (constructed in 2004) and the new storm water retention Pond 7 (constructed in 2005).

Regulatory approval for the tailings facility expansion was granted after a National Environmental Policy Act (NEPA) review by the USDA Forest Service (USFS) and other Federal, State and Local Agencies. With the USFS as the lead agency, a FEIS was issued on October 24, 2003 with a Record of Decision supporting Alternative C of the tailings disposal expansion plan. The tailings facility is operated under a Waste Management Permit # 0211-BA001 issued by the Alaska Department of Environmental Conservation (ADEC) on November 7, 2003 (ADEC, 2003), as well as other Federal, State, and Local permits.

2. DESCRIPTION OF TAILINGS FACILITY EXPANSION

2.1 Tailings Pile Geometry

The tailings pile will be constructed with maximum 3H:1V final external slopes. The maximum elevation of the tailings pile will be up to El. 330 ft (Drawing D-41005), not including the final cover.

2.2 Expansion Schedule

The individual expansion and infrastructure areas are shown on Drawing D-41002 and their expected in-service dates are given in Table 2.1. The expansion plans past 2005 are estimates and could change based on the actual mine production and further optimization of the tailings site, including items such as relocation of production rock to the Tailings Facility. Selected drawings are provided in Appendix IX for the expansion areas listed in Table 2.1.

Table 2.1 Planned Storage and Infrastructure Development/In-Service Schedule

AREA	DEVELOPMENT DATE	IN-SERVICE DATE GOAL	GEO- MEMBRANE LINED AREA*	COMMENTS
Southeast 1 (Truckwash) Expansion	2004	October, 2004	Yes	Received tailings as of October, 2004
Southeast 2 (Tank No. 6) Expansion	2005	September, 2005	Yes	Received tailings as of September 2005
Pond No. 7	2004-2005	January, 2006	Yes	Storm Water Pond
Northeast Expansion	2006	2006	Partial	Excavation to remove a loose sand and gravel layer will be carefully planned in final design to control impacts to the groundwater, and adjacent existing slurry walls and to provide a safe excavation.
Pond No. 8	2007-2008	-	Yes	Optional Pond
Pond No. 9	2006-2008	-	Yes	Storm Water Pond – required for Northeast expansion
Northwest Expansion	2006-2008	-	Yes	KGCMC reviewing layout
Pond No. 6 Tailings Expansion	2006-2008	-	No	In Detailed Design Phase
Southwest Expansion	2007-2008	-	Yes	In Detailed Design Phase

* The requirement for a geomembrane liner is specified by KGCMC.

2.3 Construction Considerations

Tailings from the milling process are de-watered to approximately 12% to 14% gravimetric moisture content at the mill with a portion (about 50%) placed underground as backfill and the remainder (currently up to 350,000 dry short tons (DST) per year) trucked approximately seven miles, and placed on the surface in a "dry" configuration at the TSF. The tailings are placed according to the KGCMC General Plan of Operation (GPO), Appendix 3 – Tailings Impoundment.

The tailings are placed in "cells", which are placement regions defined by KGCMC. KGCMC determines which cells receive tailings on a given day, and when placement will be moved to a different cell or group of cells. A KGCMC drawing of the tailings pile is included in Appendix IX which shows the schematic arrangement of cells.

The GPO states that the tailings should be spread in approximate 1 ft to 2 ft lifts using a Caterpillar D6 bulldozer and compacted using a drum roller to a minimum of 90% of the maximum standard Proctor dry density. KGCMC does periodic density testing, with a nuclear gauge or balloon densometer to determine whether the minimum density is being achieved.

In 2003 Klohn Crippen (KC) completed a detailed analysis of in situ density testing methods at the TSF. This included construction of a mold into which tailings at a known density could be placed. The nuclear gauge overestimated the density of the tailings in the mold by 6% to 12%. This assessment included a re-evaluation of the nuclear gauge calibration constants by the manufacturer Troxler. Side by side nuclear gauge and Washington balloon density testing was also carried out for this assessment and the nuclear gauge was found to give values about 8% to 12% higher than the Washington balloon method. Based on the Klohn Crippen comparison of density testing data completed in 2003 (KC 2003b), the average density at the test sites using various methods (e.g. nuclear gauge and Washington balloon) varied from 81% to 114% of the maximum standard Proctor dry density with an average of 100%, and a standard deviation of 8.4%. KGCMC 2005 test data using the nuclear gauge and Washington Balloon give a range of density values from 68% to 120% with an average of 99% and a standard deviation of 12.9%. Test data prior to 2003 were not considered, as the nuclear gauge calibration constants used were inaccurate.

KGCMC believe there is some bias in the reported density test results for the following reasons:

- KGCMC report that they periodically remove material placed below the specified minimum density and recompact it. This would tend to result in an under estimate of density since it was not possible for KC to identify tests from areas subsequently removed and recompacted; and
- KGCMC report that, especially in wet weather, trafficability difficulties require that tailings be placed on sloping lifts up to 5ft thick with nominal compaction by dozers. Subsequently density tests are done on the surface of the thick lifts. This would tend to cause an over estimate of the density based on the field test results.

Because of the uncertainty in the in situ density test data, KC has commenced several seismic liquefaction laboratory tests with starting densities, as low as 85% Proctor. In addition a field program is being planned for implementation during excavation of tailings in the North West of the pile in 2006. The field program will include in situ density and undrained strength tests, plus undisturbed sampling to allow assessment of a large section of the pile. This section has been placed over a long time period and in a variety of weather conditions and should provide an opportunity to achieve a detailed appreciation of in situ conditions.

In general, the as-placed moisture content varies with daily rainfall. From drill hole data, the gravimetric moisture content in the pile at the 2004 and 2005 hole locations varied from about 10% to 26% (See Figures VII-15 and VII-16 in Appendix VII).

The lifts are generally spread on inclines in the cells (up to 3H:1V), and slope toward the outer edges of the pile to promote surface runoff. The spreading and compaction technique can leave the placed tailings with a smooth, shiny surface. The compacted tailings surface is not specifically scarified prior to placement of the next lift, but heavy equipment is used to spread the tailings during placement. Lateral continuity of layering is not evident in the tailings because of the cell placement method. Nevertheless, a direct shear testing of tailings with a smooth, polished shear plane was done by KC in October 2005 - see Appendix VI. The test shows that there is minimal reduction in the tailings friction angle due to this placement method.

2.4 Water Management Facilities

Containment Pond No. 6 collects surface water runoff from the tailings pile, perimeter collection ditches, and pile underdrains, for routing to the water treatment plant. Containment Pond No. 6 also is a storm water surge pond. The North Retention Pond collects surface contact water from the northeast portion of the Tailings Facility and Pit 5 and routes it to Containment Pond No. 6. Containment Pond No. 6 is connected to Containment Pond No. 7 by an overflow ditch at El. 141 ft. Containment Pond 7 was constructed in summer 2005, and is intended to provide additional capacity to Pond 6 in the short term. Upon ultimate closure of Pond 6 (which will be used for tailings storage), Pond 7 will be the main collection pond for surface water runoff from the tailings pile, perimeter collection ditch, pile underdrains, and other facilities.

3. PHYSIOGRAPHIC SETTING

3.1 Physiography

The physiography of Admiralty Island is characterized by mountains that rise steeply from Hawk Inlet to El. 4,700 ft amsl. The TSF is located on a relatively flat lying terrace at about natural ground level El. 140 ft to 200 ft, with the majority of the facility located in the upper part of Tributary Creek Valley. The tailings pile is bounded to the north by a bedrock knoll and the headwaters of Cannery Creek, to the east by a steep mountain slope, to the west and southwest by a gently-sloped peat wetland, and to the south by the Tributary Creek Valley.

3.2 Climate

Greens Creek Mine is located in the southeastern portion of the Alaska Coastal Maritime climatic zone. The climate is characterized by moderate temperatures and abundant precipitation.

Temperature

The mean monthly temperatures at the site are comparable to those recorded at Juneau, as follows (SRK, 1982):

- mean annual temperature at Juneau is 40°F;
- mean daily maximum temperature ranges from 29°F in January to 64°F in July;
- mean daily minimum temperature ranges from 18°F in January to 48°F in July;
- record low temperature is minus 22°F and the record high temperature is 90°F.

Precipitation

Historical precipitation at the mine is reportedly similar to that recorded at Juneau.

Total precipitation data (combined rain and snow) at the Tailings Facility from 1997 to 2004 (provided by KGCMC) is summarized in Table 3.1.

Table 3.1 Total Annual Precipitation* at Tailings Facility, 1997-2004

MONTH	YEAR								MEAN	MAX.	MIN.
	1997	1998	1999	2000	2001	2002	2003	2004			
Jan	1.6	1.5	5.1	3.0	5.8	3.0	5.1	5.9	3.6	5.9	1.5
Feb	5.3	1.3	3.1	0.9	3.2	5.3	2.2	3.9	3.1	5.3	0.9
Mar	3.2	2.6	1.7	3.7	2.7	1.1	3.6	6.2	2.6	6.2	1.1
Apr	3.6	2.2	5.6	4.3	3.2	0.4	0.7	2.5	2.9	5.6	0.4
May	1.9	2.2	4.8	2.5	3.6	2.7	3.1	1.1	3.0	4.8	1.1
Jun	2.2	2.3	2.4	3.8	1.9	3.2	3.7	1.5	2.8	3.8	1.5
Jul	6.4	4.4	4.3	4.0	3.3	4.5	2.5	4.3	4.2	6.4	2.5
Aug	4.1	5.6	6.6	4.3	2.9	7.3	4.1	1.9	5.0	7.3	1.9
Sep	5.6	5.8	7.9	8.3	7.9	5.1	10.9	7.9	7.4	10.9	5.1
Oct	3.7	9.3	8.7	6.0	4.9	7.7	5.7	6.3	6.6	9.3	3.7
Nov	2.3	2.0	5.4	4.4	3.2	6.6	4.9	6.7	4.1	6.7	2.0
Dec	5.9	4.4	8.7	3.5	3.4	6.3	4.7	10.0	5.3	10.0	3.4
Total	**45.7**	**43.5**	**64.3**	**48.7**	**45.9**	**53.1**	**51.2**	**58.3**	**51.3**	**64.3**	**43.5**

* data are in inches rainfall equivalent

The moderate temperatures and high levels of precipitation at the mine result in a low evaporation rate. The average annual gross open water evaporation at the mine has been estimated at 20 inches (SRK, 1982).

3.3 Groundwater and Surface Hydrology

Groundwater flow is from the steep mountain slope east of the TSF westward toward Hawk Inlet. Artesian groundwater pressure (relative to the pre-facility topography) has been encountered in the bedrock (Unit 1) and silt/sand till (Unit 2) beneath the facility. The upper peat (Unit 6) and the immediately underlying shallow sand and gravel layer

(Unit 5) are typically saturated to the pre-facility topographic surface, and are considered hydraulically connected to each other. The lateral movement of groundwater into the area beneath the TSF is limited by perimeter slurry walls, see Drawing D-41002.

Surface water from the east is diverted around the tailings area to reduce water contact with tailings. Natural surface water features near the TSF include Cannery Creek to the north, Tributary Creek toward the south, and wetland areas northeast and southwest.

4. SITE GEOLOGY AND SEISMICITY

4.1 Bedrock and Surficial Geology

Bedrock at Greens Creek Mine area consists primarily of greywacke, argillite, phyllite, mafic tuffs, gneiss and schist. These rocks are typically folded along northwest-southeast striking axes, and are dissected by steep, strike-slip faults and less frequent low-angle thrust faults. The bedrock in the vicinity of the tailings facility consists of argillite and graphitic or sericite/chlorite phyllite.

Surficial stratigraphy beneath the TSF, from bottom to top, consists of the following units:

- Unit 1: bedrock at depths from surface to more than 140 ft;
- Unit 2: dense marine sandy-clay up to about 60 ft thick sometimes referred to as Till;
- Unit 3: firm to very-soft lacustrine and/or marine clay up to about 50 ft thick;
- Unit 4: dense fluvial or shallow marine sand up to about 24 ft thick;
- Unit 5: loose sand or sand and gravel immediately below the peat up to abut 14 ft thick; and
- Unit 6: amorphous to fibrous peat and organic matter to more than 20 ft thick.

Most soil layers lense in and out, vary erratically in thickness, and are not always present.

4.2 Seismicity

The major faults and geological structures that are potentially significant to seismicity at Greens Creek Mine include:

- Fairweather-Queen Charlotte Fault system (68.4 miles west);
- Chatham Strait Fault (6.2 miles west); and
- Coast Range Mega-lineament (18.6 miles east).

A seismic hazard assessment for Greens Creek Mine is presented in Klohn Crippen (1998). The report recommends that the TSF during operating life should be designed for the more severe of two identified Design Basis Earthquakes (DBE). Pile design for closure should be based on a Maximum Design Earthquake (MDE). The recommended peak ground acceleration and representative earthquake magnitude for each of the Design Basis Earthquakes (DBE1 and DBE2) and the Maximum Design Earthquake (MDE) are given in Table 4.1.

Table 4.1 Recommended Design Ground Motions at Greens Creek Mine

DESIGN CRITERION	PEAK FIRM GROUND ACCELERATION (PGA) (g)	REPRESENTATIVE EARTHQUAKE MAGNITUDE
Design Basis Earthquake 1	0.15	M6.5
Design Basis Earthquake 2	0.08	M8.0
Maximum Design Earthquake	0.30	M7.0

There are no published design criteria for return period earthquake selection for tailings deposits such as the TSF. However, for the Greens Creek TSF, KC reviewed criteria related to Dams, specifically ICOLD 1995 and the more recent Alaska Dam Safety Guidelines (Alaska, 2003). Based on the Alaska Dam Safety Guidelines the TSF would be categorized as a Class II structure with return periods of 70 to 200 years for a Design Basis Earthquake and 1,000 to 2,500 year for the Maximum Design Earthquake. The recommended design events from KC 1998, listed in Table 4.1 meet or exceed these guidelines as indicated below.

The DBE 1 parameters are based a probabilistic analyses at the 1/475 year probability level and the DBE 2 parameters are based on a deterministic analyses on the Fairweather Fault (1/130 year probability level). A check using Seed's simplified liquefaction analysis indicates that DBE1 gives a lower Factor of Safety (FOS) and hence, DBE2 is not considered further in this design report.

The MDE parameters are based on a PGA equal to 75% of the MCE (Maximum Credible Earthquake). Reference to KC 1998 indicates that 75% of the MCE would have a return period of about between 2000 years and 10,000 years and hence meets the requirements of a Class II structure. The MCE is based on the deterministic analysis of random floating crustal earthquakes in the vicinity of the site. The calculated source to site distance for the MCE is 9.3 miles. Recent large Alaskan earthquakes (listed below) have been considered relative to the design events recommended in KC 1998 and do not affect the results of the 1998 seismic hazard assessment:

- June 2004 - M6.8 on the Queen Charlotte Fault (208 miles from site);
- November 2002 - M7.9 on the Denali Fault (572 miles from site); and
- January 2000 - M6.2 (77 miles from site).

Generally the TSF is underlain at shallow depth by rock (Unit 1) or competent till (Unit 2), hence, amplification of ground motions through the natural ground is unlikely and the design ground surface PGA's of 0.15 g and 0.30 g are appropriate for the DBE and MDE, respectively.

Amplification of ground motions through the pile is considered in pseudostatic displacement analyses since it is implicit in the Hynes-Griffin methodology (Hynes-Griffin and Franklin, 1984).

This report considers the seismic liquefaction and softening potential in the tailings. A fundamental concept of seismic liquefaction is that the potential is generally only considered in saturated, or very near saturated, materials. KC's understanding (EDE, Appendix I) is that saturation in the tailings is confined to zones close to the base of the pile. In these areas, close to the foundation, amplification or damping of vibrations is not expected to vary significantly from assumptions inherent in the simplified liquefaction analyses (Youd, et al., 2001). This approach is considered sufficiently conservative for the design of the tailings pile and meets the requirements of the GPO.

5. SITE INVESTIGATIONS

Sub-surface site investigations have been carried out on and around the tailings facility since the early 1980's. Data obtained primarily between 1994 and 2005 have been used to support ongoing development and geotechnical/environmental assessment. The site investigations consist primarily of rotary or auger drill holes with Standard Penetration Tests (SPT). Piezometers and lysimeters were installed in selected drill holes. Table 5.1 provides a summary of the geotechnical drill holes that were used in this stability assessment.

Table 5.1 Summary of Geotechnical Drill Holes at Tailings Facility

BOREHOLE	LOCATION	DATE	DRILL TYPE	DRILL METHOD
DH-05-06	Pond 7	March 19, 2005	CME 850	Mud Rotary
DH-05-07	Pond 7	March 20, 2005	CME 850	Mud Rotary
DH-05-08	Old Tailings Pile	March 21–23, 2005	CME 850	Mud Rotary
DH-05-09	Old Tailings Pile	May 1–10, 2005	CME 850	Mud Rotary
DH-05-10	West Buttress	May 12–13, 2005	CME 850	Mud Rotary
DH-05-11	South Side	May 14–15, 2005	CME 850	Mud Rotary
DH-05-12	West Buttress	May 16, 2005	CME 850	Mud Rotary
DH-05-13	West Buttress	May 17, 2005	CME 850	Mud Rotary
DH-05-14	Pond 6	May 19, 2005	CME 850	Mud Rotary
DH-05-15	Southeast Corner	May 18, 2005	CME 850	Mud Rotary
DH-05-16	Southeast Corner	May 18, 2005	CME 850	Mud Rotary
DH-05-17	Southeast Corner	May 18, 2005	CME 850	Mud Rotary
DH-05-18	Southeast Corner	May 18, 2005	CME 850	Mud Rotary
DH-05-20	Southeast Corner	May 18, 2005	CME 850	Mud Rotary
DH-04-01	Northeast Expansion	Nov. 11 – Dec. 1, 2004	CME 850	Mud Rotary
DH-04-02	Northeast Expansion	Dec. 1, 2004	CME 850	Mud Rotary
DH-04-03	Northeast Expansion	Dec. 2, 2004	CME 850	Mud Rotary
DH-04-04	Northeast Expansion	Dec. 2–4, 2004	CME 850	Mud Rotary
DH-04-05	Northeast Expansion	Dec. 4–5, 2004	CME 850	Mud Rotary
DH-04-06	Pond 7	Dec. 5–7, 2004	CME 850	Mud Rotary
DH-04-07	Pond 7	Dec. 7–8, 2004	CME 850	Mud Rotary
DH-04-08	Pond 7	Dec. 8–9, 2004	CME 850	Mud Rotary
DH-04-09	Pond 7	Dec. 10, 2004	CME 850	Mud Rotary
DH-04-11	Pond 6	Dec. 12, 2004	CME 850	Mud Rotary
DH-02-04	Old Tailings Pile	Sept. 24, 2002	Longyear 38	Hollow Stem Auger
DH-02-05	East Side	Sept. 25, 2002	Longyear 38	Mud Rotary
DH-02-06	South Side	Sept. 26, 2002	Longyear 38	Mud Rotary
DH-02-07	Pond 6	Sept. 26-27, 2002	CME 75	Hollow Stem Auger / Mud Rotary / HQ Core

Table 5.1 Summary of Geotechnical Drill Holes at Tailings Facility (cont'd)

BOREHOLE	LOCATION	DATE	DRILL TYPE	DRILL METHOD
DH-02-08	Old Tailings Pile	Sept. 27-28, 2002	Longyear 38	Mud Rotary / HQ Core
DH-02-10	Old Tailings Pile	Sept. 30 – Oct. 2, 2002	Longyear 38	Mud Rotary
DH-01-01	West of Pond 7	Feb. 4–5, 2001	CME-75	Mud Rotary/ Hollow Stem Auger/ NQ Core
DH-01-02	West of Pond 7	Feb. 6, 2001	CME-75	Mud Rotary/ Hollow Stem Auger/ NQ Core
DH-01-03	West of West Buttress	Feb. 7–8, 2001	CME-75	Mud Rotary/ Hollow Stem Auger/ NQ Core
DH-01-04	West of West Buttress	Feb. 8, 2001	CME-75	Hollow Stem Auger/ HQ Core
DH-01-11	West of West Buttress	March 3-4, 2001	CME-45	Hollow Stem Auger/ NQ Core
DH-00-04	East Side	June 17, 2000	unknown	Hollow Stem Auger
DH-00-05	East Side	June 17, 2000	unknown	Mud Rotary
DH-00-06	South Side	June 18, 2000	unknown	Mud Rotary
DH-00-11	South Side	June 24, 2000	unknown	Mud Rotary
DH-00-12	West Buttress	June 25, 2000	unknown	Hollow Stem Auger
DH-00-13	West Buttress	June 25, 2000	unknown	Hollow Stem Auger
BH-97-01	West Buttress	October 20, 1997	CME-75	Hollow-Stem Auger
BH-97-02	South Side	October 23–24, 1997	CME-75	Mud Rotary
BH-97-03	Old Tailings Pile	October 24–25, 1997	CME-75	Mud Rotary
TA-1	Old Tailings Pile	August 9, 1994	unknown	Hollow-Stem Auger
TA-2	Old Tailings Pile	August 8–9, 1994	unknown	Hollow-Stem Auger
TA-3	Old Tailings Pile	August 13–14, 1994	unknown	Hollow-Stem Auger
TA-4	Old Tailings Pile	August 12–13, 1994	unknown	Hollow-Stem Auger
TA-5	Old Tailings Pile	August 5–6, 1994	unknown	Hollow-Stem Auger
TB-1	Old Tailings Pile	August 16–17, 1994	unknown	Hollow-Stem Auger
TB-2	Old Tailings Pile	August 14–15, 1994	unknown	Hollow-Stem Auger
TB-3	Old Tailings Pile	August 10–11, 1994	unknown	Hollow-Stem Auger
TB-4	Old Tailings Pile	August 17–18, 1994	unknown	Hollow-Stem Auger
TB-5	Old Tailings Pile	August 6–7, 1994	unknown	Hollow-Stem Auger

Cone penetration tests (CPT) were carried out on the tailings pile in 1997. An assessment of these data is provided in Appendix IV.

5.1 Standard Penetration Tests

The SPT is performed inside boreholes by advancing a spoon sampler beyond the bottom of the borehole by blows from a hammer with a standard weight of 140 lbs free falling from a height of 30 inches. The number of blows required to advance the sampler from 0" to 18" is recorded over 6" increments, and the blows summed over the final distance of 1 ft are correlated to soil density, strength, and structure.

The blow counts depend on the equipment and procedures used to perform the test (Chen, 1995). The SPT's in the 2001, 2002, 2004 and 2005 site investigations were advanced with automatic (safety) hammers. A safety hammer using a rope and cathead was used in the 1997 and 1994 site investigations. The hammer weight was measured at 140 lb in the 2002, 2004 and 2005 site investigations. The hammer drop height in the 2005 site investigations was measured at 29¾ inches.

SPT's were typically carried out at nominal 5 ft to 10 ft intervals, or more frequently when assessing soil layers of particular interest. The majority of the SPT's were conducted using a standard 18-inch long, 1½-inch inside diameter split spoon. The exception was the 1994 "TA" and "TB" series boreholes, in which a 2½-inch inside diameter split spoon was used. The SPT test assumes there is no soil disturbance 6 inches beyond the bottom of the borehole, which is not always the case if loose soils begin to flow into the borehole annulus.

5.2 SPT Energy Calibration

SPT energy transfer calibration measurements were carried out in 1997 and in 2005. The calibration data is used to correct energy input in the soil liquefaction analyses presented in this report. Normally, in the absence of measurements, industry practice (Seed, et al. 1985 and Drumright et al., 1996) is to assume hammer efficiency of about 60% to 70% for rope and cathead driven hammers and 80% to 90% for automatic hammers.

The average hammer efficiency measured in the 1997 SPT program was 35% (ConeTec, 1997). This low efficiency results in a very large correction which calls into question the 1997 SPT values. The results from these tests should be used with considerable caution.

In 2005, the measured average hammer transfer efficiency was 68% (Robert Miner Dynamic Testing, 2005). In addition, the hammer velocity was measured with a PDA radar unit as a check on the automatic hammer consistency (Klohn Crippen, 2005). The hammer velocity (at impact on the SPT anvil) ranged from 11.5 ft/s to 12.4 ft/s with an average velocity of 12.0 ft/s, that is, about 95% of the theoretic hammer velocity.

A hammer energy transfer efficiency of 60% was assumed for all other SPT tests. This assumption is considered to be conservative for assessing liquefaction since automatic hammers were used in 2001, 2002 and 2004.

5.3 Undrained Strength of Tailings

Field observations and instrument readings backed by analyses suggest that the tailings strength properties in the TSF are governed by drained behavior. That is, historic placement rates are sufficiently slow to dissipate most construction related pore pressure.

However, it is of interest to assess the undrained strength (S_u). Over the years KC has accumulated data on the undrained strength including:

- pocket penetrometer readings;
- CPT data; and
- laboratory triaxial test data.

The lower bound for old and new tailings S_u is 750 psf to 1000 psf based on the pocket penetrometer readings. The upper bound S_u is 2500 to 3000 psf based on CPT data on old tailings. An undrained triaxial test on new tailings, consolidated to a confining pressure equivalent to about 30 ft depth, gave an S_u value of 1500 psf. This value is just under the average range from the field data of 1625 psf to 2000 psf.

An average undrained strength of 1500 psf has been used for the new tailings in the analysis of temporary slopes.

6. LAB TESTING

6.1 Static Lab Tests

Tables 6.1 to 6.7 summarize material properties based on laboratory testing for new and old tailings, silty sand (till) (Unit 2), silt/clay (Unit 3), sand and gravel (Unit 4), sand (Unit 5), and peat (Unit 6). Plots of gradation, moisture content, and plasticity (Atterberg Limits) for each material are in Appendix VII.

Table 6.1 New Tailings (After 1996) Material Properties

MATERIAL PROPERTY	DATA SUMMARY	REFERENCES
Gradation	70% to 88% by weight passing No. 200 Sieve	Klohn Crippen 2003a, 2005b, 2005c
Moisture Content (in situ)	10% to 19%	Klohn Crippen 2003a, 2005b
Atterberg Limits Soil Classification	ML to CL-ML	Klohn Crippen 2003a, 2005b
Specific Gravity of Soil Solids (Gs)	Average Gs = 3.44 (Min. = 3.37, Max = 3.51)	Klohn Crippen 2003a, 2005c
Standard Proctor	Optimum Moisture Content = 11.5% to 12.5% Max. Dry Density = 130.2 pcf to 139.8 pcf	Klohn Crippen 2003a, 2005c
Direct Shear Test Results	Lower Bound Peak $\phi' = 38.7°$ Average Peak $\phi' = 42.0°$ Lower Bound Residual $\phi' = 31.6°$ Average Residual $\phi' = 40.9°$ (Normal Stress = 0.8 to 10.4 tsf)	Appendix VI, Klohn Crippen 2003a
Direct Shear Test Results (with smoothed failure plane)	$\phi' = 37.6°$ Normal Stress = 10.4 tsf	Appendix VI, Klohn Crippen 2005
One Dimensional Consolidation Test	Average $C_v = 3.3 \times 10^{-3}$ in/sec^2 (Normal Stress = 0 to 10.4 tsf)	Appendix VI
Undrained strength	1500 psf average	Section 5.3
Tempe Cell Test	Air Entry Value (AEV) = 0.422 (equilibrium volumetric water content in fraction of soil volume)	Klohn Crippen 2005c

Table 6.2 Old Tailings (Prior to 1996) Material Properties

MATERIAL PROPERTY	DATA SUMMARY	REFERENCES
Gradation	78% to 96% by weight passing No. 200 Sieve	SRK 1987c, Klohn Crippen 2003a, 2005b
Moisture Content (in situ)	14% to 26%	Klohn Crippen 2003a, 2005b
Atterberg Limits Soil Classification	CL-ML	Klohn Crippen 2003a, 2005b
Standard Proctor	Optimum Moisture Content = 13.5% Max. Dry Density = 129.8 pcf	SRK 1987c
Direct Shear Test Results	Average ϕ' = 36º (Min. = 35.6º, Max = 40.5º) (Stress Range = 0 to 1.6 tsf)	SRK 1987c
	Average Peak ϕ' = 33.6º Average Residual ϕ' = 31.8º (Normal Stress = 2.6 to 10.4 tsf)	Appendix VI

Table 6.3 Silty Sand (Till) (Unit 2) Material Properties

MATERIAL PROPERTY	DATA SUMMARY	REFERENCES
Gradation	60% to 100% by weight passing No. 4 Sieve	Geo-Recon 1981, SRK 1988, 1989b, 1992, 1993a, 1996
	25% to 50% by weight passing No. 200 Sieve	
Moisture Content (in situ)	6% to 15%	SRK 1993a, Klohn Crippen 2003a, 2005a, 2005b
Atterberg Limits Soil Classification	ML, CL-ML and CL	SRK 1988, 1993a, Klohn Crippen 2003a, 2005a, 2005b
Standard Proctor	Optimum Moisture Content = 18%	SRK 1982
	Max. Dry Density = 112 pcf	
Modified Proctor	Optimum Moisture Content = 11% to 13%	SRK 1982, 1988
	Max. Dry Density = 120 pcf to 130 pcf	
Consolidated-Undrained Triaxial Test	ϕ' = 32º	SRK 1989c

Table 6.4 Silt/Clay Material Properties (Unit 3)

MATERIAL PROPERTY	DATA SUMMARY	REFERENCES
Gradation	75% to 98% by weight passing No. 200 Sieve	Geo-Recon 1981, SRK 1981b, 1989a, 1992, Klohn Crippen 2005a
Moisture Content (in situ)	15% to 25%	SRK 1981b, 1992, Klohn Crippen 2005a, 2005b
Atterberg Limits Soil Classification	CL to CI	SRK 1981a, 1981b, Klohn Crippen 2005a, 2005b
Consolidated – Undrained Triaxial Tests	$\phi' = 30^\circ$ (Min. = 27.1°, Max = 33.7°)	Geotechnical Engineers 1983
Direct Shear Test	$\phi' = 31^\circ$	SRK 1982
Monotonic Loading After Consolidation	$\phi' = 32.1^\circ$	Geotechnical Engineers 1983
One Dimensional Consolidation Testing	Average Cv = 2.4×10^{-3} in/sec^2 (Stress range: 1 tsf to 11 tsf)	SRK 1992

Table 6.5 Sand and Gravel Material Properties (Unit 4)

MATERIAL PROPERTY	DATA SUMMARY	REFERENCES
Gradation	50% to 80% by weight passing No. 4 Sieve 0% to 20% by weight passing No. 200 Sieve	Geo-Recon 1981, SRK 1981b, 1987a, 1988, 1989c, 1992, 1993a, 1996, Klohn Crippen 2003a
Moisture Content (in situ)	8% to 19%	SRK 1987a, 1993a, Klohn Crippen 2003a
Modified Proctor	Optimum Moisture Content = 8.5% to 9.5% Max. Dry Density = 130 pcf to 136 pcf	SRK 1981b, 1987d

Table 6.6 Sand (SW) Material Properties (Unit 5)

MATERIAL PROPERTY	DATA SUMMARY	REFERENCES
Gradation	55% to 75% by weight passing No. 4 Sieve 5% to 15% by weight passing No. 200 Sieve (3 samples only)	SRK 1992, Klohn Crippen 2005a
Moisture Content (in situ)	10% to 17%	Klohn Crippen 2003a, 2005a, 2005b

Table 6.7 Peat Material Properties (Unit 6)

MATERIAL PROPERTY	DATA SUMMARY	REFERENCES
Gradation	60% to 80% by weight passing No. 4 Sieve 5% to 20% by weight passing No. 200 Sieve (2 samples only)	SRK 1988, 1989b
Moisture Content (in situ)	25% to 35% (2 samples only)	Klohn Crippen 2005b
Direct Shear Tests	Average $\phi' = 27^{\circ}$	SRK 1987b, 1993b
	(Min. = 26.1°, Max = 27.5°)	
Vane Shear Tests	Mean Peak Su = 207 psf	SRK 1987b
	Mean Residual Su = 32 psf	
One Dimensional Consolidation Tests	Testing completed on 2 Peat samples. Results highly variable.	SRK 1987b

6.2 Cyclical Shear Testing

Cyclical triaxial and cyclic simple shear tests were carried out on new tailings from the West Buttress area of the TSF to evaluate behavior under seismic loading (see Appendix II). The results are plotted in Figure 6.1.

Earthquake vibrations are often assumed to consist of vertically propagating shear waves causing horizontal strains. This is not the loading path in cyclic triaxial testing, and therefore, the cyclic triaxial test results require a significant correction (Appendix II) to adjust to field conditions. The magnitude of this correction is a subject of debate in the profession and hence use of cyclic triaxial testing has fallen out of favor, in KC's practice, in recent years. The laboratory data and field corrected results are plotted as liquefaction curves, see Appendix II, and Figure 6.1.

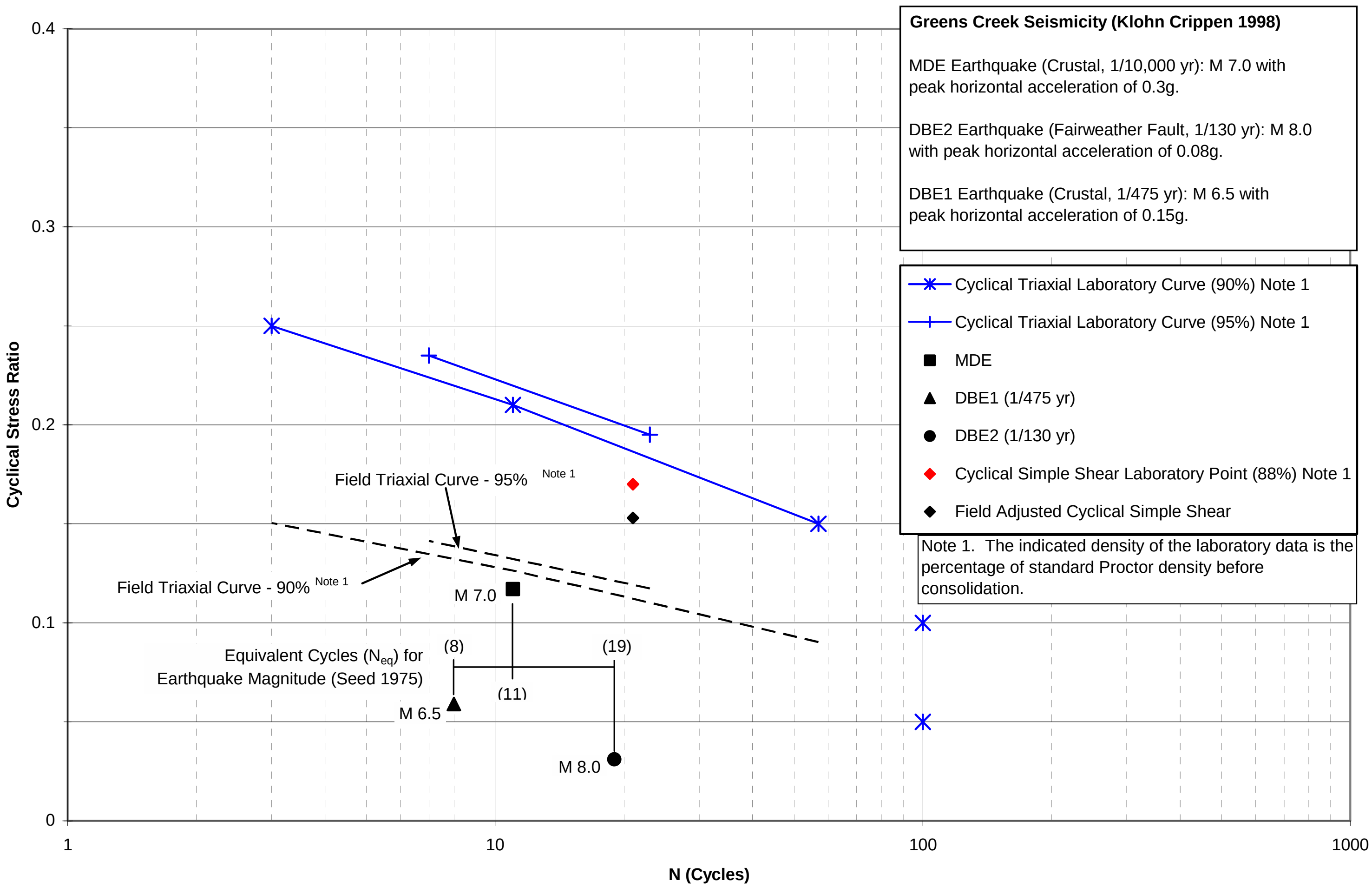

Figure 6.1 Cyclical Test Results - West Buttress Tailings

Cyclic simple shear tests model the earthquake shaking stress path more closely and hence require a much smaller correction (Appendix II). Generally, KC's practice favors cyclic simple shear testing due to the small correction needed to simulate field conditions. At the time of submission of this report a single cyclic simple shear test had been carried out as a check on the laboratory and adjusted cyclic triaxial curves and a second test was underway.

Both the cyclic shear and cyclic triaxial test field corrected results plot well above the DBE loading indicating a very low risk of liquefaction for this case. The safety factor against liquefaction for the DBE1 case is over 2 for the most severe case based on the laboratory testing (i.e., the field corrected triaxial curve for 90% Proctor).

The corrected triaxial cyclic test results also plot above the MDE loading and the cyclic shear tests plot significantly above the MDE loading. For the MDE, the safety factor against liquefaction varies from about 1.1 (for the triaxial data at 90% Proctor, corrected to field loading conditions) to about 1.5 (for the cyclic simple shear corrected to field loading conditions). The correction of laboratory test data to field loading conditions is discussed further in Appendix II.

Due to the presence of low in situ density measurements, see Section 2.3, it is intended to undertake additional cyclic simple shear box testing on loose samples. At the time of submission of this report a test was in progress on a moist tamped sample starting at a density equivalent to about 85% standard Proctor density and consolidated to 700 kPa vertical stress prior to application of cyclic loading.

7. WATER LEVELS

7.1 Current and Historical

Current and historical water levels in the tailings pile were provided by KGCMC in the form of piezometric data obtained from monitoring wells, standpipe piezometers, and vibrating wire piezometers. The water level was plotted versus time and the results are presented in Appendix I for each stratigraphic unit.

7.2 Closure Conditions

Closure water levels were provided by EDE based on maximum historical water levels in the tailings pile (Appendix I). The water levels were applied to the stability analyses assuming a hydrostatic condition. This is a conservative assumption for static analyses as there is a downward drainage gradient in the tailings pile, as indicated by readings at nested piezometers DH05-11A, DH05-11B, and DH05-11C (Figure I-1, Appendix I). The downward gradient is due to substantial underdrainage features (i.e., blanket drains, finger drains and liner drains).

Perched water tables could also exist from time to time in the pile, as wetting fronts migrate downward. Addressing the impact of perched water tables for closure conditions will be important. However, investigations into the presence of perched water is part of an on-going study not considered in this report.

Similarly, the distribution of saturation percentage above the water table may be important for closure design. This is the subject of on-going study not considered in this report. Both of these factors will be considered in the final closure evaluation.

7.3 Construction Pore Pressures

Observations during tailings placement suggest that, at least in some cases, relatively high pore pressures are induced during spreading and compaction, especially in wet weather. These observations include: a shiny surface, a soft surface inaccessible to heavy equipment, and a wavy surface under traffic loading. If the high pore pressures were to persist in the pile, there would be a reduction in the stability safety factor until pore pressure dissipation leads to the longer term water levels predicted by EDE. However, visual observations some time (weeks rather than months) after tailings placement indicate that the elevated pore pressures at the surface are not sustained; that is the surface no longer appears shiny, equipment can access dry areas of the pile, and the surface appears firm and hard.

One further indication of pore pressure response in the pile is available in the period 1995 to 1996. The mine was shut down from 1993 to 1996, and in August 1995, a geomembrane was placed over the TSF. The mine re-opened, the geomembrane was removed, and tailings placement resumed in July 1996. A general trend in water elevation reduction can be noted starting in August 1995 in piezometers P42, P43, P44, P46, P47, P50, and P51 (Appendix I, Figure I-1). The reduction in water elevation ceases around July 1996, corresponding to the time when the geomembrane cover was removed. Water level elevations have subsequently increased gradually over the years. While this may be a response to pile loading, it is considered more likely to be due to mounding of water in the TSF as the footprint expands (localized spikes in the gradual rise may be due to temporary construction activity or changes in atmospheric pressure). The overall trend in these piezometers suggests that: construction does induce some level of local pore pressure increase but this is generally confined to shallow depth and dissipates quite quickly; and dissipation of the pore pressures has occurred in the past when construction was stopped.

There is no obvious indication of sustained high construction pore pressures in the piezometers in the TSF. For example, piezometer readings (Appendix I, Figure I-1) for P46 and P47 show a spike and subsequent decrease in the measured water elevation between January and August 2005. This trend is likely due to active construction in the tailings pile near the piezometers, followed by a period of inactivity in the same area.

Both the 1995 and 1996 response to cover placement and the localized response to construction suggest pore pressure dissipation occurs over a time frame of months to one or two years. An analysis based on laboratory test data was undertaken to assess whether field observations were realistic.

A finite difference model (Gibson, 1958) was used to determine the time to dissipate construction-induced pore water pressures with a typical worst case location selected to be at the base of the new (post-1996) tailings. Coefficient of volume consolidation (c_v) values from consolidation tests on the tailings (Appendix VI) were used for corresponding normal stresses. The pile topography at various dates was studied and pore pressures resulting from the average historic overall placement rate of 0.6 ft/month and a historic local placement rate of 4.5 ft/month, sustained for 6 months were calculated.

The impact on the average placement or rise rate of 0.6 ft/month sustained over the approximate remaining mine life of 19.5 years, (i.e., about 140 ft increase in height) was analyzed for two conditions as follows:

- Assuming a thickness of 33 ft of tailings already exists above the base drain layer (represents the sides of the pile).
- Assuming a thickness of 66 ft of tailings already exists above the base drain layer (represents the middle of the pile).

The excess pore pressure calculated in the middle of the 33 ft and 66 ft layers are shown on Figure 7.1. The results show that for the 33 ft case very little excess pore pressure (i.e., 10 ft head vs. a pile rise of 140 ft) is expected while for the 66 ft case 40 ft excess pore pressure is expected. These pore pressure rises seem to be slightly higher than measured in the various piezometers but well below the pore pressure trigger levels discussed in Section 10.3.3. The analysis also shows a 50% drop in excess pore pressure within 1 to 2 years, which is consistent with piezometer instrument observations.

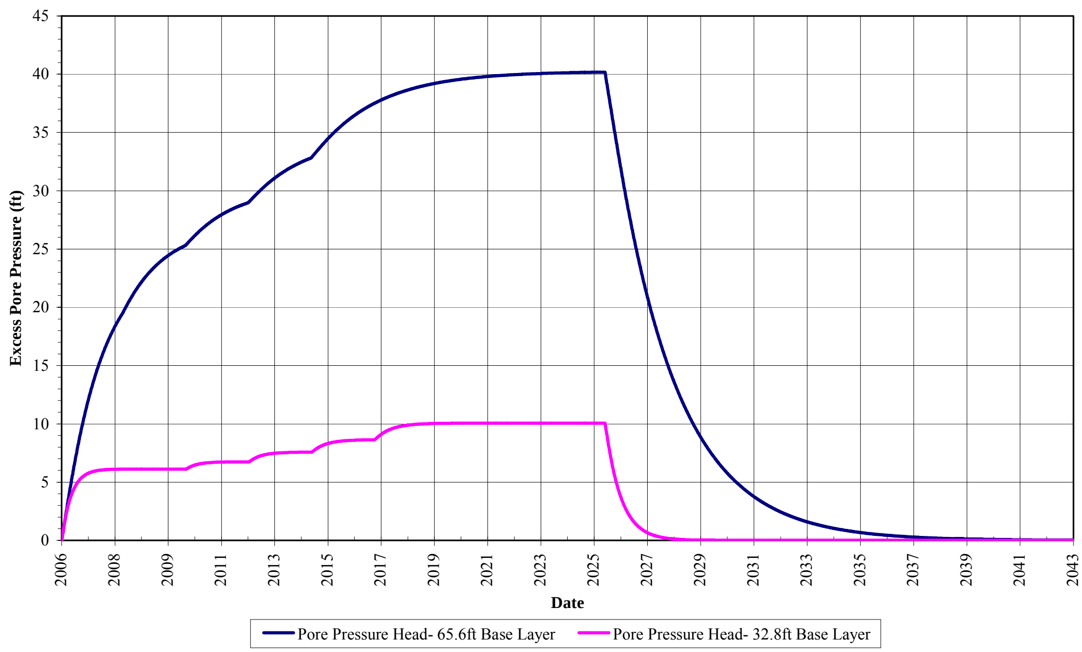

Figure 7.1 Pore Pressure Model (Double Drainage) Tailings Placement Rate 0.6 ft/month (19.5 yrs)

An additional pore pressure response assessment was made to model the conditions for the Southeast area. Two placement rates were analyzed as follows:

- 4.5 ft per month for 7 months, this is the fastest historic placement rate; and

- 9 ft per month for 7 months, this is a possible maximum localized placement rate in 2006.

Two locations were analyzed for an initial base layer thickness of 5 ft and 10 ft above the service layer. The excess pore pressure at the mid-point of the base layer is shown in Figure 7.2. The results show that for the 4.5 ft placement rate case, 18 ft of excess pore pressure is generated by the 30 ft (4.5 ft x 7 months) of fill placement at 10 ft above the base, while 40 ft of excess pore pressure is generated by the 63 ft of fill placed in the 9 ft/month placement rate case. Both of those generated excess pore pressure heads are within the safe limit for stability as discussed in Section 10.

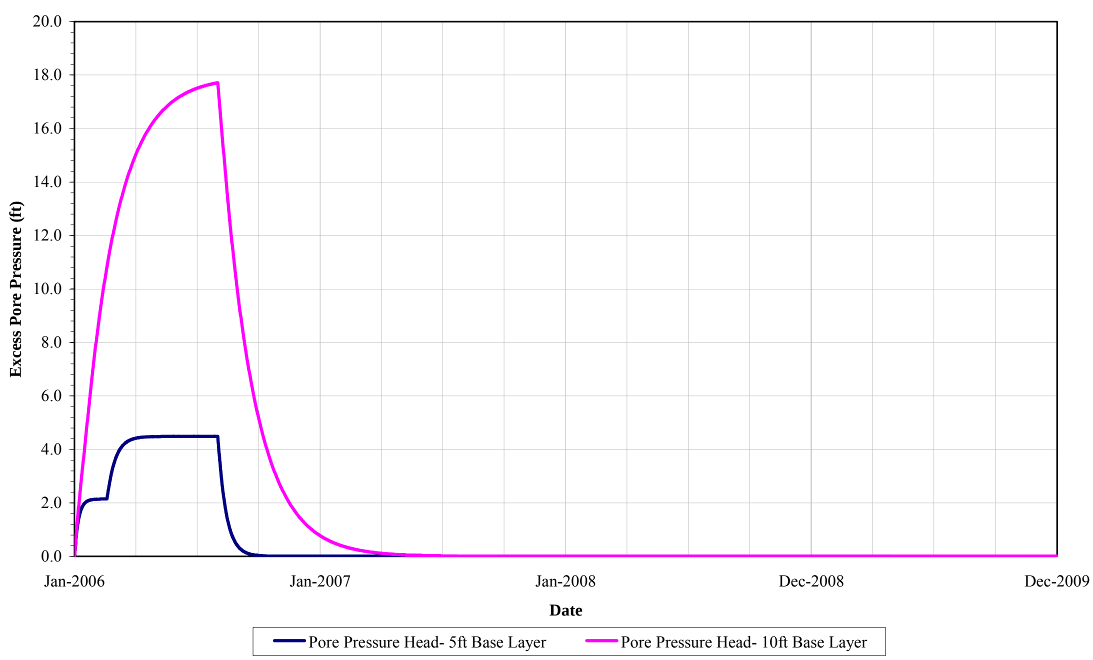

Figure 7.2 Pore Pressure Model (Double Drainage) Tailings Placement Rate 4.5 ft/month (7 months)

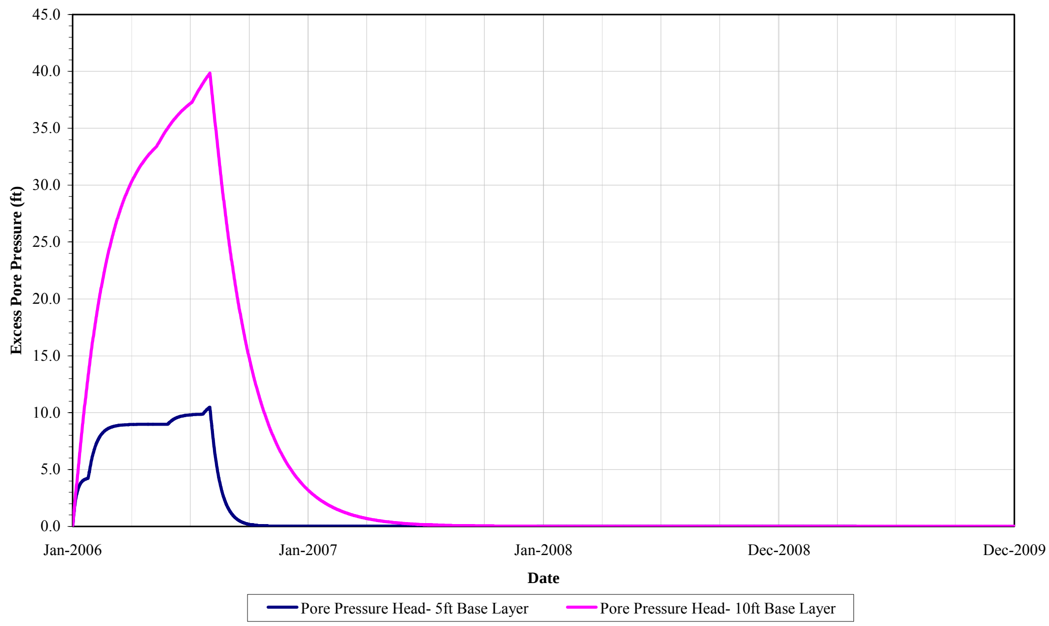

Figure 7.3 Pore Pressure Model (Double Drainage) Tailings Placement Rate 9 ft/month (7 months)

Pore pressure response is very difficult to accurately model and hence a plan to measure pore pressure by piezometers will be recommended for the Southeast expansion and for other areas whose placement rate exceeds 4.5 ft/month for 6 months or more in any 1 year.

Analyses and observations agree that construction pore pressures could exist for a few months to a few years. But due to dissipation and the sequential fill placement across the pile, construction pore pressures should be low or very localized. Careful monitoring of water levels should be continued. If pore pressures are measured that are significantly above the EDE (2005) estimated water table, the pile stability should be re-assessed. Mitigation could include adjustment of placement rate and location to control build-up of construction pore pressures.

Two of the foundation soil units, the marine clay/till (Unit 2) and the softer marine clay (Unit 3) may have the potential to generate pore pressure under the TSF loading. No laboratory consolidation test data are available to assess this case, however there are a number of piezometers in these units. Similar to the tailings, there has been no indication of a sustained construction-induced pore pressure increase in the foundation clay (Unit 3) or till (Unit 2) from the piezometers in these units. Piezometer readings in these units are shown on Figure I-2 (Appendix I). Laboratory data on similar clay found at Site E suggest that the coefficient of volume consolidation (c_v) which controls the rate of pore pressure dissipation, is very similar to the tailings c_v. Hence, the observed low pore pressure response to construction is consistent with the results of the tailings analysis.

Continued monitoring of piezometers will be required to check that current assumptions of low construction pore pressure response continue into the future.

8. DESIGN BASIS

The design criteria for the project were most recently summarized in a Klohn Crippen letter dated December 10, 2003. The following criteria are extracted from that letter. Changes to the 2003 design basis are noted in Section 8.7.

8.1 Maximum Elevation and Slope Geometry for Tailings Facility

The maximum elevation of the tailings pile (without closure cover) is El. 330 ft. Any closure cover constructed on the pile would raise the ultimate elevation of the pile by several feet. The design of a closure cover is still being developed (by others) and is not part of this report.

The tailings pile will be constructed with overall external slopes not steeper than 3H:1V.

8.2 Design Factors of Safety for Liquefaction Assessment

For design purposes, if the liquefaction analysis shows a FOS less than 1.1 against liquefaction, then full liquefaction is assumed, and post-liquefaction strength is used for stability analyses. We assume there is no pore pressure rise in material that has a FOS against liquefaction greater than 1.4, and use the static strength for stability analysis in this case. However, if the liquefaction FOS is between 1.1 and 1.4, the material strength for use in post-liquefaction stability analyses is assumed to vary between post-liquefied strength and static strength.

8.3 Design Factors of Safety for Geotechnical Stability

The minimum Factors of Safety for stability analyses will be as follows:

- 1.5 for long-term static conditions using peak strength values;
- 1.3 for long-term static conditions using residual strength values;

- 1.3 for temporary (construction) conditions;

- 1.1 for pseudo-static conditions; if the pseudo-static safety factor is less than 1.1 a deformation analysis using the Hynes-Griffin and Franklin (1984) inertial model will be completed; and

- Greater than 1.0 for post-liquefaction. This post liquefaction target safety factor is common practice. Provided the factor of safety is greater than 1.0, flow failure should not occur and the slope will only suffer relatively minor deformation (see Section 11.3). Over a short period of time, seismically induced pore pressure will start to dissipate and the slope safety factors should return to pre-earthquake long term static values.

8.4 Seismicity

See Section 4.2 and Table 4.1.

8.5 Unit Weight and Strength Parameters

Based on the laboratory testing summarized in Section 6, the unit weights along with static and post-liquefaction strength parameters of the tailings and foundation soil are summarized in Table 8.1.

Table 8.1 Material Properties used in Stability Analyses

SOIL TYPE	TOTAL UNIT WEIGHT (pcf)	STATIC (EFFECTIVE STRENGTHS)			POST-EARTHQUAKE (Appendix VIII)		APPLICABLE STABILITY SECTIONS
		Peak Friction Angle (degrees)	Residual Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	
New Tailings	128	39	32	0	Function [1]		All
Old Tailings	120	33	32	0	Function [2]		1,2,3,4a,4b
Geosynthetic Liner System	125	24.2	12.5	0	N/A[3]	N/A	5a,5b,6
Sand Drainage Blanket	120	40	N/A[4]	0	N/A	N/A	2
Peat (Unit 6)	67	27	N/A	0	N/A	N/A	1,2,3,4a,4b,6
Sand and Gravel (Unit 5)	120	33	N/A	0	Function [3]		1,2,3
Sand (Unit 4)	120	33	N/A	0	N/A	N/A	1,4a,4b,6
Silty Clay (Unit 3)	120	30	N/A	0	N/A	N/A	1,2,3,4a,4b,6
Silty Sandy Till (Unit 2)	120	33	N/A	0	N/A	N/A	All
Main Embankment Till	120	33	N/A	0	N/A	N/A	3
Compacted Rockfill/Road Fill	120	40	N/A	0	N/A	N/A	1,2,5a,5b,6
Roadfill/Native	130	36	N/A	0	N/A	N/A	6

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. For sensitivity analysis, as presented in Appendix VIII, if full liquefaction were to occur, undrained strength (S_u) 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) see Section 8.7.
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) see Section 8.7.
3. The maximum shear strength is 1640 psf. See Section 8.7.
4. N/A indicates that the soil does not liquefy during the MDE, therefore static properties were maintained in the post-liquefaction analysis.

Based on the results of the liquefaction assessment in Section 9, post-liquefaction strengths are required for certain portions of the sand and gravel (Unit 5) and tailings units lying below the water table (on applicable sections as noted in Table 8.1). Post-liquefaction undrained strength of the potentially liquefiable tailings is based on cyclical lab testing data for material at depth (>100 ft) and is reduced to values proposed by Seed and Harder (1990) at the surface. The post-liquefaction strength of the sand and gravel layer (Unit 5) is based on SPT data (Seed and Harder, 1990).

8.6 Piezometric Surfaces

EDE (2005) predicted long term water levels in the tailings pile based on historical piezometric levels. These water pressures were applied to the current stability analyses by setting the water levels within the pile as recommended by EDE, and then linearly reducing the water levels towards the toe of the pile. This assumes that drainage control features (i.e., drainage blankets and foundation drains) are installed and operate as designed. To date, the measured water levels have been consistent with or below levels as described above.

The water levels were applied to the stability analyses assuming a hydrostatic condition. This may be a conservative assumption for static analyses as there is likely a downward drainage gradient present in the tailings pile.

8.7 Changes to Design Criteria

The following modifications were made to the Design Criteria since 2003:

- The critical interface friction angle in the liner system was reduced from 26º to a peak of 24.2° and a residual of 12.5° based on results of testing done in 2005 (Appendix X). This change decreased the factor of safety (FOS) against sliding for the Stage 2 Southeast expansion. To compensate for this reduction in liner strength a rockfill berm was constructed along the toe of the Stage 2 Southeast expansion to raise the safety factor against sliding back to design requirements.

- The effective static peak friction angle for the new tailings was increased from 36º to 39º based on direct shear testing completed in 2005 (Appendix VI). This change increased the FOS of the stability sections around the tailings pile between 2% and 5% based on the results of the sensitivity analysis discussed in Appendix VIII.

- The post-liquefaction strength for the potentially liquefiable new and old tailings were updated. Previously the post-liquefaction undrained shear strength was based only on empirical relationships developed from SPT data (S_u = 2800 psf on new tailings and S_u = 400 psf on old tailings). Now, the post-liquefaction undrained strength of the new and old tailings is based on cyclical lab testing data, an empirical SPT method suggested by Idriss, 2004 that accounts for overburden pressure and a lower bound strength based on SPT data as per Seed and Harder (1990). The strength is varied from 2297 psf at depths of 140 ft or greater to 324 psf at the surface. The liquefied strengths are now based on multiple methods using lab and SPT data rather than the single empirical SPT method used previously.

- The effective static peak friction angle for the shallow sand and gravel layer (Unit 5) was increased from 27º to 33º based on SPT data from recent investigations. Where required, post-liquefaction strength of the gravelly sand layer, based on recent SPT data and using Seed and Harder, 1990, gives an undrained residual strength of about S_u = 1640 psf. In most areas Unit 5 is shown to be non liquefiable or will be removed.

- Piezometric surfaces are based on EDE's 2005 recommendations, updated from EDE 2002. The EDE 2005 piezometric levels are generally higher than those used in previous analyses, which were based on EDE 2002. EDE's 2005 recommendations are based on historical piezometric data, as opposed to the 2002 levels which were determined by modeling the piezometric surface considering long-term conditions. Since EDE's 2005 recommendations are based on actual field data rather than theoretical models they are considered more reliable. The new recommendations are also more conservative than the previous since they project a higher water level which decreases the stability of the TSF as evident from the results of the sensitivity analysis in Appendix VIII.

9. LIQUEFACTION ASSESSMENT

9.1 General

Liquefaction potential was assessed for the foundation soils and for the tailings. Generally, the till/marine sequence of silt, clay and granular soil, Units 2 to 4 inclusive, are either too dense or too plastic to be considered liquefiable. A detailed assessment was made of the shallow sand and gravel layer (Unit 5), which is sporadically present immediately below the original ground peat layer (Unit 6), and a detailed assessment was also made of the tailings.

All materials were found to be safe against liquefaction under the DBE (see Appendix III) and hence only the MDE loading case is considered in the following assessment. In general, the safety factor against liquefaction under the DBE is high enough that no significant strain softening or pore pressure rise is expected. This is a key finding in this report.

9.2 Liquefaction of Sand and Gravel (Unit 5)

A shallow sand and gravel layer (Unit 5) is present under about half of the TSF. The layer is present principally in the eastern half but also originally extended under part of the West Buttress area, as shown on Drawing D-41006. The natural shallow sand and gravel layer (Unit 5) was removed prior to construction of the West Buttress, however granular fill for drainage and trafficability was placed during construction of the West Buttress and is present in the 2005 drill hole logs. The Unit 5 layer, where present, is typically only a few feet thick but can be up to 24 ft thick.

The liquefaction potential of the shallow sand and gravel (Unit 5) was evaluated using the methods recommended by Youd, et al. (2001) and Boulanger and Idriss (2004) using the Standard Penetration Test (SPT) data collected during site investigations from 1997 through 2005. Details of the liquefaction assessment and results are in Appendix III. Table 9.1 presents a summary of the liquefaction assessment on the sand and gravel (Unit 5).

Previous liquefaction assessment (by Klohn Crippen) of the sand and gravel (Unit 5), based on SPT data, indicated that significant portions of the layer would be liquefiable under the MDE. However, we suspected that many of the low blow counts in the layer were either affected by drill technique (such as hollow stem auger drilling) or due to the sand and gravel layer being thin. Often the first blow in the layer was partly in peat and by the time the second SPT was attempted drilling was through the Unit 5 layer. Consequently, a very careful SPT program was conducted in 2004/2005 that targeted the sand and gravel layer. Continuous SPT's were started as soon as the overlying peat was encountered. This gave as many SPT values as possible in the sand and gravel layer. Further, great care was taken to keep the holes as full of mud as possible to reduce heave and loosening. Lastly, hammer velocity and energy measurements were taken. These careful tests and a critical re-assessment of previous tests resulted in an increase in the average layer blow count, as summarized in Table 9.1.

Table 9.1 Liquefaction Assessment Under MDE of the Sand and Gravel (Unit 5) Based on SPT Testing

LOCATION	AVERAGE $(N_1)_{60\text{-cs}}$	AVERAGE FOS AGAINST LIQUEFACTION (MDE)	NO. OF SPT's CONDUCTED
East Side	34.9	1.7	12
Northeast Expansion	29.5	0.9	12
Old Tailings Pile	30.0	1.9	8
South Side	30.1	2.1	2
West Buttress	22.2	1.9	5
Pond 7	29.1	1.3	5
	Average = 30.7	Average = 1.5	Total = 44

Notes: 1. $(N_1)_{60\text{-cs}}$ = Field SPT N corrected to an overburden stress of 1 tsf; a hammer energy ratio of 60%; and to an equivalent clean sand value.
2. Blow counts in excess of 50 blow/ft were assumed equal to 50 blows/ft for calculation of averages.

The simplified liquefaction assessment procedure (Youd et al., 2001) specifies the use of average SPT for a particular geologic layer. Based on average values, the analysis shows that the sand and gravel deposit will not liquefy under the DBE or under the MDE, except in the Northeast expansion area, as shown in Table 9.1. Figure 9.1 shows the scatter of SPT values. Removal of the sand and gravel layer from the Northeast expansion area is planned (KC 2004). The impact of the resulting excavation on the groundwater system in the Northeast area, the temporary stability of the excavation and the impact of the excavation on existing slurry walls and infra-structure will be addressed during final design. Design features will be incorporated as needed to maintain the closure drainage arrangement.

The average factor of safety of 1.5 against liquefaction (Table 9.1) indicates that no significant strain softening or pore pressure response is expected under DBE or MDE. This is a key finding of this report.

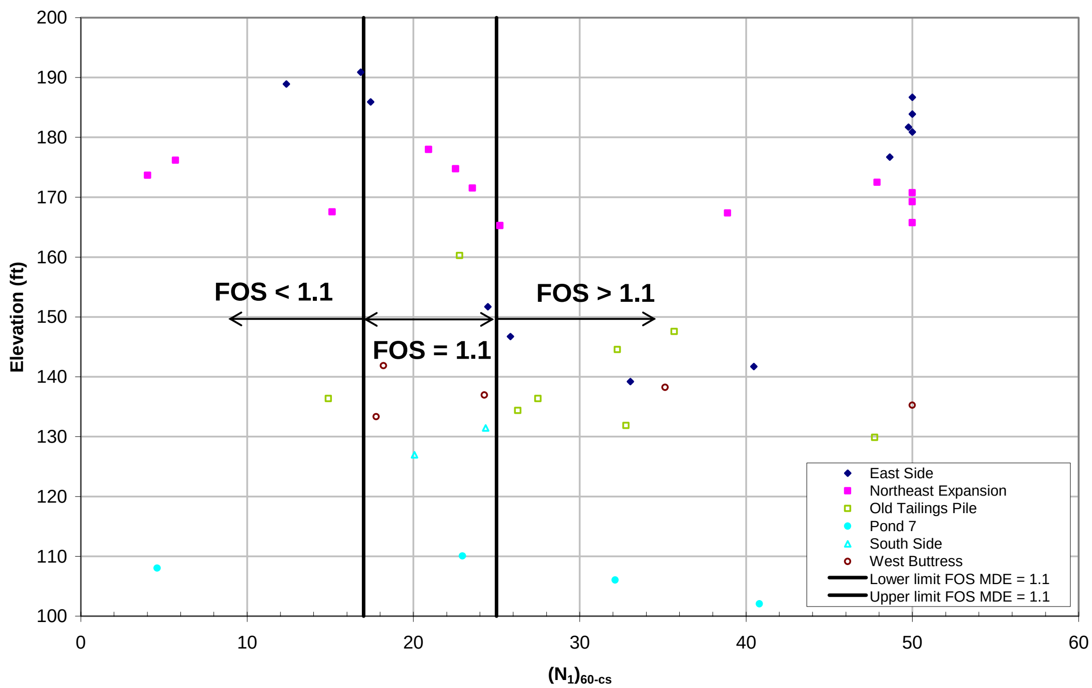

Figure 9.1 $(N_1)_{60\text{-}cs}$ vs. Elevation for Sand and Gravel (Unit 5) under TSF for the MDE

9.3 Liquefaction of Tailings

The tailings are a man made material which behave like a low to non-plastic silt. This type of material presents a most difficult challenge in assessing earthquake related performance. A number of methods were used for evaluating the liquefaction potential of the tailings. Commonly used evaluation methods are dominated by empirical relationships based on natural sands and silty sands, and extension of these methods to evaluation of tailings materials requires caution. The methods used in this study are listed below, and the results of each were considered, to obtain an overall assessment of the liquefaction potential of the tailings. For the assessment, the results were plotted and reviewed to see if there were patterns relating liquefaction potential to elevation, depth, date of placement, geographic regions and the like. The only apparent pattern was a slight difference in average SPT values between "old" and "new" tailings. The old tailings are those materials within the "old" pile footprint, placed prior to cessation of mining in 1993. The new tails were placed post mining restart in 1996.

Methods used in this liquefaction assessment include:

- Standard Penetration Test (SPT) data;
- Cone Penetration Test (CPT) data;
- Three common methods for evaluating the liquefaction susceptibility of fine-grained soils based on laboratory index property tests;
- Laboratory testing – cyclical triaxial and cyclical simple shear on new tailings; and
- Shear wave velocity data.

Each of the above methods is discussed below.

9.3.1 Liquefaction Assessment using SPT Data

The SPT test is an empirical test developed for granular materials and extended to silty soils. In highly silty and clayey soil SPT data are not reliable for a number of reasons including generation of pore pressure during the test which tends to give lower blow counts than in sandy soil. The evaluation methods described in Section 9.2 for the sand and gravel (Unit 5) were also used for liquefaction assessment of the tailings. The liquefaction assessment method includes a correction for silt content up to 35%. This silt content is well below the average 80% silt content of the tailings, however, SPT testing of the tailings was carried out for Greens Creek because it is one of the most widely used methods for liquefaction assessment. The details of the liquefaction assessment and results are in Appendix III. Table 9.2 presents a summary of the liquefaction assessment of the tailings based on SPT testing.

The SPT data were split into old and new tailings and into pile regions to check if there was spatial or time variability. There is some indication that the old tailings may be less dense (lower SPT) than the new tailings.

Table 9.2 MDE Liquefaction Assessment of Tailings based on SPT Testing

		EXCLUDING $(N_1)_{60cs}$ > 50		
LOCATION	**NEW OR OLD TAILINGS**	**Average $(N_1)_{60cs}$**	**Average FOS (MDE)**	**No. of SPT's**
East Side	New	23.1	1.8	6
Old Tailings Pile	New	23.5	1.9	11
	Old	16.2	1.2	198
South Side	New	18.9	1.7	34
	Old	28.4	2.4	13
West Side	New	22.8	1.7	26
	Old	18.7	1.2	4
Southeast Corner	New	25.0	2.1	13
New and Old Tailings		Average = 18.4	Average = 1.5	Total = 305
New Tailings		Average = 21.7	Average = 1.6	Total = 93
Old Tailings		Average = 17.0	Average = 1.2	Total = 215

Notes:
1. $(N_1)_{60cs}$ = Field SPT N corrected to an overburden stress of 1 tsf; a hammer energy ratio of 60%; and to an equivalent clean sand value.
2. $(N_1)_{60cs}$ > 50 were excluded because these points likely indicate penetration through materials other than tailings (e.g. roadfill).

While average values tend to suggest that the tailings would not liquefy under the MDE, there is quite a scatter of results with many SPT values falling into the liquefiable category (see Figure 9.2 for old tailings and Figure 9.3 for new tailings). As indicated previously, because of the cell construction methodology, continuous weak layers are unlikely within the pile. The distribution of $(N_1)_{60cs}$ throughout the TSF is irregular and local variation within holes are equal or larger than regional variations.

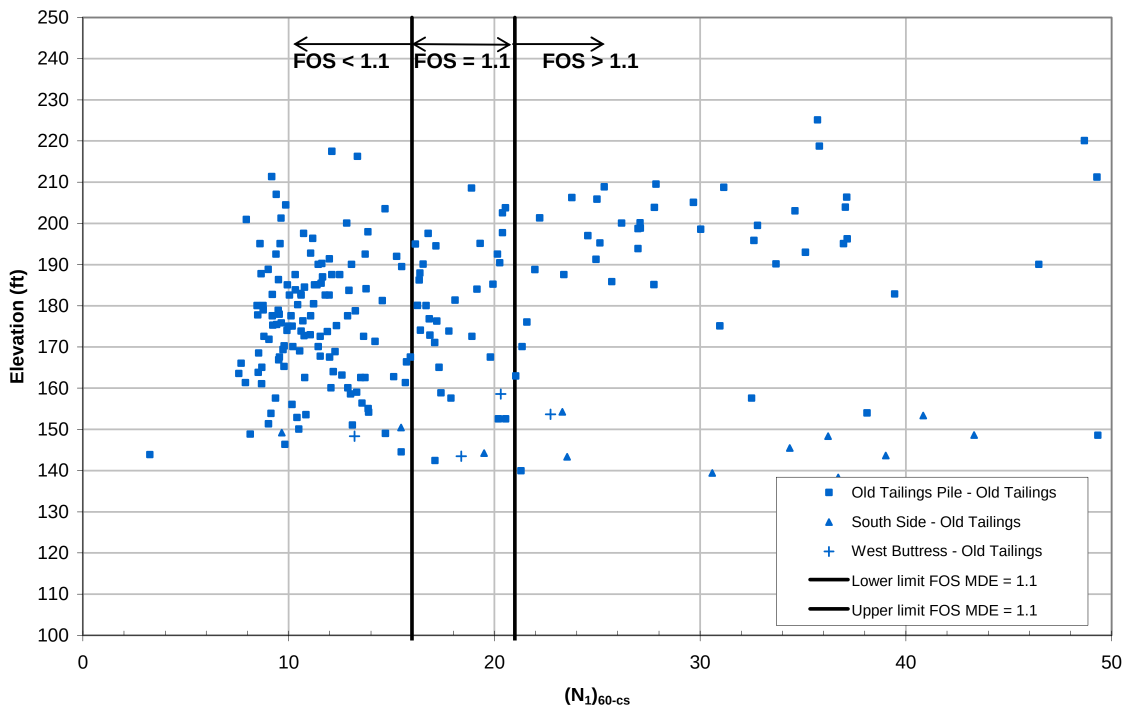

Figure 9.2 $(N_1)_{60cs}$ vs. Elevation for Old Tailings for the MDE

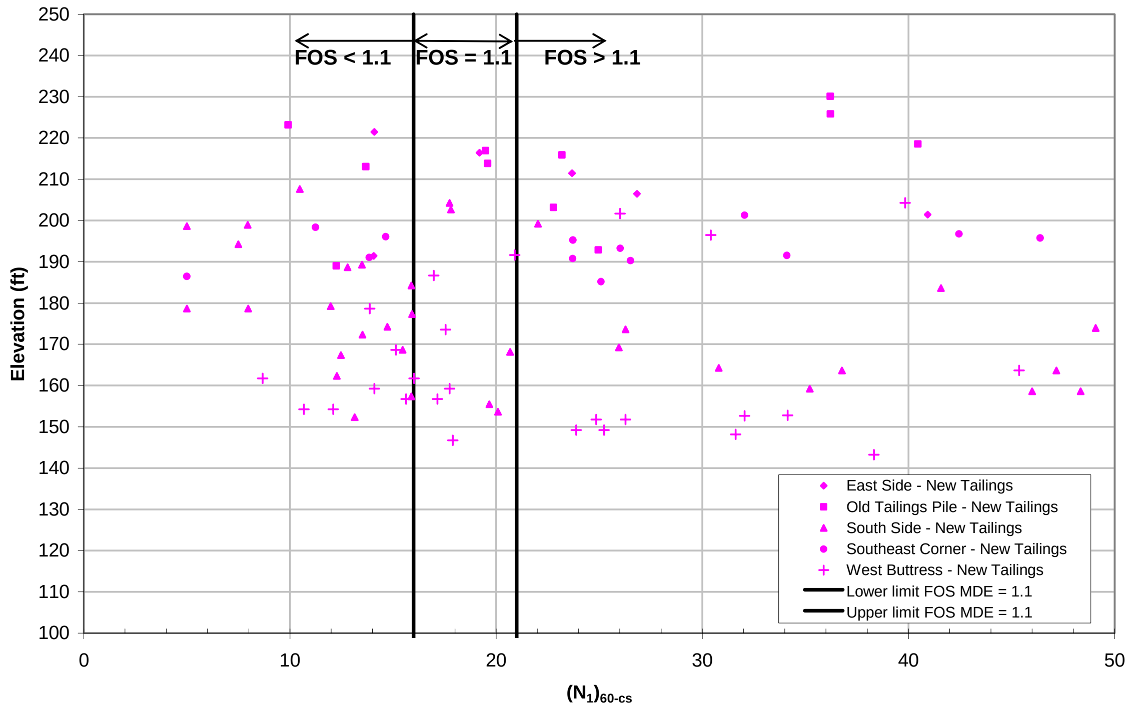

Figure 9.3 $(N_1)_{60cs}$ vs. Elevation for New Tailings for the MDE

The drill hole logs used for the SPT analysis are in Appendix V.

9.3.2 Liquefaction Assessment using CPT Data

The liquefaction resistance of the tailings was determined using the methods which convert Cone Penetration Test (CPT) to equivalent SPT data and by directly using the CPT data. The methods recommended in Youd, et al. (2001) were mainly followed. To complete this assessment two close-by pairs of CPT and borehole SPT locations were selected.

In this assessment, the test hole data on the old tailings from CPT97-16 and BH97-3 were mainly used, with a second check being made on hole pair CPT97-14 and DH02-08. The intent of this work was to assess whether CPT and SPT-based analyses gave similar estimates of liquefaction susceptibility. Details of this assessment are in Appendix IV. Careful evaluation of the SPT data were made to account for the low hammer efficiency recorded during the 1997 field work, see Appendix III.

CPT data converted to equivalent SPT agree well with SPT data from boreholes BH97-3 and DH02-08. Both the SPT derived from CPT and the direct SPT data indicate that most of the old tailings below the water table in those locations would liquefy under MDE. However, the direct CPT analyses (i.e., if not converted to SPT) indicated that the tailings were not susceptible to liquefaction due to high values of soil behavior type index, I_C. I_C was greater than 2.6 which suggested that the tailings may be too clay-rich to liquefy. Note that Youd, et al. (2001) suggested that soils classified as "clayey" according to the CPT based method should be tested in the lab to confirm the soil type and liquefaction resistance. Laboratory testing was undertaken and did not agree with the soil type classification of "clayey" indicated by I_c>2.6.

9.3.3 Index Test Criteria for Liquefaction Potential of Fine Grained Soils

The tailings generally contain more than 80% fines (see Section 6.1). Liquefaction susceptibility of fine-grained soils (silts and silty clays) has been assessed using the Modified Chinese liquefaction criteria (Finn, et al., 1994), Andrews and Martin criteria (2000), and Bray, et al. (2004) criteria. These assessments, described in Appendix XI, are used as screening tools (i.e., if materials are shown to be not susceptible to liquefaction then no further analysis is required). Conversely, where materials are shown to be susceptible to liquefaction, further analyses using SPT, CPT, laboratory testing, or other methods are suggested.

The Chinese Criteria show that the majority of the new tailings are classified as non-liquefiable, and additional laboratory testing is recommended for the old tailings to confirm the liquefaction potential. The Andrews and Martin criteria indicates that both old and new tailings are classified as potentially susceptible to liquefaction. According to the criteria by Bray, et al., about half of the new tailings samples are classified as potentially susceptible to liquefaction, and all but two of the old tailings samples are susceptible.

Note, these methods make no reference to the induced cyclic stress, density or saturation levels but simply provide a screening tool to help with a decision on whether to proceed with more analyses.

9.3.4 Liquefaction Assessment using Cyclical Lab Test Data

A field-equivalent cyclical stress ratio (CSR) was calculated for each of the design earthquakes (Table 4.1) and for a stress level equivalent to the laboratory test level. In the literature, each earthquake magnitude is assigned a number of significant cycles of loading. Hence, laboratory data can be compared to the predicted field loading. The predicted cyclic stress ratios and number of cycles for MDE and DBE were compared to the field-adjusted laboratory liquefaction curves to assess the potential for liquefaction in new tailings as shown on Figure 6.1. The results are as follows (details in Appendix II):

- The predicted CSR's for the DBE are less than one-half the calculated CSR required to initiate liquefaction (as defined by the field-adjusted laboratory liquefaction curves). With the tailings compacted to 90% of standard Proctor maximum dry density, the tests indicate that liquefaction will not be initiated by a DBE event.

- The CSR required to induce liquefaction based on the field adjusted cyclic triaxial test exceed the predicted MDE CSR by a safety factor of 1.1 to 1.2 for the 90% Proctor and 95% Proctor samples respectively. The cyclic shear field adjusted CSR, completed on a sample with a starting density of 88% Proctor, exceeds the predicted MDE CSR by 1.5. Therefore liquefaction of the new tailings will not be initiated by an MDE, but some cyclic softening of the tailings is expected.

The tests were conducted at stress levels predicted at the base of the pile since, theoretically, soil of a given density becomes more susceptible to liquefaction at higher stress. Further, most test samples were prepared at about 90% optimum standard Proctor density. The sample tested at 95% Proctor initial density showed slightly higher resistance to liquefaction.

There is considerable uncertainty in achieved density, as previously discussed in Section 2.3. In situ density nuclear gauge testing by KGCMC averages almost 100% standard Proctor density. However, calibration of the nuclear gauge and comparison with balloon density testing showed that the nuclear method may overestimate the density of the tailings by 6% to 12% (Klohn Crippen, 2003b). In addition, KGCMC consistently report that in poor weather there are zones within the tailings pile that fail to meet the 90% standard Proctor density criteria. These zones are identified as far as possible, removed, and recompacted. Nevertheless, there is some risk that soft material could be left in the TSF. Thus, it is considered important to check what the impact of loose material would be on liquefaction potential. Consequently, a cyclic simple shear test is underway on a loose tamped sample with an initial placed density of approximately 85% standard Proctor density.

Based on the laboratory testing it appears that the tailings are not liquefiable under the MDE, although on average the factor of safety against liquefaction could be about 1.3 (midway between cyclic triaxial and cyclic shear box predictions). Based on that approximation, an MDE-induced design pore pressure rise (Δu) of about 33% of the initial effective stress is expected for saturated new tailings below the water table.

The preceding conclusions are based on five cyclical triaxial tests and one cyclical simple shear test carried out on new tailings from one bulk sample, and it is proposed that additional data are collected during the operating life of the mine to confirm the assessment.

As part of the laboratory test program, samples strained to liquefaction were maintained in an undrained state then sheared monotonically in extension. The average post liquefaction undrained strength of the tailings as measured in extension was 2297 psf (110 kPa).

9.3.5 Liquefaction Assessment using Shear Wave Velocity

The empirical method for evaluating liquefaction resistance using shear wave velocity was applied to the tailings, refer to Figure 9.4 (Youd et al., 2002). Average shear wave velocities for the tailings were collected during the geophysics site investigation program completed by KC in July, 2005. The shear wave velocities were corrected for overburden stress and used to calculate the cyclic resistance ratio for the tailings. Based on Youd et al, 2002, liquefaction is not expected if the corrected shear wave velocity is greater than 656 ft/s (200 m/s) or if the cyclic resistance ratio is greater than the cyclic stress ratio. Table 9.3 summarizes the field and corrected shear wave velocities, cyclic resistance ratio and the cyclic stress ratio for the DBE and MDE.

The shear wave velocity analysis indicates that the tailings are not liquefiable for the DBE or MDE seismic event.

Table 9.3 Summary of Liquefaction Resistance Using Empirical Shear Wave Velocity Method

LOCATION	MEASURED SHEAR WAVE VELOCITY	CORRECTED SHEAR WAVE VELOCITY[1] (V_s1)	CYCLIC RESISTANCE RATIO	CYCLIC STRESS RATIO	
				DBE	MDE
Centre of Pile	951.4 ft/s	821.2 ft/s	n/a (V_s1 > 656 ft/s)	0.06	0.15
West Buttress	876.0 ft/s	639.7 ft/s	0.63	0.05	0.07
South Side	767.7 ft/s	638.5 ft/s	0.59	0.06	0.06

Notes:
1) Corrected values were based on ReMi (Refraction Microtremor Shear Wave Soundings) shear wave velocities collected during field tests completed by KC in July, 2005 (Klohn Crippen, 2005e). ReMi velocities are diagnostic of the average shear wave velocity with depth through the tailings.

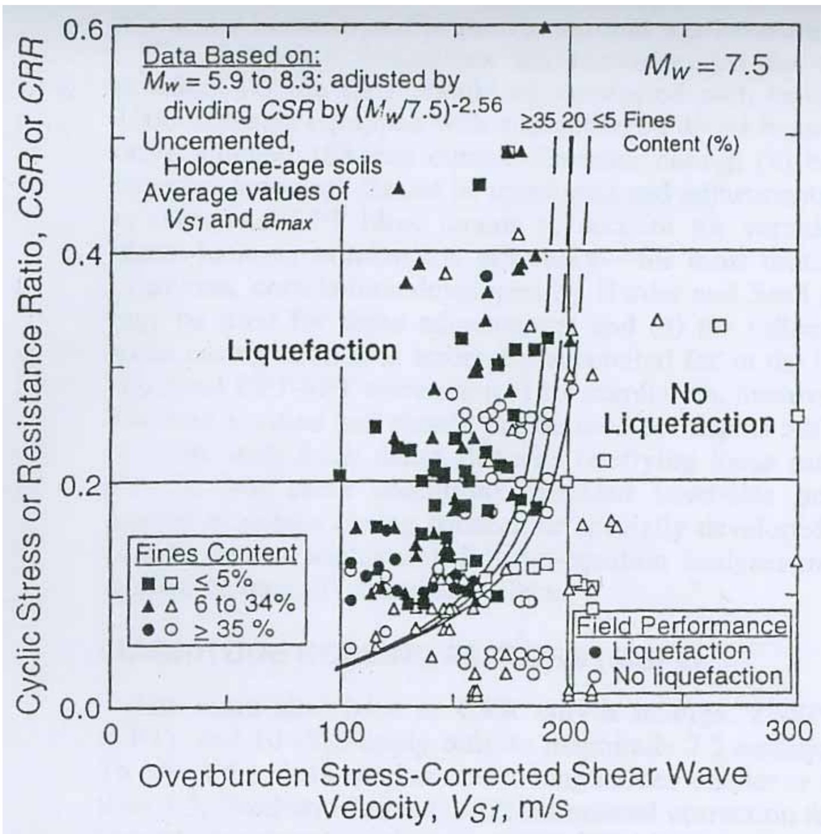

Figure 9.4 Liquefaction Relationship Recommended for Clean, Uncemented Soils with Liquefaction Data from Compiled Case Histories (Reproduced from Youd, et al. 2001)

9.3.6 Summary of Liquefaction Potential of Tailings

The liquefaction potential of the tailings was considered using a variety of methods: SPT testing; CPT testing; criteria for fine-grained soils; in situ shear wave measurements; and laboratory testing.

None of the methods can be expected to give a definitive answer since most methods are based on empirical evaluations of native, granular soils. Secondly, laboratory tests cannot adequately reproduce in situ conditions. Table 9.4 lists some of the issues controlling the reliability of each method.

Table 9.4 Liquefaction Assessment Methods

METHOD	RELIABILITY FOR LIQUEFACTION ASSESSMENT		CONCLUSIONS[1]
	Points in Favor	Points Against	
SPT	• Large database and extensive empirical correlations • Large number of tests available	• Case histories strongly focused on natural granular soils • Not considered reliable for very fine soils	• Based on average values, the new tailings are not liquefiable, and the old tailings are liquefiable.
CPT	• Mainstream method, good database, interpretation method well established • More repeatable than SPT • Accounts for pore pressure developed during testing	• CPT based interpretation gives opposite result from interpretation where CPT is converted to $(N_1)_{60\text{-}cs}$ • Non natural soil, with high S.G., low modulus	• CPT based interpretation shows not liquefiable • SPT derived from CPT gives results as per SPT method
Criteria for Fine Grained Soils	• Simple screening test	• Developed for natural soils • Methods give conflicting results	• Conflicting results, but most show liquefaction potential
Laboratory test	• Test done on actual material • Test on remolded soil simulated field conditions	• Undisturbed sample was deemed too difficult to obtain so remolded sample used • Large corrections needed for cyclic triaxial to compare with field conditions • Small number of tests available	• Not liquefiable but pore pressure response expected • Fairly high post liquefaction shear strength measured
In situ Shear Wave Measurements	• In situ test does not rely on destructive sampling	• Small strain not necessarily representative of earthquake strain • Method developed for natural soils, very high S.G. of tails not directly modeled • Soil density varies with the inverse square of velocity making this method a coarse estimate of liquefaction potential. • Measures bulk properties	• Not liquefiable

Notes: 1. All tests conclude that the tailings are not liquefiable under DBE. The above conclusions relate to MDE.

Based on the data available from the five approaches to liquefaction assessment, Klohn Crippen's conclusions are as follows:

- The tailings are not liquefiable and should not experience significant softening under the DBE. All methods point to this conclusion.
- There is a possibility that water saturated tailings could liquefy in some portions of the pile under the MDE. The old tailings appear to be more susceptible to liquefaction than the new tailings.
- Where saturated new tailings do not liquefy, a pore pressure increase is expected because the factor of safety against liquefaction is less than 1.4. Field corrected laboratory triaxial tests suggest pore pressure rise up to 96% of overburden stress. Cyclic shear testing indicates very limited pore pressure rise would occur. On average, based on all tests, a pore pressure increase of 33% of the effective overburden stress is estimated as the design condition.
- Under liquefied conditions, even in the most susceptible material prepared at low initial density (88% standard Proctor density) and tested at high stress, the post liquefaction strength of the new tailings is quite high indicating that the soil is dilatant under post liquefaction extension strain (worst case condition), which means that flow failure is not likely.

Groundwater levels in the tailings pile should be monitored after an earthquake equal to or larger than the DBE to assess if there are measurable increases in pore pressure.

Liquefaction is not expected under DBE type loading and is, thus, not a significant risk during operation. Liquefaction is considered to be primarily a long-term closure risk. Further, the impact of tailings liquefaction, if any, on stability of the pile is highly dependent on the extent of saturation in the tailings. As the tailings pile is constructed, data from field observations and instrumentation will improve the understanding of the extent of long term TSF water and saturation levels and will allow a better assessment of this long term risk. In the meantime, for the purposes of this report, a reasonable design scenario for closure is to assume that new tailings experience a pore pressure rise of 33% of pre-earthquake effective stress and the old tailings liquefy.

10. STATIC AND DBE STABILITY

Static stability analyses of the ultimate tailings pile at the overall Stage 2 configuration were carried out using the limit equilibrium method (Morgenstern-Price) with the Slope/W component of the GeoStudio 2004 computer program (Geo-Slope, 2005). Since no liquefaction or softening is expected under the DBE, the static analyses were used to calculate yield acceleration and deformation under the DBE.

Ten representative sections through the pile were analyzed. Dwg. D-41005 shows a plan of the tailings pile and the location of each stability section. The stability sections were selected to represent the more critical sections of the TSF. The lowest factor of safety was determined for each case, although in some instances very shallow surface slides of low consequence were ignored, as is common practice.

10.1 Results of Static and DBE Stability Analyses

The results of the static stability analyses using effective stress analysis and peak and residual strengths are presented in Table 10.1. The sections with stratigraphy, material properties, and critical slip surfaces for peak and residual conditions are in Appendix VIII.

Table 10.1 Static Stability Results Effective Stress Analysis

STABILITY SECTION	LOCATION	MINIMUM FOS		COMMENTS
		PEAK STRENGTH	RESIDUAL STRENGTH	
MINIMUM REQUIRED FOS		**1.5**	**1.3**	
1a	Northeast	1.7	1.6	Assumes partial excavation of Sand and Gravel at toe (Klohn Crippen 2004).
1b	Northeast	1.7	1.6	Based on Section 1 but with tailings slope extended to ultimate elevation 330 ft at 3H:1V slope.
2	North	2.3	1.9	
3	South Slope	2.3	2.0	
4	West Buttress	1.9	1.8	
4b	West Buttress	1.7	1.6	
5a	Northwest[1]	2.7	1.8	Failure along liner interface assuming residual strength of 16°.
5b	Northwest[1]	1.8	1.3	Failure along liner interface assuming residual strength of 16°.
5c	Northwest[1]	2.5	1.7	Based on Section 5a; Tailings slope increased to 3H:1V, from toe to elevation 280 ft. Failure along liner interface assuming residual strength of 16°.
6	Southeast	2.1	1.3	Failure along liner interface

Note 1. Northwest liner system will be designed to have 16° minimum residual strength.

Analysis of Southeast Section 6 using peak and residual strengths for the tailings and liner gives factors of safety of 2.1 and 1.3, respectively, which meets the design criteria. This analysis includes the current rock fill toe berm, and assumed access road at El. 185 ft on the section,

10.2 Seismic Deformation under DBE

Although the tailings pile meets acceptable safety factors for limit equilibrium slope stability during the DBE, some deformation is expected. Seismic deformation of the tailings pile was assessed using pseudo-static methods (Hynes-Griffin, et al. 1984) assuming static peak strengths in the tailings and liner systems and then assuming static residual strengths for the tailings and liner. The analysis was applied to stability Sections 3 (South side) and 6 (Southeast). Section 6 is representative of the tailings pile areas with a geosynthetic liner system, and Section 3 represents those areas of the tailings pile without the liner system. Newmark's sliding block model, which provides the basis for the Hynes-Griffin deformation prediction, is a good representation of tailings sitting on a lined foundation.

The yield acceleration (the acceleration at which the calculated factor of safety is 1.0) for static peak conditions, is 0.30 g in Section 3 (no liner) and 0.28 g in Section 6 (with liner). The yield acceleration for static residual strength conditions is 0.26 g in Section 3 (no liner) and 0.08 g in Section 6 (with liner). The design ground acceleration for the DBE is 0.15 g (Klohn Crippen, 1998). A summary of yield accelerations and estimated deformations is in Table 10.2.

Table 10.2 Predicted Deformations under the DBE

SECTION	STATE	STATIC FOS	YIELD ACCEL. FOR FOS = 1.0	PREDICTED DEFORMATIONS (inches)	
				DBE (PGA=0.15g)	
				Mean	**Upper Bound**
Section 3 (no liner)	Static Peak	2.3	0.30	< 4 [1]	6
	Static Residual	2.0	0.26	< 4 [1]	8
Section 6 (with liner)	Static Peak	2.1	0.26	< 4 [1]	8
	Static Residual	1.3	0.07	< 4 [1]	43

Note: 1. Below Hynes-Griffin deformation curve limits.

The Hynes-Griffin method assumes a relatively large base amplification factor based on case histories from many sites. The South and Southeast areas of the pile are largely founded on rock or dense till, and hence, the base amplification is expected to be low and calculated deformation in these areas is expected to be at the low end of the ranges presented in Table 10.2.

The expected deformation is not expected to significantly affect the pile stability. However, there could be some disruption to the liner system and to underdrain pipes. In general, the underdrains are designed to function without the drain pipes in place, so disruption of piping is not a major concern. The drain gravel zone around the pipes is sized to handle the design flows without the pipes. Localized tears in the liner may occur but the risk of significant liner disruption is considered to be low. Liner performance following the 1989 M6.9 Northridge earthquake, in California, was studied in detail by Augello et al (1995) and they concluded that liner damage was minimal for earthquake loading similar to the Green's Creek DBE.

10.3 Temporary Construction Conditions

10.3.1 Southeast 2 Expansion Area

Temporary geometry in the Southeast 2 expansion area was reviewed to assess if there were short term conditions (prior to filling in Pond 6) which could result in stability problems. The critical case was found to be Section 7 (See Figure VIII-11, Appendix VIII) where a section running down the steepest portion of the liner daylights into Pond 6. The static safety factors for this condition were 2.1 using peak strengths for the tailings and liner system, and 1.3 using residual strengths on the tailings and liner system. This section meets the criteria described in Section 8.3.

Similar assessments of temporary slopes will be made during final design of each expansion area.

10.3.2 Undrained Strength

Stability analysis was carried out to represent a temporary condition in which pore pressures are elevated due to construction. Based on the data presented in Section 5.3, a design average undrained strength of 1500 psf appears appropriate for the tailings.

Southeast expansion area Section 6 and Section 7 were analyzed with the area built to El. 240 ft. Elevation 240 ft is an estimate of the maximum height that Southeast 2 may be raised to in 2006 based on information by KGCMC. The sections are shown in plan on Drawing D-41005, and the stability sections and failure surfaces are on Figure VIII-12 (Appendix VIII).

The minimum FOS based on undrained strength analysis for both Section 6 and Section 7 are greater than the temporary condition design criteria FOS of 1.3.

Table 10.3 Static Undrained Strength Stability Results

STABILITY SECTION	LOCATION	MINIMUM FOS
6	Southeast	1.6
7	Southeast	1.5

10.3.3 Elevation of Tailings Piezometric Surface

To model a temporary pore pressure condition in the tailings during construction, the water level in the tailings was raised to the surface of the ultimate tailings pile for stability Sections 1 through 6. When using peak strengths on the tailings and liner the FOS against a major slope failure through the foundation was greater than 1.3 for all sections, except Section 4 (West Buttress). The minimum FOS for Section 4 was 1.2. When using residual strengths in both the tailings and liner, where present, the FOS against a major slope failure through the foundation was greater than 1.0 for all sections, except for Sections 5b (Northwest) and 6 (Southeast 2) with liner systems. The minimum FOS for Sections 5b and 6 was 0.8. For the sections where the temporary safety factor was below design criteria, the analysis was re-run assuming the construction pore pressure was 70% of the height of the fill. All sections passed the criteria for 1.3 safety factor based on peak strength and exceeded 1.0 based on residual strength.

Thus, as construction proceeds in the West Buttress and Southeast areas, KGCMC must take special care to monitor the pore pressures in the tailings. If the pore pressure increases and approaches a level of 70% of the thickness of the rising tailings layer at any time, work must be stopped in this area to allow the pore pressures to dissipate and an analysis done to assess when construction can restart. For example, if the tailings thickness is 100 ft above the measurement point, the water pressure at that point in the column should not exceed 70 ft (i.e., 30 ft below the top of the pile) without conducting a stability assessment. This same calculation can be done for any point in the column. A typical earthfill monitoring program could include, piezometers placed roughly at 1/3 from the base, at mid height and at 1/3 from the top of the planned final height (H) of the pile during that construction season. The piezometers would be allowed to read 2/3*0.7*H; 0.5*0.7*H and 1/3*0.7*H respectively, without needing further assessment of stability.

11. POST-EARTHQUAKE (MDE) STABILITY

West Buttress Section 4b was analyzed for the post-earthquake (MDE) stability case, as it contains the largest volume of old tailings, and the highest water table among the sections (see Figure VIII-6, Appendix VIII) and as such represents the most critical area of the TSF.

11.1 Stability Analysis

Post-earthquake stability analyses of the tailings pile was carried out using the limit equilibrium method (Morgenstern-Price) with the Slope/W component of the GeoStudio 2004 computer program (Geo-Slope, 2005).

As per the SPT liquefaction assessment presented in Section 9, we concluded that the old tailings would liquefy under the MDE. Thus, post-liquefaction strength was applied to the old tailings located below the water table.

The average FOS against liquefaction under the MDE for the new tailings is 1.3 based on cyclic laboratory testing. This is greater than 1.1, so the new tailings are not expected to liquefy under the MDE event. However, the FOS is less than 1.4 and so the tailings will likely experience some cyclic softening. In this case, the new tailings below the water table was assigned a pro-rated strength that is 33% less than the peak static strength.

Tailings (old and new) above the water table were assigned residual static (drained) strengths.

The resulting FOS against slope stability failure calculated for this post-earthquake condition on Section 4b, was 1.1, indicating that a flow slide failure of the slope will not occur. However, some deformation is expected. The design criteria requirement is for

FOS ≥ 1.0 for the post-liquefaction condition as described in Section 8.3. This analysis is very sensitive to the elevation of the water table and therefore we recommend that post-closure water levels be monitored.

11.2 Sensitivity Analysis

We conclude from the current analyses that the new tailings will not liquefy under the MDE. However, liquefaction assessment practices can change over the years, as more data is collected and as the state of practice evolves. Consequently, post-liquefaction stability analyses were completed for 10 stability sections to check that there are reasonable contingency measures available if future data or liquefaction assessment practices indicate that liquefaction of the new tailings is possible.

Stability analysis which assumes that all tailings (old and new) below the water table will liquefy is presented in Appendix VIII. The FOS for this hypothetical condition is greater than 1.0, except for Section 4 (West Buttress). A rock fill toe berm would be required on the West Buttress to raise the FOS above 1.0 for this hypothetical full liquefaction case.

11.3 Seismic Deformation under MDE

Although the tailings pile generally meets acceptable safety factors against limit equilibrium slope stability failure during the MDE, some deformation is expected. KC current practice in these types of cases is to undertake a FLAC analysis of the pile as the best means of assessing deformations. However FLAC is a complex tool, and in our experience needs to be used only when the design conditions are well known. Some critical parameters such as final ground water levels, and liquefaction susceptibility of tailings are being studied and will continue to be studied through the life of the project. When these and other uncertainties are better defined a FLAC analysis could be run to provide confidence in the predicted displacements for closure. In the meantime simple analyses were run to obtain a gross estimate of possible deformations based on current assumed closure conditions. These methods are discussed in the following sections.

11.3.1 Hynes-Griffin Deformations

Seismic deformations of the tailings pile were assessed using pseudo-static methods (Hynes-Griffin, et al. 1984) assuming liquefaction in saturated old tailings (post-liquefaction condition), and reduced strength (cyclic softening) in the saturated new tailings. While this is a non typical application of the Hynes-Griffin Franklin analysis, there is significant precedent for using this approach for cases where liquefaction is expected but a post liquefaction slope stability safety factor of greater than 1 is predicted. This simplified approach to assessing deformation in these specific conditions, which apply at Green's Creek, is described in a later paper by Hynes-Griffin Franklin (Marcusson, W.F. III, Hynes, M.E., Franklin A.G. Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankment; Earthquake Spectra Vol 6 No 3, 1990 pp 529 to 572). 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. 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. However for the case where the method is being used with post earthquake softened strengths this may not always be the case.

Using the design ground acceleration of 0.3 g for the Maximum Design Earthquake (Klohn Crippen, 1998). The predicted deformations for the mean and mean plus 1 standard deviations range from 3 ft to 6 ft.

The Hynes-Griffin method assumes a relatively large base amplification factor based on case histories from many sites. The south and southeast areas of the pile are largely founded on rock or dense till, and hence, the base amplification is expected to be low and calculated deformation in these areas is expected to be at the low end of the predicted deformation range.

11.3.2 Lab Based Deformations

The maximum shear strain required to reach post-liquefaction strength developed during the cyclic triaxial test was 14.1%. The equivalent peak post-cyclic shear strain was 18.9% (See Appendix II). Testing was carried out on new tailings.

The West Buttress (Appendix VIII, Figure VIII-5) has the thickest layer of tailings (~50 ft) below the water table (old plus new tailings were considered).

Using the shear strains and liquefied tailings thickness quoted above (applied on both old and new tailings), the range of expected deformations for the West Buttress, based on lab data, would be approximately 7 ft to 9.5 ft.

11.3.3 Comparison of Seismic Deformations

Table 11.1summarizes the results of the two methods that were used to estimate seismic deformations resulting from liquefaction of the tailings below the water table.

The expected range of deformation is between 3 ft and 10 ft.

Table 11.1 Comparison of Seismic Deformation Estimates

METHOD	MEAN (ft)	MEAN PLUS 1 STD DEV (ft)
Hynes-Griffin	3	6
Laboratory Cyclic Shear Strain	9.5	not applicable

The expected deformation will not significantly affect the pile stability. However, there could be some disruption to the liner system and to underdrain pipes. Reference to Augello et al (1995), see Section 10.2 suggests that some liner tears could occur but this is not expected to be a major concern for pile stability as discussed in Section 10.2.

12. CONCLUSIONS

The following conclusions are based on current standard analysis, measurements, and data received prior to January 20, 2006:

- All materials were found to be safe against liquefaction under the DBE.
- Two materials have some potential for liquefaction or softening under the MDE in zones where they are saturated: an intermittent shallow sand and gravel layer (Unit 5) and the tailings. The MDE loading case is critical in the seismic stability assessment.
- Careful assessment of 2004 and 2005 drill hole data and review of previous data, indicates that the sand and gravel (Unit 5) deposits are generally not liquefiable, except in the Northeast region of the pile. The Northeast area has not yet been fully constructed, and remaining Unit 5 sand and gravel deposits will be removed beneath the expansion of the tailings pile in this area.
- The liquefaction potential of the tailings is difficult to determine as it is a manufactured material. The majority of research on liquefaction pertains to clean sands and with corrections for silty sands (up to 35% silt). Five methods were used to assess the liquefaction potential of the tailings. The most reliable method is considered to be the cyclical laboratory testing, which indicates that the new tailings are not liquefiable, even when placed at initial density as low as 88% standard Proctor dry density.
- SPT and CPT data indicate that the old tailings are less dense than the new tailings, and are susceptible to liquefaction under the MDE. This could be confirmed by cyclic testing of the old tailings.
- Based on our judgment of the extent of liquefaction and softening in the tailings under the MDE, the pile will be stable under MDE with deformations in the likely range of 3 ft to 10 ft.
- The tailings pile meets the peak static stability criteria in all sections.
- The tailings pile meets the residual static stability criteria in all sections.
- The performance of tailings under seismic loading is the subject of research in many universities and the understanding of behavior of silt

under seismic loading is expected to improve over the years. Consequently, while KC believe that the new tailings will not liquefy under the MDE, we believe that it is sensible to assess the consequence of such liquefaction, if subsequent data or the evolving state of practice were to result in a different conclusion. Consequently this report includes, in Appendix VIII, a detailed analysis of the stability of the TSF under the assumption that the tailings below the water table liquefies. Our conclusion is that in this case a modest rock toe berm around the West Buttress would suffice to prevent flow failure of the pile and limit deformation to the order of several feet. On other sides of the pile berms are not needed.

13. RECOMMENDATIONS

Based on the preceding discussions, KC recommends the following:

Annual Plan

- Prepare an annual plan to maintain placement rates below about 4.5 ft per month for no more than 6 months in any one area. If a faster or longer rate of rise is planned, undertake an analysis of pore pressure increase and stability.

- Maintain and improve tailings placement techniques so that the majority of the pile is placed at no less that 90% standard Proctor density. Monitor this by regular inspection and testing.

Piezometer / Instrument Installations

- Install and maintain sufficient piezometers and lysimeters to accomplish the following:

  - Measure water levels during construction to identify construction pore pressures, possible perched water tables and tailings saturation levels. Tailings placement rate and location should be adjusted accordingly to avoid build-up of excessive construction pore pressures.

  - Install piezometers in areas of high placement rates to confirm that pore water pressures rise no higher than 70% of the thickness of tailings above the measuring point. In areas of higher planned placement rates, the pile stability should be reassessed and mitigative measures for reduction of tailings water levels may need to be applied.

  - Measure long term water and saturation levels. This will confirm the possible extent of liquefaction, which is restricted to saturated materials.

  - We recommend the installation of an accelerometer on rock. The accelerometer may be installed within an existing building founded on rock for protection. If an earthquake occurs, check to see if any pore pressure increase is measured.

- Additional piezometers should be installed above and below the geomembrane liner during the construction of future lined expansion areas. Readings from these instruments will help to confirm that under drainage features continue to operate as designed.

Modeling

- Consider developing a 3-D model to keep track of tailings placement location and depth over time and to improve the assessment that there are no systematic regional variations in tailings density or SPT. Other attributes such as geochemistry, geotechnical drilling and laboratory data, and in situ density test results can be added to the model. Once the model is developed, it can be updated using survey data provided by KGCMC.

- A site response analysis using the program SHAKE (SHAKEEDIT) should be carried out to check the assumed amplification factor for the design earthquake and provide a more reliable means of calculating deformations.

- To reduce the uncertainty in water levels assumed for long term closure stability analyses, we recommend that the following conditions be modeled using a selection of 2D and 3D programs such as SEEP/W, SoilCover and/or SoilVision:

  - Operating Condition - develop a model and calibrate the model using observations made during operation; and

  - Closure Condition - use calibrated model to determine the phreatic surface and saturation contours for normal rainfall and 10-yr wet cases;

- When long term ground water levels, saturation conditions, and possibility of tailings liquefaction and/or softening can be more accurately predicted review the need for a FLAC deformation analysis.

- Liaise with designers of the tailings closure cover to stress the importance of a low water level in the pile.

Testing / Correlations

- Periodically check the undrained strength of the tailings as placed. The undrained strength can be tested using a hand vane or pocket penetrometer. An average strength of 1500 psf should be maintained. If the strength is less than this, KGCMC should consider removing and recompacting the tailings.

- Continue to carry out periodic strength testing (direct shear or triaxial) of the tailings to confirm static and cyclic strength properties used in the analyses.

- Conduct additional cyclic simple shear tests to assess the liquefaction susceptibility of old and new tailings compacted to less than 90% Standard Proctor density (i.e., to represent loosely placed tailings).

- Develop a correlation between SPT and other field testing and in situ density testing by test-pitting to measure in situ density at 2004/2005 SPT test locations. This may also be achieved by carefully testing and sampling tailings exposed in the proposed excavation for the Northwest corner. A program of sampling and testing for the Northwest corner excavation is being prepared.

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REFERENCES

Alaska Department of Natural Resources – Dam Safety and Construction Unit (2003), "Guidelines for Cooperation with the Alaska Dam Safety Program", September 2003.

Andrews, D. C. A. and G.R. Martin (2000). Criteria for Liquefaction of Silty Soils. 12th World Conference in Earthquake Engineering, Paper No. 312.

Augello, A.J., Matasovic, N. Bray, J.D., Kavazanjian, E., Jr., and Seed, R.B. (1995), "Evaluation of Solid Waste Landfill Performance During the Northridge Earthquake," In: Earthquake Design and Performance of Solid Waste Landfills, ASCE Geotechnical Special Publication No. 54, pp. 17-50.

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.

Bowles, J.E. (1988). Foundation Analysis and Design. 4th Edition.

Bray, J.D., R.B. Sancio, M.F. Riemer, and T. Durgunoglu (2004). Liquefaction Susceptibility of Fine-Grained Soils. 11th International Conf. on Soil Dynamics and Earthquake Engineering and 3rd International Conf. on Earthquake Geotechnical Engineering, Berkeley, CA, Jan. 7-9, 2004.

Chen, W.F. (1995). The Civil Engineering Handbook. CRC Press.

Cone Tec (1997). Field Report – Cone Penetration Testing and SPT Energy Calibration at Green Creek Mine. November 5, 1997.

Drumright, E.E., C.W. Pfingsten, and R.G. Lukas (1996). Influence of Hammer Type on SPT Results. Journal of Geotechnical Engineering. Vol. 122, No. 7. July. pp. 598-599.

Environmental Design Engineering (EDE) (2002). *"Kennecott Greens Creek Mine – Stage II Tailings Expansion, Hydrologic Analysis"*. Engineering Report submitted to KGCMC. 61 pages; attachments.

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

Finn, W.D.L., R.H. Ledbetter and G. Wu (1994). Liquefaction in Silty Soils: Design and Analysis. Ground Failures under Seismic Conditions, Geotechnical Special Publication 44, ASCE, New York, pp. 51-76.

Geo-Recon International Ltd. (1981). Report of Geophysical Investigation Tailings Damsites, Greens Creek Project Admiralty Island, Alaska. July 21, 1981.

Geo-Slope International (2005). GeoStudio 2005 Slope/W computer program.

Geotechnical Engineers Inc. (1983). Project No. 82989. Addendum to Report on Laboratory Testing. January 1983.

Gibson, R.E. (1958). The Progress of Consolidation in a Clay Layer Increasing in Thickness with Time Geotechnique. Vol. VIII. Pp. 171-182.

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.

Idriss, I. M. and R.W. Boulanger (2004). "Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes", 11th International Conf. on Soil Dynamics and Earthquake Engineering and 3rd International Conf. on Earthquake Geotechnical Engineering, Berkeley, CA, Jan. 7-9, 2004.

Idriss, I. M. (2004), "Resolved and Unresolved Issues in Soil Liquefaction", International Conference on Cyclic Behaviour of Soils and Liquefaction Phenomena, Bochum, Germany, April 1, 2004.

International Commission on Large Dams (ICOLD, 1995). "Tailings Dams and Seismicity", Bulletin 98, ICOLD, Paris, France.

Jeffries, M. G. and M.P. Davies (1993). Use of CPTu to Estimate Equivalent SPT N_{60}. Geotechnical Testing Journal, Vol. 16, No. 4, pp. 458-468.

Klohn Crippen Consultants Ltd. (1998). Seismic Hazard Assessment at Greens Creek Mine Site, Admiralty Island, Alaska, August.

Klohn Crippen Consultants Ltd. (1999). Kennecott Greens Creek Mining Company Evaluation of Tailings Pile Final Report. July.

Klohn Crippen Consultants Ltd. (2003a). Kennecott Greens Creek Mining Company Tailings Project 2002 Geotechnical Investigation – Field Report. April.

Klohn Crippen Consultants Ltd. (2003b). Kennecott Greens Creek Mining Company Tailings Compaction Results – Final. May.

Klohn Crippen Consultants Ltd. (2003c). Kennecott Greens Creek Mining Company Tailings Project Summary of Klohn Crippen Reports – 2003 Update. October.

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 Consultants Ltd. (2005a). Kennecott Greens Creek Mining Company 2004 Geotechnical Investigation – Field Data Summary. April 20, 2005.

Klohn Crippen Consultants Ltd. (2005b). Kennecott Greens Creek Mine 2005 Geotechnical Investigation – Field Data Summary. August 16, 2005.

Klohn Crippen Consultants Ltd. (2005c). Kennecott Greens Creek Mine Evaluation of Co-Disposal of Production Rock and Filter Pressed Tailings. May 16, 2005.

Klohn Crippen Consultants Ltd. (2005d). Kennecott Greens Creek Mining Company Stage 2 Tailings Facility Expansion - Stability Update. September 27, 2005.

Klohn Crippen Consultants Ltd. (2005e). Kennecott Greens Creek Mining Company Seismic Investigation of Greens Creek Mine Tailings Area – Draft. December 9, 2005.

Koester, J. P. (1992). The Influence of Test Procedure on Correlation of Atterberg Limits with Liquefaction in Fine Grained Soils, Geotechnical Testing Journal, Vo. 15, No. 4, pp. 352-361.

Kulhawy, F. H. and P.W. Mayne. (1990). Manual on Estimating Soil Properties for Foundation Design, Electric Power Res. Inst. EL-6800; Prof. 1493-6, Electric Power Res. Inst., Palo Alto, Calif., 2-38.

Liao, S. and R.V. Whitman (1986). Overburden Correction Factors for SPT in Sand. ASCE Journal of Geotechnical Engineering, Vol. 112, No. 3, pp373-377.

Lysmer, J., T. Udaka, C.F. Tsai, H.B. Seed (1975). FLUSH: a computer program for approximate 3-D analysis of soil-structure interaction problems. UCB/EERC-75/30, Berkeley: Earthquake Engineering Research Center, University of California, Nov. 1975, 139 pages.

Marcusson, W.F. III, Hynes, M.E., Franklin A.G. Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankment; Earthquake Spectra Vol 6 No 3, 1990 pp 529 to 572

Robert Miner Dynamic Testing Inc. (2005). Standard Penetration Test Energy Measurements, Drill Hole DH05-08, March 21, 22, & 23, 2005, CME 850 Drill Rig, CME Auto Hammer, Cascade Drilling, Greens Creek Mine, Admiralty Island, Alaska, April 2, 2005.

Robertson , P. K., R.G. Campanella. and A. Wightman (1983). SPT-CPT Correlations, ASCE Journal of Geotechnical Engineering, Vo. 109, No. 11, pp. 1449-1459.

Robertson, P. K. (1990). Soil Classification Using CPT, Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 151-158.

Robertson, P. K., and C.E. Wride. (1998). Evaluation of Cyclic Liquefaction Potential Using the Cone Penetration Test, Canadian Geotechnical Journal, Vol. 35, No. 3, pp. 442-459.

Schnabel, P.B., J. Lysmer and H.B. Seed (1972). SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. Report No. EERC 72-12, Earthquake Engineering Research Centre, University of California, Berkeley, Calif., Feb.

Seed, H.B., K. Tokimatsu, L.F. Harder and R.M. Chung (1985). The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, ASCE Journal of Geotechnical Engineering, Vol. 111, No. 12, pp1425-1445.

Seed, H.B. and P. De Alba, (1986). Use of SPT and CPT tests for Evaluating the Liquefaction Resistance of Sand, Proc. Insitu ’86, ASCE Spec. Conf. On Use of Insitu Testing in Geotech Engrg., ASCE Special Publication No. 6.

Seed, H.B. and L.F. Harder (1990). SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength. Proc. H. Bolton Seed Memorial Symposium, J. Michael Duncan, Ed. Vol.2, pp. 351-376.

Seed, R.B., K.O. Cetin, R.E.S Moss, A.M. Kammerer, J. Wu, J.M Pestana, M.F. Riemer, (2001). Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation, in: Proceedings: 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 26-31, 2001.

Steffen, Robertson, and Kirsten (SRK) (1981a). Report No. 51501/1. Greens Creek Project Tailings Impoundment Preliminary Design Cannery Site. October, 1981.

Steffen, Robertson and Kirsten (SRK) (1981b). Feasibility Investigations For The Greens Creek Tailings Storage Site. February 4, 1981.

Steffen, Robertson, and Kirsten (SRK) (1982). Report No. 51505/2. Greens Creek Joint Venture Project Noranda Mining Cannery Tailings Impoundment Summary Report. December, 1982.

Steffen, Robertson and Kirsten (SRK) (1987a). Report No. 62203/2. Report on further Geotechnical Site Investigation at the Greens Creek Mill Site for Greens Creek Mining Company. September, 1987.

Steffen, Robertson and Kirsten (SRK) (1987b). Triaxial and Consolidation Report Greens Creek Mine – Saddle Tailings Pile, May 7, 1987.

Steffen, Robertson and Kirsten (SRK) (1987c). Report No. 62202/B. Technical Memorandum, Upper Cannery Tailings Impoundment, Laboratory Testing of Tailings for "Dry Tailings Disposal. January, 1987.

Steffen, Robertson and Kirsten (SRK) (1987d). Report No. 62203. Mill Site Geotechnical Investigation. May 1987.

Steffen, Robertson and Kirsten (SRK) (1988). Report No. 62202/A. Upper Cannery Tailings Impoundment Geotechnical Investigation and Materials Testing for Greens Creek Mining Company. July, 1988.

Steffen, Robertson and Kristen (SRK) (1989a). Report No. 62206/2. As-Built Report on the Cannery Muskeg Tailings Impoundment Greens Creek Project. February, 1989.

Steffen, Robertson and Kirsten (SRK) (1989b). Report No. 62210/1. As-Built Report on the Piezometer and Horizontal Drainage System at the Mill Site Backslope Greens Creek Project. April, 1989.

Steffen, Robertson and Kirsten (SRK) (1989c). Report No. 62210/2. Development and Remediation of Stability Problems at the Mill Site Backslope Greens Creek Project. May, 1989.

Steffen, Robertson and Kristen (SRK) (1992). USFS Response to Proposed Operations and Maintenance Plan for Waste Area "E". February, 1992.

Steffen, Robertson and Kirsten (SRK) (1993a). Report No. G106208. Report on Geotechnical Investigations Related to an Expansion of the Kennecott Greens Creek Mine Near Juneau, Alaska. January, 1993.

Steffen, Robertson and Kirsten (SRK) (1993b). Report No. G106102. Draft Report, Stability Analysis Related to Proposed Tailings Pile Expansion. January 1993.

Steffen, Robertson and Kirsten (SRK) (1996). Report No. G106113. Geotechnical Review and Analysis of the Greens Creek Tailings Pile. February, 1996.

Stark, T. M. and S.M. Olson (1995). Liquefaction Resistance Using CPT and Field Case Histories, ASCE Journal of Geotechnical Engineering, Vo. 121, No. 12, pp. 856-869.

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.

Youd, T.L., C.M. Hansen, and S.F. Bartlett. (2002) Revised multilinear regression equations for prediction of lateral spread displacement. Journal of Geotechnical and Geoenvironmental Engineering Vol. 128, No. 12, pp.1007-1017, December.

APPENDIX I

Piezometric Data and EDE Water Levels

Figure I-1. Tailings Piezometers

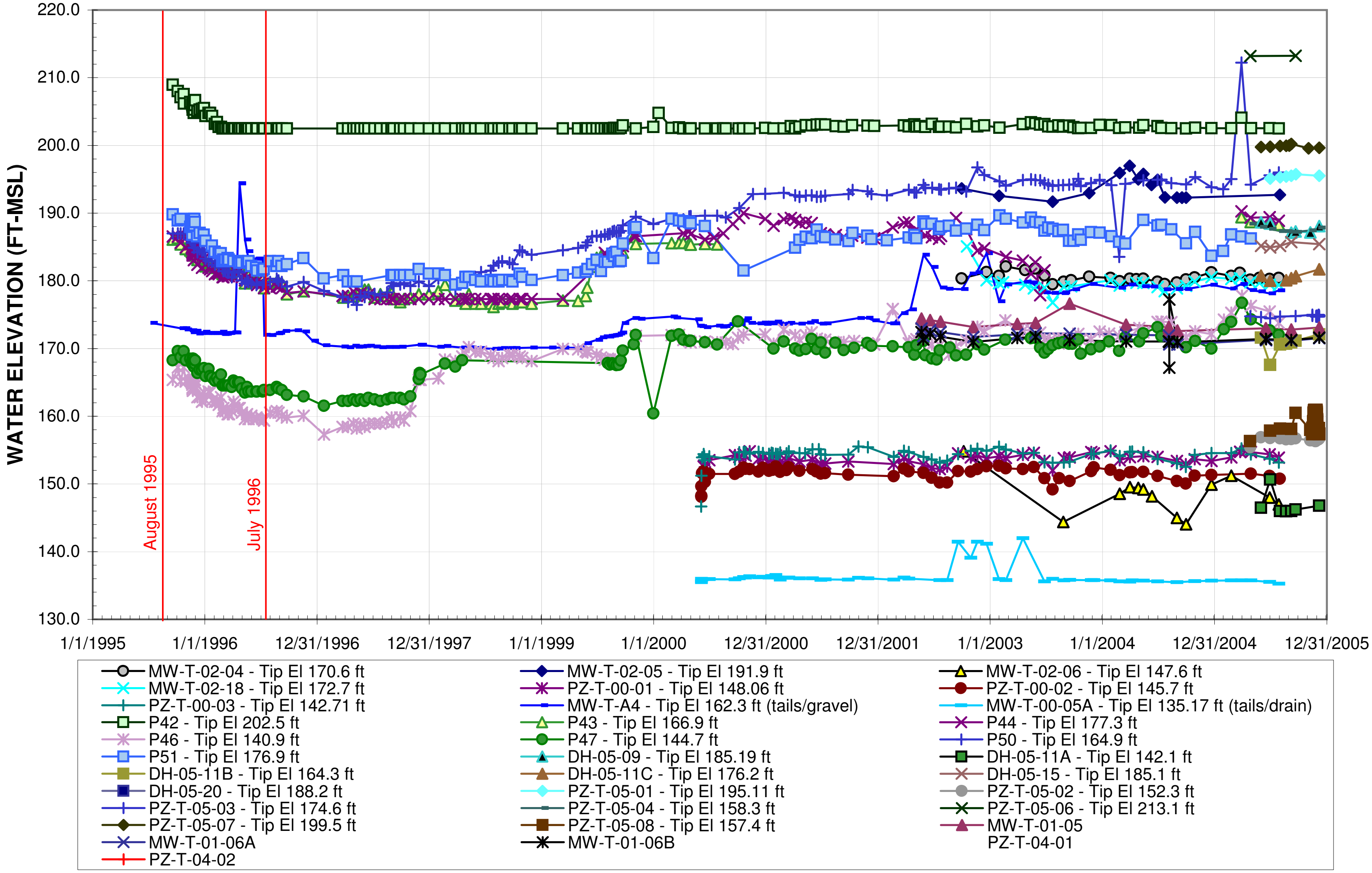


Figure I-2. Till Piezometers

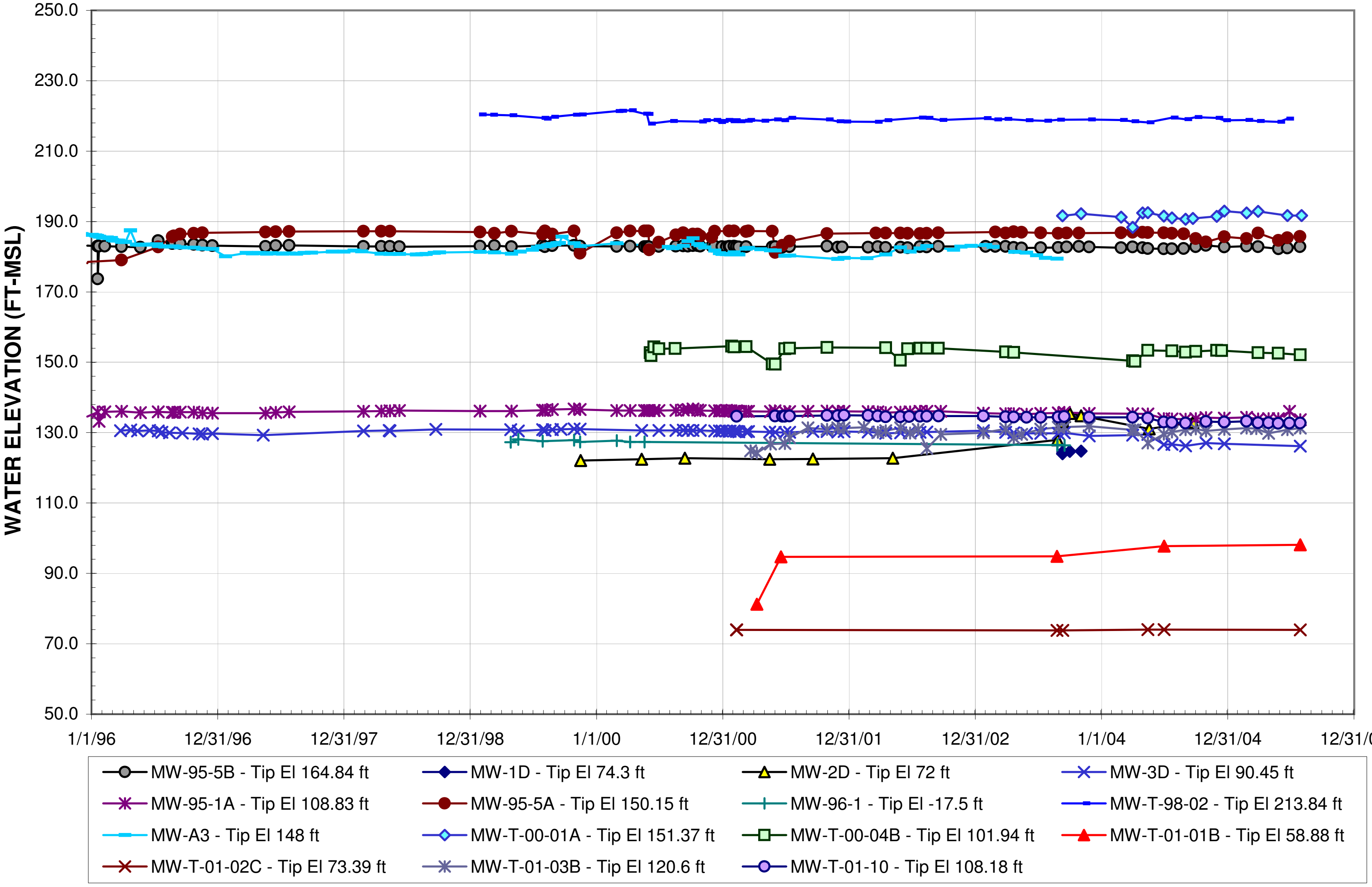


Figure I-3. Other Piezometers

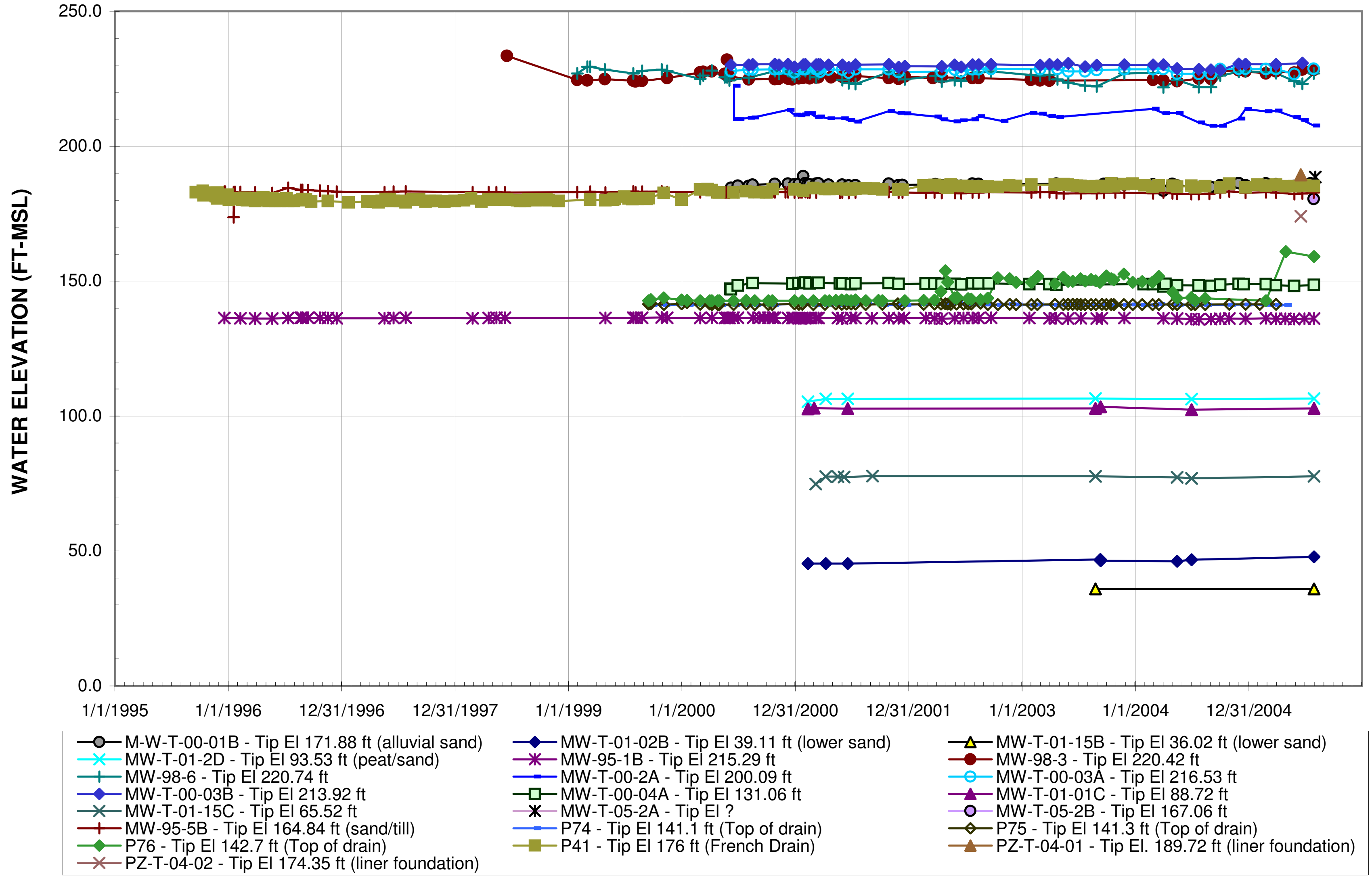


Figure I-4. Peat Piezometers

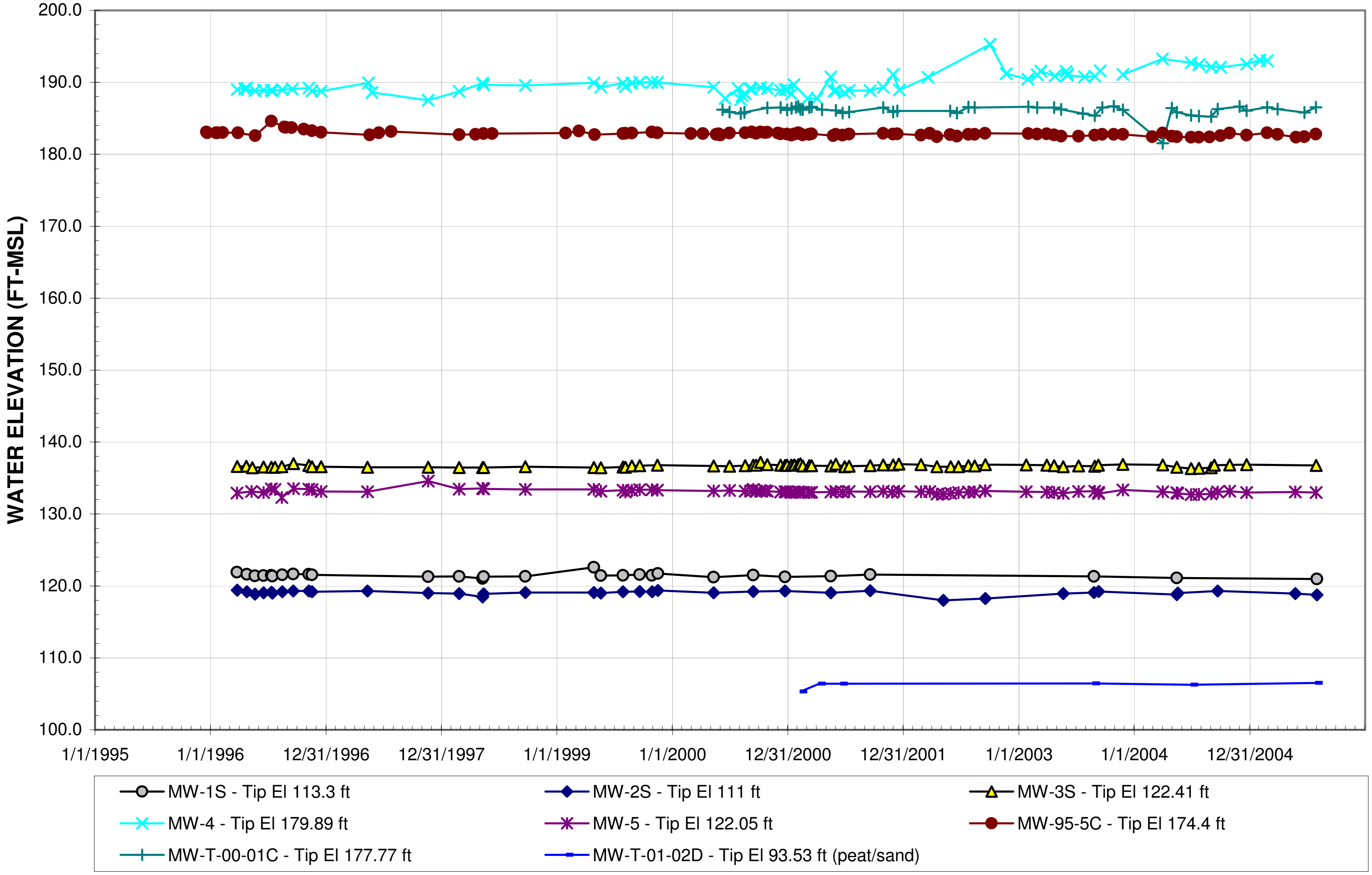
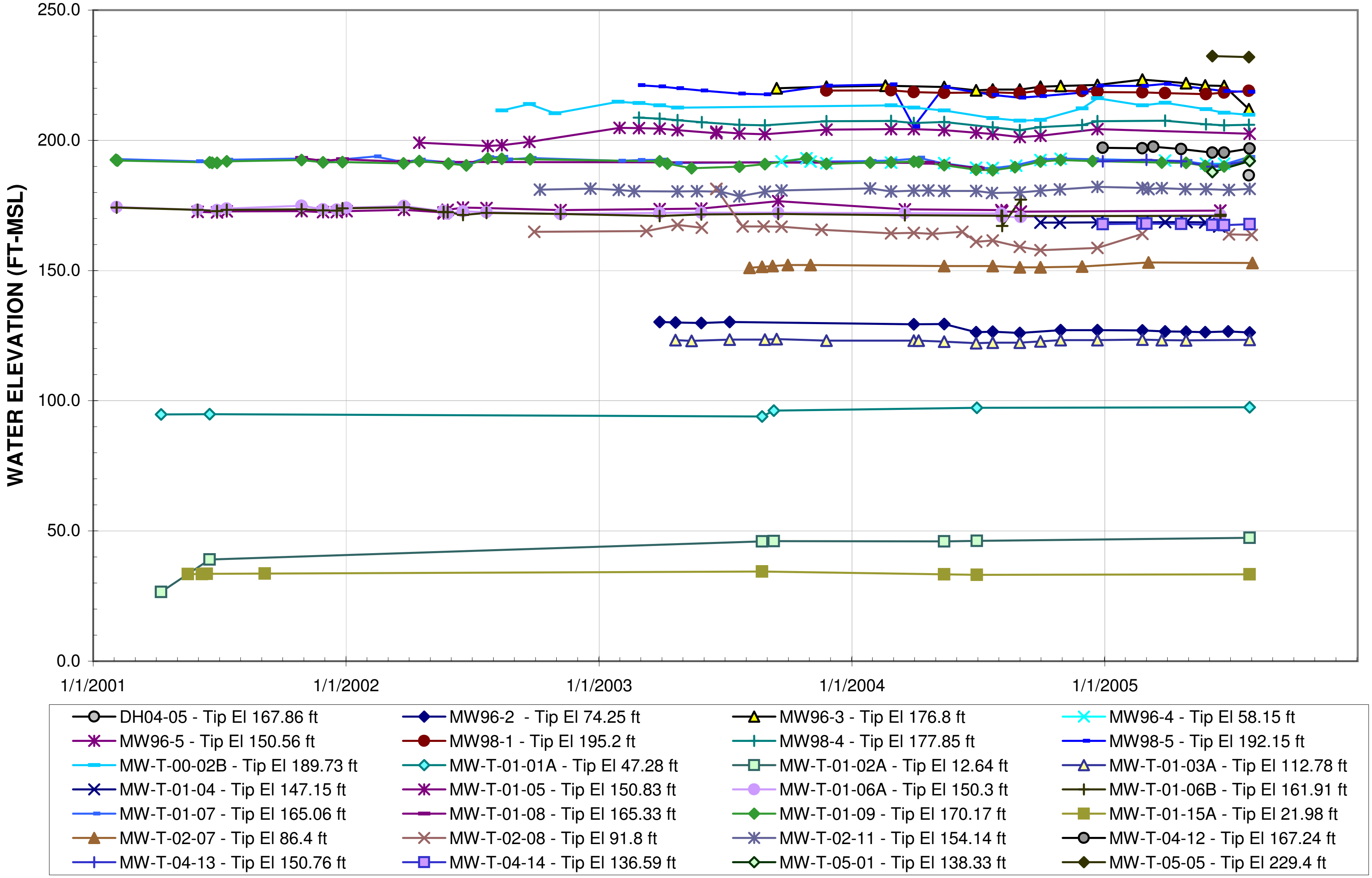


Figure I-5. Bedrock Piezometers



EDE CONSULTANTS – MEMO

EDE Consultants
Environmental Engineering * Hydrology * Water Resources Engineering

DATE:	8/8/05
TO:	BOB CHAMBERS, AL MORRISON, GEORGIA LYSAY; KLOHN CRIPPEN LTD.
FROM:	BRUCE NELSON, CHERYL NAUS; EDE
CC:	TOM ZIMMER, PETE CONDON, KERRY LEAR, ERIC SUNDBERG; KGCMC
RE:	PROJECTED PIEZOMETRIC HEADS IN THE TAILINGS SE2 AREA

BACKGROUND

The following is an evaluation of the projected piezometric heads within the SE2 expansion area. On July 29th EDE was requested by Greens Creek Mining Company and Klohn-Crippen Consultants Ltd. to provide predictions of long term and expected worst case piezometric heads in the SE2 tailings area for the purpose of conducting a stability analysis. It is understood that the urgent need for the stability analysis was triggered following a State of Alaska (DNRC) inspection of the SE2 area and subsequent finding that tailings placement in this area is denied until such time as a stability analysis is provided to the state inspector to his satisfaction. It is further understood that the SE2 site now has the bedding layer installed and the membrane/drain layers installed and is lacking only the service layer prior to placement of tailings.

OBJECTIVES

EDE has been in the process of updating the entire tailings site hydrologic assessment based upon new drilling, additional monitoring, and future expansion plans since the last update in 2002. This assessment is not complete and completion is not expected until late September. The current situation with respect to the prohibition of tailings placement in the SE2 area has redirected our attentions to this specific area with the purposes of:

1) Estimating the expected long term water levels in the tailings pile.

2) Estimating the expected long term water levels in the liner system (above and below).

3) Estimating the maximum probable water levels in the pile and the liner system.

ESTIMATION METHODS

Lacking a current, complete, tailings site hydrologic model, and further, lacking the time to complete that model under the urgency of the current circumstances, EDE has taken an

analog approach to assessing the water levels (heads) within the tailings and within the liner system (above and below the liner) for the SE2 area. This method uses measured field data to examine the post construction water level responses and current conditions principally within the East Expansion immediately to the north of the SE1 area and the SE1 expansion area as probable analogs of the SE2 area. Another potentially analogous area is the SW corner of the tailings South Expansion which is underlain by a blanket drain.

The SE1 area is constructed in a similar manner to the proposed construction of the SE2 area with apparent minor differences in the gradation of the bedding layer and potentially the service layer of the liner (not placed yet). Both the SE1 and SE2 areas overlie highly fractured quarried bedrock as a foundation condition.

The projection of water levels (heads) based upon an uncapped pile condition is expected to provide a conservative estimation of the performance of the under-drain layer and French drain systems as compared to the long term capped closure system incorporating a water balance cap. The East and South expansion areas have some utility as analogs, though they are underlain by peat and clay till and controlled by blanket and finger drain systems as opposed to a geo-synthetic liner system as is the case with the SE1 and SE2 areas.

<u>HEAD ESTIMATIONS</u>

Existing potentiometric head data from selected stand pipe piezometers (SP) and vibrating wire piezometers (VWP) were compiled for the life of the devices. Piezometer hydrographs were plotted to graphically determine if the water levels at a piezometer had achieved equilibrium with the rate of sub-drainage out of the pile and the rate of infiltration into the pile and/or the re-establishment of equilibrium in the underlying bedrock following liner and/or blanket drain placement. Suitable data exist for three bedrock VWP completions (beneath the liner) as well as a single SP within the tailings of the East Expansion area. These plots are contained in Attachment "A". A location map showing the groundwater monitoring network including piezometer and well locations and hydrologic controls is in Attachment "B".

Data from these 4 monitoring points extend from early 2001 to present for the VWP data and 2002 to present for the SP data. Examination of the hydrographs of the bedrock VWP data indicate that water level fluctuations of approximately 1 ft. to 2 ft are superimposed over an overall trend of a 1 to 3 ft. rise in head from 4/01 until approximately 6/02 and subsequently, a slight trend downward for the past 3 years. The bedrock appears to have reached an equilibrium water elevation condition in the East Expansion area. The tailings SP indicates a typical ofabout 2 ft. Within the tailings, the data covers the period of 9/02 to present. The data suggest a slight downward trend, or an apparent drain-down effect, and do not indicate any rise in head despite the fact that this area of the tailings is uncapped and under what can be accurately considered to be maximum infiltration exposure.

The above elevation head analysis does not indicate any increasing head conditions in either the tailings, or the bedrock immediately beneath the tailings. The following analysis is important and relevant to the geotechnical analysis examines the pressure head in relation to the foundation of the liner, within the liner system, and within the tailings material. Cross sections provided by Klohn-Crippen Ltd. (sections 3, 8, 10, 11; see map Attachment B for

locations) through the existing SE1, projected SE2 area, and East Expansion areas were used in conjunction with the piezometric measurements in the SE1 area and the East Expansion area to examine the head levels relative to the tailings pile and hydrologic controls within the tailings pile. The well and piezometer locations were superimposed on the cross sections and the maximum recorded water elevation plotted on the well/cross section. With regards to bedrock measurements, all bedrock water elevation measurements are below the bottom of the liner by 8 to 10 ft. Bedrock measurement points include MW-T-01-04, MW-T-01-06A, MW-T-01-06B, and MW-T-01-05. The low pressures beneath the liner is not unexpected since the bedrock in the SE1 and SE2 areas was quarried prior to tailings placement and in doing so, highly fractured by blasting and heavy equipment. The shallow bedrock beneath the tailings would be expected to have a hydraulic conductivity several orders of magnitude greater than the in-tact bedrock at depth. Therefore, artesian heads within the deep bedrock are dissipated within the fractured near surface bedrock of the quarried areas. This condition will remain so long as the fractured bedrock is daylighted to unconfined drainage.

With respect to head in the tailings, measurements were plotted for DH-T-05-20PZ, DH-T-05-15PZ, and PZ-T-00-03. DH-T-05-15PZ and DH-T-20PZ show near zero or slightly negative pressure head within the tailings. PZ-T-00-03 is in the southeast corner of the South tailings expansion, and outside the area of geo-synthetic lining. The area monitored by PZ-T-00-03 is lacking the presence of a continuous drainage layer such as a blanket drain or sand service layer. This monitoring point depicts a maximum expected head condition as it is relatively poorly drained and is uncapped and under the influence of maximum expected infiltration. The maximum measured head within the tailings at this point is approximately 13 ft. and is currently approximately 11 ft. The maximum head measured in the tailings at MW-T-02-05 is 3.65 ft. which was recorded just after completion. Current tailings head measured in this piezometer, which is located in an area with a continuous under-drain system, is 2.42 ft.

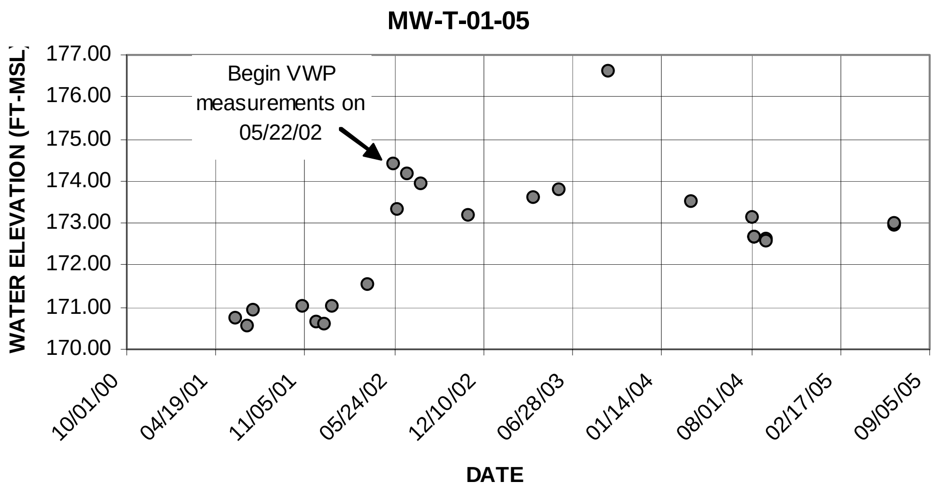
No measurements of pressure head or elevation head are available at this time for points within the liner system (ie: within the service layer or the geo-grid drainage layer). However, it is expected that due to hydraulic conductivity gradients, little or no pressure head accumulation would occur. Basic groundwater theory backs this contention. Hydraulic gradients are driven by a number of factors. Flow occurs from higher head to lower head. Flow or flux rate is a function of the head and the hydraulic conductivity. For any given cross section perpendicular to the flow, the volume of flow can be computed by Darcys law . The tailings pile whether capped or uncapped is a heterogeneous system due to distinct layers of materials that comprise the pile. From the top down these layers are 1) the cap (layered in itself), 2) tailings, 3)sand service layer, 4)geo-grid drainage layer 5) impermeable membrane layer. Each layer has a distinctively different hydraulic conductivity. The flow rate though the system will be controlled by the layer of lowest hydraulic conductivity. The formation of a saturated zone is dependent upon the position of the layer of lowest hydraulic conductivity within the system. The tailings system as it currently exists has a strongly downward gradient as driven by increasing hydraulic conductivity with depth up to the membrane layer. Inclusive of the cap, the system will go from a low conductivity in the cap, to relatively higher in the tailings, higher yet in the service layer and very high in the drainage layer. These relationships assure that infiltration waters will be transported away from/out of the tailings more rapidly than can be replenished, therefore the formation of a significant saturated layer within the tailing or the liner system is not expected.

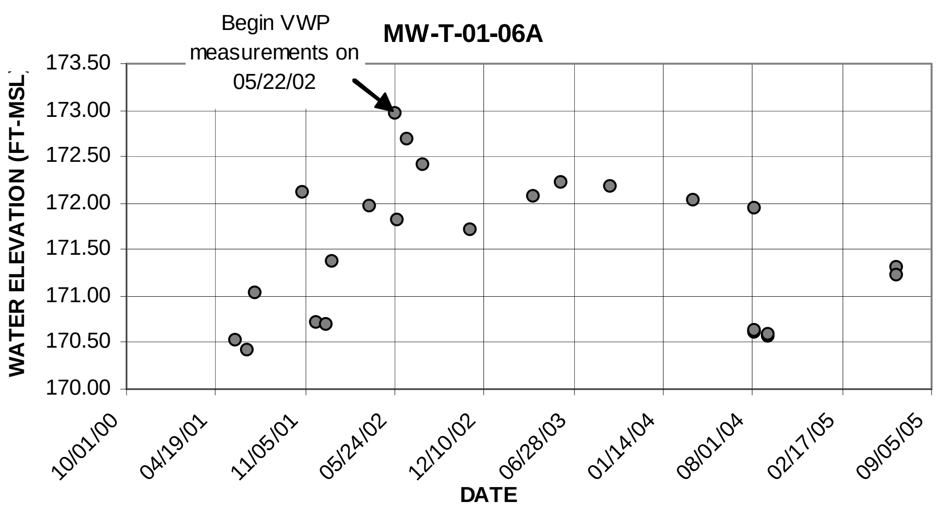
For purposes of design and modeling of the least favorable stability condition with respect to head within the tailings and within the liner system, the conditions within PZ-T-00-03 provides the most conservative analog with a tailings pressure of approximately 13 ft. The maximum expected head within the service or drainage layer of the liner system as reflected by MW-T-02-05 within the blanket drain of the East Expansion is approximately 4 ft. Use of these values in the modeling of the stability of the SE1 and SE2 areas would provide a maximum head case under uncapped conditions and presumes that there has been some compromise or alteration of the hydraulics of the under-drain system or within the tailings (ie; loss of permeability). More realistically, it is expected that due to hydraulic conductivity gradients between layers of the tailings design, zero or near zero head will accumulate in the tailings or in the liner system and beneath the liner system.

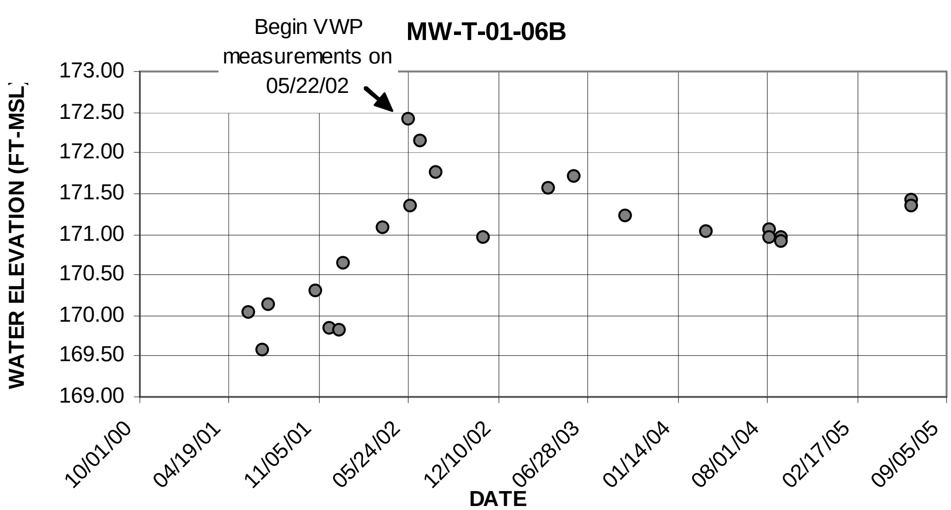
BNN/BNN

ATTACHMENT A – PIEZOMETER HYDROGRAPHS

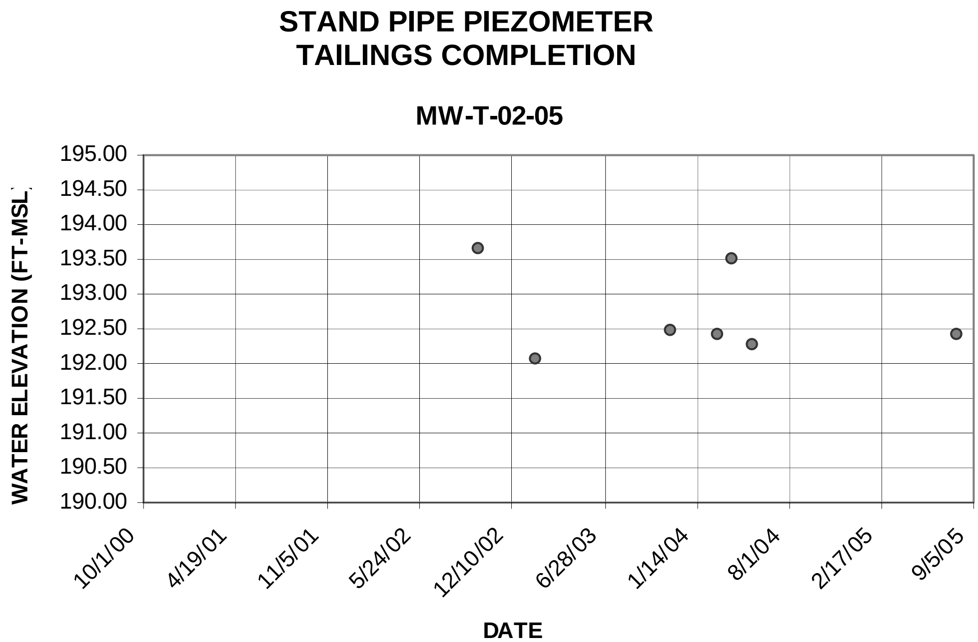
VIBRATING-WIRE PIEZOMETERS (FORMERLY MONITORING WELLS)
BEDROCK COMPLETION







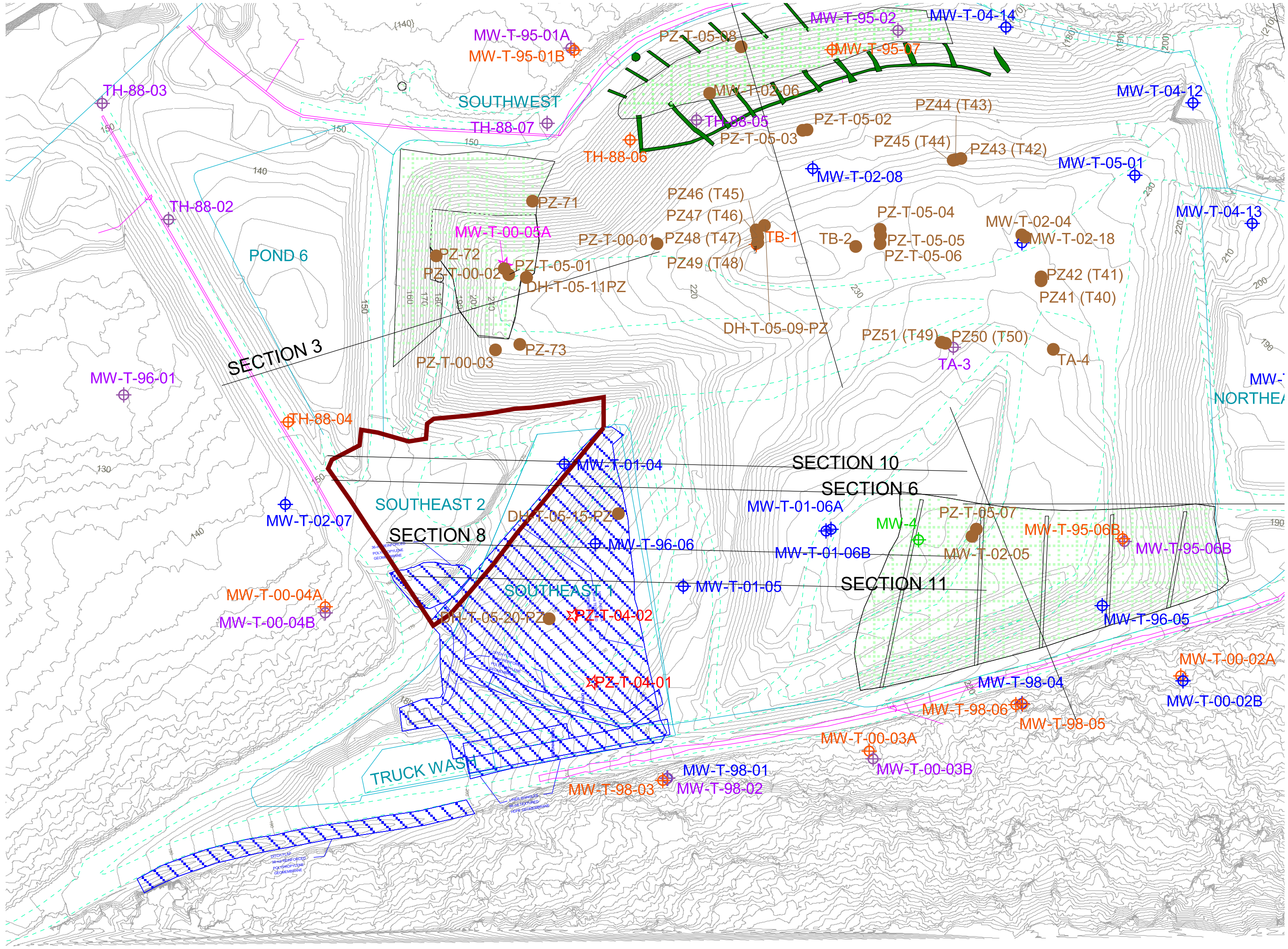
ATTACHMENT A – PIEZOMETER HYDROGRAPHS



ATTACHMENT B

SOUTHEAST 2 LINED AREA
SOUTHEAST 1 LINED AREA
SLURRY WALL
BLANKET DRAIN

TAILINGS
LINER FOUNDATION
PEAT
SAND
TILL
BEDROCK
UNDERDRAIN AND TAILINGS



SCALE = 1:3000

EDE CONSULTANTS – MEMO

DATE:	9/9/2005
TO:	BOB CHAMBERS, GEORGIA LYSAY; KLOHN CRIPPEN LTD.; TOM ZIMMER, ERIC SUNDBERG, PETE CONDON; KGCMC
FROM:	CHERYL NAUS; EDE
CC:	BRUCER NELSON, RUSS HAMILTON; EDE
RE:	PROJECTED PIEZOMETRIC HEADS IN THE TAILINGS SE2 AREA

Executive Summary

Based on review of maximum observed heads in tailings, bedrock, and till monitoring wells and piezometers in the tailings area, it is recommended that higher heads for tailings be used for stability sections 1, 2, parts of sections 3 and 4, and sections 5a and 5b for conservative modeling values. Monitored data for bedrock and till suggest that for conservative stability modeling, the lower piezometric surface on sections 1, 2, 4, and perhaps 5b should be increased in parts of the sections.

On stability sections 1 and 2, the upper piezometric surface (tailings) is everywhere at or below the base of the tailings pile, and historically monitored data at points near these sections suggest the use of between 6 and 20 feet of head in tailings for a conservative value. Portions of stability sections 3 and 4 depict a tailings water level commensurate with maximum observed heads, but data from monitoring points near the center of the tailings pile indicate that a more conservative stability analysis would result from use of higher water levels for tailings in this part of the pile. Stability sections 5a and 5b show about 5 feet of head in the tailings. Comparison to other areas, combined with consideration of the relatively steep hydraulic gradient in the underdrain planned for this area, indicates that a head value of less than 20 feet may be appropriate for this area. However, 5 feet of head in the tailings is lower than the lowest conservative estimate based on observed maximum water levels in existing tailings.

The lower piezometric surfaces (bedrock/till) depicted on sections 1 and 2 near the limited bedrock and till monitoring points adjacent to these sections are about 26 and 16 feet lower, respectively,

than the maximum observed head data. An increase in bedrock/till water levels in these sections corresponding to the observed maximum heads would be consistent with an increase in tailings heads in these sections. Similarly, limited maximum head observations at points near section 4 suggest an increase in head of between 5 to about 25 feet. depending on location in the section. The 140 foot elevation of the lower piezometric surface on the left side of section 5b may need to be increased substantially. This elevation is substantially lower than any bedrock heads measured in the tailings area, with the exception of the far western part of the tailings lease area, and it is not likely that excavation in preparation for tailings placement would decrease the pressure head by such a large magnitude.

Introduction

This memorandum describes a review conducted by EDE Consultants of assumed long-term design water levels depicted on stability sections 1 through 5b, provided for review by Klohn Crippen Consultants on August 10, 2005. Sections were reviewed in the context of historical heads in tailings, corresponding the upper piezometric surface on the stability sections, and in bedrock or till, corresponding to the lower piezometric surface on the stability sections. Updated monitor well and piezometer data provided by Greens Creek Mine (GCM) between August 4 and August 22, 2005 were used for the review when possible. Additional data for wells and piezometers that are not currently monitored were on file with EDE and were used when necessary (when updated data were not available from GCM). For those wells and piezometers for which GCM provided updated data, no graphs are provided with this document because memo recipients are in possession of graphs included in the updated files provided by GCM. Graphs are provided for wells and piezometers for which data on file at EDE were used.

The overall approach was to examine maximum water levels in tailings, bedrock, and till as the most conservative design water levels for long-term conditions. This reasoning is derived from review of data recorded in wells and piezometers for which the historical data include the response to placement and removal of an engineered tailings cover (piezometers in place prior to late 1999). These wells and piezometers include PZ-46 through PZ-49, PZ-51, PZ-51, PZ-71 through PZ-73, MW-B1, MW-B2, and MW-A3. Equilibrium values prior to cover placement are not available for these wells, preventing the comparison of pre-cover values to maximum values observed following cover removal. In all cases, however, the maximum heads observed following cover removal were greater than the maximum heads observed during the covered period, indicating that the use of maximum observed heads is a conservative estimate providing a generous factor of safety.

A second reason to use maximum recorded heads is that no data are available to examine the response to cover placement and removal in many parts of the tailings pile. Thus the observed, uncovered conditions in these parts of the pile are the most representative data available.
A third advantage of using maximum recorded heads rather than those observed during covered conditions is that heads in tailings will likely increase in response to placement of additional tailings. However, this increase in pressure head should be offset by the reduced infiltration induced by placement of an engineered cover such that the maximum recorded heads for uncovered

conditions may be a reasonable approximation of covered conditions. EDE suggests considering the use of equilibrium heads observed during covered periods only if the results of stability modeling using more conservative heads are not acceptable in terms of tailings pile stability. In that case, estimates of heads in and beneath tailings under covered conditions for the new pile geometry will be necessary.

For this review, water elevations were hand-plotted on hard copies of the cross sections provided by Klohn Crippen Consultants. Reproductions of the annotated cross sections are not provided with this memo because of the lack of time that would be required to draft the sections. However, a map of the tailings area that shows the locations of monitor wells and piezometers relative to the plan-view locations of the cross sections is provided (figure 1). Cross section locations were provided electronically by Klohn Crippen. Table 1 lists wells and piezometers that were used for review of the cross sections, and table 2 provides the maximum water elevations recorded at these wells and piezometers.

Stability Section 1

Monitoring points that can be used to evaluate the design water levels depicted on stability section 1 include PZ-T-05-07, MW-T-02-05, MW-T-95-06A, MW-T-05-02A/B, and MW-T-00-01A. The maximum recorded data for PZ-T-05-07 and MW-T-02-05 (both tailings completions) indicate heads in the tailings above the piezometric surface shown on stability section 1, perhaps warranting the use of more conservative design water levels. However, the history of well MW-T-02-05 has been complicated by the addition of tailings to the pile and corresponding changes in casing length, and it is possible that ground-water elevations measured in this well are in error because of use of an incorrect vertical datum. Additional uncertainty is introduced by the lack of a long-term period of record for recently installed piezometer PZ-T-05-07. Available data for PZ-T-05-07 indicate a ground-water elevation of approximately 200 feet, about 6 feet above the bottom of the pile at that location as indicated in section 1 (estimation of cross section elevations are very approximate due to the scale of the drawings).

The monitoring points used to evaluative the southern part of section 1 are underlain by a blanket and finger drain system, similar to West Buttress tailings monitoring points MW-T-02-06 and PZ-76. The hydrograph for piezometer PZ-76 exhibits intermittent saturation, varying from zero to near zero head to as much as 20 feet of head in the tailings. Water elevation data and the apparent base of the tailings pile (as indicated on stability section 4) indicate that head in the tailings at the location of well MW-T-02-06 has varied from about 12 to 14 feet. By analogy, these monitoring points, coupled with the maximum heads measured in PZ-T-05-07 and MW-T-02-05, indicate that it may be appropriate to use higher heads in tailings, perhaps 20 feet, for stability section 1.

Evaluation of heads in till and bedrock in stability section 1 is difficult given available data. Water-level data for till well MW-T-95-06A are not available after the East Expansion of 2000, so no information regarding water level response to tailings placement is available. The maximum water elevation in this well prior to tailings placement was approximately 206 feet (figure 2), or about 6 feet above the base of the tailings indicated on section 1. No bedrock wells are located near section 1; however bedrock well MW-T-96-05 is, like section 1, located in the East Expansion area. The maximum head measured in this well was 210 feet; about 25 feet higher than the lower piezometric surface shown on section 1 at the location at which well MW-T-96-05 would plot if projected westward onto the section. That measurement, however, may be inaccurate due to poor documentation of the history of extension of the casing to accommodate tailings placement. The well does have a history of data before and after tailings placement, and measurements seem to indicate a slight initial rise in head following tailings placement (approximately 2 feet using the most probable data), followed by an overall downward trend to pre-tails levels. This suggests that, by analogy, the head in MW-T-95-06A prior to tails placement may be an appropriate design level. For a conservative design estimate, the bedrock/till piezometric surface on section 1, which appears to be approximately 180 feet near well MW-T-95-06A could be increased to around 206 feet.

Recently installed monitoring wells MW-T-05-02 A and B, located near the northern end of stability section 1, document artesian conditions in the sand in this area. These wells are located outside the tailings placement footprint, and heads measured in these wells generally are in agreement with the upper piezometric surface shown on stability section 1. Well MW-T-00-01A (till completion), also located outside the tailings placement footprint, is artesian, indicating that the lower piezometric surface on the northern end of stability section 1 should perhaps be increased to land surface.

Stability Section 2

In addition to monitoring points PZ-T-05-07 and MW-T-02-05, useful monitoring points corresponding to the location of stability section 2 include MW-T-96-05, MW-T-98-04, and MW-T-98-05. The upper piezometric surface shown on section 2 is nearly everywhere at the base of, or below the base of, the tailings. A more conservative surface would reflect some head in the tailings. As previously mentioned, data collected from PZ-T-05-07, considered to be more reliable than MW-T-02-05, indicate a ground-water elevation of approximately 200 feet, about 6 to 8 feet above the bottom of the pile as indicated in section 2. No other tailings monitoring points are located adjacent to section 2.

Well MW-T-96-05, completed in bedrock, was also discussed previously. The maximum head of 210 feet measured in this well indicates a need for higher bedrock/till head values on this stability section. The next highest, and apparently more reasonable from examination of the hydrograph, head in this well is 206 feet. The lower piezometric surface appears to be approximately 190 feet on stability section 2 at this location. Based on available data (recognizing that data are limited), it is recommended that the bedrock/till potentiometric surface be increased by as much as 16 feet for conservative modeling purposes.

Wells MW-T-98-04 (bedrock completion) and MW-T-98-05 (till completion) are located outside of the tailings placement footprint. The maximum recorded water elevations in these wells are approximately 210 feet and 230 feet, respectively. These water elevations indicate the appropriate values are being used east of the slurry wall in stability section 2.

Stability Section 3

Nearly all monitoring points near stability section 3 (figure 1, table 1) are completed in tailings. Exceptions are well MW-T-02-08, a bedrock monitoring well, and wells MW-T-00-05A and MW-B1, which are completed in tailings and underdrain materials. The maximum recorded head in bedrock well MW-T-02-08 appears to be erroneous due to extension uncertainties. Normal water elevations are around 165 feet, indicating that the lower piezometric surface shown on the stability section is appropriate. The maximum recorded water elevation in MW-T-00-05A indicates about 4 feet of head above the monitoring point at approximately 138 feet (near the base of the tailings shown in stability section 3). In well MW-B1, the maximum elevation of about 157 feet (figure 3) is slightly below the piezometric surface depicting tailings head on the section, indicative of reasonable values for modeling in this area.

Maximum water elevations monitored in pneumatic piezometers PZ-46, PZ-47, PZ-48 (figure 4), PZ-49 (figure 5), and MW-B2 (figure 6) are approximately 178, 177, 187, 175, and 184 feet, respectively. The design piezometric surface, approximately 160 feet in this area, apparently reflects the heads measured by these piezometers while the cover was in place in 1997 (heads were approximately 158 to 162 feet in these piezometers), which may be appropriate for modeling purposes. However, for conservative design estimates, tailings heads in the northern part of the section could be increased to between 175 and 187 feet.

Maximum water levels for other monitoring points corresponding to stability section 3 (including piezometers PZ-71, PZ-72, and PZ-73; figure 7-9) generally correspond to the piezometric surfaces shown on the section at those locations or otherwise indicate suitable design water levels for conservative stability modeling. An exception is recently installed vibrating-wire piezometer DH-T-05-11c, in which measurements equate to a ground-water level of approximately 180 feet, substantially higher (about 30 feet) than the piezometric surfaces shown on stability section 3. It is likely that the historical data for nearby monitoring points, which indicate that the design piezometric surface for tailings corresponds to the maximum observed water elevations, justify the use of the surface shown on the section rather than a higher design level based on limited data from this piezometer.

Stability Section 4

The tailings piezometric surface on the portion of stability section 4 that is east of MW-T-02-08 corresponds to the part of the tailings pile previously discussed in review of stability section 3. As with section 3, heads measured in MW-B2, PZ-46, PZ-47, PZ-48, and PZ-49 when the pile was covered indicate that the design piezometric surface shown in the stability section is appropriate for covered tailings in this area, but higher heads may be warranted for a more conservative stability estimate. Additional monitoring points near stability section 4 are piezometers PZ-50 and PZ-51, further east and near the center of the tailings pile. In PZ-50, heads appear to have stabilized at around 194 feet following cover removal. The maximum value measured in PZ-51 was almost 190 feet. These values indicate that an increase of about 20 feet in the piezometric surface would provide a conservative water-level estimate for tailings in this part of the section. Design values near the toe of the pile as depicted in the stability section are adequate for conservative modeling, based on maximum observed heads in MW-T-02-06 and PZ-T-05-08.

The bedrock/till piezometric surface could be increased in the eastern part of the section as indicated by the maximum head of approximately 188 feet in well MW-A3 (a till completion) as compared to the lower piezometric surface of approximately 162 feet shown on the stability section. However, this maximum head was measured shortly after well installation, and water levels have declined since that time. The maximum head following cover removal in this area was almost 186 feet. Further west in the section, the bedrock/till piezometric surface could be represented by the apparently normal head of approximately 165 feet in bedrock well MW-T-02-08, which would be a more conservative modeling value than the value of approximately 160 feet shown on the section in that location. Maximum recorded heads in wells MW-T-01-03A (bedrock well) and MW-T-01-03B (till well) are about 125 and 132 feet, respectively. The lower piezometric surface of about 120 feet shown on stability section 4 could be adjusted upward 5 to 7 feet for a more conservative estimate. At the far west end of this section, the bedrock/till piezometric surface is more than adequate based on the maximum head in bedrock well MW-T-01-15A of approximately 34 feet.

West of well MW-T-02-08, vibrating wire piezometers PZ-T-05-02 and PZ-T-05-03 indicate that conservative tailings head values are shown on the stability section. The head measured in PZ-T-05-02 is below the tailings piezometric surface, and PZ-T-05-03, with an instrument tip elevation approximately equal to the tailings piezometric surface, has recorded heads very near zero. Similarly, the maximum water elevation of approximately 156 feet in MW-T-02-06 indicates that the design piezometric surface of approximately 160 is adequate. The maximum head in PZ-76 is slightly higher than the design piezometric surface. However, this piezometer historically has recorded intermittent saturation, varying from zero to near zero head to as much as 20 feet of head in the tailings, and the design piezometric surface is only about 2 feet below the maximum head recorded to date.

Piezometric surfaces representing head in the underdrain material are not shown on the stability sections; however, piezometers PZ-74 and PZ-75 provide an opportunity to evaluate heads in this material in the West Buttress expansion area. These are the only two piezometers completed entirely within the underdrain material. Both piezometers have recorded normal fluctuations of less than a foot around a baseline no more than 0.5 foot above the instrument elevation, and both instruments commonly record pressure heads of zero in the underdrain materials.

Stability Sections 5a and 5b

No data are available for direct comparison of historical tailings water elevations in the Northwest 1 and 2 expansion areas. By analogy to other areas, it may be appropriate to use a conservative value of between 6 and 20 feet of head in the tailings. Stability sections 5a and 5b show about 5 feet of head in the tailings. Drainage from these areas will, however, be enhanced by the relatively steep hydraulic gradient in the underdrain, which may warrant the use of a head estimate lower than 20 feet for conservative purposes.

By plotting and contouring the maximum head values in bedrock wells in the vicinity of stability sections 5a and 5b, an estimate of maximum heads along the sections was obtained. Along section 5a, the maximum head estimated in this manner varies from about 200 feet on the western end of the section to about 210 feet near DH-05-21 to about 197 feet near the eastern end of the section. These estimates agree quite well with the lower piezometric surface shown on section 5a. Along section 5b, the maximum estimated head varies from about 200 feet on the western end of the section (as plotted on figure 1) to about 215 to 220 feet near DH-05-21 to about 197 feet near the eastern end of the section. Comparison to stability section 5b is difficult because the section is shown to be over 700 feet long whereas the plan view line(figure 1) is about 510 feet in length. The lower piezometric surface is at an elevation of nearly 230 feet on the right side of stability section 5b, which seems to be more than adequate. The 140 foot elevation of the lower piezometric surface on the left side of the section, however, is substantially lower than any bedrock heads measured in the tailings area, with the exception of the far western part of the tailings lease area. Excavation in preparation for tailings placement may decrease the pressure head in the northwest expansion area, but a decrease of nearly 30 feet does not seem likely.

Table 1. Wells and Piezometers Used for Stabilty Section Reviev

Name	Other name(s)	Stability Section No.	Completion Zone	Instrument Type	Comment
MW-T-00-01A	MW-001A	1	TILL	MONITOR WELL	North of slurry wall, not in tailings placement area
MW-T-05-02A		1	SAND	MONITOR WELL	North of slurry wall, not in tailings placement area
MW-T-05-02B		1	SAND	MONITOR WELL	North of slurry wall, not in tailings placement area
MW-T-95-06A	MW95-6A	1	TILL	MONITOR WELL	Underlain by blanket and finger drains
MW-T-02-05	DH-02-05	1, 2	TAILINGS	MONITOR WELL	Underlain by blanket and finger drains
PZ-T-05-07	DH-T-05-04A, SL-T-05-09	1, 2	TAILINGS	VIBRATING WIRE PIEZO	Underlain by blanket and finger drains
MW-T-96-05	MW96-5	2	BEDROCK	MONITOR WELL	Underlain by blanket and finger drains
MW-T-98-04	MW98-04	2	BEDROCK	MONITOR WELL	East of slurry wall, not in tailings placement area
MW-T-98-05	MW98-05	2	BEDROCK	MONITOR WELL	East of slurry wall, not in tailings placement area
DH-T-05-11-PZ-A	DH-05-11-PZ	3	TAILINGS	VIBRATING WIRE PIEZO	Underlain by blanket (?) and finger drains
DH-T-05-11-PZ-B	DH-05-11-PZ	3	TAILINGS	VIBRATING WIRE PIEZO	Underlain by blanket (?) and finger drains
DH-T-05-11-PZ-C	DH-05-11-PZ	3	TAILINGS	VIBRATING WIRE PIEZO	Underlain by blanket (?) and finger drains
MW-T-00-05A	MW-005A	3	TAILINGS AND UNDERDRAIN	MONITOR WELL	Underlain by blanket and finger drains
PZ-71		3	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by blanket and finger drains
PZ-72		3	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by blanket and finger drains
PZ-73		3	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by finger drains
PZ-T-00-01	PZAT-1	3	TAILINGS	STANDPIPE PIEZOMETER	Underlain by finger drains
PZ-T-00-02	PZAT-2	3	TAILINGS	STANDPIPE PIEZOMETER	Underlain by blanket and finger drains
PZ-T-00-03	PZAT-3	3	TAILINGS	STANDPIPE PIEZOMETER	Underlain by finger drains
PZ-T-05-01	DH-T-05-01C, SL-T-05-02	3	TAILINGS	VIBRATING WIRE PIEZO	Underlain by blanket and finger drains
DH-T-05-09-PZ	DH-05-09-PZ	3, 4	TAILINGS	VIBRATING WIRE PIEZO	Underlain by finger drains
MW-B1	TB-1	3, 4	TAILINGS AND UNDERDRAIN	MONITOR WELL	Underlain by finger drains
MW-B2	TB-2	3, 4	TAILINGS	MONITOR WELL	Underlain by finger drains
MW-T-02-08	DH-02-08	3, 4	BEDROCK	MONITOR WELL	
PZ-46		3, 4	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by finger drains
PZ-47		3, 4	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by finger drains
PZ-48		3, 4	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by finger drains
PZ-49		3, 4	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by finger drains
MW-A3	TA-3	4	TILL	MONITOR WELL	Underlain by finger drains
MW-T-01-03A	DH-01-04	4	BEDROCK	MONITOR WELL	West of slurry wall, not in tailings placement area
MW-T-01-03B		4	TILL	MONITOR WELL	West of slurry wall, not in tailings placement area
MW-T-01-15A	DH-01-11	4	BEDROCK	MONITOR WELL	West of slurry wall, not in tailings placement area
MW-T-02-06	DH-02-06	4	TAILINGS	MONITOR WELL	Underlain by blanket and french drains

Table 1. Wells and Piezometers Used for Stabilty Section Review-concluded.					
Name	**Other name(s)**	**Stability Section No.**	**Completion Zone**	**Instrument Type**	**Comment**
PZ-50		4	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by finger drains
PZ-51		4	TAILINGS	PNEUMATIC PIEZOMETEF	Underlain by finger drains
PZ-74		4	UNDERDRAIN	PNEUMATIC PIEZOMETEF	Underlain by blanket and french drains
PZ-75		4	UNDERDRAIN	PNEUMATIC PIEZOMETEF	Underlain by blanket and french drains
PZ-76		4	TAILINGS AND UNDERDRAIN	PNEUMATIC PIEZOMETEF	Underlain by blanket and french drains
PZ-T-05-02	DH-T-05-02A, SL-T-05-03	4	TAILINGS	VIBRATING WIRE PIEZO	Underlain by finger drains
PZ-T-05-03	DH-T-05-02B, SL-T-05-04	4	TAILINGS	VIBRATING WIRE PIEZO	Underlain by finger drains
PZ-T-05-08	DH-T-05-05, SL-T-05-11	4	TAILINGS	VIBRATING WIRE PIEZO	Underlain by blanket and french drains
MW-T-01-07		5 A/B	BEDROCK	MONITOR WELL	not in tailings placement area
MW-T-01-08		5 A/B	BEDROCK	MONITOR WELL	not in tailings placement area
MW-T-01-09		5 A/B	BEDROCK	MONITOR WELL	not in tailings placement area
MW-T-04-12		5 A/B	BEDROCK	MONITOR WELL	
MW-T-04-13		5 A/B	BEDROCK	MONITOR WELL	
MW-T-04-14		5 A/B	BEDROCK	MONITOR WELL	
MW-T-05-04		5 A/B	BEDROCK	MONITOR WELL	not in tailings placement area
MW-T-05-05		5 A/B	BEDROCK	MONITOR WELL	not in tailings placement are:
MW-T-96-03	MW96-3	5 A/B	BEDROCK	MONITOR WELL	

Table 2. Maximum Water Elevations in Wells and Piezometers Used for Stabilty Section Review

Name	Stability Section No.	Completion Zone	Instrument elevation	Maximum Water Elevation	Date Measured
MW-T-00-01A	1	TILL		192.87	3/29/2005
MW-T-05-02A	1	SAND		188.57	8/2/2005
MW-T-05-02B	1	SAND		180.41	7/28/2005
MW-T-95-06A	1	TILL		206.48	10/23/1996
MW-T-02-05	1, 2	TAILINGS		196.99	3/30/2004
PZ-T-05-07	1, 2	TAILINGS	199.5	200.16	6/30/2005
MW-T-96-05	2	BEDROCK		210.25	12/19/2001
MW-T-98-04	2	BEDROCK		210.2	11/15/1999
MW-T-98-05	2	BEDROCK		229.35	3/12/1999
DH-T-05-11-PZ-A	3	TAILINGS	142.1	150.67	6/30/2005
DH-T-05-11-PZ-B	3	TAILINGS	164.3	171.6	6/1/2005
DH-T-05-11-PZ-C	3	TAILINGS	176.2	180.75	6/1/2005
MW-T-00-05A	3	TAILINGS AND UNDERDRAIN		141.97	4/17/2003
PZ-71	3	TAILINGS	144	153.05	7/26/1999
PZ-72	3	TAILINGS	138.43	155.04	7/26/1997
PZ-73	3	TAILINGS	141.4		
PZ-T-00-01	3	TAILINGS		155.01	4/15/2005
PZ-T-00-02	3	TAILINGS		152.68	1/30/2003
PZ-T-00-03	3	TAILINGS		155.54	10/29/2001
PZ-T-05-01	3	TAILINGS	195.11	195.12	6/30/2005
DH-T-05-09-PZ	3, 4	TAILINGS	185.19	188.65	6/1/2005
MW-B1	3, 4	TAILINGS AND UNDERDRAIN		156.53	10/14/1995
MW-B2	3, 4	TAILINGS		184.29	12/22/2000
MW-T-02-08	3, 4	BEDROCK		181.34	6/20/2003
PZ-46	3, 4	TAILINGS	140.9	177.8	3/31/2005
PZ-47	3, 4	TAILINGS	144.7	176.76	3/31/2005
PZ-48	3, 4	TAILINGS	154.9	187.3	5/11/2001
PZ-49	3, 4	TAILINGS	169.5	174.58	11/28/1997
MW-A3	4	TILL		188.2	10/25/1995
MW-T-01-03A	4	BEDROCK		125.19	9/23/2002
MW-T-01-03B	4	TILL		131.85	11/26/2003
MW-T-01-15A	4	BEDROCK		34.47	8/25/2003
MW-T-02-06	4	TAILINGS		156.5	2/24/2005
PZ-50	4	TAILINGS	164.9	212.24	3/29/2005
PZ-51	4	TAILINGS	176.9	189.82	9/19/1995
PZ-74	4	UNDERDRAIN	141.1	142.72	1/23/2001
PZ-75	4	UNDERDRAIN	141.3	142.04	1/18/2001
PZ-76	4	TAILINGS AND UNDERDRAIN	142.7	160.93	4/29/05
PZ-T-05-02	4	TAILINGS	152.3	156.89	6/1/2005

Table 2. Maximum Water Elevations in Wells and Piezometers Used for Stabilty Section Review-con					
Name	**Stability Section No.**	**Completion Zone**	**Instrument elevation**	**Maximum Water Elevation**	**Date Measured**
PZ-T-05-03	4	TAILINGS	174.6	174.77	4/26/2005
PZ-T-05-08	4	TAILINGS	157.4	157.85	6/30/2005
MW-T-01-07	5 A/B	BEDROCK		193.57	7/28/2005
MW-T-01-08	5 A/B	BEDROCK		193.36	2/2/2001
MW-T-01-09	5 A/B	BEDROCK		193.07	10/28/2003
MW-T-04-12	5 A/B	BEDROCK		197.56	3/12/2005
MW-T-04-13	5 A/B	BEDROCK		192.16	7/29/2005
MW-T-04-14	5 A/B	BEDROCK		168.01	2/24/2005
MW-T-05-04	5 A/B	BEDROCK		237.94	6/7/2005
MW-T-05-05	5 A/B	BEDROCK		231.96	7/28/2005
MW-T-96-03	5 A/B	BEDROCK		227.45	5/19/2005

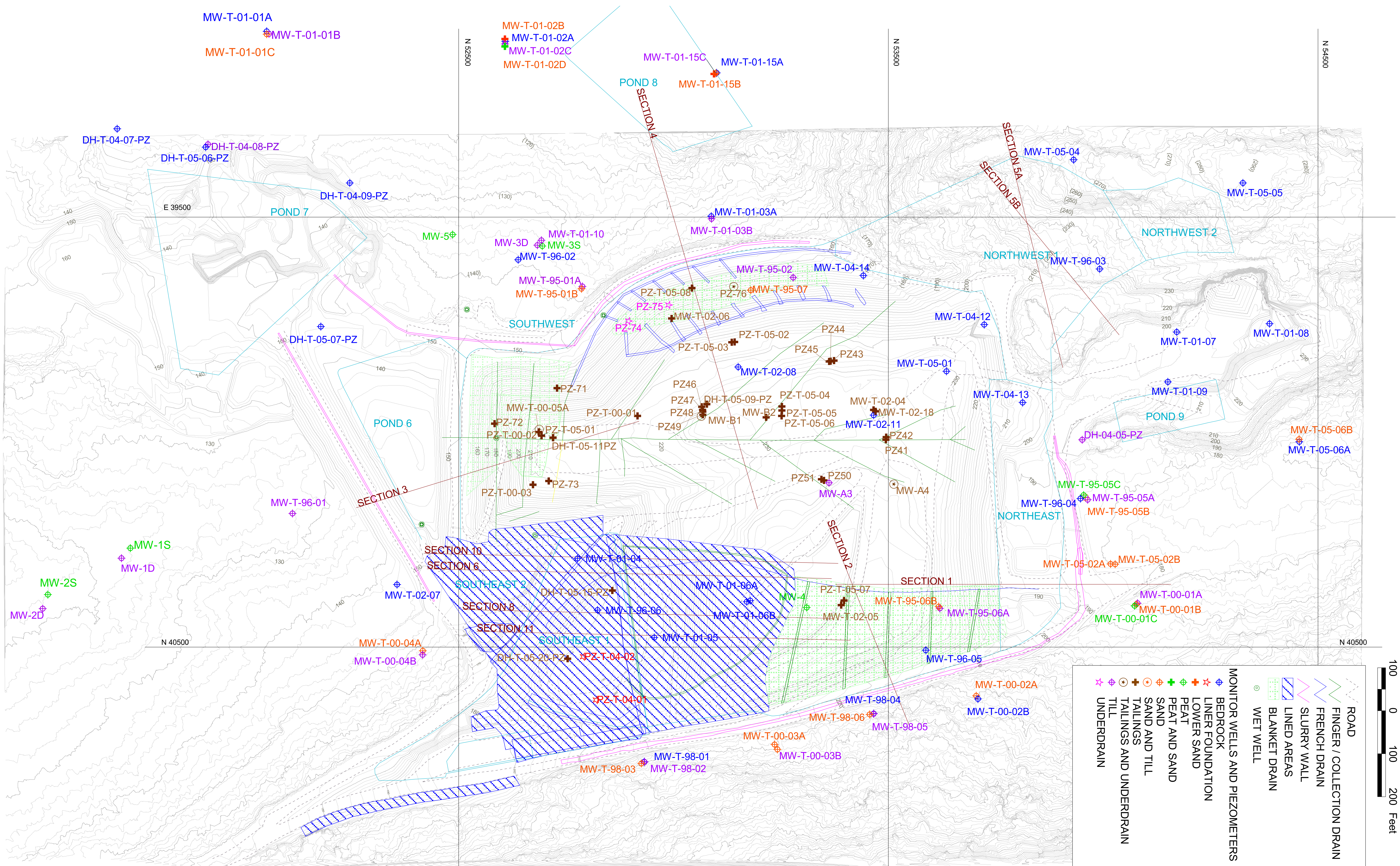


Figure 1. Locations of stability sections, monitor wells, and piezometers in the tailings area

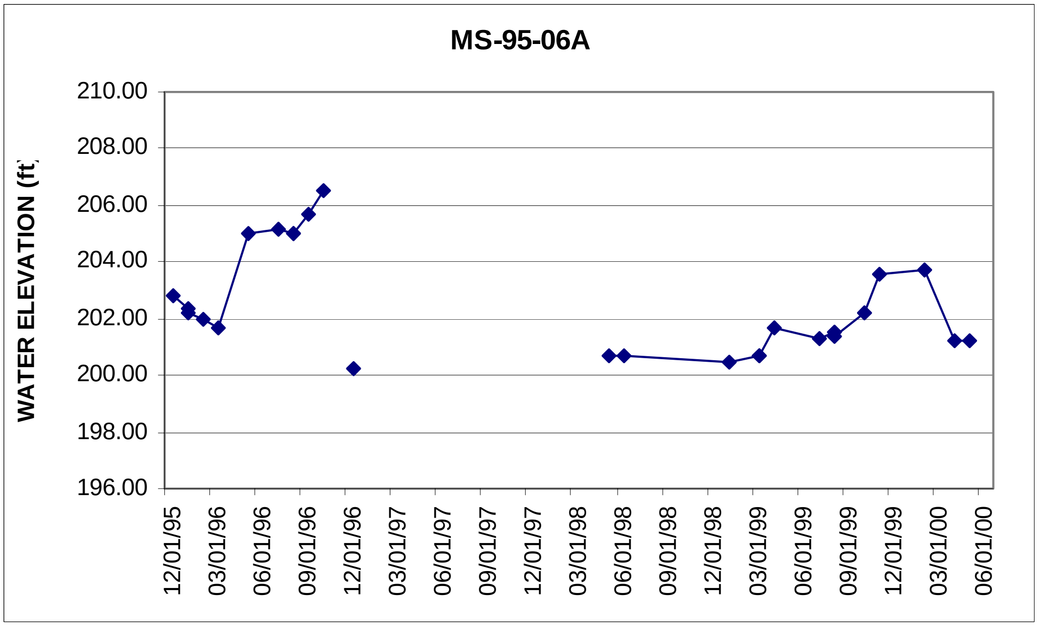


Figure 2

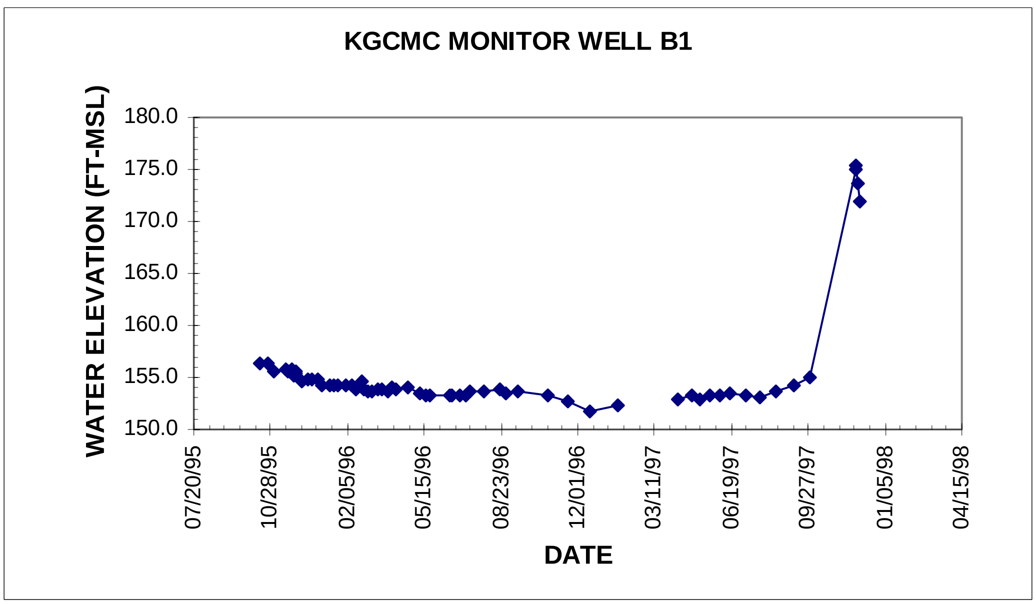


Figure 3

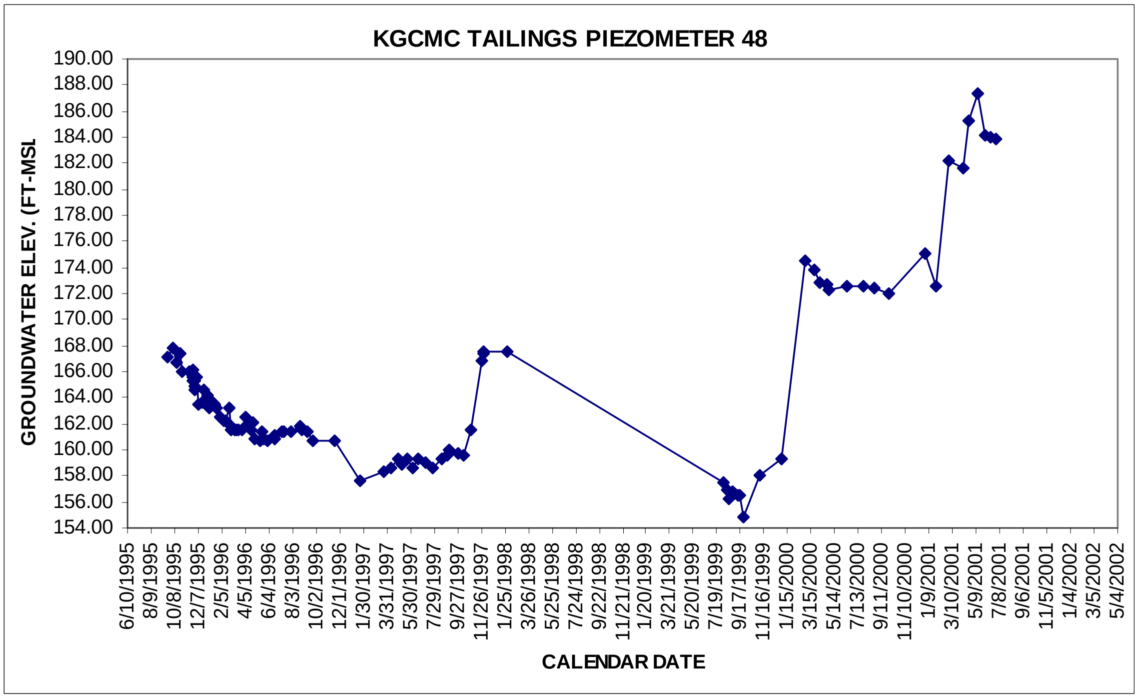


Figure 4

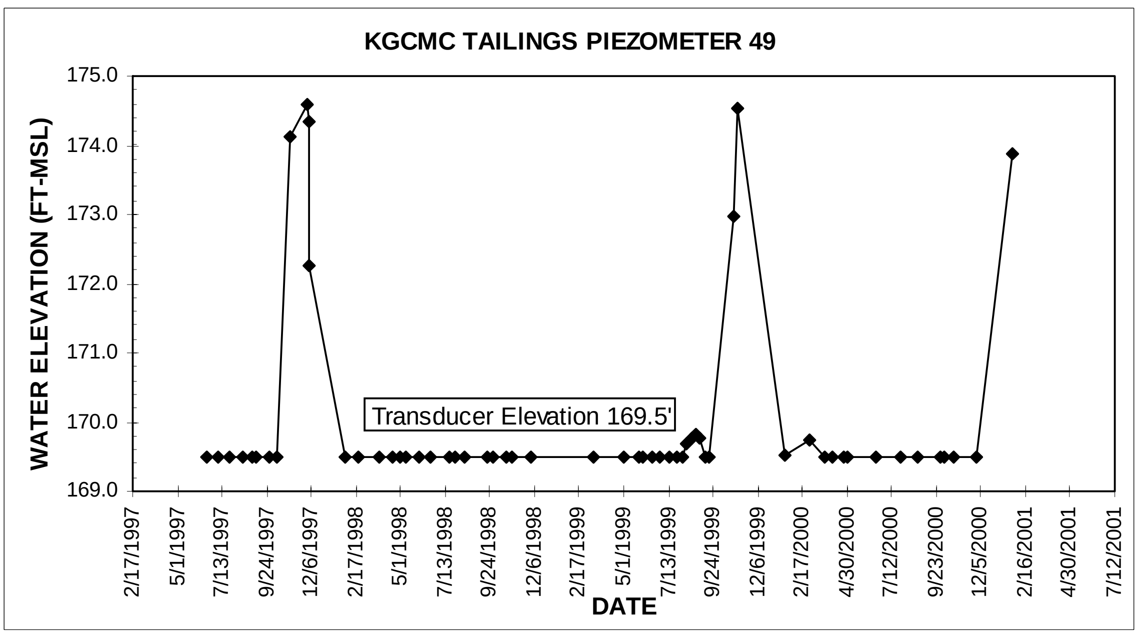


Figure 5

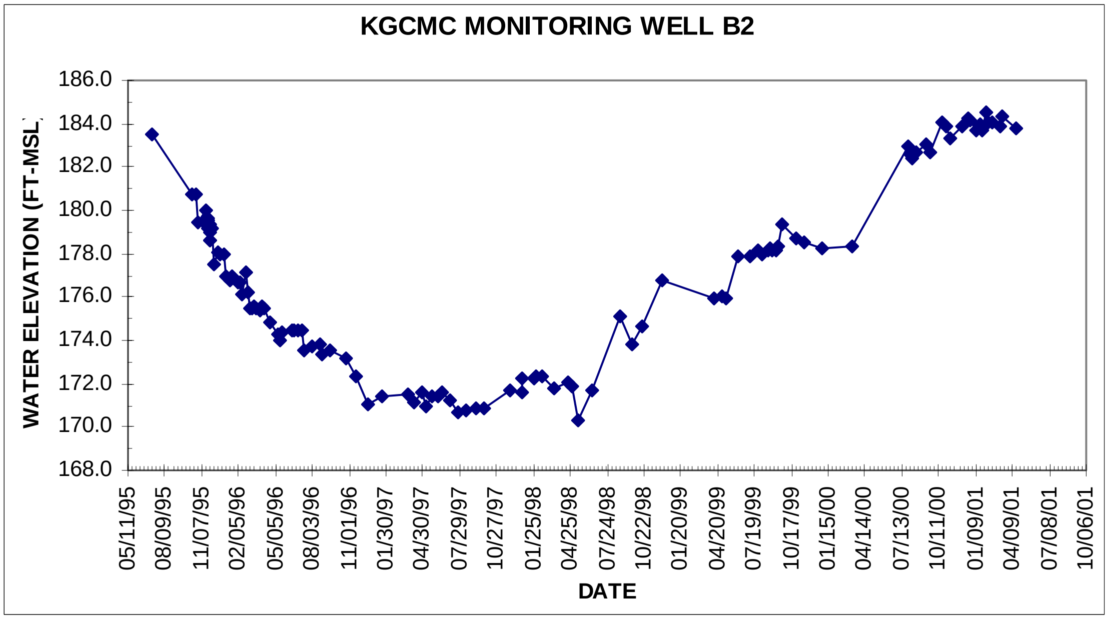


Figure 6

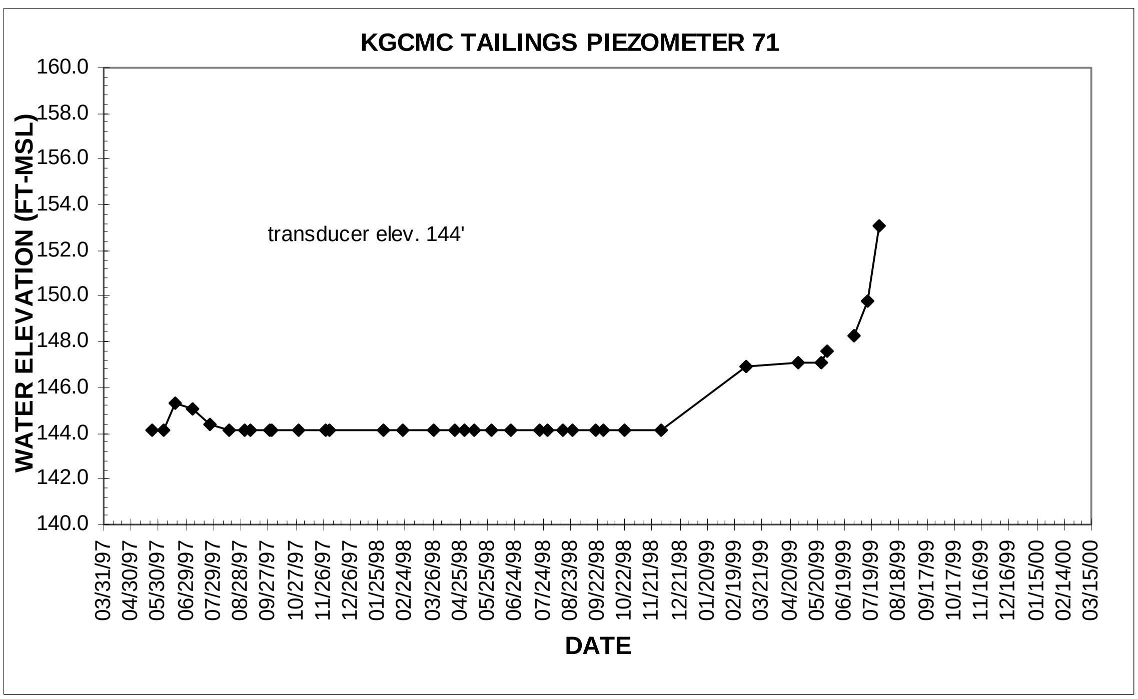


Figure 7

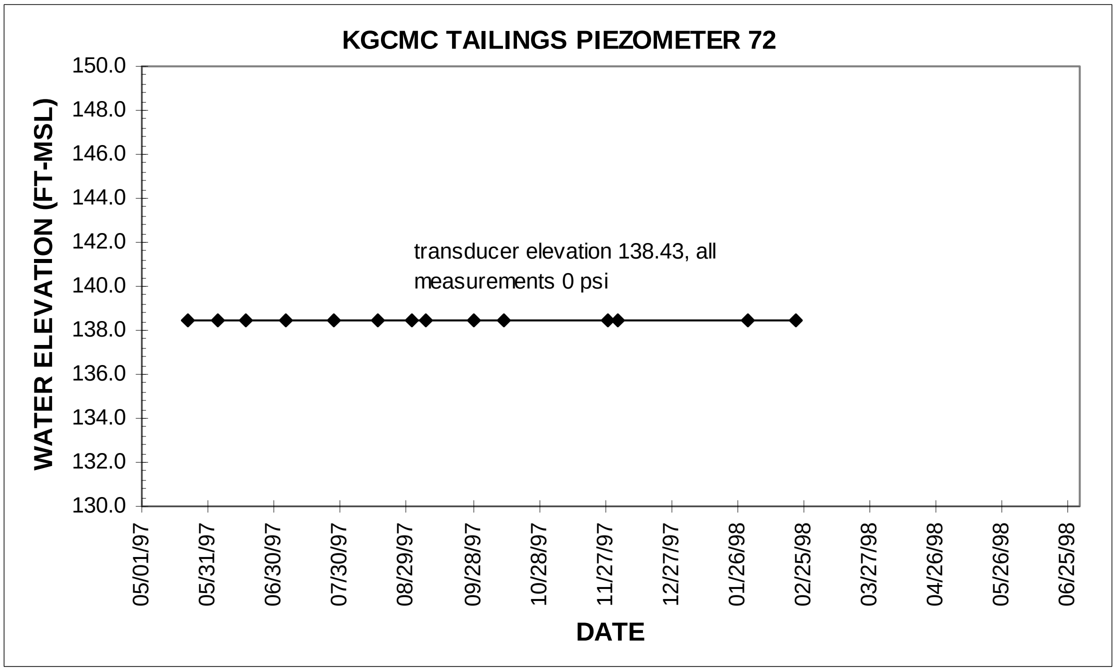


Figure 8

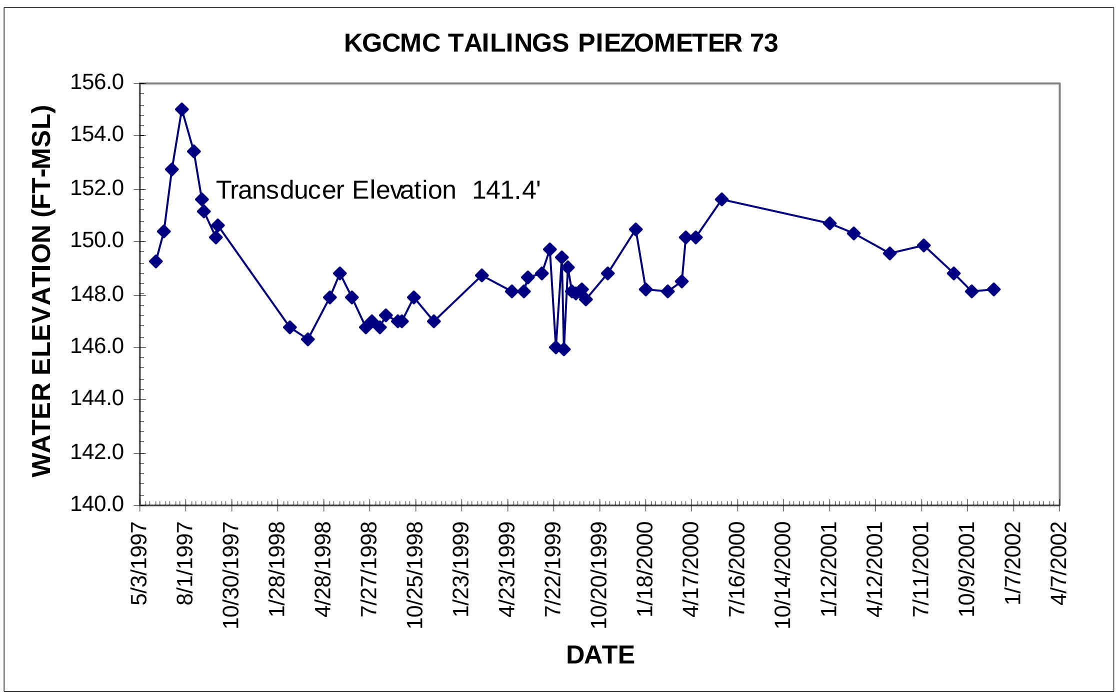


Figure 9

APPENDIX II

Cyclical Testing

APPENDIX II
Summary of Results for Tailings Cyclical Testing Programme

II-1. INTRODUCTION

At the request of Mr. Tom Zimmer of Kennecott Greens Creek Mining Company (KGCMC), Klohn Crippen has carried out a laboratory cyclical test programme on tailings samples from KGCMC's Tailings Storage Facility (TSF).

The objective of this assessment is to investigate whether cyclical loading due to the design earthquake(s) could initiate liquefaction of KGCMC tailings in conditions equivalent to those estimated near the bottom of the TSF at closure. Tailings at depth are more likely to be saturated, plus, for a given sample density, material at high stress is more susceptible to liquefaction.

This letter provides background comments, a description of the laboratory testing programme, and a summary of results and conclusions of the study.

II-2. BACKGROUND AND TESTING RATIONALE

II-2.1 Background

Previous geotechnical stability studies of the TSF have generally evaluated the liquefaction susceptibility through the use of empirical correlations to field data such as standard penetration tests and cone penetration tests (e.g. SRK, 1996; Klohn Crippen, 1999). The previous studies concluded that there are potentially liquefiable zones within the tailings pile and foundation soils. SRK (1996) carried out cyclical triaxial tests on Shelby Tube samples of tailings, and concluded that the tailings would experience a strength reduction due to cyclical softening during a M7.0 maximum design earthquake with a peak horizontal acceleration of 0.3 g. The SRK tailings samples were obtained by hand pushing a Shelby tube into the waste pile of an excavated test pit and, as such, likely were less compacted than normal tailings in the pile. Therefore, the results have not been used in this assessment. The test results are in Attachment 4.

After a review of the previous geotechnical stability analyses, Klohn Crippen (2002) recommended additional laboratory testing to evaluate the behaviour of the tailings under seismic loading, and to confirm the results of the previous studies.

II-2.2 Testing Rationale

The liquefaction potential of a soil may be evaluated in terms of the amplitude and number of shear stress loading cycles. The loading conditions that are required to trigger liquefaction can be described in terms of cyclical shear stress, and can be expressed in terms of the cyclical stress ratio (CSR). CSR is the ratio of the resolved cyclical shear stress on a horizontal plane to the initial effective vertical stress on that plane (in this case, it is assumed that the shear stress is induced by an earthquake). By definition, CSR varies with depth and pore pressure (among other factors) for a given cyclical shear stress input.

The ultimate (closure) crest of the TSF under KGCMC's current permit is at El. 330 ft.[1], and the original ground surface under the crest is at about mean El. 160 ft. EDE Consultants Ltd. (EDE) assessed the hydrology of the TSF and concluded that with a low-permeability permanent cover, the post-closure water table position under the ultimate crest of the TSF will be at about El. 175 ft to 180 ft (EDE, 2001).

At El. 160 ft (a depth of about 170 ft) in the TSF, the effective vertical stress is estimated to be 130 psi (900 kPa) based on the laboratory index properties of the tailings reported in Klohn Crippen (2003), and assuming that the tailings are compacted to 90% of standard Proctor maximum dry density.

The recommended seismic design criteria and associated ground motion parameters for Greens Creek Mine were assessed in Klohn Crippen (1998), and are summarized in Table 2.1. Using an empirical method presented in Youd et al. (2001), the field CSR for each design earthquake was calculated at 170 ft depth under the expected post-closure conditions (Table 2.1).

Table 2.1 Recommended Seismic Parameters for Greens Creek Mine

DESIGN CRITERIA	PEAK HORIZONTAL GROUND ACCELERATION (g)	REPRESENTATIVE EARTHQUAKE MAGNITUDE	CALCULATED FIELD CSR
Design Basis Earthquake 1 Crustal (1/475 year)	0.15	M6.5	0.06
Design Basis Earthquake 2 Fairweather Fault (1/130 year)	0.08	M8.0	0.03
Maximum Design Earthquake Crustal (1/10,000 year)	0.30	M7.0	0.12

[1] All elevations referenced in this report are the height above mean sea level (amsl).

II-3. TESTING METHODOLOGY

II-3.1 Tailings Description

Bulk surficial tailings samples (disturbed) of about 20 kg were collected by KGCMC staff in October, 2002 and shipped to Klohn Crippen's laboratory in Vancouver, British Columbia. Two samples were collected from the TSF: one from the west buttress area, and one from an operationally-active portion of the southeastern area. As shown in Table 3.1, the index properties of these two samples are similar (Klohn Crippen, 2003). The laboratory data sheets for the index tests are included in Attachment 1.

Table 3.1 Summary of Index Properties

INDEX PROPERTY	WEST BUTTRESS	SOUTHEAST ACTIVE AREA
Gradation (sand % / fines %)	16.5 / 83.5	17.0 / 83.0
Atterberg Limits (W_L / W_P)	21 / 17	20 / 15
Specific Gravity (average)	3.44	3.37
Standard Proctor Maximum Dry Density (kg/m^3)	2085	2090
Standard Proctor Optimum Moisture Content	12.1	12.8

The tailings material from the west buttress area was selected for the cyclical tests. Separate tailings specimens were taken from the bulk sample for each cyclical test; no tailings were re-used in subsequent tests.

II-3.2 Triaxial Test Specimen Preparation and Testing Procedure

The cyclical triaxial test specimens were moist-tamped into a triaxial mold at a density equivalent to about 90% (CTX-01, CTX-02 and CTX-03) or 95% (CTX-04 and CTX-05) of maximum Standard Proctor dry density at about the optimum moisture content (12.1%). Carbon dioxide gas was added to the tailings during moist tamping to increase pore saturation in the specimen during the consolidation phase of the test.

Each specimen was water saturated and isotropically consolidated at 900 kPa (1,000 kPa cell pressure with a 100 kPa back pressure) for about 24 hours in a triaxial test frame. After consolidation, the test cell drain was closed to maintain cell pressure, and the cell was transferred to a cyclical testing frame.

Each specimen was cyclically loaded at a frequency of 0.1 Hz at a pre-determined CSR value. The test CSR values were selected in a range that spans the calculated field CSR values for each of the design earthquakes (Table 2.1) to define a laboratory liquefaction curve for the tailings sample. The liquefaction criteria that were used for the tests are:

- A pore pressure increase in the specimen equal to the effective isotropic cell pressure (900 kPa); *or*,

- Cyclical strain of 5% single-amplitude (½ cycle) or 10% double-amplitude (1 cycle).

If a specimen did not meet the liquefaction criteria in 100 loading cycles, the test was halted and the CSR was increased by about 0.05. The specimen was then reloaded at the increased CSR for a further 100 cycles or until a liquefaction criteria was achieved. Only one specimen (CTX-01) required multiple loading phases.

After each specimen had met the liquefaction criteria, the cell was transferred back into a normal triaxial frame, and the specimen was monotonically loaded (one specimen in compression, four specimens in extension) in an undrained condition to determine the post-liquefaction large-strain shear strength of the soil (commonly called *residual strength*) (see Photos 1 and 2, Attachment 3).

II-3.3 Simple Shear Test Specimen Preparation and Testing Procedure

The simple shear test specimen (CSS-01) was moist-tamped into a shear box mold at a density equivalent to about 88% of maximum Standard Proctor dry density at a moisture content of 16.3%. The specimen was monotonically consolidated at 900 kPa for over 12 hours in the shear box.

After consolidation, the specimen was cyclically loaded in an undrained state at a frequency of 0.1 Hz and a CSR of 0.17. The liquefaction criteria that were used for the test are:

- A pore pressure increase in the specimen equal to the effective cell pressure (900 kPa); *or*,

- Cyclical strain of 3.75% single-amplitude (½ cycle).

The specimen met one of these criteria in less than 100 cycles; no incremental loading phases were required. After the cyclical loading phase, the specimen was monotonically loaded in an undrained condition at a strain rate of 10% per hour to determine the post-liquefaction large-strain shear strength of the soil (*residual strength*).

II-4. TEST RESULTS AND DISCUSSION

Five cyclical triaxial tests were carried out at Klohn Crippen's laboratory in Vancouver, British Columbia between April 24 and October 3, 2003. One cyclical simple shear test was carried out at MEG Technical Services' laboratory in Richmond, British Columbia on May 20-21, 2005.

The results from the laboratory testing programme are summarized below. The laboratory data for the cyclical tests (cyclical stress ratio, pore pressure ratio and axial strain) and for the monotonic tests (shear stress and pore pressure ratio) are attached in Attachment 2.

II-4.1 Cyclical Test Results

The loading sequence and test results for each of the cyclical test specimens is given in Table 4.1. These test results are plotted on Figure 4.1 as laboratory liquefaction curves.

Table 4.1 Summary of Cyclical Triaxial and Cyclical Simple Shear Test Results

TEST NO.	FINAL DENSITY (kg/m^3)	DATA TYPE	TEST STAGE 1	TEST STAGE 2	TEST STAGE 3	MAXIMUM STRAIN (%)
CTX-01	2075	CSR	0.05	0.10	0.15	-5.6
		No. Cycles	100	100	57	
CTX-02	2062	CSR	0.21	-	-	-9.1
		No. Cycles	11	-	-	
CTX-03	2058	CSR	0.25	-	-	-14.1
		No. Cycles	3	-	-	
CTX-04	2104	CSR	0.195	-	-	-5.1
		No. Cycles	23	-	-	
CTX-05	2091	CSR	0.235	-	-	-7.5
		No. Cycles	7	-	-	
CSS-01	2071	CSR	0.17	-	-	-4.5
		No. Cycles	21	-	-	

The laboratory CSR's must be adjusted to equivalent field conditions for comparison with calculated CSR's for the design earthquakes. Seed, et al. (1975a, in Kramer, 1996) suggest that the CSR required to initiate liquefaction in the field is about 10% less than that required in cyclical simple shear:

$$CSR_{field} = \tau_{cyc} / \sigma'_{Vo} = 0.9 \times CSR_{simple\ shear} = 0.9 \times c_r \times CSR_{triaxial} \quad (1)$$

where a correction factor, c_r, is required to account for the difference between simple shear and triaxial loading conditions (viz., $CSR_{simple\ shear} = c_r \times CSR_{triaxial}$);

$c_r = (1+K_o)/2$ (Finn et al., 1971) (2)

$c_r = 2\times(1+2K_o)/3\sqrt{3}$ (Castro, 1975) (3)

$c_r = 0.57$ (Idriss, pers.comm., 2003) (4)

and the coefficient of earth pressure at rest, K_o, is given by;

$K_o = 1 - \sin(\phi')$ (5)

Using a conservative value of 38° for the effective friction angle of the tailings, ϕ', the mean field-adjusted CSR values for the triaxial data are about 60% of the laboratory CSR values. The field-adjusted liquefaction curves are shown in Figure 4.1. The field adjusted CSR value for the simple shear data is 90% of the laboratory CSR value.

The calculated field CSR's for the recommended DBE and MDE design earthquakes (Table 2.1) plot below the field-adjusted liquefaction curves shown on Figure 4.1. The equivalent number of loading cycles for each of the design earthquakes was taken from an empirical relationship proposed by Seed et al. (1975b). Theoretically, if the CSR for a design earthquake plots below the field-adjusted liquefaction curves, then the design earthquake is considered insufficient to trigger liquefaction. These results are discussed further in Section II-5 of this report.

II-4.2 Monotonic Test Results

The monotonic loading test results are summarized in Table 4.2. The residual friction angle given in Table 4.2 is based on the total stress state after liquefaction.

Table 4.2 Summary of Monotonic Triaxial Test Results

TEST NO.	LOADING STATE	PEAK STRAIN (%)	UNDRAINED STRENGTH, S_u (kPa)
CTX-01	Compression	+11	265
CTX-02	Extension	-17	110
CTX-03	Extension	-17	130
CTX-04	Extension	-11.5	155
CTX-05	Extension	-18.9	135
CSS-01	Unidirectional Shear	18	106

The post-liquefaction undrained strength in the triaxial specimens ranged from 110 kPa to 155 kPa when tested in extension, and was 265 kPa in compression. The post-liquefaction undrained strength in the simple shear specimen was estimated at 106 kPa.

II-5. CONCLUSIONS AND RECOMMENDATIONS

Cyclical triaxial tests were carried out on tailings from the west buttress area of the Greens Creek TSF to evaluate their behaviour under cyclical shear loading. The triaxial test results were adjusted to their theoretical field-equivalent and plotted as liquefaction curves (Figure 4.1).

A single cyclical simple shear test was carried out as a check on the laboratory and adjusted cyclical triaxial curves. The field-adjusted simple shear data point (Figure 4.1) plots well above the equivalent portion of the triaxial data curves. In our judgement, this difference indicates that the triaxial adjustment factor, c_r (equations 2, 3 and 4, in Section II-4), only partially accounts for differences between the triaxial and simple shear tests. Therefore, the field-adjusted triaxial liquefaction curves are considered conservative.

A field-equivalent cyclical stress ratio (CSR) was calculated for each of the Greens Creek Mine design earthquakes (Table 2.1). These data were compared to the field-adjusted liquefaction curves to assess the potential for the initiation of liquefaction in tailings near the base of the Greens Creek TSF, as follows:

- The calculated CSR's for the adopted design basis earthquakes (DBE's) are less than one-half the calculated CSR required to initiate liquefaction (as defined by the field-adjusted liquefaction curves). With the tailings compacted to 90% of standard Proctor maximum dry density, the tests indicate that liquefaction will not be initiated by a DBE event.

- The calculated CSR for the adopted maximum design earthquake (MDE) is less than, but close to, the field-adjusted liquefaction curves. In our judgement, because the position(s) of the liquefaction curves are considered conservative, the tests indicate that liquefaction of the new tailings will not be initiated by an MDE event.

On this basis, the post-earthquake stability of the TSF for the Stage 2 final design should be evaluated on the basis of the calculated pseudo-static factor of safety. Some softening of the tailings strength may occur and post earthquake strength will vary between peak strength and a conservative value of 110 kPa for the undrained (residual) strength of the tailings at the base of the pile. Tailings undrained strength will likely reduce at lower confining stress and the use of residual values proposed by Seed and Harder (1990) is recommended for use at shallow depth.

The preceding conclusions and recommendations are based on five cyclical triaxial tests and one cyclical simple shear test carried out on tailings from one bulk sample. However, the tailings index testing shows considerable consistency of the new tailings product over time and therefore we believe these cyclic tests are representative of the new tailings in the TSF.

REFERENCES

Castro, G. (1975). *Liquefaction and Cyclic Mobility of Sands.* Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT6, pp. 257-276.

EDE Consultants Ltd. (2001). *Kennecott Greens Creek Mining Company, Stage II Tailings Expansion, Hydrologic Analysis.* Engineering report prepared for KGCMC. 61 pages; attachments.

Finn, W.D.L., Pickering, D.J. & P.L. Bransby (1971). *Sand Liquefaction in Triaxial and Simple Shear Tests.* Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM4, pp. 639-659.

Klohn Crippen (2003). *Kennecott Greens Creek Mining Company, 2002 Geotechnical Investigation – Field Report*. Engineering report prepared for KGCMC. 16 pages; attachments.

Klohn Crippen (2002). *Existing Tailings Facility Stability*. Review memorandum prepared for KGCMC. 6 pages; attachment.

Klohn Crippen (1999). *Kennecott Greens Creek Mine, Evaluation of Tailings Pile*. Engineering report prepared for KGCMC.

Klohn Crippen (1998). *Kennecott Greens Creek Mine, Seismic Hazard Assessment – Final Report*. Engineering report prepared for KGCMC. 29 pages; attachments.

Kramer, S.L. (1996). *Geotechnical Earthquake Engineering.* Prentice Hall International Series in Civil Engineering and Engineering Mechanics; *William J. Hall, Editor*. 653 pages.

Seed, H.B., Lee, K.L., Idriss, I.M., & R. Makdisi (1975a). *The Slides in the San Fernando Dams During the Earthquake of Feb. 9, 1971*. Journal of Geotechnical Engineering Division, ASCE, Vol. 101, No. GT7, pp. 651-688.

Seed, H.B., Idriss, I.M., Makdisi, F. & N. Banerjee (1975b). *Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analyses.* EERC Report No. 75-29 (National Science Foundation). 13 pages; figures.

Seed, H.B. and Harder, L.F. (1990). *SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength*. Proc. H. Bolton Seed Memorial Symposium, J. Michael Duncan, Ed. Vol.2, pp. 351-376.

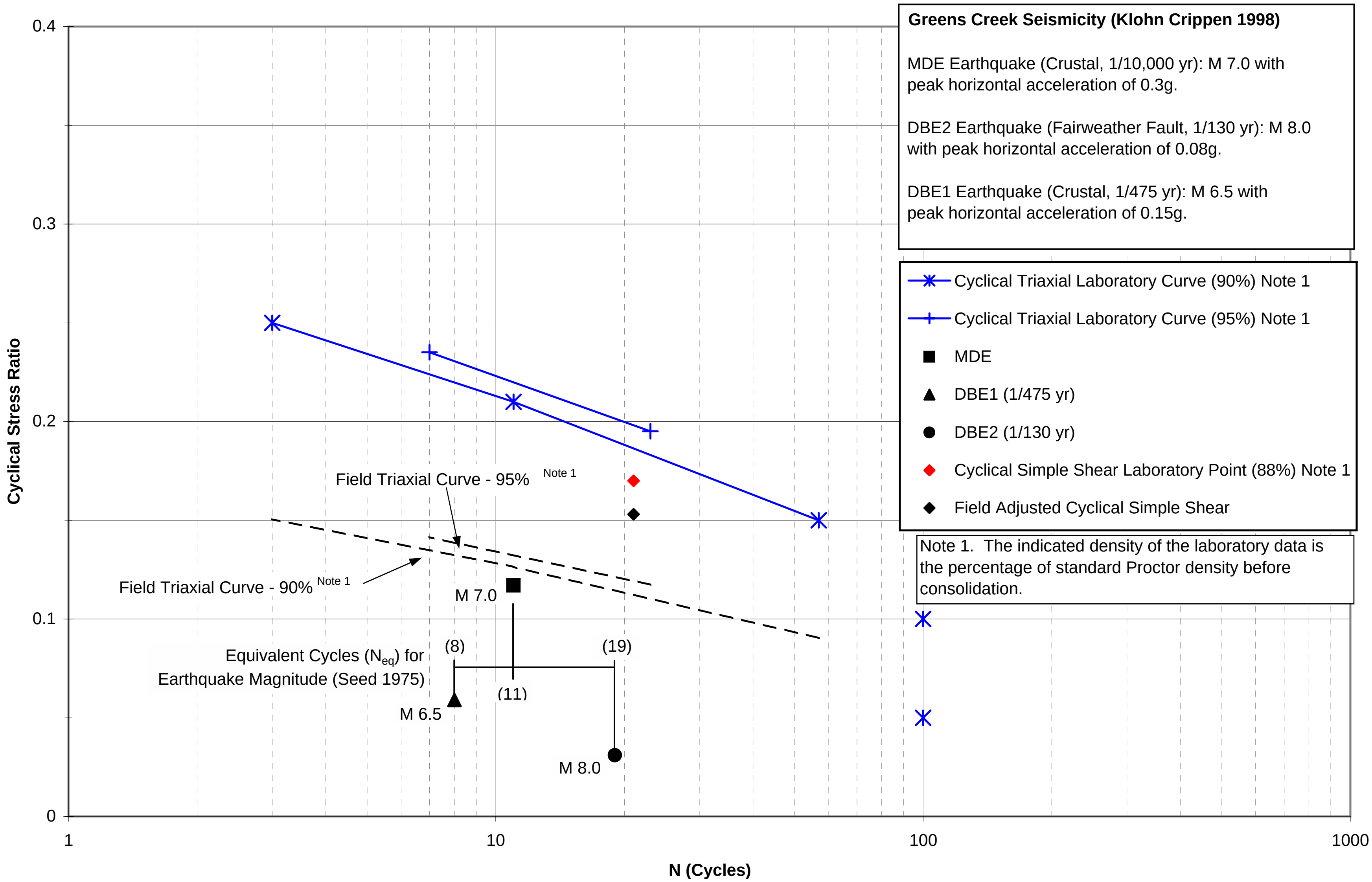
Steffen Robertson & Kirsten (1996). *Addendum to Report No. G106113, Geotechnical review and Analysis of the Greens Creek Tailings Pile.* Engineering report prepared for KGCMC. 19 pages; attachments.

Youd, T.L., I.M. Idriss, R. Andrus, I. Arango, G. Castro, J. Christian, R. Dobry, L. Finn, L. Harder Jr., M.E. Hynes, K. Ishihara, J. Koester, S. Liao, W. Marcuson III, G. Martin, J. Mitchell, Y. Moriwaki, M. Power, P.K. Robertson, R.B. Seed & K. Stokoe II (2001). *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils.* Journal of Geotechnical and Geoenvironmental Engineering. October, 2001. 17 pages.

Figures

Figure 4.1 Cyclical Testing Test Results – West Buttress Tailings

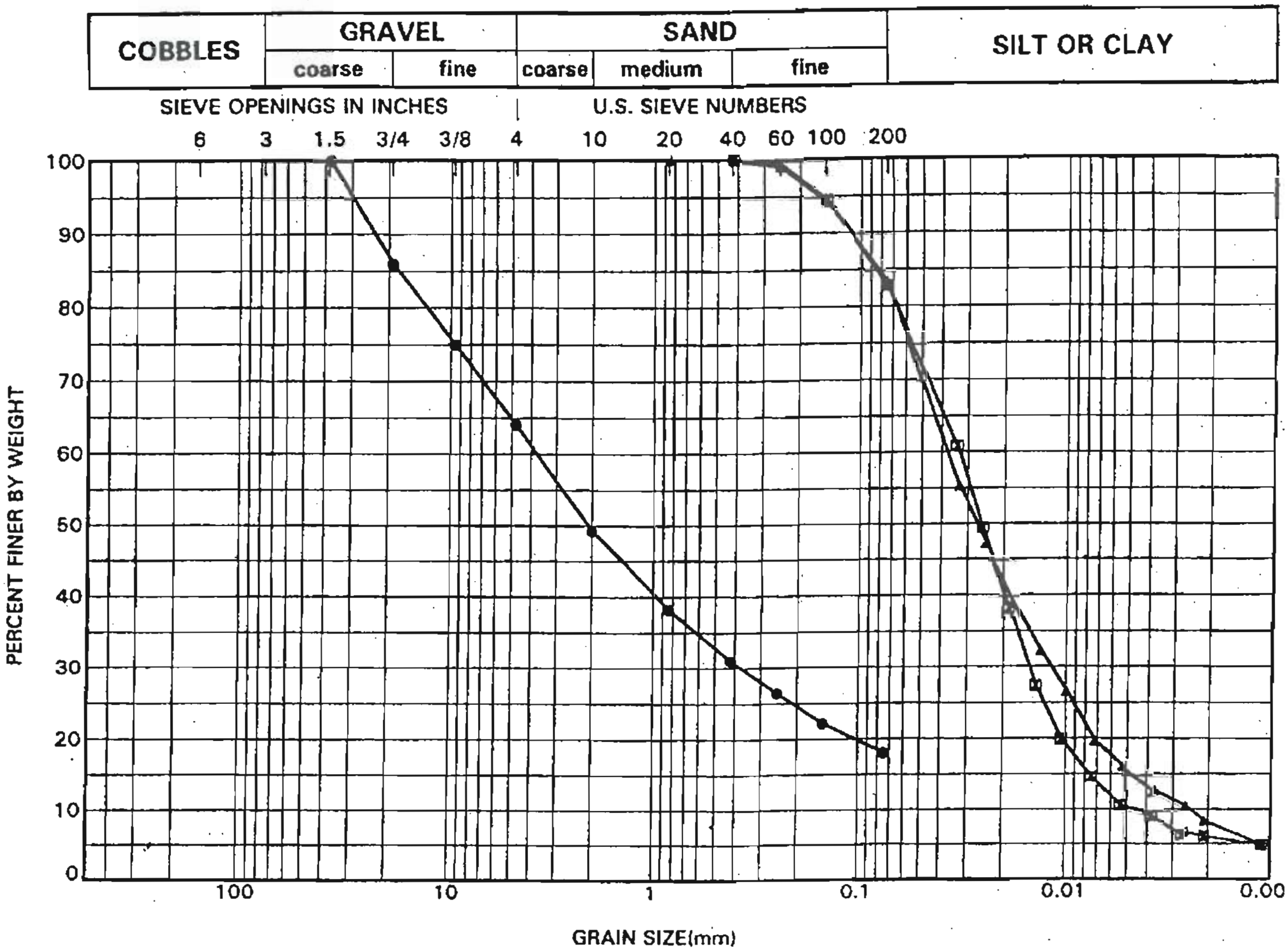
Figure 4.1
Cyclical Testing Test Results - West Buttress Tailings



Attachment 1

Laboratory Data
Tailings Index Property Tests

GRAIN SIZE DISTRIBUTION



	HOLE/SAMPLE	DEPTH(ft)	D85	D60	D50	D15	D10	CU	%GRAVEL	%SAND	%FINES
●	DH02-17	43.75	18.040	3.766	2.096				36.0	46.0	18.0
⊠	EASTTAIL	0.00	0.084	0.033	0.026	0.008	0.005		0.0	17.0	83.0
▲	WESTTAIL	0.00	0.082	0.038	0.027	0.005	0.003		0.0	16.5	83.5

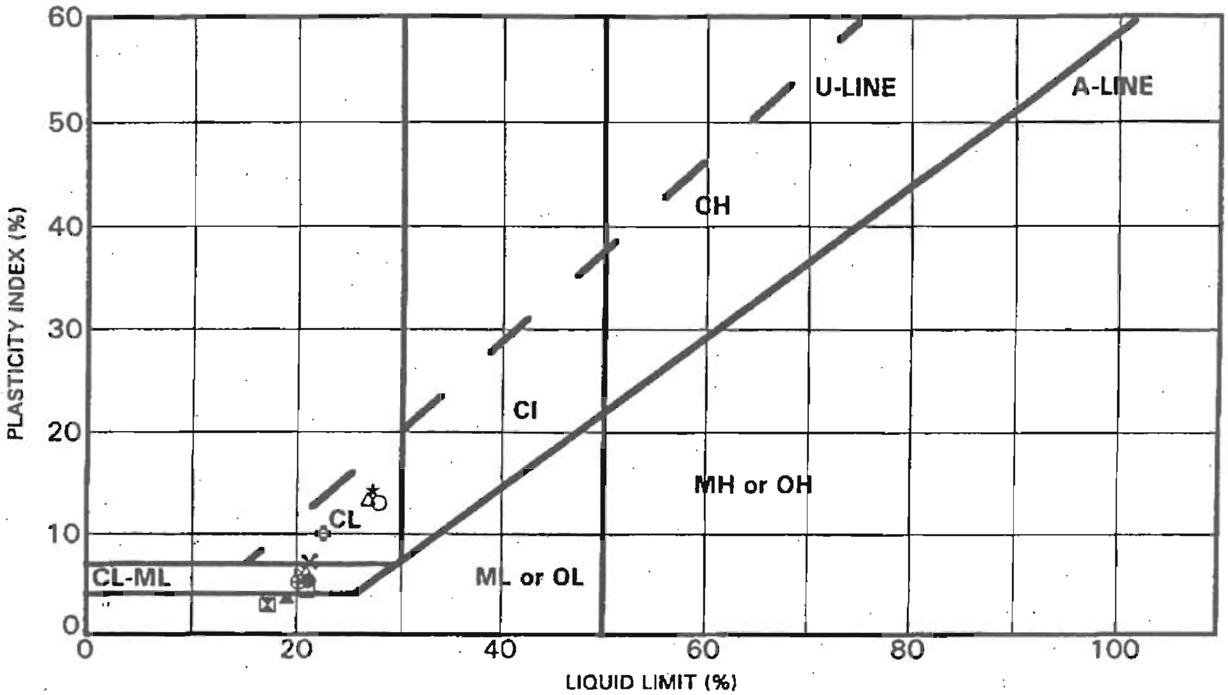
	HOLE/SAMPLE	DEPTH(ft)	W%	W_L	W_P	PI	REMARKS / SAMPLE DESCRIPTION
●	DH02-17	43.75	14.7				
⊠	EASTTAIL	0.00	12.1	20	15	5	
▲	WESTTAIL	0.00	20.2	21	17	4	

CU = Coefficient of Uniformity = D60/D10 Particle Sizes, e.g. D85, in mm Mechanical sieving(ASTM D422) after wash & dry(D1140) or hydrometer(D422)

KLOHN CRIPPEN

PROJECT NO.:	PM7802 29 02
PROJECT:	KGCMC 2002 Geotech.Investigation
LOCATION:	Admiralty Island, Alaska
FIGURE:	
DRAWN BY: Ganan	CHECKED BY: JUAN

PLASTICITY CHART



	HOLE	SAMPLE	DEPTH(ft)	%W	W_L	W_P	PI	% FINES	REMARKS
●	DH02-04		60.75	20.5	21	16	5	96.3	
⧗	DH02-05		10.25	14.9	17	14	3	85.0	
▲	DH02-06		10.75	19.4	19	15	4	87.0	
★	DH02-07		20.75	19.5	27	13	14	92.0	
✕	DH02-08		15.75	21.3	21	14	7	86.0	
✪	DH02-09		84.25	10.4	23	12	10	35.5	
○	DH02-09		109.25	17.0	28	15	13	70.7	
△	DH02-12		94.25	12.2	27	13	13		
⊗	DH02-17		13.75	14.1	21	14	6		
⊕	EASTTAIL		0.00	12.1	20	15	5	83.0	
□	WESTTAIL		0.00	20.2	21	17	4	83.5	

KLOHN CRIPPEN

PROJECT NO.: PM7802 29 02	
PROJECT:	KGCMC 2002 Geotech.Investigation
LOCATION:	Admiralty Island, Alaska
FIGURE:	
DRAWN BY: Ganan	CHECKED BY: JUM

SPECIFIC GRAVITY OF SOIL SOLIDS
(ASTM-D854)

Hole No.						
Depth						
Sample No.			West Tails			
Flask No.		6	5	11		
Volume of Flask @ 20° C	ml					
Evaporating Dish No.						
Method of Air removal		Boiling	Boiling	Boiling		
De-airing Period	hr	0.5	0.5	0.5		
Test temperature	°C	24	24	24		
Mass of Flask+Water (M_a)	g	640.26	677.58	679.21		
Mass of Flask+Water+Soil (M_b)	g	694.68	735.96	727.05		
Mass of Dish/Flask+Soil		213.56	261.03	248.02		
Mass of Dish/Flask		136.66	179.15	180.57		
Mass of Dry Soil (M_o)	g	76.90	81.88	67.45		
Correction factor (K) @ Test Temperature		0.9973	0.9973	0.9973		
Specific Gravity of Solids @ 20° C		3.411	3.475	3.430		

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

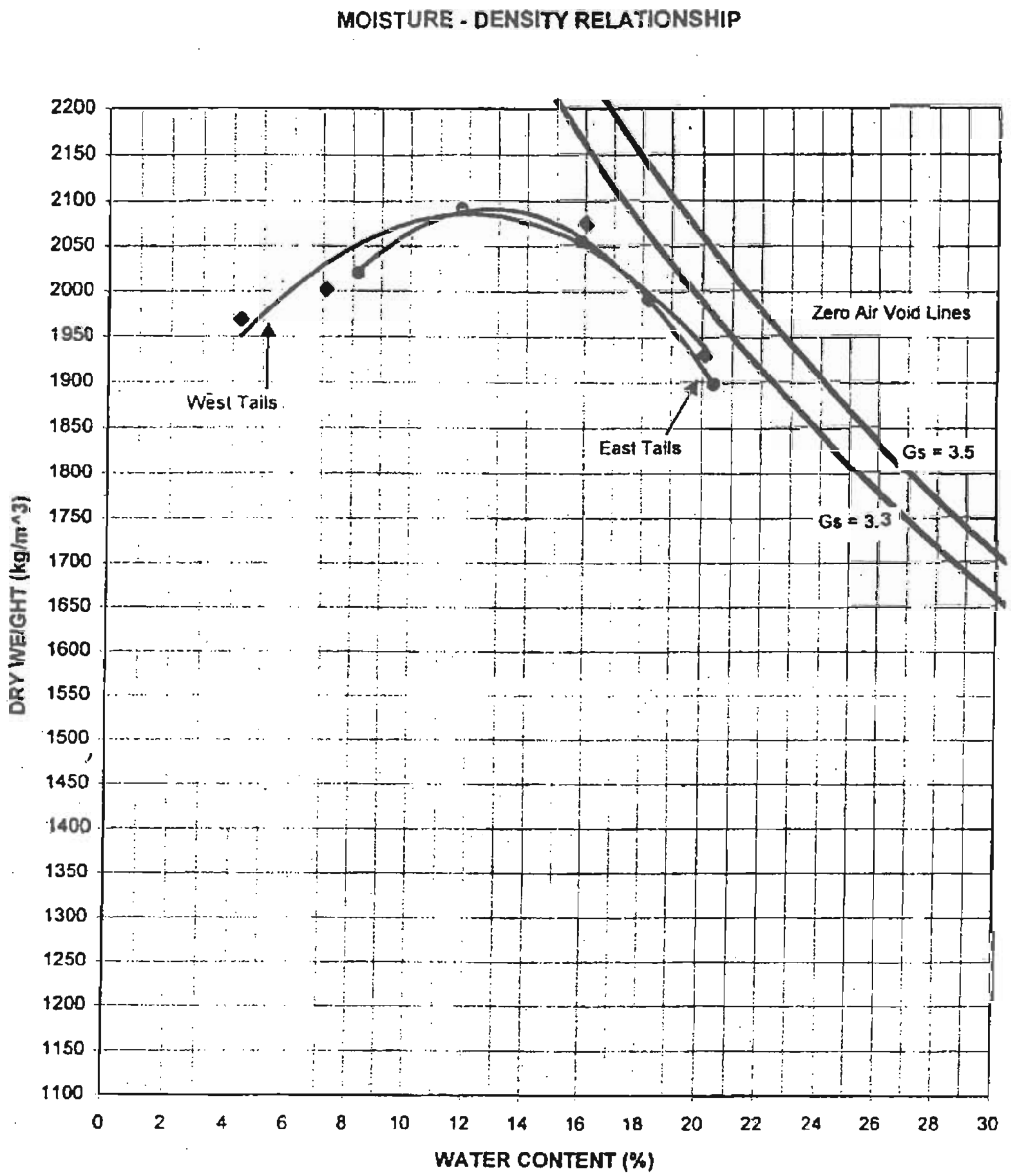
Average Specific Gravity of Solids @ 20° C = 3.44

Hole No.						
Depth						
Sample No.			East Tails			
Flask No.		12	4	8		
Volume of Flask @ 20° C	ml					
Evaporating Dish No.						
Method of Air removal		Boiling	Boiling	Boiling		
De-airing Period	hr	0.5	0.5	0.5		
Test temperature	°C	24	24	24		
Mass of Flask+Water (M_a)	g	680.64	679.09	676.65		
Mass of Flask+Water+Soil (M_b)	g	732.14	719.14	715.85		
Mass of Dish/Flask+Soil		254.99	237.76	233.92		
Mass of Dish/Flask		182.30	180.69	177.99		
Mass of Dry Soil (M_o)	g	72.69	57.07	55.93		
Correction factor (K) @ Test Temperature		0.9973	0.9973	0.9973		
Specific Gravity of Solids @ 20° C		3.421	3.344	3.334		

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

Average Specific Gravity of Solids @ 20° C = 3.37

KLOHN CRIPPEN	JOB NO.: PM7802 29 02	
	PROJECT: KGCMC 2002 Geo.Investigation	
	LOCATION: Alska	
	DATE: January 8, 2003	
	TESTED BY: Ganan	CHECKED BY: JUAN



Sample ID	OWC	Maximum Dry Density
West Tails	12.1	2085
East Tails	12.8	2090

KLOHN CRIPPEN

PROJECT No.:	PM7802 29 02
PROJECT:	KGCMC 2002 Geo.Investigation
LOCATION:	Alaska
FIGURE:	
DRAWN BY:	Ganan CHECKED BY:

Attachment 2

Laboratory Data
Cyclical Triaxial & Simple Shear and Monotonic Loading Tests

TABLE 1. TEST INFORMATION AND SUMMARY RESULTS

KGCMC 2002 GEOTECHNICAL INVESTIGATION

Test No	Sample ID	W_c	σ'_{3c} (kPa)	γ_d (kg/m^3)	Cyclic Tests						Post Cyclic Tests	
					Stage	1	2	3	ε_{1max} (%)	$\Delta u_r/\sigma'_{vc}$	S_u (kPa)	ε_{1peak} (%)
CTX01	WEST TAILS	0.197	900	2075	$\sigma_{d\,cyc}/2\sigma'_{3c}$	0.05	0.10	0.15	-5.6	0.96	265*	+11*
					N	100[a]	100[a]	57[b]				
CTX02	WEST TAILS	0.196	900	2062	$\sigma_{d\,cyc}/2\sigma'_{3c}$	0.21			-9.1	0.94	110*	-17*
					N	11[b]						
CTX03	WEST TAILS	0.198	900	2058	$\sigma_{d\,cyc}/2\sigma'_{3c}$	0.25			-14.1	0.92	130*	-17*
					N	4[b]						

* Visually estimated form the s_u vs ε_1 plots
a Cyclic loading continued with higher Cyclic Stress Ratio
b Liquefaction criterion reached

Notations:

N	Number of cycles required to exceed 5.0 % Axail strain
ε_1	Axial strain
ε_{1peak}	Axial strain corresponding to S_u
ε_{1max}	Maximum Axial strain during cyclic loading
e	Void ratio at the end of consolidation
W_c	Water content at end of test
γ_d	Dry Density at the end of consolidation
σ'_3	Effective confining stress
σ'_{3c}	Effective confining consolidation stress
$\sigma_{d\,cyc}$	Cyclic deviator stress
Δu	Excess pore pressure
Δu_r	Residual excess pore pressure at the end of cyclic test
S_u	Peak undrained shear stress in post-cyclic loading

TABLE 2. TEST INFORMATION AND SUMMARY RESULTS

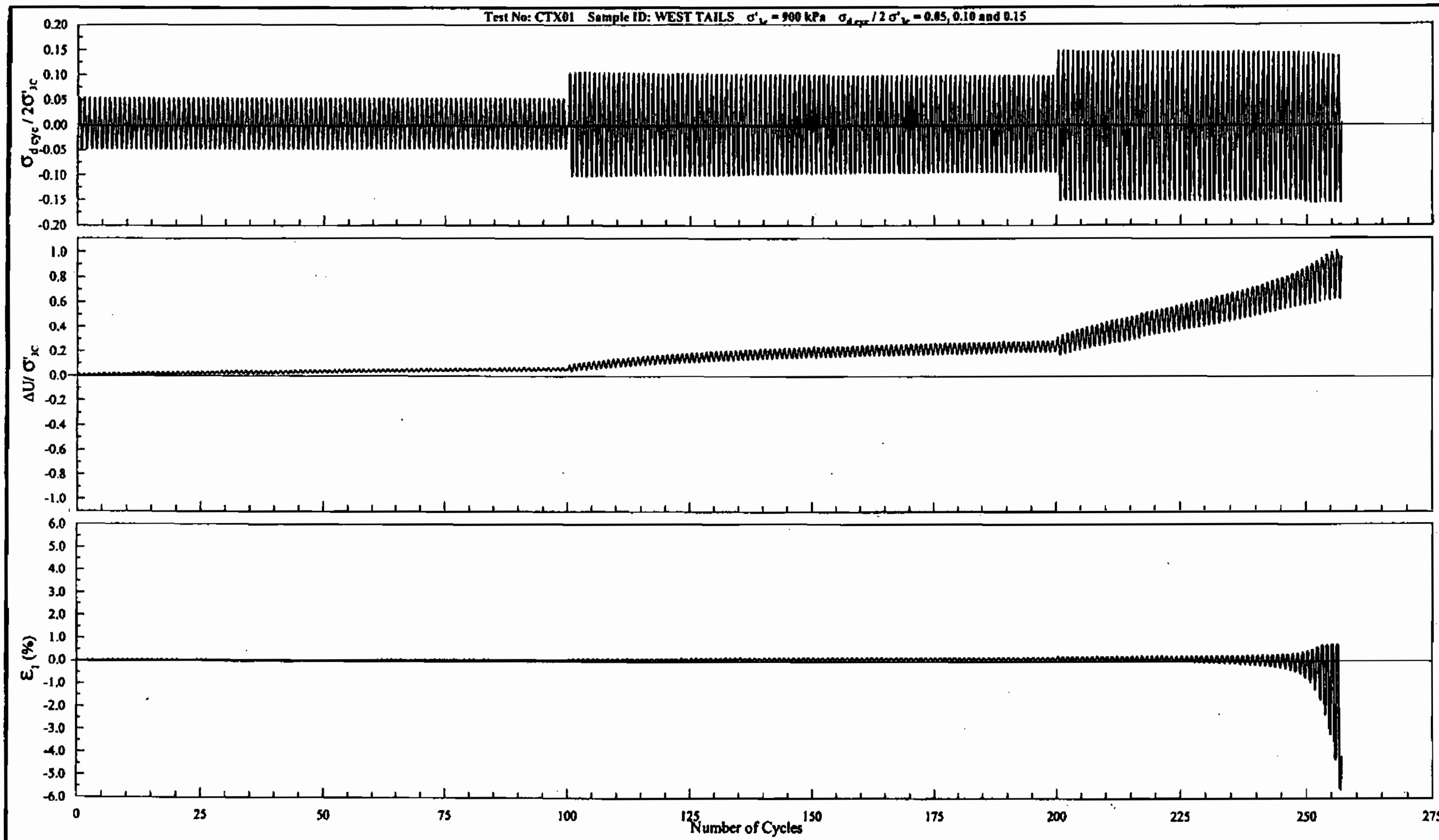
KGCMC 2002 GEOTECHNICAL INVESTIGATION

Test No	Bore Hole ID	w_c	σ'_{3c} (kPa)	γ_d (kg/m^3)	Cyclic Tests				Post Cyclic Tests	
					Stage	1	ε_{1max} (%)	$\Delta u_r/\sigma'_{vc}$	S_u (kPa)	ε_{1peak} (%)
CTX04	WEST TAILS	0.192	900	2104	$\sigma_{d\,cyc}/2\sigma'_{3c}$	0.195	-5.14	-	155	-11.5
					N	23[b]				
CTX05	WEST TAILS	0.197	900	2091	$\sigma_{d\,cyc}/2\sigma'_{3c}$	0.235	-7.54	0.94	135	-18.9
					N	7[b]				

* Visually estimated form the s_u vs ε_1 plots
a Cyclic loading continued with higher Cyclic Stress Ratio
b Liquefaction criterion reached

Notations:

N	Number of cycles required to exceed 5.0 % Axial strain
ε_1	Axial strain
ε_{1peak}	Axial strain corresponding to S_u
ε_{1max}	Maximum Axial strain during cyclic loading
e	Void ratio at the end of consolidation
w_c	Water content at end of test
γ_d	Dry Density at the end of consolidation
σ'_3	Effective confining stress
σ'_{3c}	Effective confining consolidation stress
$\sigma_{d\,cyc}$	Cyclic deviator stress
Δu	Excess pore pressure
Δu_r	Residual excess pore pressure at end of cyclic test
s_u	Peak undrained shear stress in post-cyclic loading



Klohn Crippen		Date
Designed		
Drawn		
Recommended		
Checked		

Kennecott Minerals

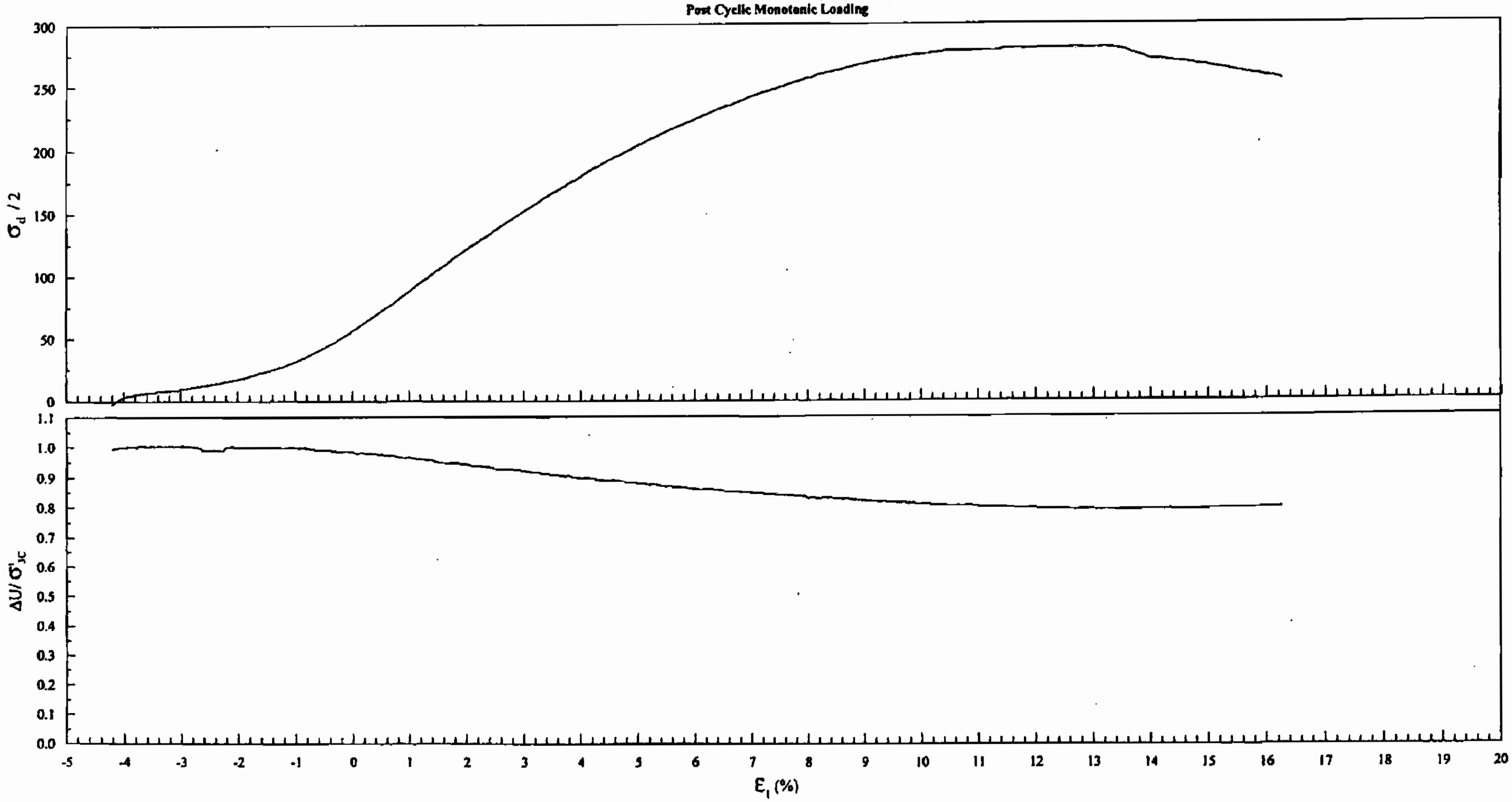
KLOHN CRIPPEN

KGCMC 2002 Geotechnical Investigation

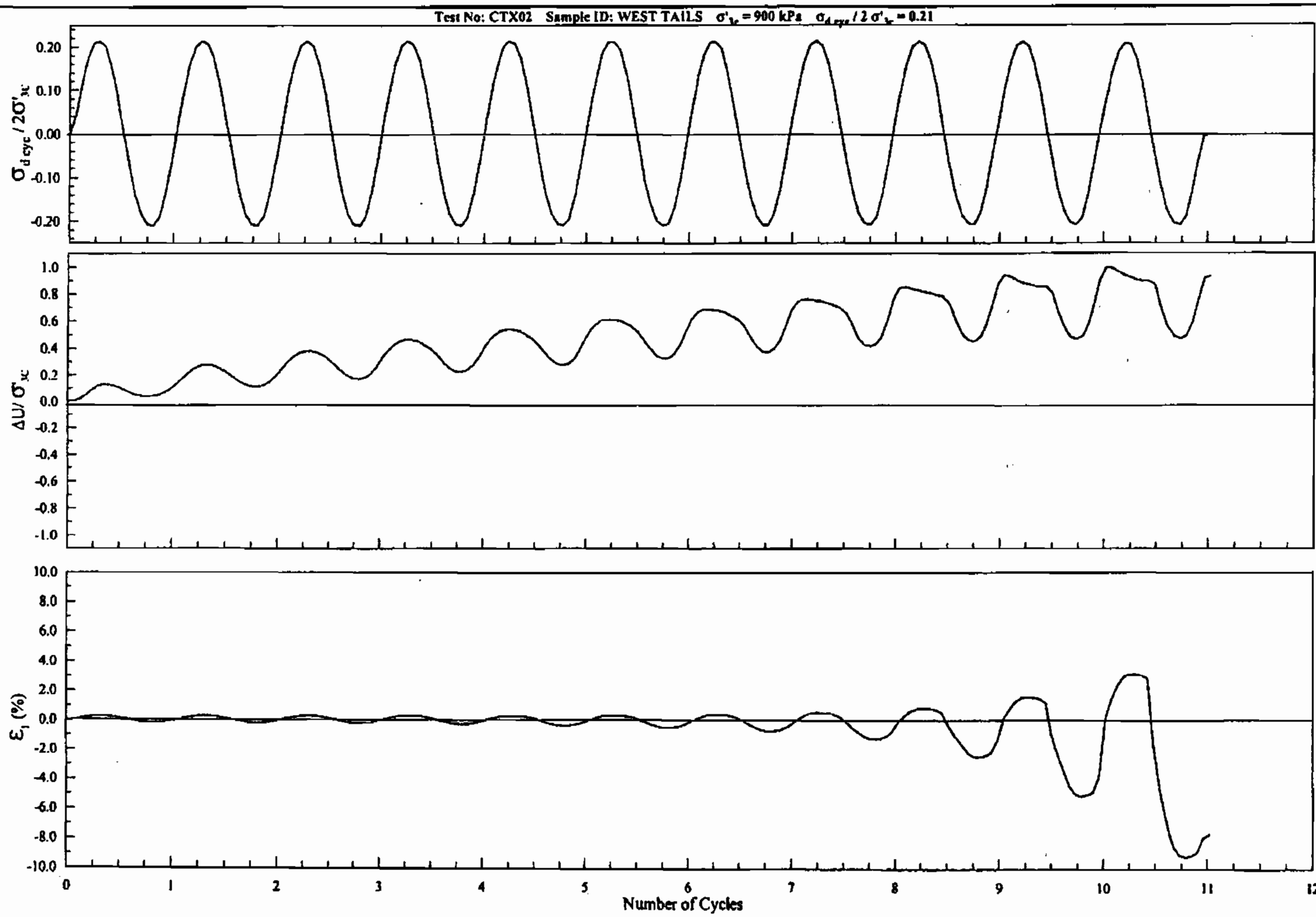
Cyclic Triaxial Test
West Tails

April 24, 2003 | PM7802 29

Test No: CTX01 Sample ID: WEST TAILS σ'_{3c} = 900 kPa $\sigma_{d\,cyc} / 2\,\sigma'_{3c}$ = 0.05, 0.10 and 0.15
Post Cyclic Monotonic Loading



Klohn Crippen		Date	Client	Project
Designed			Kennecott Minerals	KGCMC 2002 Geotechnical Investigation
Drawn				Title: Cyclic Triaxial Test West Tails
Recommended			KLOHN CRIPPEN	
Checked				Date: April 24, 2003 — Project No.: PM7802 29



Klohn Crippen		Date
Designed		
Drawn		
Recommended		
Checked		

Client: Kennecott Minerals

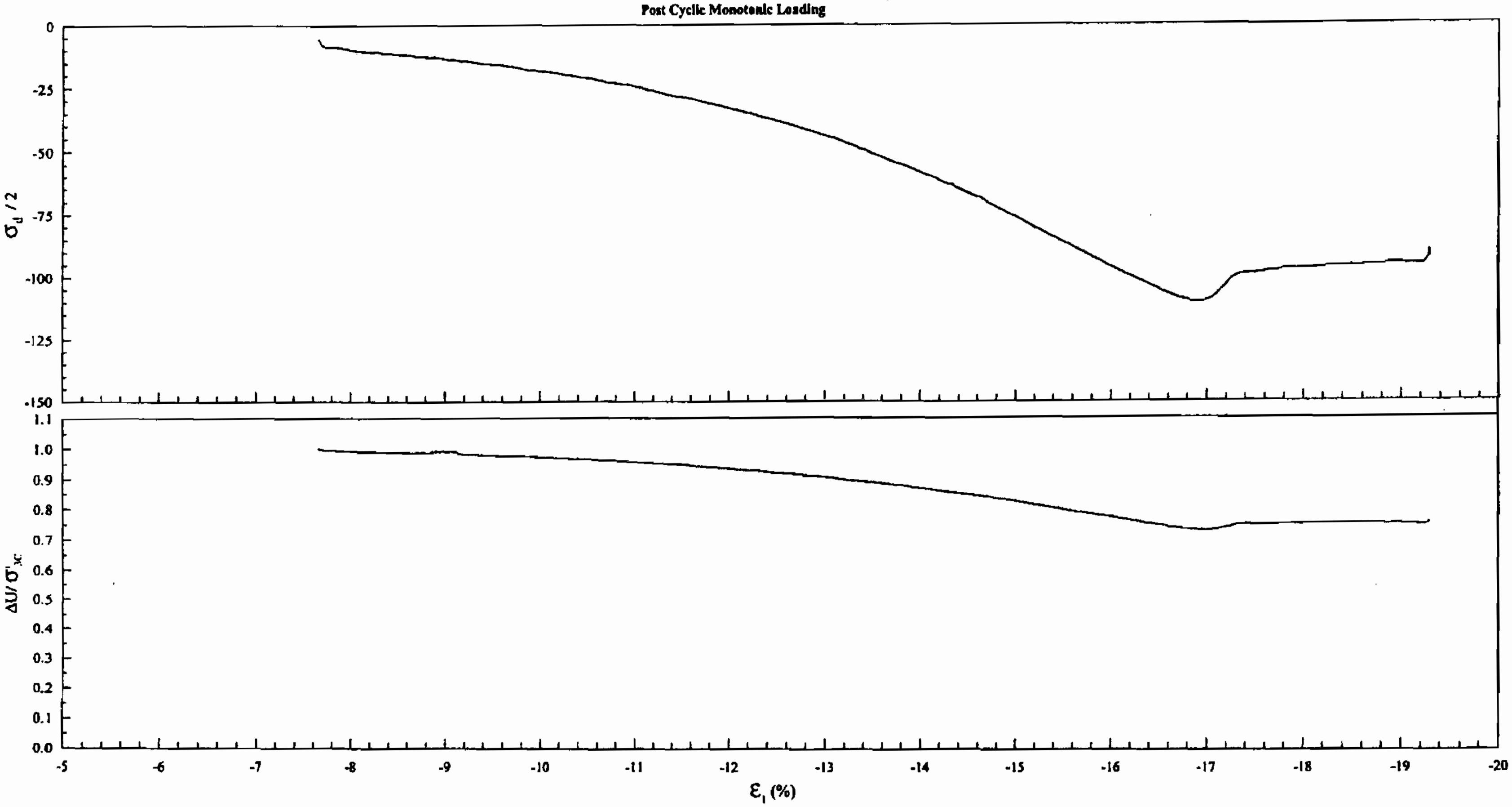
KLOHN CRIPPEN

Project: KGCMC 2002 Geotechnical Investigation

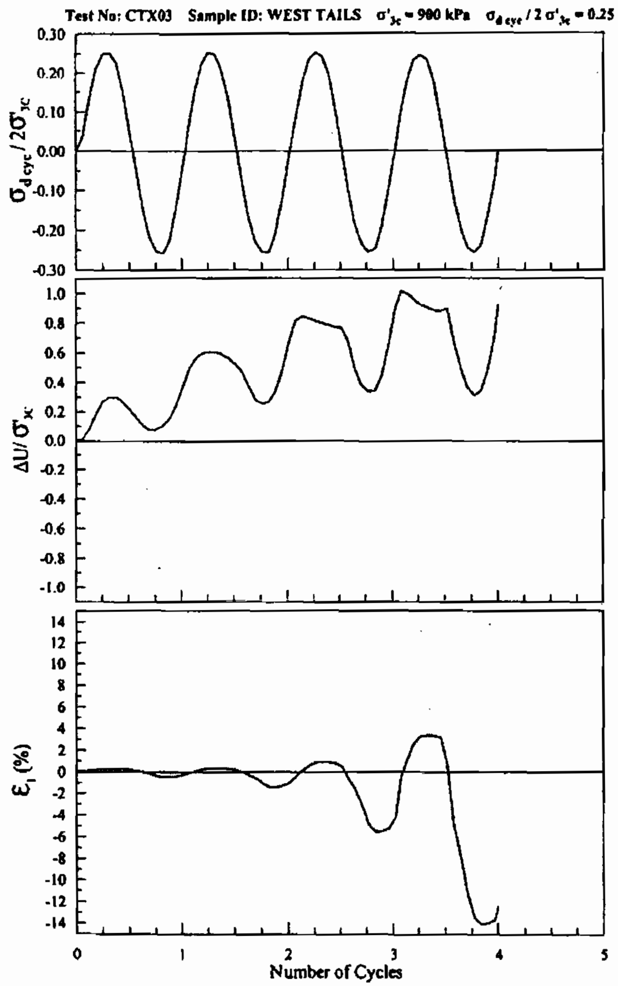
Title: Cyclic Triaxial Test
West Tails

Date of Issue	Project No.	Figure	Rev
May 1, 2003	PM7802 29		

Test No: CTX02 Sample ID: WEST TAILS σ'_{3c} = 990 kPa $\sigma_{d\,cyc} / 2\sigma'_{3c}$ = 0.21
Post Cyclic Monotonic Loading



Klohn Crippen		Date	Kennecott Minerals	KGCMC 2002 Geotechnical Investigation
Designed				Cyclic Triaxial Test
Drawn				West Tails
Recommended				May 1, 2003
Checked			KLOHN CRIPPEN	PM7802 29



Klohn Crippen		Date
Designed		
Drawn		
Recommended		
Checked		

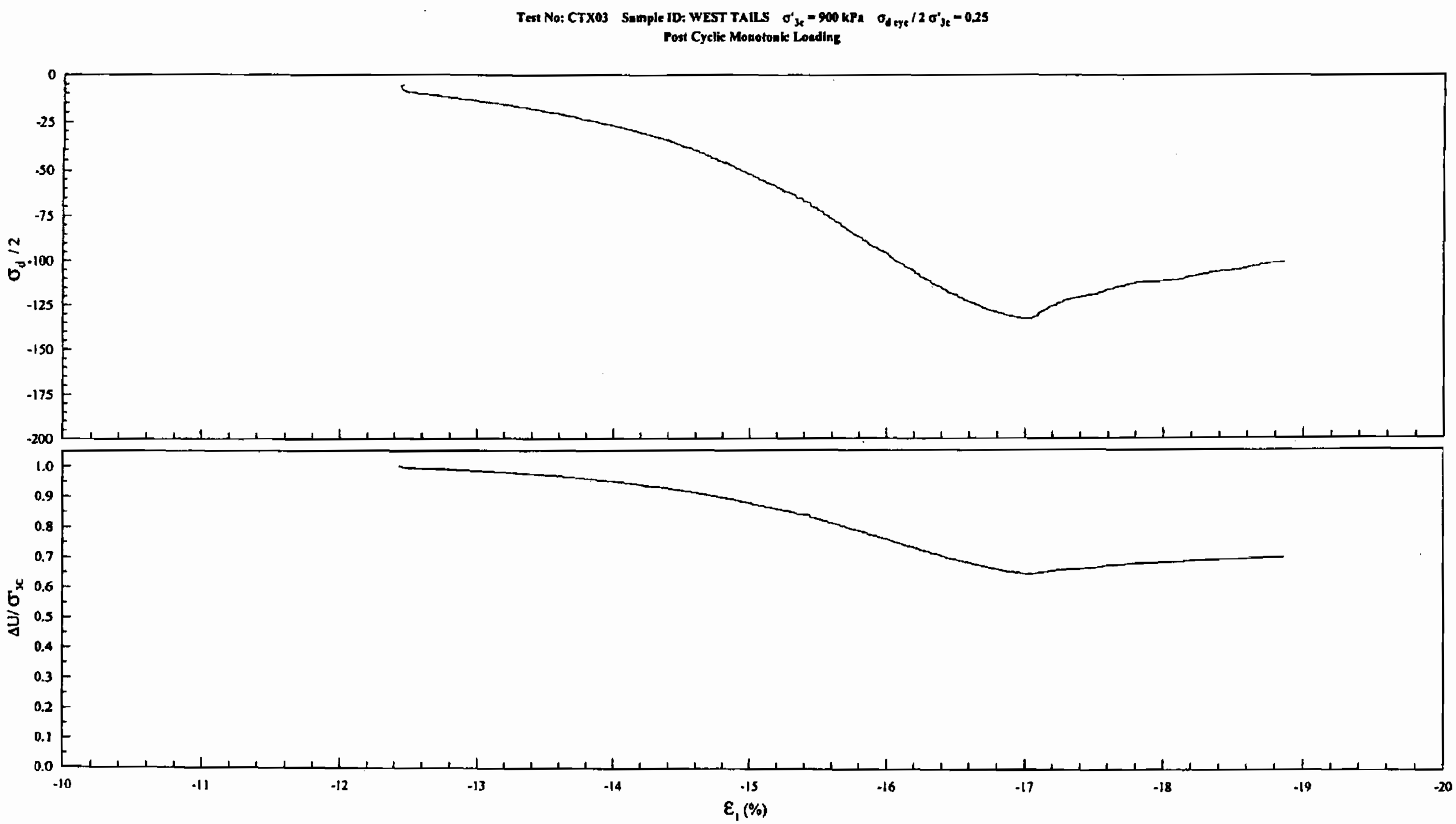
Kennecott Minerals

KLOHN CRIPPEN

KGCMC 2002 Geotechnical Investigation

Cyclic Triaxial Test
West Tails

May 9, 2003 | PM7802 29



Klohn Crippen		Date
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Drawn		
Recommended		
Checked		

CLIENT: Kennecott Minerals

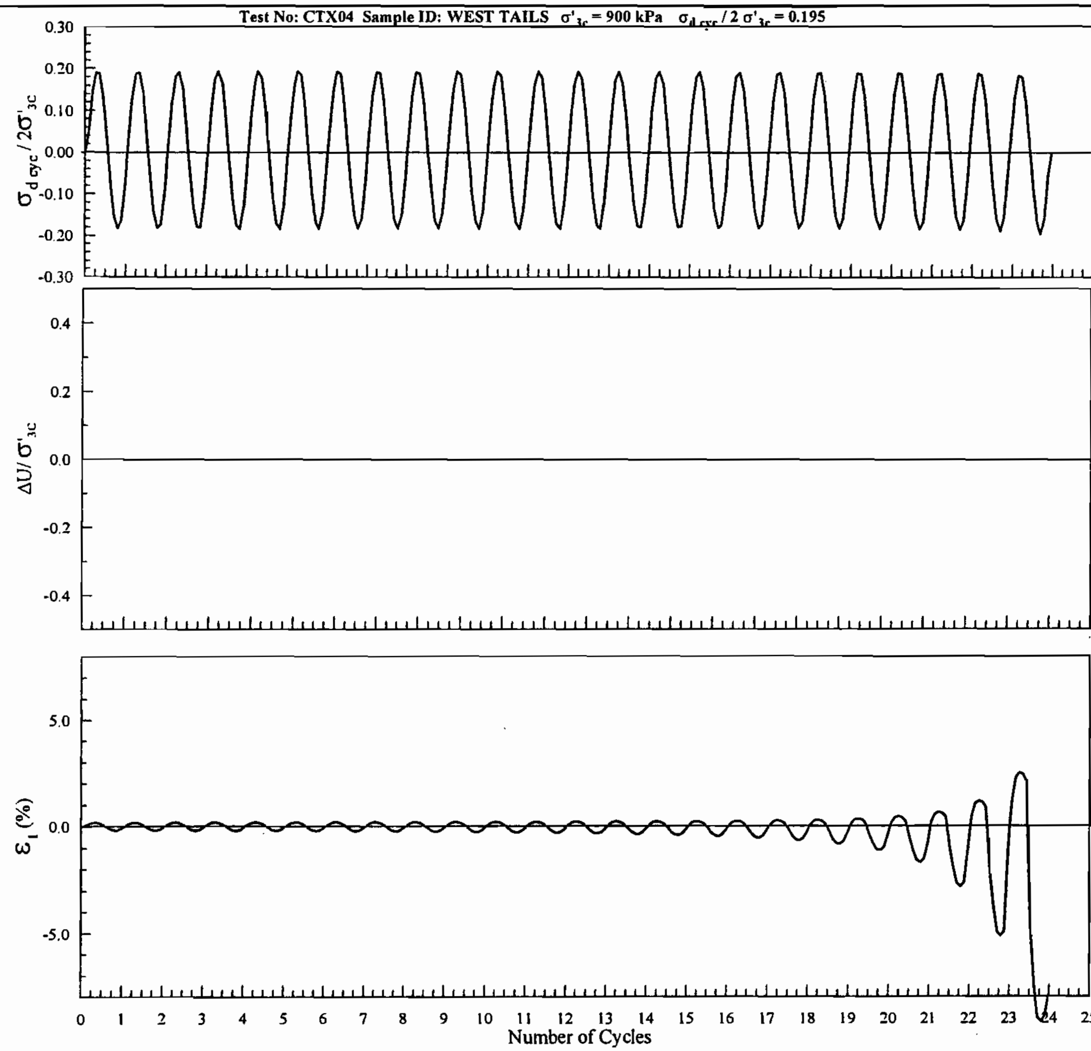
KLOHN CRIPPEN

PROJECT: KGCMC 2002 Geotechnical Investigation

TITLE: Cyclic Triaxial Test
West Tails

DATE OF ISSUE: May 9, 2003

PROJECT No: PM7802 29



Klohn Crippen		Date
Designed		
Drawn	Ganan	10/03
Recommended		
Checked		

CLIENT
Kennecott Minerals

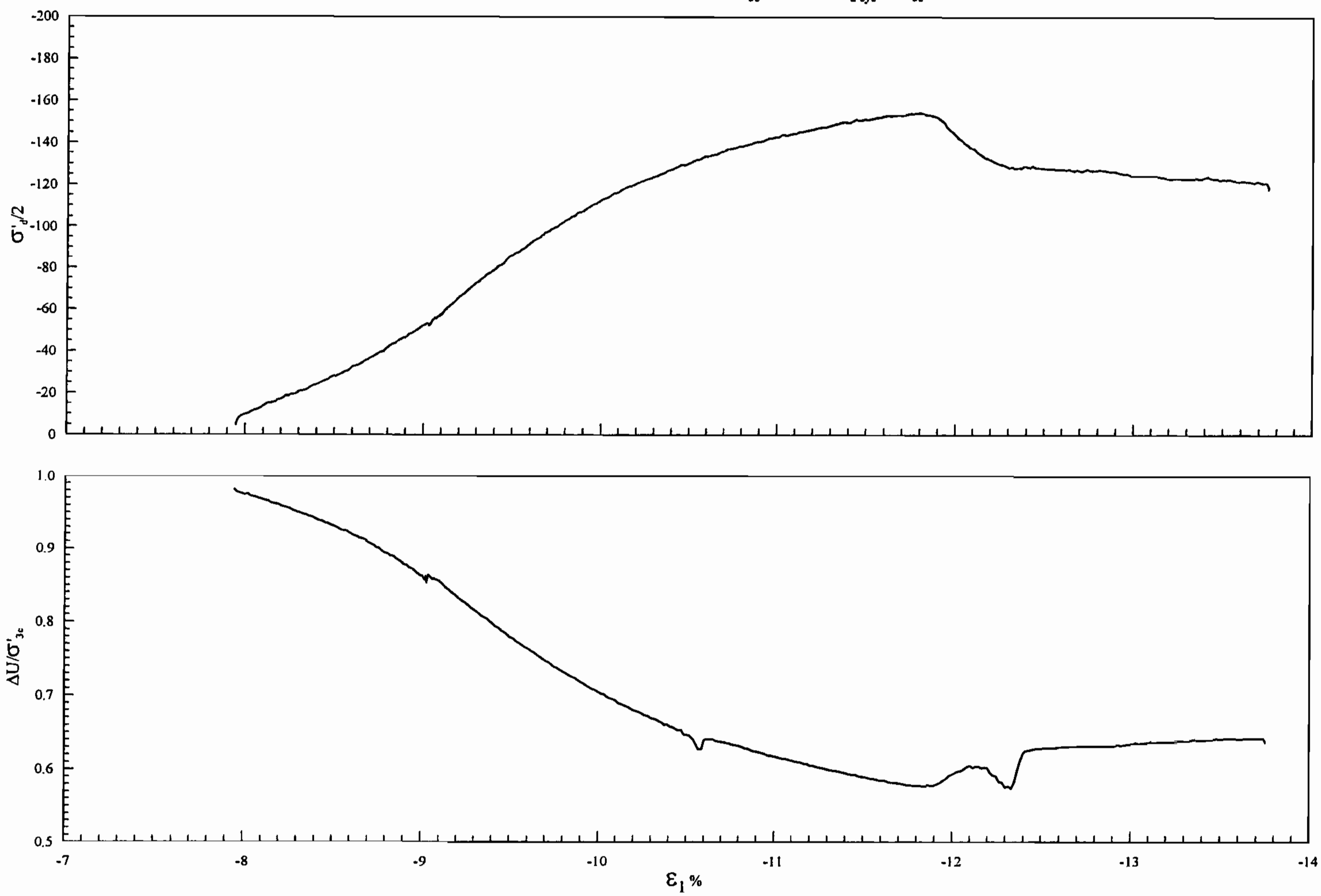
KLOHN CRIPPEN

PROJECT
KGCMC 2002 Geotechnical Investigation

TITLE
Cyclic Triaxial Test
WEST TAILS

DATE OF TEST	PROJECT No.	FIG No.	REV.
Sept 29, 2003	M07802		

Test No: CTX04 Sample ID: WEST TAILS σ'_{3c} = 900 kPa $\sigma_{d\,cyc}/2\,\sigma'_{3c}$ = 0.195



Klohn Crippen		Date
Designed		
Drawn	Ganan	10/03
Recommended		
Checked		

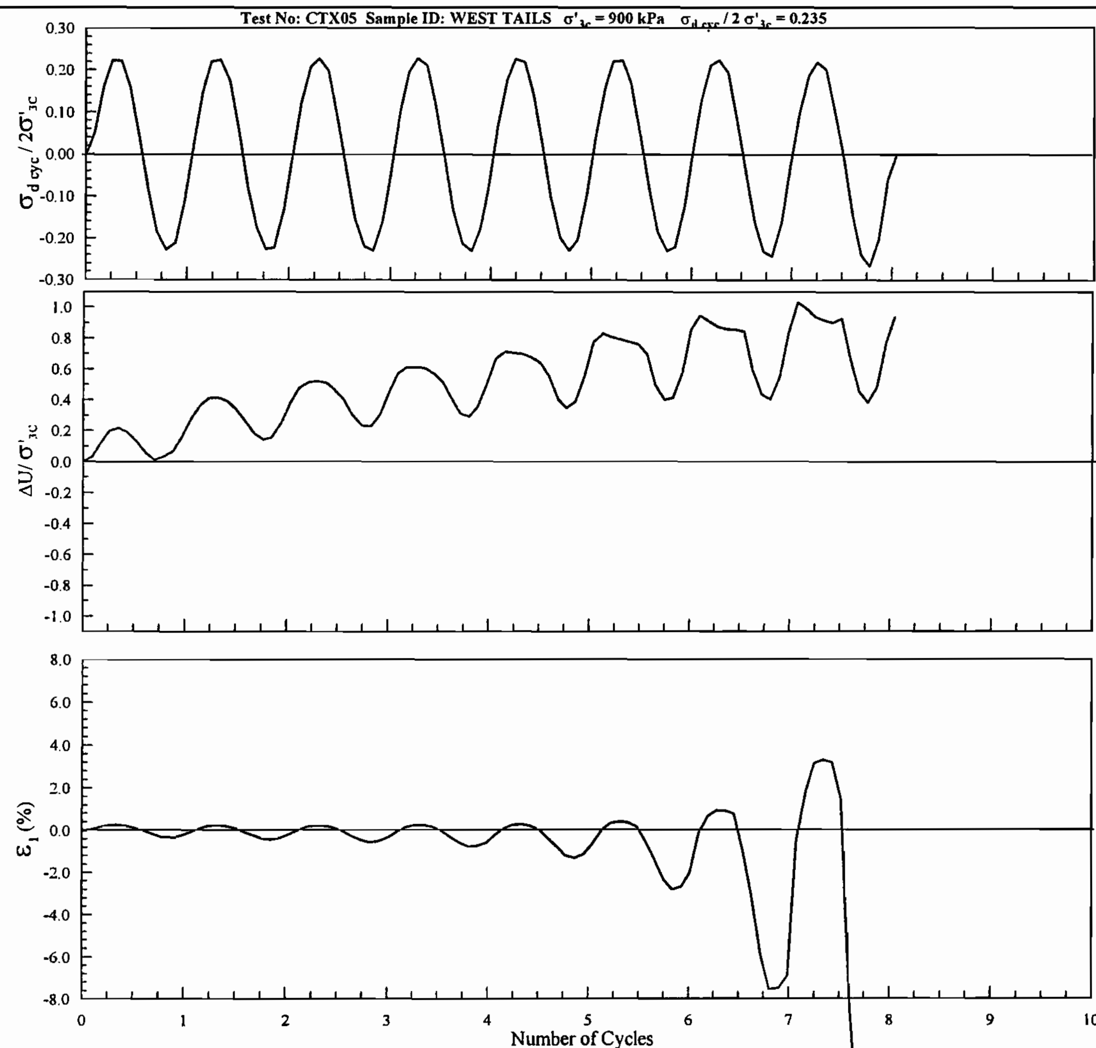
CLIENT: Kennecott Minerals

KLOHN CRIPPEN

PROJECT: KGCMC 2002 Geotechnical Investigation

TITLE: Post Cyclic Triaxial Test
WEST TAILS

DATE OF TEST	PROJECT No.	FIG No.	REV.
Sept 29, 2003	M07802		



Klohn Crippen		Date
Designed		
Drawn	Ganan	10/03
Recommended		
Checked		

CLIENT
Kennecott Minerals

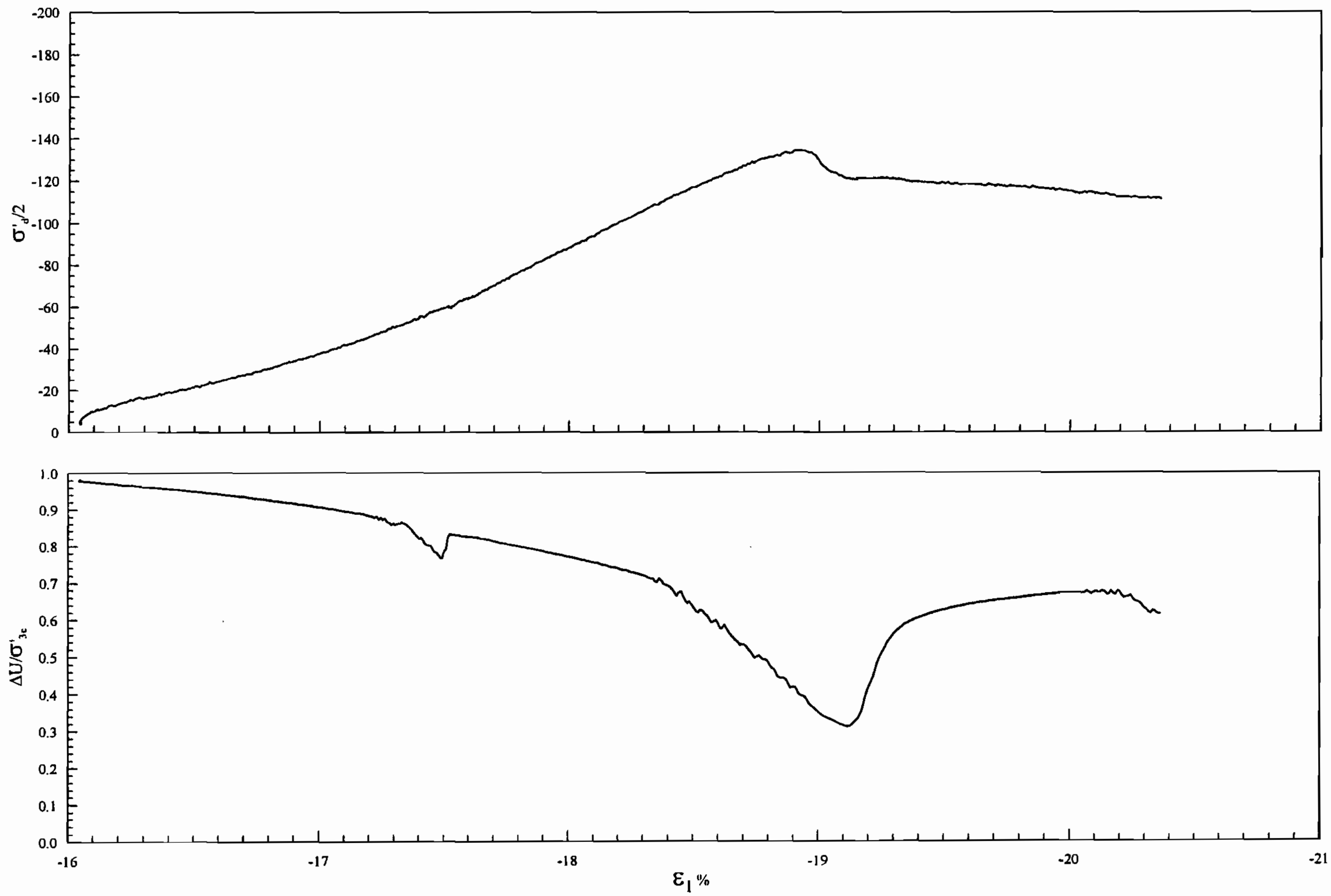
KLOHN CRIPPEN

PROJECT
KGCMC 2002 Geotechnical Investigation

TITLE
Cyclic Triaxial Test
WEST TAILS

DATE OF TEST	PROJECT No.	FIG No.	REV.
Oct 03, 2003	M07802		

Test No: CTX05 Sample ID: WEST TAILS σ'_{3c} = 900 kPa $\sigma_{d\,cyc}$ / 2 σ'_{3c} = 0.235



Klohn Crippen		Date
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Drawn	Ganan	10/03
Recommended		
Checked		

CLIENT
Kennecott Minerals

KLOHN-CRIPPEN

PROJECT
KGCMC 2002 Geotechnical Investigation

TITLE
Post Cyclic Triaxial Test
WEST TAILS

DATE OF TEST	PROJECT No.	FIG No.	REV.
Oct 03, 2003	M07802		

MEG TECHNICAL SERVICES

A Division of MEG Consulting Limited

CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	Klohn Crippen - DSS Testing	Test ID:	05-154-01
Location:	Unknown	Borehole: Unknown	Depth: Unknown
Sample:	Disturded Tailings	Station: Unknown	Testing Date: May 20-21, 2005

Sample Visual Description: Dark Grey Fine Silty Sand

Summary of initial sample conditions:

Diameter of the Sample	53.5 mm
Initial Height of the Sample	17.99 mm
Initial Effective Vertical Stress	900 kPa
Initial Dry Density	1832 kg/m^3
Initial Water Content	16.4%

DSS Test parameters:

Cyclic Stress Ratio	0.17
Maximum Cyclic Shear Stress	152.6 kPa
Limiting Cyclic Shear Strain	3.75%
Post-Cyclic Static Shear Strain Rate	10% / hour

Sample Conditions after Consolidation:

Consolidated Height of Sample	15.91 mm
Consolidated Dry Density	2071.3 kg/m^3

Summary of final sample conditions:

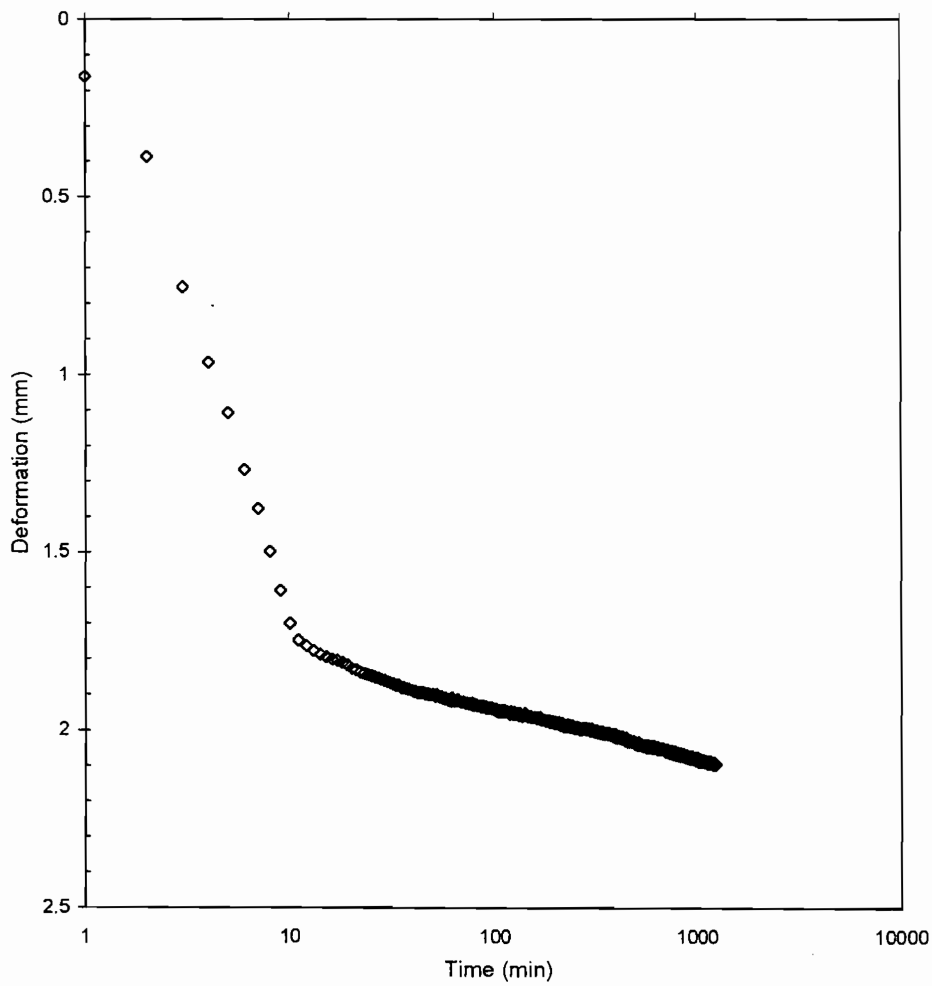
Final Water Content	17.3%

MEG TECHNICAL SERVICES

A Division of MEG Consulting Limited

CYCLIC DIRECT SIMPLE SHEAR TEST - CONSOLIDATION STAGE

Project:	Klohn Crippen - DSS Testing			Test ID:	05-154-01
Location:	Unknown	Borehole:	Unknown	Depth:	Unknown
Sample:	Disturbed Tailings	Station:	Unknown	Date:	May 20, 2005

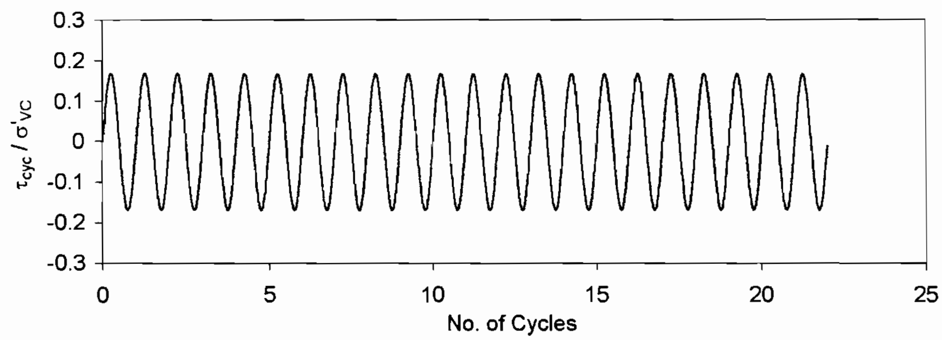


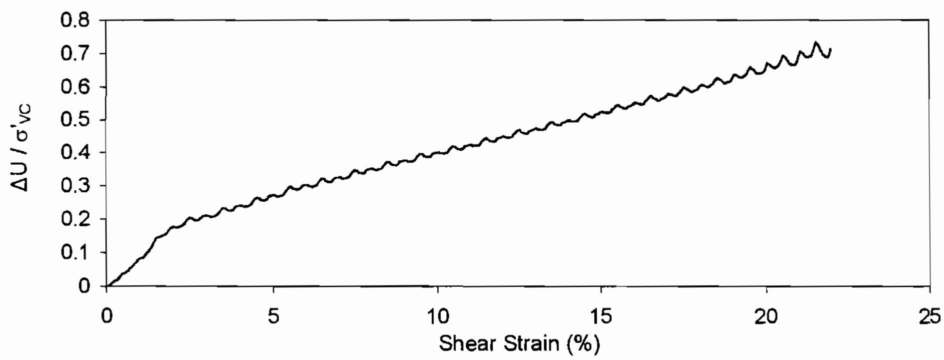
MEG TECHNICAL SERVICES

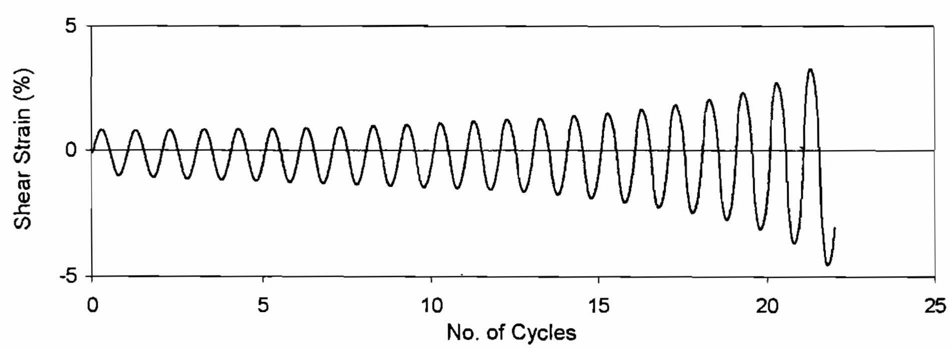
A Division of MEG Consulting Limited

CYCLIC DIRECT SIMPLE SHEAR TEST - STRESS CONTROL

Project:	Klohn Crippen - DSS Testing			Test ID:	05-154-01
Location:	Unknown	Borehole:	Unknown	Depth:	Unknown
Sample:	Disturded Tailings	Station:	Unknown	Date:	May 20, 2005





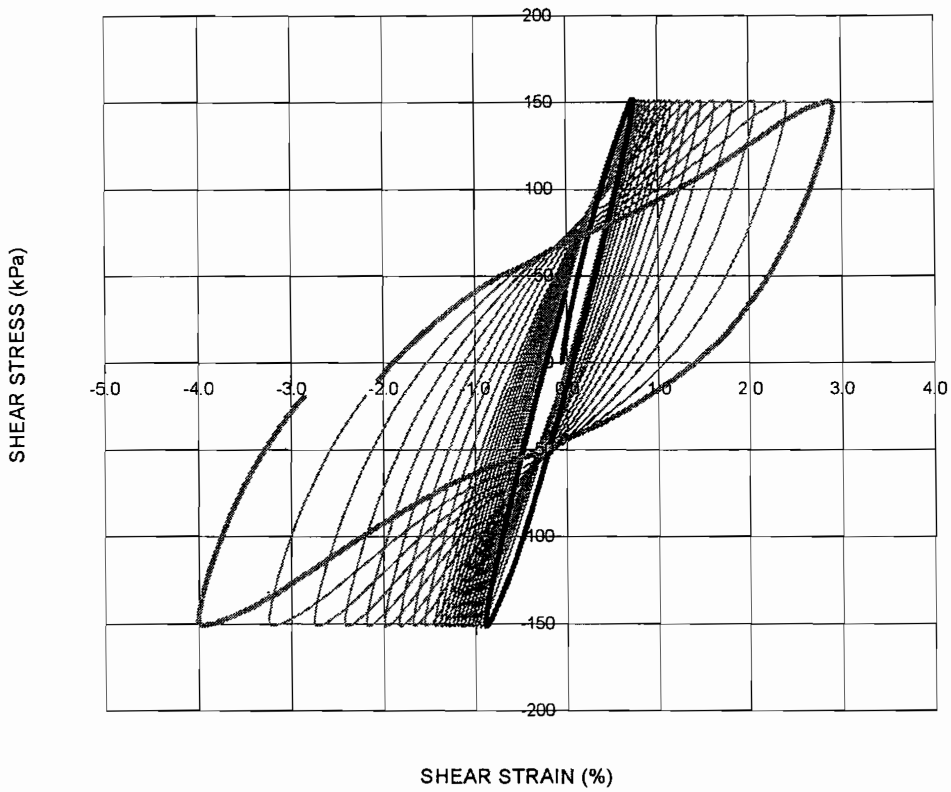


MEG TECHNICAL SERVICES

A Division of MEG Consulting Limited

CYCLIC DIRECT SIMPLE SHEAR TEST - STRESS CONTROL

Project:	Klohn Crippen - DSS Testing			Test ID:	05-154-01
Location:	Unknown	Borehole:	Unknown	Depth:	Unknown
Sample:	Disturded Tailings	Station:	Unknown	Date:	May 20, 2005

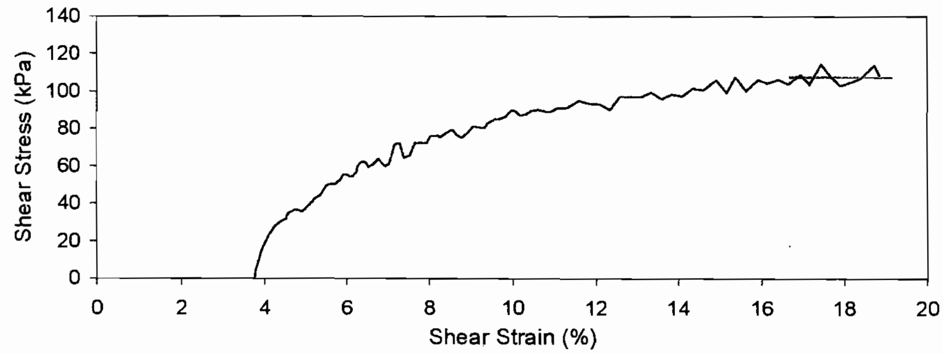


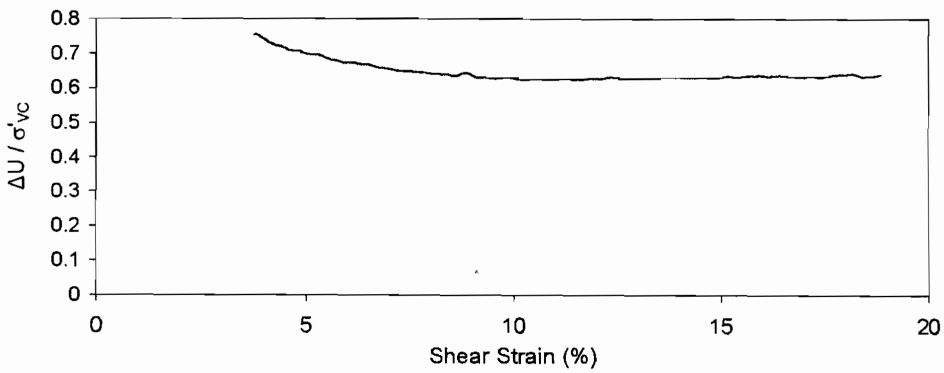
MEG TECHNICAL SERVICES

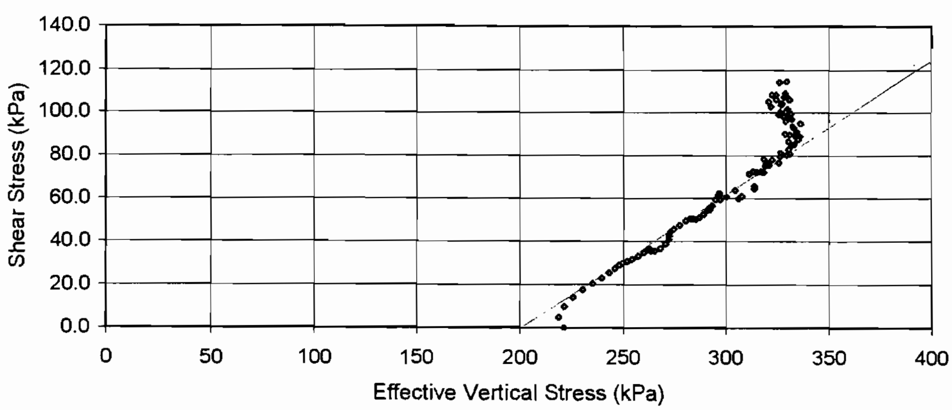
A Division of MEG Consulting Limited

CYCLIC DIRECT SIMPLE SHEAR TEST - POST-CYCLIC STATIC SHEAR

Project:	Klohn Crippen - DSS Testing			Test ID:	05-154-01
Location:	Unknown	Borehole:	Unknown	Depth:	Unknown
Sample:	Disturded Tailings	Station:	Unknown	Date:	May 20, 2005







Attachment 3

Photographs

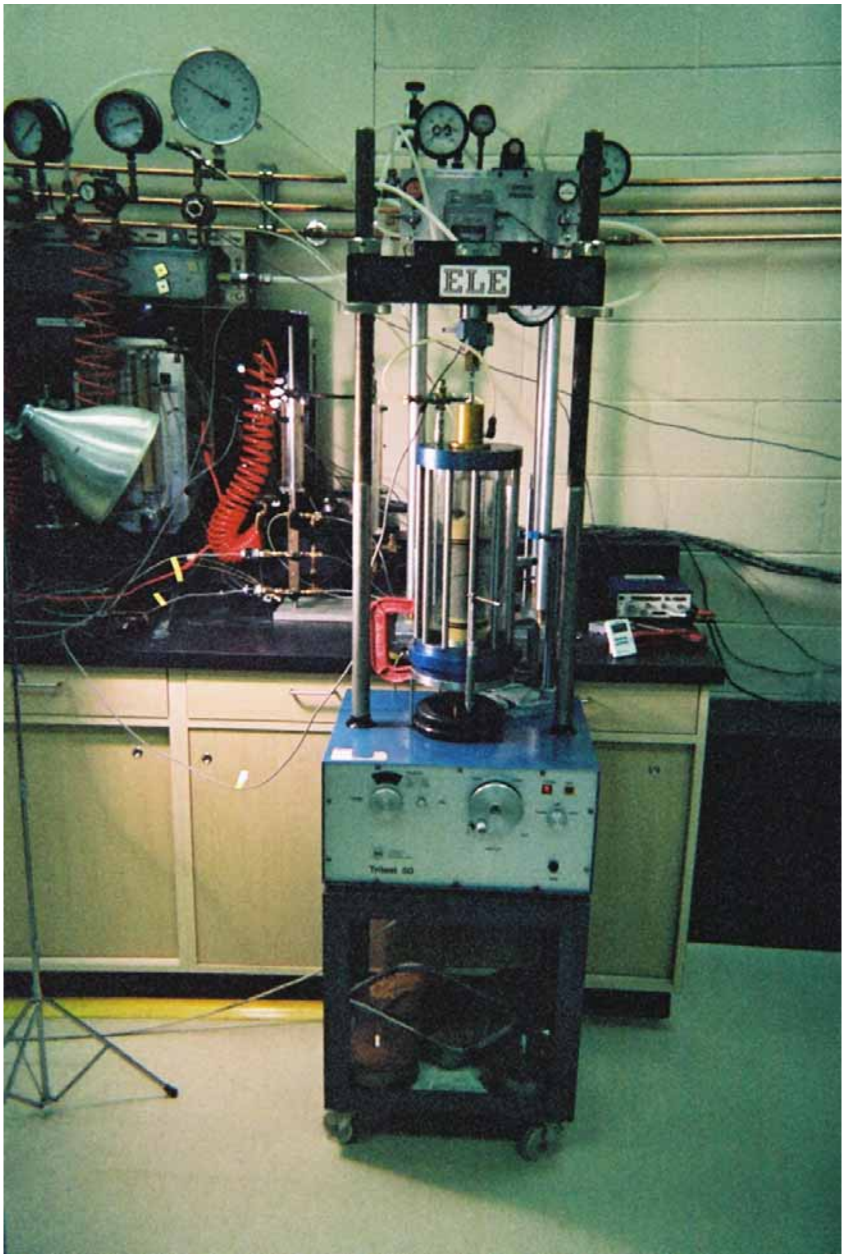


Photo 1 Specimen CTX 02 in Monotonic Triaxial Test Frame.

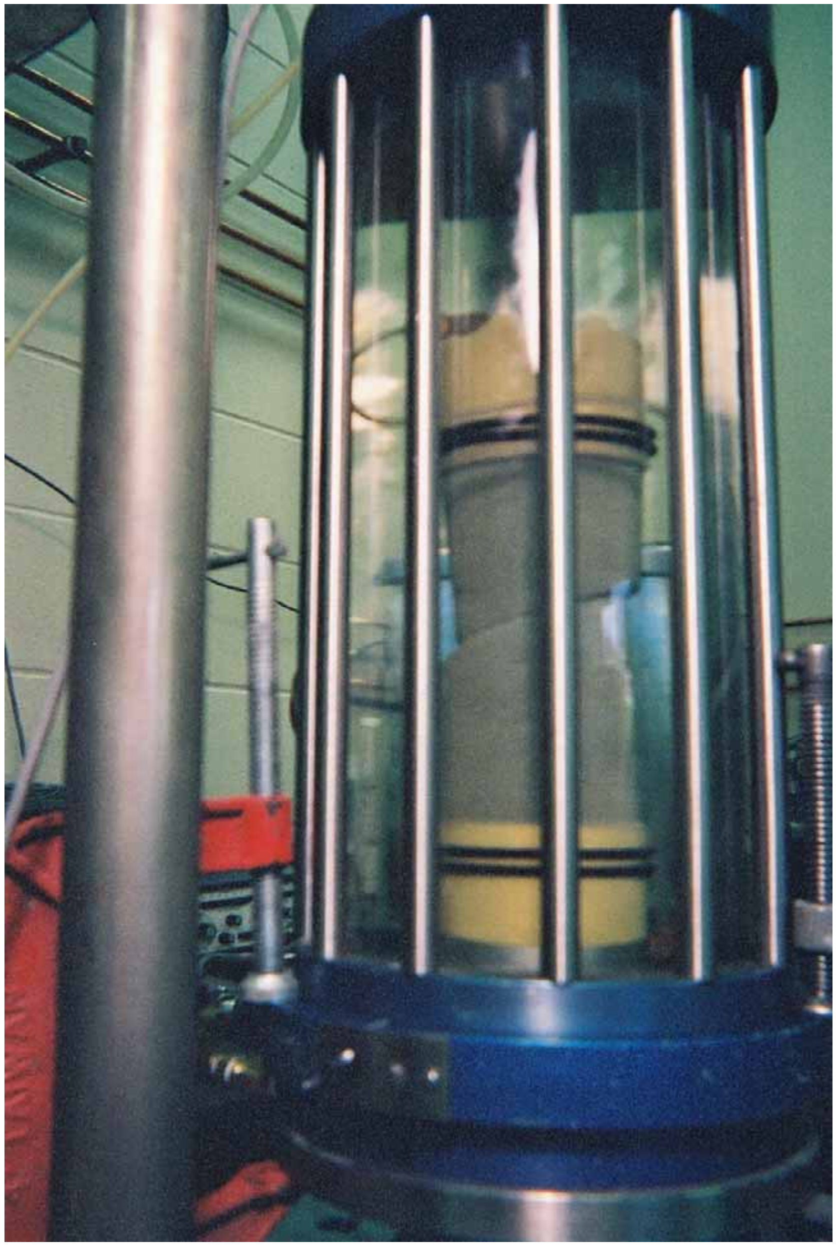


Photo 2 Specimen CTX 02 after large-strain Monotonic Extension.

Attachment 4

Cyclical Triaxial Test Data
SRK 1996

Dr. Y.P. Vaid
10559 Yarmish Drive
Richmond, B.C. V7E 5E6

8 March 1996

Steffen Robertson and Kirsten (Canada) Inc.
#800, 580 Hornby Street
Vancouver, B.C. V6C 3B6
Attention: Cam Scott

Dear Mr. Scott:

Re: Triaxial Tests on Tailings

Cyclic undrained triaxial tests on Shelby tube samples were performed at the Soil Mechanics Lab of the University of British Columbia in order to determine the development of cyclic strains and pore pressures with number of cycles as well as post-cyclic undrained strength. The tests were performed under my supervision by a qualified graduate student experienced in such testing.

Full size 3" Shelby tube samples supplied were used for the triaxial tests. Sample extrusion and sample selection for the tests was done in your presence. To avoid disturbance due to possible slumping under self-weight, specimens were extruded directly into the membrane lined forming jacket.

Two cyclic triaxial tests were carried out. The test variables are given in Table 1. One specimen was hydrostatically consolidated under a confining pressure $\sigma'_{vc} = \sigma'_{hc} = 75$ kPa. The other specimen was consolidated anisotropically under a vertical effective stress of 75 kPa and radial effective stress of 50 kPa. Consolidation was allowed for a period of about one day prior to cyclic loading. A back pressure of at least 300 kPa was used. B-values achieved were in excess of 0.99.

Cyclic loading was applied at a frequency of 0.1 Hz. This consisted of a symmetric sinusoidal pulse at constant stress ratio $\sigma_{dcy}/2\sigma'_{hc}$. Cyclic loading was terminated when a maximum single amplitude axial strain of ≥2½ % was recorded. A continuous record of test data was obtained by a computer interfaced data acquisition system. The test variables monitored consisted of full time histories of deviator stress, axial strain and induced pore pressure. Post-cyclic monotonic undrained compression tests were carried out at an axial strain rate of about 10%/hour.

...2/

Anisotropically consolidated specimen was stage cyclically loaded. Successively increasing levels of $\sigma_{dcy}/2\sigma'_{hc}$ amplitude pulses were applied for about 10 cycles each until the maximum strain amplitude exceeded 2½%. The specimen was then brought to the hydrostatic stress state by undrained unloading of the static deviator stress, prior to post-cyclic loading. Hydrostatically consolidated specimen was post-cyclically loaded with multiple cycles. The initial loading imposed was in the compression mode followed by reversal into the extension and finally again into the compression mode.

A summary of test results is presented in Table 1. Plots of test results are shown in Figs. 1 and 2. Results of post-cyclic monotonic tests show plots of deviator stress and pore pressure versus axial strain as well as effective stress paths.

The data for the cyclic tests show the development of maximum axial strain as well as pore pressure versus number of cycles.

Please do not hesitate to contact me if there are any questions regarding the test procedures or the results.

Sincerely yours,

Y. P. Vaid

Dr. Y.P. Vaid

:kl
Encl.

<u>Table 1: Test Information and Summary of Results</u>

Sample Tube	Test No.	Water Content (%)		σ'_{vc} (kPa)	σ'_{hc} (kPa)	Cyclic Loading				Precyclic Loading		Postcyclic Loading		Remarks
		w_i	w_c			$\sigma_{dcy}/2\sigma'_{hc}$	N	ε_{1max} (%)	$\Delta u_r/\sigma'_{3c}$	σ_d (kPa)	$^*\varepsilon_{1peak}$ (%)	σ_d (kPa)	$^*\varepsilon_{1peak}$ (%)	
Site 3 - Shelby 3	1	21.6	20.9	75	50	0.124	11	0.04				121	13.3	Post-cyclic compression
						0.144	11	0.08						
						0.192	10	0.40						
						0.243	18	2.63	0.759					
Site 3 - Shelby 3	2	23.1	20.6	75	75	0.150	12	-2.51	0.920			131	12.2	Post-cyclic compression-extension-compression

*no peaks in stress-strain curves. σ_d values noted at strain levels indicated during first compression cycle.

w_i = Initial water content

w_c = Water content at end of consolidation

σ'_{vc} = Vertical consolidation stress in triaxial test

σ'_{hc} = Radial consolidation stress in triaxial test

σ_v-σ_h = σ_d = Deviator stress (+)compression, (-)extension

$\sigma_{dcy}/2\sigma'_{hc}$ = Cyclic stress ratio in triaxial test

N = Number of cycles

ε_{1max} = Maximum amplitude of axial strain during cyclic loading in triaxial test, (+)compression, (-)extension

Δu_γ = Residual pore pressure at the end of cyclic loading

ε_{1peak} = Peak axial strain during pre-cyclic or post-cyclic monotonic loading in triaxial test

$Gw = Se$

$G = 3.2$

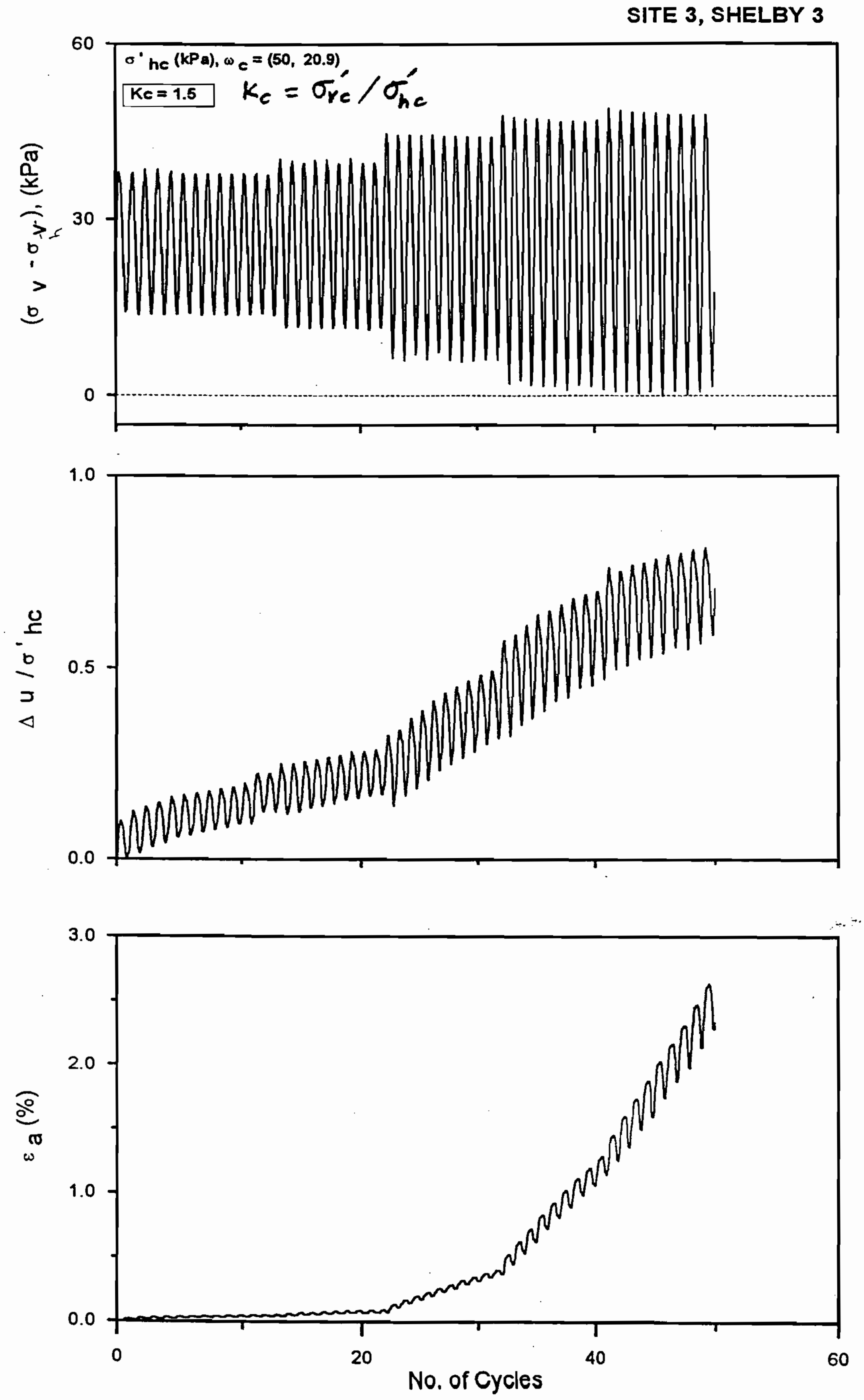
$S = 1$

$3.2 \times .209$; $e = 0.6688$

$\gamma_d = \frac{G}{1+e}\gamma_w = \frac{3.2}{1+.6688} \times 62.4 = \underline{119\,pcf} \approx 120\,pcf$ ok

$CSR = 0.65\, A_{max} \frac{\sigma_{vo}}{\sigma'_{vo}}\, r_d$

$\approx 0.65 \times 0.3 \times \frac{375}{300} \times 0.95 \approx 0.23$



SITE 3, SHELBY 3
σ ' hc (kPa), ω c = (50, 20.9)
Kc = 1.5
$K_c = \sigma'_{vc} / \sigma'_{hc}$
(σ v - σ h), (kPa)
60
30
0
Δ u / σ ' hc
1.0
0.5
0.0
ε a (%)
3.0
2.0
1.0
0.0
0
20
40
60
No. of Cycles

fig. 1 (a)

SITE 3, SHELBY 3

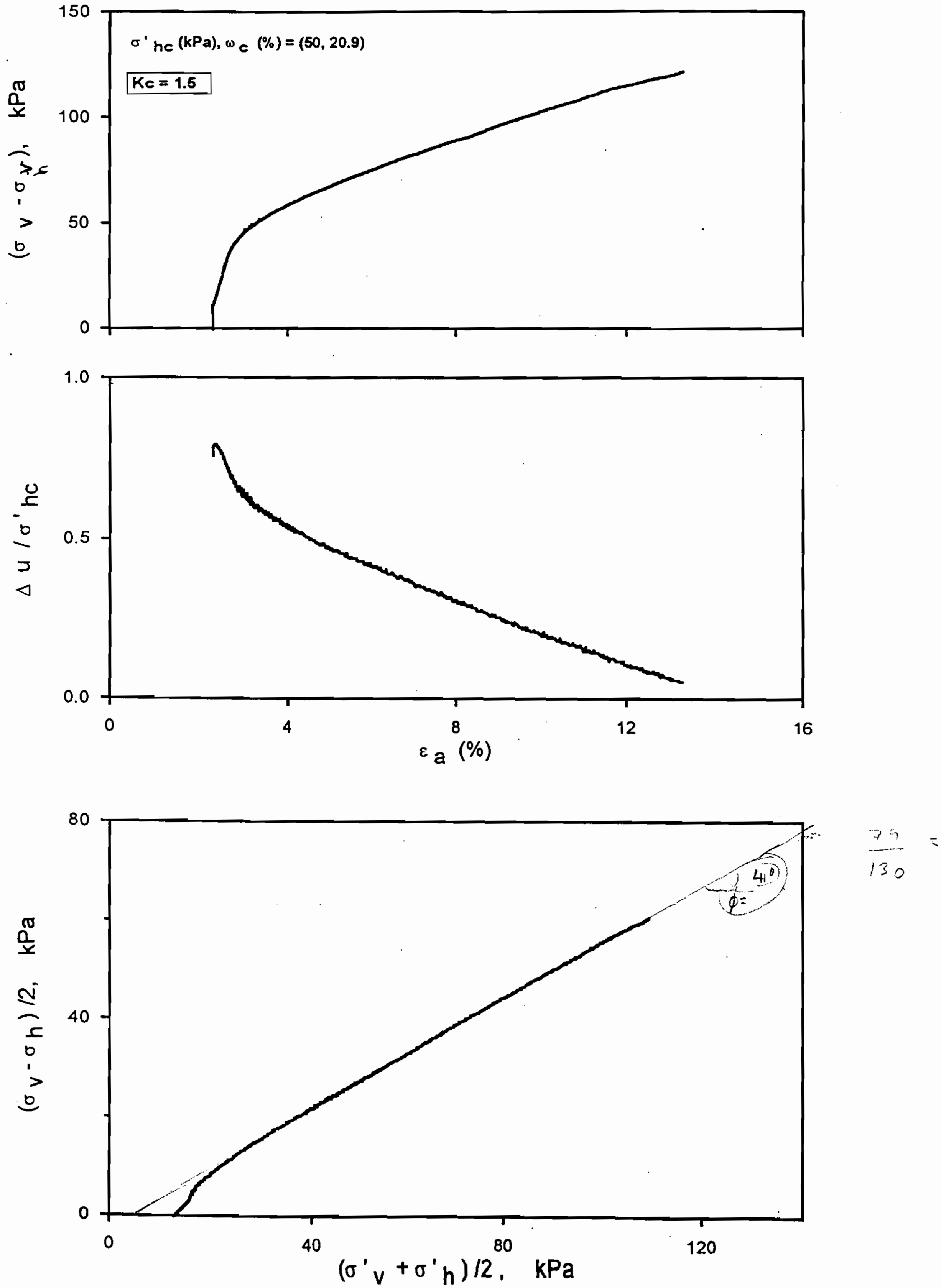


Fig 1(b)

SITE 3, SHELBY 3

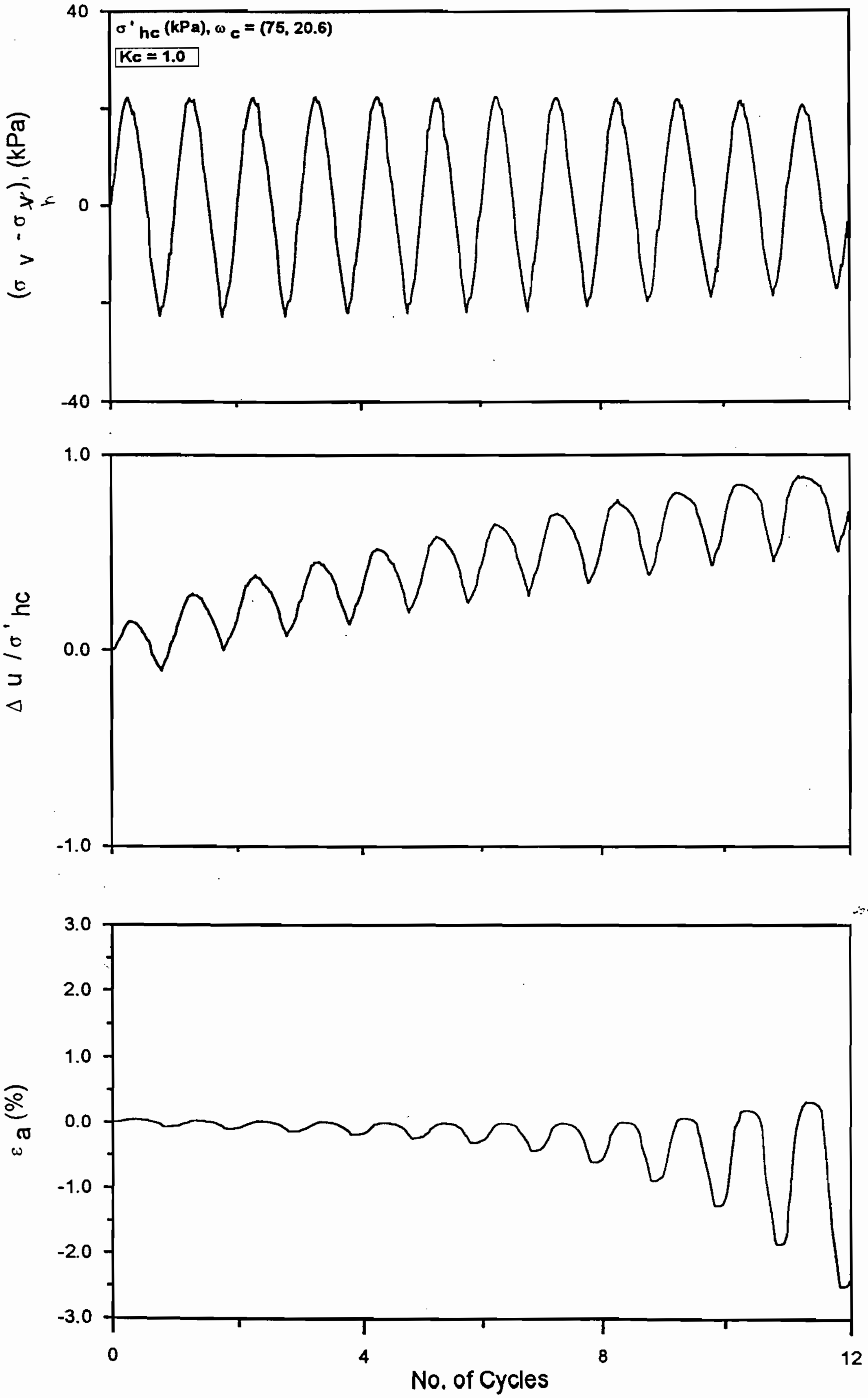


Fig. 2(a)

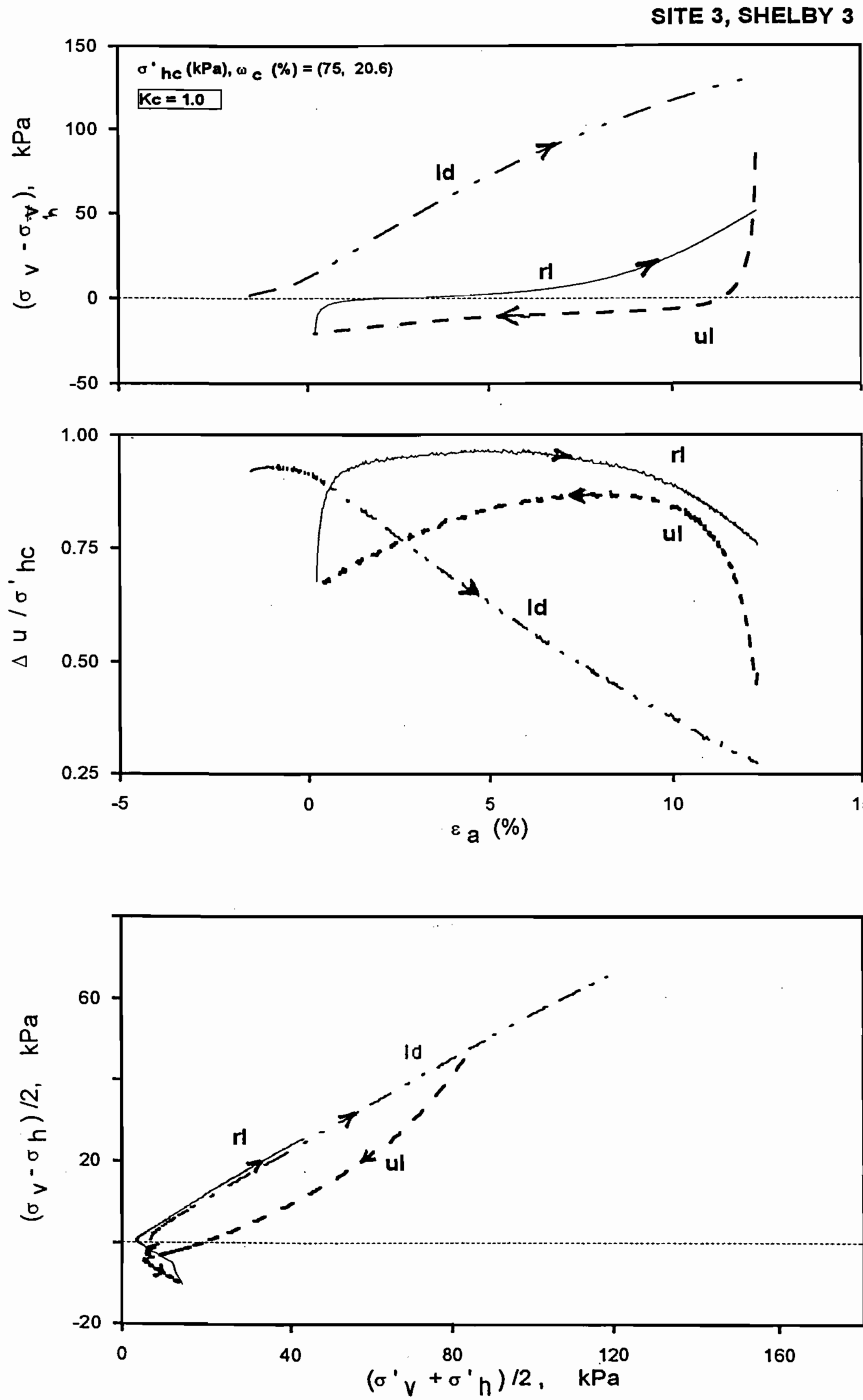

SITE 3, SHELBY 3
σ' hc (kPa), ω c (%) = (75, 20.6)
Kc = 1.0
(σ v - σ h), kPa
ld
rl
ul
Δ u / σ' hc
ε a (%)
(σ v - σ h) /2, kPa
(σ' v + σ' h) /2, kPa
150
100
50
0
-50
1.00
0.75
0.50
0.25
-5
5
10
15
60
20
-20
40
80
120
160

Fig. 2 (b)

APPENDIX III

Liquefaction Assessment - SPT

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TABLES

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(continued)

FIGURES

APPENDIX III
Tailings Liquefaction Potential Assessment SPT

III-1. INTRODUCTION

An assessment of liquefaction susceptibility of tailings and native sand at the Greens Creek Mine was evaluated under the Maximum Design Earthquake (MDE) and Design Basis Earthquake (DBE) loading. The procedure recommended in Youd et al. (2001) and Boulanger and Idriss (2004) were followed. The liquefaction resistance of the tailings and sand discussed in this appendix uses the methods based on Standard Penetration Test (SPT) data.

III-2. DESIGN EARTHQUAKE

The liquefaction evaluation was carried out for the MDE and the governing DBE. Klohn Crippen carried out a detailed probabilistic and deterministic seismic hazard analyses for the Greens Creek mine site in 1998 (Klohn Crippen, 1998). Based on these analyses, the MDE is a magnitude 7 (M7.0) earthquake with Peak Firm Ground Acceleration (PGA) of 0.3 g, and the governing DBE for liquefaction assessment is a magnitude 6.5 (M6.5) with a PGA of 0.15 g. The alternate DBE (M8.0, PGA 0.08 g) was also considered, but yields a FOS 1.1 times higher than the governing DBE based on the method recommended by Youd et al (2001).

Generally, the tailings storage facility is founded on rock or very shallow soil and no amplification of firm ground acceleration was applied to the calculations based on type and depth of soil overburden[1], as agreed upon by the Klohn Crippen design engineer and KGCMC.

III-3. PIEZOMETRIC CONDITIONS

The water level at the time of drilling is needed to calculate the effective stress at the test depth and was estimated either directly or indirectly. If a piezometer was installed in the borehole, the water level measured shortly after it was installed was applied to the SPT correction. Alternatively, if no piezometer was installed, any water level observations made during drilling were considered. If no water level information was noted on the borehole log, water levels in monitoring wells or piezometers located near the borehole at

[1] Ground surface accelerations could vary from borehole to borehole and depend on the type and depth of overburden for that hole, based on Figure 30 in Seed et al. (2001).

the time of drilling were assessed and applied to the SPT correction. This was done only if the water levels were found to be comparable in depth to the borehole in question, and the water table appeared to be typical for the main soil type in the borehole. For boreholes that had peat at the ground surface, the water level was assumed to be at the base of the peat layer. This avoids a mathematical nuance in the effective stress equations, since peat is buoyant, but does not significantly impact the effective stress calculation.

The liquefaction susceptibility of tailings was evaluated for the piezometric level estimated as discussed. No downward gradient was considered in the evaluation and hydrostatic condition was assumed.

III-4. ESTIMATION OF $(N_1)_{60CS}$ FROM MEASURED SPT BLOW COUNTS

$(N_1)_{60\text{-}cs}$[2] values were calculated from the field blow counts (N) with corrections including those for hammer efficiency, overburden pressure, and for fines content. $(N_1)_{60\text{-}cs}$ results for each SPT are presented in Table 6.1. Field blow counts (N) are shown on the drill hole logs in Appendix V.

Soil unit weights used in the analyses are presented in Table 4.1 and were previously reported in Table 5.1 of the Forest Service Submission Report (Klohn Crippen, 2004).

Table 4.1 Geotechnical Properties of Soil and Tailings

MATERIAL	IN SITU TOTAL UNIT WEIGHT (pcf)
Tailings in Old Facility	120
New Tailings (Current and Future)	128
Peat (Consolidated)	67
Gravelly Sand	120
Silty Clay	120
Silty Sandy Till	120
Compacted Rockfill	120
Sand and Geomembrane	125

(Klohn Crippen, 2004)

The measured SPT blow count N_m are corrected to obtain $(N_1)_{60}$ as follows (Youd et al., 2001):

$$(N_1)_{60} = N_m\, C_N C_E C_B C_R C_S$$

[2] $(N_1)_{60\text{-}cs}$ is the $(N_1)_{60}$ of an "equivalent clean sand" material.

in which C_N is the factor to normalize N_m to an effective overburden stress of 1 tsf, C_E is the correction for hammer energy ratio (ER), C_B is the correction factor for borehole diameter, C_R is the correction factor for rod length and C_S is the correction factor for samplers with or without liners.

The corrected $(N_1)_{60}$ values are then corrected for the effect of fines content to obtain the clean sand corrected SPT blow count $(N_1)_{60cs}$ as follows (Youd et al. 2001):

$$(N_1)_{60cs} = \alpha+\beta\ (N_1)_{60}$$

in which α and β are coefficients depending on the fines content (FC). For FC greater than or equal to 35% , the values of α and β are 5 and 1.2, respectively.

III-4.1 Correction Factor for Overburden Stress, C_N

Since SPT blow count value increases with increasing overburden stress, overburden correction factor, C_N is applied to normalize the measured SPT blow count to an effective overburden stress of approximately 1 tsf (Seed and Idriss, 1985). SPT during the field testing have been conducted at overburden stresses up to about 5 tsf.

Youd et al. (2001) recommends using the Liao and Whitman (1986) relationship for effective pressures less than 200 kPa:

$$C_N = (P_a/\sigma'_{vo})^{0.5}$$

Where P_a is atmospheric pressure of approximately 100 kPa, (1 tsf), and $/\sigma'_{vo}$ is the effective vertical overburden pressure. For effective vertical overburden pressures between 200 kPa and 300 kPa (2 to 3 tsf) Youd et al. (2001) recommends using the Seed and Idriss (1982) relationship:

$$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a)$$

although both relationships are acceptable.

Youd et al. (2001) indicates that the C_N correction factor is uncertain for effective overburden pressures greater than 3 tsf, particularly for the Liao and Whitman (1986) relationship.

An alternative method of calculating the C_N correction factor is presented in Boulanger and Idriss (2004):

$$C_N = (P_a/\sigma'_{vo})^{0.784-(0.0768)*\sqrt{[(N_1)_{60}]}}$$

where $(N_1)_{60}$ is limited to a maximum value of 46. Because $(N_1)_{60}$ is dependent on C_N as a correction factor, some iteration is involved in the calculation. $(N_1)_{60\text{-}cs}$ values calculated from, $(N_1)_{60}$, based on the Boulanger and Idriss (2004) (Id & B) method, are also presented on Tables 6.1 and 7.1 for comparison.

III-4.2 Correction for Hammer Energy Ratio (ER), C_E

An important factor in the interpretation of SPT blow counts is the energy transferred to SPT sampler from the falling hammer. An energy ratio (ER) of 60% is considered as the reference value for energy corrections and the correction factor for energy, C_E is defined as C_E =ER/60. The energy ratio delivered to the sampler depends on several factors including the type of hammer, hammer weight, drop height, lifting mechanism, size of the sampler etc.

Automatic (safety) hammers were used in the 2001, 2002, 2004 and 2005 SPT's. A safety hammer using a rope and cathead was used in 1997. SPT hammer energy calibration measurements were done in 2005, and indicated an average measured hammer efficiency of 68% (Robert Miner Dynamic Testing, 2005). An average hammer efficiency of 35% was measured for the 1997 SPT tests (ConeTec, 1997). A hammer efficiency of 60% was assumed for all remaining SPT tests.

III-4.3 Correction Factors, C_B, C_R, and C_S

The recommendations given in Youd et al. (2001) were closely followed to estimate the correction factors, C_B, C_R, and C_S. These factors are all close to 1 for the drilling methods used at Greens Creek.

III-4.4 Correction Factors for Fines Content (FC), α and β

The fines content for each SPT was based on either the laboratory grain size distribution results for that SPT, or an estimate based on typical laboratory results for that soil type.

III-5. LIQUEFACTION POTENTIAL

The earthquake induced Cyclic Stress Ratios (CSR) are compared to the Cyclic Resistance Ratios (CRR) to determine whether the liquefaction will be triggered or not during the design earthquake.

III-5.1 Earthquake Induced Cyclic Stress Ratio (CSR)

Seed's simplified method was used to determine the Cyclic Stress Ratio (CSR) induced by the earthquake. The peak firm ground acceleration (PGA) under MDE is 0.3 g and under DBE is 0.15 g. This value was applied at the surface of the tailings pile in all the analyses presented in this report.

The CSR is expressed as:

$$CSR = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\sigma_{vo}')\ r_d$$

in which a_{max} is the peak horizontal acceleration at the ground surface, g is the acceleration of gravity , σ_{vo} and σ_{vo}' are total and effective vertical stresses, respectively, and r_d is the stress reduction coefficient. The variation of r_d with depth proposed by Liao and Whitman (1986) and recommended in Youd et al. (2001) was used.

III-5.2 Cyclic Resistance Ratio (CRR)

III-5.2.1 Cyclic Resistance Ratio (CRR)

The $CRR_{7.5}$ was calculated from $(N_1)_{60\text{-}cs}$ using the relationship given in Youd et al (2001).

III-5.2.2 Correction Factors (K_m, K_σ and K_α) for CRR

The CRRs calculated using Youd et al. (2001) are applicable for magnitude M7.5 earthquake and for an overburden stress of 1 tsf. Therefore, the CRRs obtained from this calculation were corrected for the design earthquake magnitudes (MDE = M7.0, DBE = M6.5) and for the location specific overburden pressure. The earthquake magnitude correction factor, K_m, and overburden correction factor, K_σ, recommended in Youd et al. (2001) were used. In the estimation of $K_{\sigma,}$ the relative density, D_r of the tailings was taken as 50%. The correction factor for the initial static shear stress, K_α was assumed to be 1.0 as in common practice.

III-6. LIQUEFACTION OF SAND AND GRAVEL LAYER

A summary of the SPT tests done in the shallow sand and gravel layer below the peat, as well as the results of the liquefaction assessment are presented in Table 6.1 and Table 6.2.

Table 6.1 Sand and Gravel – SPT Reduction and Liquefaction Assessment Summary

Drill hole	SPT Name	Middle Depth (m)	Elevation (ft)	Field N	Effective Stress (tsf)	% Fines (used for correction)	$(N_1)_{60-cs}$ (NCEER)	$(N_1)_{60-cs}$ (Id&B)	Drill Rig	FOS - DBE (NCEER)	FOS - MDE (NCEER)	FOS - DBE (Id&B)	FOS - MDE (Id&B)	Location
DH-00-05	SPT-17	20.9	139.2	38.0	2.2	10.0	33	37	mud rotary	5.31	2.20	5.31	2.20	East Side
DH-00-05	SPT-16	20.1	141.7	46.0	2.1	10.0	40	47	mud rotary	5.20	2.15	5.20	2.15	East Side
DH-00-05	SPT-15	18.6	146.7	28.0	2.0	10.0	26	28	mud rotary	3.11	1.28	3.54	1.47	East Side
DH-00-05	SPT-14	17.1	151.7	27.0	1.8	5.0	24	26	mud rotary	2.75	1.14	3.09	1.28	East Side
DH-00-05	SPT-9	9.4	176.7	44.0	1.1	5.0	49	50	mud rotary	4.72	1.95	4.72	1.95	East Side
DH-00-04	SPT-6	4.9	180.9	59.0	0.7	5.0	50	65	hollow stem auger	5.43	2.25	5.43	2.25	East Side
DH-00-05	SPT-8	7.9	181.7	42.0	1.0	5.0	50	49	mud rotary	4.88	2.02	4.88	2.02	East Side
DH-00-04	SPT-5	4.0	183.9	48.0	0.6	5.0	50	53	hollow stem auger	5.79	2.40	5.79	2.40	East Side
DH-00-04	SPT-4	3.4	185.9	13.0	0.5	5.0	17	17	hollow stem auger	2.28	0.94	2.24	0.93	East Side
DH-00-05	SPT-7	6.4	186.7	40.0	0.8	5.0	50	49	mud rotary	5.08	2.10	5.08	2.10	East Side
DH-00-04	SPT-3	2.4	188.9	9.0	0.4	5.0	12	13	hollow stem auger	1.89	0.78	1.92	0.79	East Side
DH-00-04	SPT-2	1.8	190.9	11.0	0.4	5.0	17	16	hollow stem auger	2.68	1.11	2.63	1.09	East Side
DH-04-02	SPT-4	6.3	165.3	13.0	0.3	5.0	25	25	mud rotary	1.46	0.60	1.44	0.59	Northeast Expansion
DH-04-04	SPT-7	10.1	165.8	84.0	0.6	5.0	50	101	mud rotary	3.20	1.32	3.20	1.32	Northeast Expansion
DH-04-02	SPT-3	5.7	167.4	22.0	0.2	7.9	39	37	mud rotary	2.25	0.93	2.25	0.93	Northeast Expansion
DH-04-04	SPT-6	9.6	167.6	10.0	0.6	4.6	15	15	mud rotary	1.00	0.41	1.00	0.41	Northeast Expansion
DH-04-01	SPT-5	6.0	169.3	88.0	0.5	10.0	50	102	mud rotary	3.95	1.63	3.95	1.63	Northeast Expansion
DH-04-01	SPT-4	5.6	170.8	55.0	0.5	12.1	50	66	mud rotary	3.96	1.64	3.96	1.64	Northeast Expansion
DH-04-04	SPT-5	8.4	171.6	14.0	0.5	5.0	24	22	mud rotary	1.54	0.64	1.43	0.59	Northeast Expansion
DH-04-05	SPT-6	6.0	172.5	37.0	0.6	5.0	48	43	mud rotary	4.33	1.79	4.33	1.79	Northeast Expansion
DH-04-05	SPT-5	5.6	173.7	3.0	0.6	5.6	4	4	mud rotary	0.57	0.23	0.58	0.24	Northeast Expansion
DH-04-01	SPT-3	4.3	174.8	13.0	0.4	5.0	23	21	mud rotary	2.01	0.83	1.87	0.77	Northeast Expansion
DH-04-05	SPT-4	4.8	176.2	4.0	0.5	5.6	6	6	mud rotary	0.69	0.28	0.71	0.30	Northeast Expansion
DH-04-05	SPT-3	4.3	178.0	14.0	0.5	5.6	21	20	mud rotary	2.05	0.85	1.95	0.81	Northeast Expansion
DH-05-09	SPT-18	31.4	129.9	66.0	4.0	10.0	48	79	mud rotary	5.77	2.39	5.77	2.39	Old Tailings Pile
DH-05-09	SPT-17	30.8	131.9	35.0	3.9	29.2	33	35	mud rotary	5.82	2.41	5.82	2.41	Old Tailings Pile
DH-05-09	SPT-16	30.0	134.4	31.0	3.9	17.1	26	29	mud rotary	3.76	1.56	4.85	2.00	Old Tailings Pile
TB-1	SPT-17	13.3	136.4	12.3	1.6	15.0	15	15	hollow stem auger	1.63	0.67	1.65	0.68	Old Tailings Pile
DH-05-09	SPT-15	29.4	136.4	33.0	3.8	15.9	27	31	mud rotary	4.19	1.73	5.95	2.46	Old Tailings Pile
DH-05-08	SPT-18	27.1	144.6	40.0	3.9	13.3	32	40	mud rotary	6.50	2.69	6.50	2.69	Old Tailings Pile
DH-05-08	SPT-17	26.1	147.6	47.0	3.8	10.9	36	47	mud rotary	6.62	2.74	6.62	2.74	Old Tailings Pile
TA-3	SPT-16	12.4	160.3	23.2	1.6	5.0	23	24	hollow stem auger	2.57	1.06	2.73	1.13	Old Tailings Pile
DH-04-08	SPT-5	3.3	102.0	25.0	0.3	5.0	41	35	mud rotary	4.00	1.65	4.00	1.65	Pond 7
DH-04-08	SPT-4	2.7	104.0	34.0	0.3	10.0	50	45	mud rotary	4.06	1.68	4.06	1.68	Pond 7
DH-04-08	SPT-3	2.1	106.0	20.0	0.2	10.0	32	32	mud rotary	4.17	1.72	4.17	1.72	Pond 7
DH-04-08	SPT-2	1.5	108.0	3.0	0.2	5.0	5	5	mud rotary	0.60	0.25	0.60	0.25	Pond 7
DH-04-08	SPT-1	0.9	110.0	15.0	0.1	5.0	23	23	mud rotary	2.50	1.03	2.50	1.04	Pond 7
DH-05-11	SPT-19	26.3	127.0	26.0	4.7	13.9	20	21	mud rotary	7.43	3.07	3.47	1.44	South Side
BH97-2	SPT-7	10.3	131.5	30.0	0.9	10.0	24	24	mud rotary	2.53	1.05	2.46	1.02	South Side
DH-05-10	SPT-17	24.6	133.4	20.0	3.7	15.4	18	18	mud rotary	2.52	1.04	2.57	1.06	West Buttress
DH-05-10	SPT-16	24.0	135.3	81.0	3.7	7.1	50	117	mud rotary	6.73	2.78	6.73	2.78	West Buttress
DH-05-10	SPT-15	23.5	137.0	31.0	3.6	10.5	24	27	mud rotary	3.78	1.56	4.78	1.98	West Buttress
DH-05-13	SPT-5	6.1	138.3	28.0	1.1	7.0	35	35	mud rotary	6.84	2.83	6.84	2.83	West Buttress
BH97-1	SPT-2	2.4	141.9	19.0	0.3	10.0	18	18	hollow-stem auger	2.64	1.09	2.63	1.09	West Buttress

LOCATION	(N1)60-cs				FOS - MDE (NCEER)			
	Average	*Min*	*Max*	*Count*	*Average*	*Min*	*Max*	*Count*
East Side	34.9	12.4	50.0	12.0	1.7	0.8	2.4	12.0
Northeast Expansion	29.5	4.0	50.0	12.0	0.9	0.2	1.8	12.0
Old Tailings Pile	30.0	14.9	47.7	8.0	1.9	0.7	2.7	8.0
Pond 7	30.1	4.6	50.0	5.0	1.3	0.2	1.7	5.0
South Side	22.2	20.1	24.3	2.0	2.1	1.0	3.1	2.0
West Buttress	29.1	17.8	50.0	5.0	1.9	1.0	2.8	5.0

Table 6.2 Summary of Sand Liquefaction Assessment by Location

Location	Average $(N_1)_{60\text{-}cs}$	Average FOS (MDE)	No. of SPT's Conducted	Percentage of SPT's Indicating Liquefaction Under MDE (%)
East Side	34.9	1.7	12	17
Northeast Expansion (undeveloped)	29.5	0.9	12	67
Old Tailings Pile	30.0	1.9	8	25
South Side	30.1	2.1	2	50
West Buttress	22.2	1.9	5	40
Pond 7	29.1	1.3	5	40

Summary plots showing $(N_1)_{60\text{-}cs}$ vs. elevation and depth are on Figures 6.1 and 6.2. The FOS under the MDE vs. $(N_1)_{60\text{-}cs}$ is plotted on Figure 6.3. The figures also show that, depending on stress conditions, $(N_1)_{60\text{-}cs}$ values in the range 17 to 25 result in a factor of safety (FOS) under the MDE of 1.1.

FOS under DBE and MDE are plotted against elevation in Figures 6.4 and 6.5.

The simplified method of analysis was based on empirical results comparing average SPT results in a given layer to records of liquefaction. Hence, the liquefaction assessment in this report is also based on average SPT values. As shown on Table 6.1, liquefaction is not expected under the DBE. As also shown in Table 6.2, based on average SPT $(N_1)_{60\text{-}cs}$, the shallow sand and gravel layer is not liquefiable under MDE except in the undeveloped northeast expansion area. In the northeast expansion area, it is planned to remove the layer. Furthermore, significant softening is indicated in areas where the safety factor against liquefaction is less than 1.4, this is the case only in the Pond 7 location where the layer has been removed during the 2005 construction activities.

It should be noted that the liquefaction assessment for each borehole is based on the as-drilled elevation conditions, and does not take the ultimate tailings pile overburden stresses into consideration. In general, the impact of increased stress should be offset by tailings consolidation, however, there could also be changes in the ratio of effective and total stress which will depend on the location of the final water table. Consequently, the liquefaction assessment will need to be updated once the final site conditions are known.

III-7. LIQUEFACTION OF TAILINGS

A summary of the SPT tests done in tailings, as well as the results of the liquefaction assessment are presented in Table 7.1 and Table 7.2.

Table 7.1 Tailings – SPT Reduction and Liquefaction Assessment Summary

Drill hole	SPT Name	Middle Depth (m)	Elevation (ft)	Field N	Effective Stress (tsf)	% Fines (used for correction)	$(N_1)_{60-cs}$ (NCEER)	$(N_1)_{60-cs}$ (NCEER) <50 only	$(N_1)_{60-cs}$ (Id&B)	Stability Soil Unit	Drill Rig	FOS - DBE (NCEER)	FOS - MDE (NCEER)	FOS - DBE (Id&B)	FOS - MDE (Id&B)	Location
DH-02-05	SPT-1	1.8	221.5	5	0.4	85.0	14.1	14.1	13.2	new tails	mud rotary	2.265	0.937	2.132	0.882	East Side
DH-02-05	SPT-2	3.3	216.5	10	0.7	85.0	19.2	19.2	17.5	new tails	mud rotary	3.118	1.290	2.829	1.170	East Side
DH-02-05	SPT-3	4.8	211.5	15	1.0	85.0	23.7	23.7	21.2	new tails	mud rotary	4.152	1.717	3.574	1.478	East Side
DH-02-05	SPT-4	6.3	206.5	18	1.3	85.0	26.8	26.8	24.0	new tails	mud rotary	4.885	2.021	4.010	1.659	East Side
DH-02-05	SPT-5	7.8	201.5	33	1.6	85.0	40.9	40.9	37.8	new tails	mud rotary	7.019	2.903	7.019	2.903	East Side
DH-02-05	SPT-6	9.4	196.5	100	2.0	50.0	104.6		123.2	new tails	mud rotary	6.832	2.826	6.832	2.826	East Side
DH-02-05	SPT-7	10.9	191.5	5	2.2	85.0	14.1	14.1	12.7	new tails	mud rotary	2.027	0.838	1.852	0.766	East Side
DH-02-08	SPT-1	1.8	189.0	4	0.4	85.0	12.2	12.2	11.6	new tailings	mud rotary/HQ core	2.001	0.827	1.918	0.793	Old Tailings Pile
DH-02-10	SPT-1	1.8	230.1	17	0.3	80.0	36.2	36.2	29.6	new tailings	mud rotary	7.496	3.100	6.624	2.740	Old Tailings Pile
DH-05-08	SPT-2	3.0	223.6	42	0.6	85.0	88.5		64.0	new tailings	mud rotary	7.566	3.129	7.566	3.129	Old Tailings Pile
DH-05-08	SPT-3	4.5	218.6	20	0.9	85.0	40.5	40.5	34.6	new tailings	mud rotary	7.658	3.167	7.658	3.167	Old Tailings Pile
DH-05-08	SPT-4	6.2	213.1	6	1.2	84.1	13.7	13.7	12.7	new tailings	mud rotary	2.197	0.909	2.063	0.853	Old Tailings Pile
DH-05-09	SPT-1	3.0	223.2	3	0.6	77.0	9.9	9.9	10.0	new tailings	mud rotary	1.699	0.703	1.709	0.707	Old Tailings Pile
DH-05-09	SPT-2	4.9	217.0	10	1.0	77.0	19.5	19.5	17.7	new tailings	mud rotary	3.213	1.329	2.900	1.199	Old Tailings Pile
DH-05-09	SPT-3	5.8	213.9	11	1.1	77.1	19.6	19.6	17.8	new tailings	mud rotary	3.182	1.316	2.864	1.184	Old Tailings Pile
DH-05-09	SPT-4	9.1	203.2	15	1.8	80.0	22.8	22.8	20.6	new tailings	mud rotary	3.525	1.458	3.095	1.280	Old Tailings Pile
DH-05-09	SPT-5	12.2	192.9	18	2.3	84.4	25.0	25.0	22.7	new tailings	mud rotary	3.939	1.629	3.418	1.414	Old Tailings Pile
DH-02-04	SPT-1	3.3	225.9	22	0.7	80.0	36.2	36.2	30.6	new tails	hollow stem auger	7.585	3.137	7.585	3.137	Old Tailings Pile
DH-02-04	SPT-2	6.3	215.9	15	1.3	80.0	23.2	23.2	20.9	new tails	hollow stem auger	3.805	1.574	3.314	1.371	Old Tailings Pile
BH97-3	SPT-1	1.4	200.2	100	0.3	80.0	112.1		70.6	old tailings	mud rotary	7.477	3.093	7.477	3.093	Old Tailings Pile
BH97-3	SPT-2	3.0	195.2	17	0.6	80.0	19.3	19.3	17.7	old tailings	mud rotary	3.132	1.296	2.846	1.177	Old Tailings Pile
BH97-3	SPT-3	4.5	190.2	37	0.9	80.0	33.7	33.7	29.2	old tailings	mud rotary	7.658	3.167	6.457	2.670	Old Tailings Pile
BH97-3	SPT-4	6.3	184.2	12	1.2	80.0	13.8	13.8	12.8	old tailings	mud rotary	2.200	0.910	2.063	0.853	Old Tailings Pile
BH97-3	SPT-5	9.1	175.2	12	1.8	80.0	12.3	12.3	11.4	old tailings	mud rotary	1.865	0.771	1.744	0.721	Old Tailings Pile
BH97-3	SPT-6	12.7	163.2	14	2.5	80.0	12.6	12.6	11.4	old tailings	mud rotary	1.947	0.805	1.790	0.740	Old Tailings Pile
BH97-3	SPT-7	15.5	154.2	18	3.0	80.0	13.9	13.9	12.4	old tailings	mud rotary	2.204	0.912	1.991	0.824	Old Tailings Pile
DH-02-04	SPT-3	9.4	205.9	20	2.0	80.0	25.0	25.0	22.7	old tailings	hollow stem auger	3.993	1.651	3.447	1.426	Old Tailings Pile
DH-02-04	SPT-4	12.4	195.9	30	2.6	80.0	32.6	32.6	30.9	old tailings	hollow stem auger	7.018	2.903	7.018	2.903	Old Tailings Pile
DH-02-04	SPT-5	15.5	185.9	25	3.2	80.0	25.7	25.7	23.8	old tailings	hollow stem auger	4.519	1.869	3.973	1.643	Old Tailings Pile
DH-02-04	SPT-6	18.5	175.9	6	3.6	96.3	9.6	9.6	8.8	old tailings	hollow stem auger	1.689	0.699	1.571	0.650	Old Tailings Pile
DH-02-08	SPT-2	3.3	184.0	10	0.7	85.0	19.2	19.2	17.5	old tailings	mud rotary/HQ core	3.113	1.288	2.825	1.169	Old Tailings Pile
DH-02-08	SPT-3	4.8	179.0	3	1.0	86.0	8.8	8.8	8.7	old tailings	mud rotary/HQ core	1.573	0.650	1.566	0.648	Old Tailings Pile
DH-02-08	SPT-4	6.3	174.0	4	1.3	85.0	9.9	9.9	9.6	old tailings	mud rotary/HQ core	1.657	0.685	1.609	0.666	Old Tailings Pile
DH-02-08	SPT-5	7.9	169.0	5	1.6	85.0	10.5	10.5	9.9	old tailings	mud rotary/HQ core	1.670	0.691	1.596	0.660	Old Tailings Pile
DH-02-08	SPT-6	9.4	164.0	7	1.9	85.0	12.2	12.2	11.2	old tailings	mud rotary/HQ core	1.812	0.750	1.692	0.700	Old Tailings Pile
DH-02-08	SPT-7	10.9	159.0	8	2.0	85.0	13.3	13.3	12.1	old tailings	mud rotary/HQ core	1.870	0.774	1.725	0.713	Old Tailings Pile
DH-02-08	SPT-8	12.4	154.0	33	2.2	85.0	38.1	38.1	36.2	old tailings	mud rotary/HQ core	6.329	2.618	6.329	2.618	Old Tailings Pile
DH-02-08	SPT-9	14.0	149.0	10	2.3	85.0	14.7	14.7	13.2	old tailings	mud rotary/HQ core	1.959	0.810	1.781	0.736	Old Tailings Pile
DH-02-08	SPT-10	15.5	144.0	51	2.4	85.0	53.1		55.6	old tailings	mud rotary/HQ core	6.204	2.566	6.204	2.566	Old Tailings Pile
DH-02-10	SPT-2	3.3	225.1	21	0.6	80.0	35.7	35.7	30.1	old tailings	mud rotary	7.585	3.137	7.585	3.137	Old Tailings Pile
DH-02-10	SPT-3	4.8	220.1	34	0.9	80.0	48.7	48.7	41.5	old tailings	mud rotary	7.677	3.175	7.677	3.175	Old Tailings Pile
DH-02-10	SPT-4	6.3	215.1	40	1.2	80.0	55.1		49.1	old tailings	mud rotary	7.433	3.074	7.433	3.074	Old Tailings Pile
DH-02-10	SPT-5	7.9	210.1	42	1.5	80.0	52.2		48.6	old tailings	mud rotary	7.130	2.949	7.130	2.949	Old Tailings Pile
DH-02-10	SPT-6	9.4	205.1	24	1.8	80.0	29.7	29.7	27.1	old tailings	mud rotary	6.205	2.566	4.740	1.960	Old Tailings Pile
DH-02-10	SPT-7	10.9	200.1	22	2.1	80.0	27.1	27.1	24.8	old tailings	mud rotary	4.769	1.972	4.024	1.665	Old Tailings Pile
DH-02-10	SPT-8	12.4	195.1	34	2.4	80.0	37.0	37.0	35.7	old tailings	mud rotary	7.098	2.936	7.098	2.936	Old Tailings Pile
DH-02-10	SPT-9	14.0	190.1	13	2.7	80.0	16.5	16.5	14.8	old tailings	mud rotary	2.549	1.054	2.286	0.945	Old Tailings Pile
DH-02-10	SPT-10	15.5	185.1	27	3.0	80.0	27.8	27.8	26.0	old tailings	mud rotary	5.375	2.223	4.643	1.920	Old Tailings Pile
DH-02-10	SPT-11	17.0	180.1	14	3.3	80.0	16.3	16.3	14.4	old tailings	mud rotary	2.655	1.098	2.367	0.979	Old Tailings Pile
DH-02-10	SPT-12	18.5	175.1	33	3.5	50.0	31.0	31.0	30.0	old tailings	mud rotary	7.717	3.192	7.717	3.192	Old Tailings Pile
DH-05-08	SPT-5	7.5	208.6	10	1.5	85.0	18.9	18.9	17.1	old tailings	mud rotary	2.903	1.201	2.613	1.081	Old Tailings Pile
DH-05-08	SPT-6	9.1	203.6	8	1.8	95.0	14.7	14.7	13.3	old tailings	mud rotary	2.182	0.902	1.999	0.827	Old Tailings Pile
DH-05-08	SPT-7	10.6	198.6	22	2.1	95.0	30.0	30.0	27.6	old tailings	mud rotary	6.980	2.887	4.988	2.063	Old Tailings Pile
DH-05-08	SPT-8	12.4	192.6	14	2.4	93.6	20.2	20.2	18.1	old tailings	mud rotary	3.084	1.276	2.737	1.132	Old Tailings Pile
DH-05-08	SPT-9	13.9	187.6	18	2.7	95.0	23.4	23.4	21.2	old tailings	mud rotary	3.730	1.543	3.278	1.356	Old Tailings Pile
DH-05-08	SPT-10	15.5	182.6	5	2.8	95.0	10.0	10.0	9.2	old tailings	mud rotary	1.591	0.658	1.488	0.615	Old Tailings Pile
DH-05-08	SPT-11	17.0	177.6	8	3.0	95.0	12.9	12.9	11.5	old tailings	mud rotary	1.950	0.806	1.769	0.732	Old Tailings Pile
DH-05-08	SPT-12	18.5	172.6	9	3.1	95.7	13.6	13.6	12.1	old tailings	mud rotary	2.070	0.856	1.864	0.771	Old Tailings Pile
DH-05-08	SPT-13	20.0	167.6	16	3.2	95.0	19.8	19.8	17.7	old tailings	mud rotary	3.056	1.264	2.705	1.119	Old Tailings Pile
DH-05-08	SPT-14	21.6	162.6	9	3.4	95.0	13.5	13.5	11.9	old tailings	mud rotary	2.139	0.884	1.920	0.794	Old Tailings Pile
DH-05-08	SPT-15	23.1	157.6	31	3.5	95.0	32.5	32.5	31.9	old tailings	mud rotary	7.062	2.921	7.062	2.921	Old Tailings Pile

Table 7.1 Tailings – SPT Reduction and Liquefaction Assessment Summary (cont'd)

Drill hole	SPT Name	Middle Depth (m)	Elevation (ft)	Field N	Effective Stress (tsf)	% Fines (used for correction)	$(N_1)_{60\text{-}cs}$ (NCEER)	$(N_1)_{60\text{-}cs}$ (NCEER) <50 only	$(N_1)_{60\text{-}cs}$ (Id&B)	Stability Soil Unit	Drill Rig	FOS - DBE (NCEER)	FOS - MDE (NCEER)	FOS - DBE (Id&B)	FOS - MDE (Id&B)	Location
DH-05-08	SPT-16	24.6	152.6	18	3.7	96.1	20.2	20.2	18.1	old tailings	mud rotary	2.975	1.230	2.628	1.087	Old Tailings Pile
DH-05-09	SPT-6	15.3	182.9	33	2.6	87.0	39.5	39.5	38.8	old tailings	mud rotary	6.553	2.710	6.553	2.710	Old Tailings Pile
DH-05-09	SPT-7	18.3	172.9	12	2.8	89.7	16.9	16.9	15.0	old tailings	mud rotary	2.377	0.983	2.123	0.878	Old Tailings Pile
DH-05-09	SPT-8	21.3	163.0	17	3.1	90.0	21.0	21.0	18.9	old tailings	mud rotary	3.167	1.310	2.803	1.159	Old Tailings Pile
DH-05-09	SPT-9	24.4	152.9	6	3.4	93.0	10.4	10.4	9.4	old tailings	mud rotary	1.522	0.629	1.405	0.581	Old Tailings Pile
DH-05-09	SPT-10	25.7	148.6	50	3.5	95.0	49.3	49.3	56.8	old tailings	mud rotary	6.350	2.626	6.350	2.626	Old Tailings Pile
DH-05-09	SPT-11	26.9	144.6	12	3.7	95.0	15.5	15.5	13.6	old tailings	mud rotary	2.047	0.847	1.817	0.751	Old Tailings Pile
DH-05-09	SPT-12	27.6	142.5	14	3.7	95.8	17.1	17.1	15.1	old tailings	mud rotary	2.236	0.925	1.977	0.818	Old Tailings Pile
DH-05-09	SPT-13	28.3	140.0	19	3.8	50.0	21.3	21.3	19.3	old tailings	mud rotary	2.815	1.164	2.506	1.036	Old Tailings Pile
TA-1	SPT-1	1.4	176.1	9	0.3	80.0	21.6	21.6	19.4	old tailings	hollow stem auger	3.531	1.461	3.105	1.284	Old Tailings Pile
TA-1	SPT-2	2.2	173.6	29	0.4	80.0	53.6		40.6	old tailings	hollow stem auger	7.522	3.111	7.522	3.111	Old Tailings Pile
TA-1	SPT-3	3.0	171.1	8	0.6	80.0	17.1	17.1	15.9	old tailings	hollow stem auger	2.753	1.139	2.563	1.060	Old Tailings Pile
TA-1	SPT-4	3.7	168.6	3	0.7	80.0	8.5	8.5	8.7	old tailings	hollow stem auger	1.530	0.633	1.548	0.640	Old Tailings Pile
TA-1	SPT-5	4.5	166.1	2	0.8	80.0	7.7	7.7	7.9	old tailings	hollow stem auger	1.306	0.540	1.331	0.551	Old Tailings Pile
TA-1	SPT-6	5.3	163.6	2	0.9	80.0	7.6	7.6	7.8	old tailings	hollow stem auger	1.212	0.501	1.233	0.510	Old Tailings Pile
TA-1	SPT-7	6.0	161.1	3	1.0	80.0	8.7	8.7	8.7	old tailings	hollow stem auger	1.269	0.525	1.268	0.525	Old Tailings Pile
TA-1	SPT-8	6.8	158.6	6	1.0	80.0	13.0	13.0	12.3	old tailings	hollow stem auger	1.693	0.700	1.610	0.666	Old Tailings Pile
TA-1	SPT-9	7.5	156.1	4	1.1	80.0	10.2	10.2	9.9	old tailings	hollow stem auger	1.312	0.543	1.281	0.530	Old Tailings Pile
TA-1	SPT-10	8.3	153.6	5	1.2	80.0	10.8	10.8	10.4	old tailings	hollow stem auger	1.323	0.547	1.279	0.529	Old Tailings Pile
TA-1	SPT-11	9.1	151.1	6	1.2	80.0	13.1	13.1	12.2	old tailings	hollow stem auger	1.496	0.619	1.412	0.584	Old Tailings Pile
TA-2	SPT-1	0.8	190.1	23	0.2	80.0	34.5	34.5	40.1	old tailings	hollow stem auger	7.442	3.078	7.442	3.078	Old Tailings Pile
TA-2	SPT-2	1.6	187.6	4	0.3	80.0	5.9	5.9	11.5	old tailings	hollow stem auger	1.979	0.818	1.891	0.782	Old Tailings Pile
TA-2	SPT-3	2.4	185.1	4	0.5	80.0	5.7	5.7	11.3	old tailings	hollow stem auger	1.874	0.775	1.872	0.774	Old Tailings Pile
TA-2	SPT-4	3.1	182.6	5	0.6	80.0	6.0	6.0	11.5	old tailings	hollow stem auger	1.957	0.809	1.923	0.795	Old Tailings Pile
TA-2	SPT-5	3.9	180.1	3	0.8	80.0	3.1	3.1	8.6	old tailings	hollow stem auger	1.523	0.630	1.540	0.637	Old Tailings Pile
TA-2	SPT-6	4.6	177.6	3	0.9	80.0	3.6	3.6	9.2	old tailings	hollow stem auger	1.630	0.674	1.621	0.671	Old Tailings Pile
TA-2	SPT-7	5.7	174.1	10	1.1	80.0	9.5	9.5	15.1	old tailings	hollow stem auger	2.650	1.096	2.443	1.011	Old Tailings Pile
TA-2	SPT-8	6.2	172.6	5	1.2	80.0	5.4	5.4	11.0	old tailings	hollow stem auger	1.898	0.785	1.818	0.752	Old Tailings Pile
TA-2	SPT-9	6.9	170.1	14	1.3	80.0	13.7	13.7	19.3	old tailings	hollow stem auger	3.359	1.389	2.976	1.231	Old Tailings Pile
TA-2	SPT-10	7.7	167.6	4	1.4	80.0	3.7	3.7	9.2	old tailings	hollow stem auger	1.484	0.614	1.444	0.597	Old Tailings Pile
TA-2	SPT-11	8.5	165.1	3	1.5	80.0	2.9	2.9	8.5	old tailings	hollow stem auger	1.315	0.544	1.292	0.534	Old Tailings Pile
TA-2	SPT-12	9.2	162.6	5	1.6	80.0	4.6	4.6	10.2	old tailings	hollow stem auger	1.486	0.615	1.419	0.587	Old Tailings Pile
TA-2	SPT-13	10.0	160.1	6	1.6	80.0	5.7	5.7	11.2	old tailings	hollow stem auger	1.593	0.659	1.499	0.620	Old Tailings Pile
TA-2	SPT-14	10.7	157.6	4	1.7	80.0	3.4	3.4	9.0	old tailings	hollow stem auger	1.277	0.528	1.236	0.511	Old Tailings Pile
TA-3	SPT-1	1.0	197.8	8	0.2	80.0	20.4	20.4	18.4	old tailings	hollow stem auger	3.285	1.359	2.922	1.209	Old Tailings Pile
TA-3	SPT-2	1.8	195.3	11	0.3	80.0	25.1	25.1	22.3	old tailings	hollow stem auger	4.416	1.826	3.682	1.523	Old Tailings Pile
TA-3	SPT-3	2.5	192.8	4	0.5	80.0	11.1	11.1	11.1	old tailings	hollow stem auger	1.850	0.765	1.848	0.764	Old Tailings Pile
TA-3	SPT-4	3.3	190.3	5	0.6	80.0	11.6	11.6	11.4	old tailings	hollow stem auger	1.937	0.801	1.903	0.787	Old Tailings Pile
TA-3	SPT-5	4.0	187.8	3	0.8	80.0	8.7	8.7	8.8	old tailings	hollow stem auger	1.505	0.623	1.517	0.628	Old Tailings Pile
TA-3	SPT-6	4.8	185.3	11	0.8	80.0	19.9	19.9	18.1	old tailings	hollow stem auger	2.941	1.216	2.645	1.094	Old Tailings Pile
TA-3	SPT-7	5.6	182.8	3	0.9	80.0	9.2	9.2	9.2	old tailings	hollow stem auger	1.372	0.568	1.365	0.565	Old Tailings Pile
TA-3	SPT-8	6.3	180.3	4	1.0	80.0	10.4	10.4	10.1	old tailings	hollow stem auger	1.443	0.597	1.410	0.583	Old Tailings Pile
TA-3	SPT-9	7.1	177.8	3	1.1	80.0	8.5	8.5	8.5	old tailings	hollow stem auger	1.185	0.490	1.183	0.489	Old Tailings Pile
TA-3	SPT-10	7.8	175.3	3	1.1	80.0	9.2	9.2	9.1	old tailings	hollow stem auger	1.202	0.497	1.186	0.490	Old Tailings Pile
TA-3	SPT-11	8.6	172.8	5	1.2	80.0	10.8	10.8	10.3	old tailings	hollow stem auger	1.300	0.538	1.256	0.519	Old Tailings Pile
TA-3	SPT-12	9.4	170.3	4	1.3	80.0	9.8	9.8	9.5	old tailings	hollow stem auger	1.171	0.485	1.141	0.472	Old Tailings Pile
TA-3	SPT-13	10.1	167.8	5	1.3	80.0	11.5	11.5	10.9	old tailings	hollow stem auger	1.317	0.545	1.256	0.519	Old Tailings Pile
TA-3	SPT-14	10.9	165.3	4	1.4	80.0	9.8	9.8	9.4	old tailings	hollow stem auger	1.141	0.472	1.107	0.458	Old Tailings Pile
TA-3	SPT-15	11.7	162.8	8	1.5	80.0	15.1	15.1	13.8	old tailings	hollow stem auger	1.642	0.679	1.514	0.626	Old Tailings Pile
TA-4	SPT-1	1.3	202.6	8	0.3	80.0	20.4	20.4	18.4	old tailings	hollow stem auger	3.293	1.362	2.929	1.211	Old Tailings Pile
TA-4	SPT-2	2.1	200.1	5	0.4	80.0	12.8	12.8	12.5	old tailings	hollow stem auger	2.088	0.864	2.034	0.841	Old Tailings Pile
TA-4	SPT-3	2.8	197.6	4	0.6	80.0	10.7	10.7	10.7	old tailings	hollow stem auger	1.809	0.748	1.805	0.746	Old Tailings Pile
TA-4	SPT-4	3.6	195.1	3	0.7	80.0	8.6	8.6	8.8	old tailings	hollow stem auger	1.538	0.636	1.557	0.644	Old Tailings Pile
TA-4	SPT-5	4.3	192.6	6	0.9	80.0	13.7	13.7	13.0	old tailings	hollow stem auger	2.257	0.933	2.147	0.888	Old Tailings Pile
TA-4	SPT-6	5.1	190.1	5	1.0	80.0	11.4	11.4	11.0	old tailings	hollow stem auger	1.958	0.810	1.893	0.783	Old Tailings Pile
TA-4	SPT-7	5.9	187.6	5	1.1	80.0	10.3	10.3	10.0	old tailings	hollow stem auger	1.717	0.710	1.671	0.691	Old Tailings Pile
TA-4	SPT-8	6.6	185.1	4	1.2	80.0	9.9	9.9	9.6	old tailings	hollow stem auger	1.555	0.643	1.517	0.627	Old Tailings Pile
TA-4	SPT-9	7.4	182.6	5	1.3	80.0	10.6	10.6	10.1	old tailings	hollow stem auger	1.542	0.638	1.489	0.616	Old Tailings Pile

Table 7.1 Tailings – SPT Reduction and Liquefaction Assessment Summary (cont'd)

Drill hole	SPT Name	Middle Depth (m)	Elevation (ft)	Field N	Effective Stress (tsf)	% Fines (used for correction)	$(N_1)_{60\text{-}cs}$ (NCEER)	$(N_1)_{60\text{-}cs}$ (NCEER) <50 only	$(N_1)_{60\text{-}cs}$ (Id&B)	Stability Soil Unit	Drill Rig	FOS - DBE (NCEER)	FOS - MDE (NCEER)	FOS - DBE (Id&B)	FOS - MDE (Id&B)	Location
TA-4	SPT-10	8.2	180.1	10	1.3	80.0	16.7	16.7	15.2	old tailings	hollow stem auger	2.196	0.908	2.011	0.832	Old Tailings Pile
TA-4	SPT-11	8.9	177.6	5	1.4	80.0	11.1	11.1	10.5	old tailings	hollow stem auger	1.453	0.601	1.389	0.575	Old Tailings Pile
TA-4	SPT-12	9.7	175.1	5	1.5	80.0	10.2	10.2	9.7	old tailings	hollow stem auger	1.322	0.547	1.274	0.527	Old Tailings Pile
TA-4	SPT-13	10.4	172.6	3	1.6	80.0	8.8	8.8	8.6	old tailings	hollow stem auger	1.163	0.481	1.139	0.471	Old Tailings Pile
TA-4	SPT-14	11.2	170.1	5	1.6	80.0	10.2	10.2	9.7	old tailings	hollow stem auger	1.284	0.531	1.232	0.510	Old Tailings Pile
TA-4	SPT-15	12.0	167.6	10	1.7	80.0	15.9	15.9	14.5	old tailings	hollow stem auger	1.874	0.775	1.712	0.708	Old Tailings Pile
TA-5	SPT-1	1.0	204.0	19	0.2	80.0	37.1	37.1	34.2	old tailings	hollow stem auger	7.451	3.082	7.451	3.082	Old Tailings Pile
TA-5	SPT-2	1.9	201.0	2	0.4	80.0	7.9	7.9	8.5	old tailings	hollow stem auger	1.433	0.593	1.504	0.622	Old Tailings Pile
TA-5	SPT-3	2.8	198.0	6	0.6	80.0	13.9	13.9	13.9	old tailings	hollow stem auger	2.248	0.930	2.253	0.932	Old Tailings Pile
TA-5	SPT-4	3.7	195.0	8	0.7	80.0	16.2	16.2	15.4	old tailings	hollow stem auger	2.618	1.083	2.492	1.030	Old Tailings Pile
TA-5	SPT-5	4.3	193.0	23	0.9	80.0	35.1	35.1	30.6	old tailings	hollow stem auger	7.649	3.163	7.649	3.163	Old Tailings Pile
TA-5	SPT-6	5.1	190.5	12	1.0	80.0	20.3	20.3	18.4	old tailings	hollow stem auger	3.399	1.406	3.047	1.260	Old Tailings Pile
TA-5	SPT-7	5.9	188.0	10	1.1	80.0	16.4	16.4	15.0	old tailings	hollow stem auger	2.602	1.076	2.394	0.990	Old Tailings Pile
TA-5	SPT-8	6.6	185.5	5	1.2	80.0	11.6	11.6	11.0	old tailings	hollow stem auger	1.773	0.733	1.695	0.701	Old Tailings Pile
TA-5	SPT-9	7.4	183.0	5	1.3	80.0	10.6	10.6	10.1	old tailings	hollow stem auger	1.554	0.643	1.497	0.619	Old Tailings Pile
TA-5	SPT-10	8.2	180.5	5	1.4	80.0	11.2	11.2	10.6	old tailings	hollow stem auger	1.547	0.640	1.477	0.611	Old Tailings Pile
TA-5	SPT-11	8.9	178.0	4	1.4	80.0	9.5	9.5	9.2	old tailings	hollow stem auger	1.301	0.538	1.265	0.523	Old Tailings Pile
TA-5	SPT-12	9.7	175.5	4	1.5	80.0	9.4	9.4	9.1	old tailings	hollow stem auger	1.254	0.519	1.219	0.504	Old Tailings Pile
TA-5	SPT-13	10.4	173.0	5	1.6	80.0	11.0	11.0	10.4	old tailings	hollow stem auger	1.396	0.577	1.330	0.550	Old Tailings Pile
TB-1	SPT-1	1.0	176.9	6	0.2	80.0	16.8	16.8	15.4	old tailings	hollow stem auger	2.670	1.104	2.448	1.012	Old Tailings Pile
TB-1	SPT-2	1.9	173.9	7	0.4	80.0	17.8	17.8	16.4	old tailings	hollow stem auger	2.844	1.176	2.618	1.083	Old Tailings Pile
TB-1	SPT-3	2.5	171.9	3	0.5	80.0	9.0	9.0	9.3	old tailings	hollow stem auger	1.581	0.654	1.614	0.668	Old Tailings Pile
TB-1	SPT-4	3.3	169.4	3	0.6	80.0	9.7	9.7	9.8	old tailings	hollow stem auger	1.680	0.695	1.685	0.697	Old Tailings Pile
TB-1	SPT-5	4.0	166.9	3	0.8	80.0	9.5	9.5	9.5	old tailings	hollow stem auger	1.663	0.688	1.657	0.685	Old Tailings Pile
TB-1	SPT-6	4.8	164.4	65	0.9	80.0	91.1		73.2	old tailings	hollow stem auger	7.135	2.951	7.135	2.951	Old Tailings Pile
TB-1	SPT-7	5.7	161.4	8	1.0	80.0	15.7	15.7	14.5	old tailings	hollow stem auger	2.214	0.916	2.063	0.853	Old Tailings Pile
TB-1	SPT-8	6.5	158.9	9	1.0	80.0	17.4	17.4	16.0	old tailings	hollow stem auger	2.347	0.971	2.152	0.890	Old Tailings Pile
TB-1	SPT-9	7.2	156.4	6	1.1	80.0	13.6	13.6	12.7	old tailings	hollow stem auger	1.751	0.724	1.653	0.684	Old Tailings Pile
TB-1	SPT-10	8.0	153.9	3	1.2	80.0	9.2	9.2	9.0	old tailings	hollow stem auger	1.210	0.501	1.193	0.493	Old Tailings Pile
TB-1	SPT-11	8.8	151.4	3	1.3	80.0	9.0	9.0	8.8	old tailings	hollow stem auger	1.151	0.476	1.133	0.469	Old Tailings Pile
TB-1	SPT-12	9.5	148.9	3	1.3	80.0	8.1	8.1	8.1	old tailings	hollow stem auger	1.039	0.430	1.034	0.428	Old Tailings Pile
TB-1	SPT-13	10.3	146.4	4	1.4	80.0	9.8	9.8	9.4	old tailings	hollow stem auger	1.176	0.486	1.141	0.472	Old Tailings Pile
TB-1	SPT-14	11.0	143.9	3	1.5	0.0	3.3	3.3	3.1	old tailings	hollow stem auger	0.626	0.259	0.617	0.255	Old Tailings Pile
TB-1	SPT-15	11.8	141.4	65	1.5	0.0	63.7		73.9	old tailings	hollow stem auger	5.177	2.141	5.177	2.141	Old Tailings Pile
TB-2	SPT-1	0.8	203.1	16	0.2	80.0	34.6	34.6	30.2	old tailings	hollow stem auger	7.442	3.078	7.442	3.078	Old Tailings Pile
TB-2	SPT-2	1.8	200.1	12	0.4	80.0	26.2	26.2	22.7	old tailings	hollow stem auger	4.758	1.968	3.777	1.562	Old Tailings Pile
TB-2	SPT-3	2.5	197.6	8	0.5	80.0	16.8	16.8	15.7	old tailings	hollow stem auger	2.693	1.114	2.522	1.043	Old Tailings Pile
TB-2	SPT-4	3.3	195.1	3	0.7	80.0	9.6	9.6	9.6	old tailings	hollow stem auger	1.661	0.687	1.665	0.689	Old Tailings Pile
TB-2	SPT-5	4.0	192.6	3	0.8	80.0	9.4	9.4	9.3	old tailings	hollow stem auger	1.645	0.680	1.639	0.678	Old Tailings Pile
TB-2	SPT-6	4.8	190.1	6	1.0	80.0	13.1	13.1	12.3	old tailings	hollow stem auger	2.189	0.905	2.083	0.861	Old Tailings Pile
TB-2	SPT-7	5.6	187.6	6	1.2	80.0	12.5	12.5	11.8	old tailings	hollow stem auger	2.041	0.844	1.941	0.803	Old Tailings Pile
TB-2	SPT-8	6.3	185.1	5	1.3	80.0	11.4	11.4	10.8	old tailings	hollow stem auger	1.792	0.741	1.716	0.710	Old Tailings Pile
TB-2	SPT-9	7.1	182.6	6	1.4	80.0	12.0	12.0	11.2	old tailings	hollow stem auger	1.757	0.727	1.668	0.690	Old Tailings Pile
TB-2	SPT-10	7.8	180.1	3	1.4	80.0	8.8	8.8	8.6	old tailings	hollow stem auger	1.304	0.539	1.282	0.530	Old Tailings Pile
TB-2	SPT-11	8.6	177.6	5	1.5	80.0	10.1	10.1	9.6	old tailings	hollow stem auger	1.390	0.575	1.339	0.554	Old Tailings Pile
TB-2	SPT-12	9.4	175.1	5	1.6	80.0	10.0	10.0	9.5	old tailings	hollow stem auger	1.329	0.550	1.280	0.529	Old Tailings Pile
TB-2	SPT-13	10.1	172.6	12	1.7	80.0	18.9	18.9	17.1	old tailings	hollow stem auger	2.335	0.966	2.098	0.868	Old Tailings Pile
TB-2	SPT-14	10.9	170.1	6	1.8	80.0	11.4	11.4	10.6	old tailings	hollow stem auger	1.434	0.593	1.352	0.559	Old Tailings Pile
TB-2	SPT-15	11.7	167.6	6	1.8	80.0	12.0	12.0	11.1	old tailings	hollow stem auger	1.476	0.610	1.382	0.572	Old Tailings Pile
TB-2	SPT-16	12.4	165.1	12	1.9	80.0	17.3	17.3	15.6	old tailings	hollow stem auger	2.060	0.852	1.861	0.770	Old Tailings Pile
TB-2	SPT-17	13.2	162.6	8	2.0	80.0	13.7	13.7	12.5	old tailings	hollow stem auger	1.641	0.679	1.510	0.625	Old Tailings Pile
TB-2	SPT-18	13.9	160.1	8	2.1	80.0	12.9	12.9	11.8	old tailings	hollow stem auger	1.550	0.641	1.433	0.593	Old Tailings Pile
TB-2	SPT-19	14.7	157.6	13	2.2	80.0	17.9	17.9	16.1	old tailings	hollow stem auger	2.122	0.878	1.906	0.788	Old Tailings Pile
TB-2	SPT-20	15.5	155.1	9	2.2	80.0	13.9	13.9	12.5	old tailings	hollow stem auger	1.662	0.687	1.521	0.629	Old Tailings Pile
TB-2	SPT-21	16.2	152.6	16	2.3	80.0	20.6	20.6	18.5	old tailings	hollow stem auger	2.497	1.033	2.219	0.918	Old Tailings Pile
TB-2	SPT-22	17.0	150.1	6	2.4	80.0	10.5	10.5	9.7	old tailings	hollow stem auger	1.330	0.550	1.250	0.517	Old Tailings Pile
TB-3	SPT-1	0.8	216.4	65	0.2	80.0	123.5		78.9	old tailings	hollow stem auger	7.442	3.078	7.442	3.078	Old Tailings Pile
TB-3	SPT-2	2.4	211.4	3	0.5	80.0	9.2	9.2	9.4	old tailings	hollow stem auger	1.595	0.660	1.631	0.675	Old Tailings Pile

Table 7.1 Tailings – SPT Reduction and Liquefaction Assessment Summary (cont'd)

Drill hole	SPT Name	Middle Depth (m)	Elevation (ft)	Field N	Effective Stress (tsf)	% Fines (used for correction)	$(N_1)_{60\text{-}cs}$ (NCEER)	$(N_1)_{60\text{-}cs}$ (NCEER) <50 only	$(N_1)_{60\text{-}cs}$ (Id&B)	Stability Soil Unit	Drill Rig	FOS - DBE (NCEER)	FOS - MDE (NCEER)	FOS - DBE (Id&B)	FOS - MDE (Id&B)	Location
TB-3	SPT-3	3.1	208.9	14	0.6	80.0	25.3	25.3	22.3	old tailings	hollow stem auger	4.526	1.872	3.737	1.546	Old Tailings Pile
TB-3	SPT-4	3.9	206.4	24	0.8	80.0	37.1	37.1	31.6	old tailings	hollow stem auger	7.621	3.152	7.621	3.152	Old Tailings Pile
TB-3	SPT-5	4.6	203.9	17	0.9	80.0	27.8	27.8	24.5	old tailings	hollow stem auger	5.550	2.296	4.332	1.792	Old Tailings Pile
TB-3	SPT-6	5.4	201.4	14	1.1	80.0	22.2	22.2	20.0	old tailings	hollow stem auger	3.760	1.555	3.302	1.366	Old Tailings Pile
TB-3	SPT-7	6.2	198.9	17	1.2	80.0	27.1	27.1	24.2	old tailings	hollow stem auger	5.096	2.108	4.138	1.711	Old Tailings Pile
TB-3	SPT-8	6.9	196.4	5	1.4	80.0	11.2	11.2	10.6	old tailings	hollow stem auger	1.806	0.747	1.727	0.714	Old Tailings Pile
TB-3	SPT-9	7.7	193.9	19	1.5	80.0	27.0	27.0	24.3	old tailings	hollow stem auger	4.838	2.001	3.993	1.651	Old Tailings Pile
TB-3	SPT-10	8.5	191.4	6	1.7	80.0	12.0	12.0	11.1	old tailings	hollow stem auger	1.845	0.763	1.735	0.718	Old Tailings Pile
TB-3	SPT-11	9.2	188.9	4	1.8	80.0	9.0	9.0	8.7	old tailings	hollow stem auger	1.451	0.600	1.408	0.582	Old Tailings Pile
TB-3	SPT-12	10.0	186.4	5	2.0	80.0	9.5	9.5	9.0	old tailings	hollow stem auger	1.512	0.625	1.452	0.600	Old Tailings Pile
TB-3	SPT-13	10.7	183.9	5	2.0	80.0	10.3	10.3	9.7	old tailings	hollow stem auger	1.574	0.651	1.493	0.618	Old Tailings Pile
TB-3	SPT-14	11.5	181.4	13	2.1	80.0	18.1	18.1	16.3	old tailings	hollow stem auger	2.566	1.061	2.305	0.953	Old Tailings Pile
TB-3	SPT-15	12.3	178.9	5	2.2	80.0	9.5	9.5	9.0	old tailings	hollow stem auger	1.426	0.590	1.363	0.564	Old Tailings Pile
TB-3	SPT-16	13.0	176.4	6	2.2	80.0	10.7	10.7	9.9	old tailings	hollow stem auger	1.546	0.639	1.454	0.601	Old Tailings Pile
TB-3	SPT-17	13.8	173.9	6	2.3	80.0	10.6	10.6	9.8	old tailings	hollow stem auger	1.523	0.630	1.432	0.592	Old Tailings Pile
TB-3	SPT-18	14.6	171.4	10	2.4	80.0	14.2	14.2	12.8	old tailings	hollow stem auger	1.945	0.804	1.772	0.733	Old Tailings Pile
TB-3	SPT-19	15.3	168.9	8	2.5	80.0	12.3	12.3	11.1	old tailings	hollow stem auger	1.703	0.704	1.570	0.649	Old Tailings Pile
TB-3	SPT-20	16.1	166.4	12	2.5	80.0	15.7	15.7	14.1	old tailings	hollow stem auger	2.135	0.883	1.925	0.796	Old Tailings Pile
TB-3	SPT-21	16.8	163.9	4	2.6	80.0	8.5	8.5	8.1	old tailings	hollow stem auger	1.282	0.530	1.237	0.511	Old Tailings Pile
TB-3	SPT-22	17.6	161.4	3	2.7	80.0	7.9	7.9	7.6	old tailings	hollow stem auger	1.221	0.505	1.190	0.492	Old Tailings Pile
TB-4	SPT-1	0.8	221.3	28	0.2	80.0	57.1		48.5	old tailings	hollow stem auger	7.442	3.078	7.442	3.078	Old Tailings Pile
TB-4	SPT-2	1.6	218.8	17	0.3	80.0	35.8	35.8	30.1	old tailings	hollow stem auger	7.486	3.096	7.486	3.096	Old Tailings Pile
TB-4	SPT-3	2.4	216.3	5	0.5	80.0	13.4	13.4	13.0	old tailings	hollow stem auger	2.168	0.896	2.116	0.875	Old Tailings Pile
TB-4	SPT-4	3.1	213.8	56	0.6	80.0	89.3		65.3	old tailings	hollow stem auger	7.576	3.133	7.576	3.133	Old Tailings Pile
TB-4	SPT-5	3.9	211.3	33	0.8	80.0	49.3	49.3	40.7	old tailings	hollow stem auger	7.621	3.152	7.621	3.152	Old Tailings Pile
TB-4	SPT-6	4.6	208.8	20	0.9	80.0	31.2	31.2	27.2	old tailings	hollow stem auger	7.667	3.171	5.295	2.190	Old Tailings Pile
TB-4	SPT-7	5.4	206.3	15	1.1	80.0	23.8	23.8	21.3	old tailings	hollow stem auger	4.136	1.711	3.563	1.474	Old Tailings Pile
TB-4	SPT-8	6.2	203.8	12	1.2	80.0	20.5	20.5	18.6	old tailings	hollow stem auger	3.323	1.374	2.964	1.226	Old Tailings Pile
TB-4	SPT-9	6.9	201.3	4	1.4	80.0	9.6	9.6	9.3	old tailings	hollow stem auger	1.605	0.664	1.563	0.646	Old Tailings Pile
TB-4	SPT-10	7.7	198.8	19	1.5	80.0	27.0	27.0	24.3	old tailings	hollow stem auger	4.838	2.001	3.993	1.651	Old Tailings Pile
TB-4	SPT-11	8.5	196.3	30	1.7	80.0	37.2	37.2	34.1	old tailings	hollow stem auger	7.036	2.910	7.036	2.910	Old Tailings Pile
TB-4	SPT-12	9.2	193.8	48	1.8	80.0	55.2		53.7	old tailings	hollow stem auger	6.941	2.871	6.941	2.871	Old Tailings Pile
TB-4	SPT-13	10.0	191.3	20	2.0	80.0	24.9	24.9	22.6	old tailings	hollow stem auger	4.046	1.674	3.495	1.446	Old Tailings Pile
TB-4	SPT-14	10.7	188.8	17	2.1	80.0	22.0	22.0	19.8	old tailings	hollow stem auger	3.377	1.397	2.984	1.234	Old Tailings Pile
TB-4	SPT-15	11.5	186.3	12	2.3	80.0	16.4	16.4	14.7	old tailings	hollow stem auger	2.446	1.012	2.209	0.913	Old Tailings Pile
TB-4	SPT-16	12.3	183.8	8	2.4	80.0	12.9	12.9	11.7	old tailings	hollow stem auger	1.984	0.820	1.821	0.753	Old Tailings Pile
TB-4	SPT-17	13.0	181.3	10	2.5	80.0	14.6	14.6	13.1	old tailings	hollow stem auger	2.200	0.910	1.996	0.825	Old Tailings Pile
TB-4	SPT-18	13.8	178.8	9	2.6	80.0	13.2	13.2	11.9	old tailings	hollow stem auger	1.998	0.826	1.825	0.755	Old Tailings Pile
TB-4	SPT-19	14.6	176.3	14	2.7	80.0	17.2	17.2	15.4	old tailings	hollow stem auger	2.540	1.050	2.274	0.941	Old Tailings Pile
TB-4	SPT-20	15.3	173.8	8	2.7	80.0	11.9	11.9	10.7	old tailings	hollow stem auger	1.796	0.743	1.654	0.684	Old Tailings Pile
TB-5	SPT-1	1.1	217.5	4	0.2	80.0	12.1	12.1	11.5	old tailings	hollow stem auger	1.971	0.815	1.883	0.779	Old Tailings Pile
TB-5	SPT-2	1.9	214.8	63	0.4	80.0	117.8		71.9	old tailings	hollow stem auger	7.504	3.104	7.504	3.104	Old Tailings Pile
TB-5	SPT-3	2.7	212.0	30	0.5	80.0	50.5		39.7	old tailings	hollow stem auger	7.553	3.124	7.553	3.124	Old Tailings Pile
TB-5	SPT-4	3.5	209.5	16	0.7	80.0	27.9	27.9	24.3	old tailings	hollow stem auger	5.542	2.292	4.240	1.754	Old Tailings Pile
TB-5	SPT-5	4.3	207.0	3	0.8	80.0	9.4	9.4	9.4	old tailings	hollow stem auger	1.649	0.682	1.643	0.679	Old Tailings Pile
TB-5	SPT-6	5.0	204.5	4	1.0	80.0	9.9	9.9	9.7	old tailings	hollow stem auger	1.722	0.712	1.694	0.701	Old Tailings Pile
TB-5	SPT-7	5.7	202.3	48	1.1	80.0	62.1		54.2	old tailings	hollow stem auger	7.590	3.139	7.590	3.139	Old Tailings Pile
TB-5	SPT-8	6.6	199.5	23	1.3	80.0	32.8	32.8	29.3	old tailings	hollow stem auger	7.384	3.054	6.263	2.590	Old Tailings Pile
TB-5	SPT-9	7.3	197.0	17	1.4	80.0	24.5	24.5	22.1	old tailings	hollow stem auger	4.093	1.693	3.515	1.454	Old Tailings Pile
TB-5	SPT-10	8.1	194.5	11	1.6	80.0	17.2	17.2	15.6	old tailings	hollow stem auger	2.591	1.071	2.354	0.974	Old Tailings Pile
TB-5	SPT-11	8.8	192.0	10	1.7	80.0	15.3	15.3	13.9	old tailings	hollow stem auger	2.271	0.939	2.081	0.861	Old Tailings Pile
TB-5	SPT-12	9.6	189.5	10	1.9	80.0	15.5	15.5	14.0	old tailings	hollow stem auger	2.294	0.949	2.093	0.866	Old Tailings Pile
TB-5	SPT-13	10.4	187.0	6	2.0	80.0	11.6	11.6	10.7	old tailings	hollow stem auger	1.783	0.738	1.668	0.690	Old Tailings Pile
TB-5	SPT-14	11.1	184.5	6	2.2	80.0	10.8	10.8	10.0	old tailings	hollow stem auger	1.682	0.696	1.582	0.654	Old Tailings Pile
BH97-2	SPT-1	1.6	159.9	46	0.3	80.0	54.3		40.5	new tailings	mud rotary	7.488	3.097	7.488	3.097	South Side
BH97-2	SPT-2	3.0	155.5	18	0.6	80.0	19.7	19.7	17.9	new tailings	mud rotary	3.198	1.323	2.892	1.196	South Side
DH-00-06	SPT-1	1.8	204.3	7	0.4	80.0	10.6	10.6	16.3	new tailings	mud rotary	2.831	1.171	2.592	1.072	South Side
DH-00-06	SPT-2	3.3	199.3	12	0.7	80.0	14.2	14.2	19.8	new tailings	mud rotary	3.678	1.521	3.228	1.335	South Side

Table 7.1 Tailings – SPT Reduction and Liquefaction Assessment Summary (cont'd)

Drill hole	SPT Name	Middle Depth (m)	Elevation (ft)	Field N	Effective Stress (tsf)	% Fines (used for correction)	$(N_1)_{60\text{-}cs}$ (NCEER)	$(N_1)_{60\text{-}cs}$ (NCEER) <50 only	$(N_1)_{60\text{-}cs}$ (Id&B)	Stability Soil Unit	Drill Rig	FOS - DBE (NCEER)	FOS - MDE (NCEER)	FOS - DBE (Id&B)	FOS - MDE (Id&B)	Location
DH-00-06	SPT-3	4.8	194.3	2	1.0	80.0	2.1	2.1	7.7	new tailings	mud rotary	1.420	0.587	1.441	0.596	South Side
DH-00-06	SPT-4	6.3	189.3	7	1.3	80.0	7.1	7.1	12.5	new tailings	mud rotary	2.125	0.879	1.993	0.824	South Side
DH-00-06	SPT-5	7.8	184.3	10	1.6	80.0	9.1	9.1	14.5	new tailings	mud rotary	2.374	0.982	2.171	0.898	South Side
DH-00-06	SPT-6	9.4	179.3	7	2.0	80.0	5.8	5.8	11.0	new tailings	mud rotary	1.789	0.740	1.671	0.691	South Side
DH-00-06	SPT-7	10.9	174.3	10	2.3	80.0	8.1	8.1	13.3	new tailings	mud rotary	2.166	0.896	1.970	0.815	South Side
DH-00-06	SPT-8	12.4	169.3	23	2.6	80.0	17.5	17.5	23.8	new tailings	mud rotary	4.359	1.803	3.781	1.564	South Side
DH-00-06	SPT-9	13.9	164.3	30	2.9	80.0	21.5	21.5	29.3	new tailings	mud rotary	7.128	2.948	6.039	2.498	South Side
DH-00-06	SPT-10	15.5	159.3	37	3.2	80.0	25.2	25.2	35.0	new tailings	mud rotary	7.317	3.026	7.317	3.026	South Side
DH-00-11	SPT-1	1.8	207.6	3	0.4	80.0	10.5	10.5	10.1	new tailings	mud rotary	1.758	0.727	1.713	0.709	South Side
DH-00-11	SPT-2	3.3	202.6	9	0.7	80.0	17.8	17.8	16.4	new tailings	mud rotary	2.877	1.190	2.651	1.096	South Side
DH-00-11	SPT-3	4.5	198.6	0	0.9	80.0	5.0	5.0	5.5	new tailings	mud rotary	1.104	0.456	1.167	0.482	South Side
DH-00-11	SPT-4	6.0	193.6	50	1.3	80.0	67.3		60.5	new tailings	mud rotary	7.400	3.061	7.400	3.061	South Side
DH-00-11	SPT-5	7.5	188.6	7	1.6	80.0	12.8	12.8	11.9	new tailings	mud rotary	1.963	0.812	1.838	0.760	South Side
DH-00-11	SPT-6	9.1	183.6	36	1.9	80.0	41.6	41.6	39.3	new tailings	mud rotary	6.847	2.832	6.847	2.832	South Side
DH-00-11	SPT-7	10.6	178.6	3	2.2	80.0	8.0	8.0	7.7	new tailings	mud rotary	1.316	0.544	1.290	0.533	South Side
DH-00-11	SPT-8	12.1	173.6	23	2.5	80.0	26.3	26.3	24.1	new tailings	mud rotary	4.455	1.842	3.846	1.591	South Side
DH-00-11	SPT-9	13.6	168.6	12	2.8	80.0	15.5	15.5	13.8	new tailings	mud rotary	2.341	0.968	2.104	0.870	South Side
DH-00-11	SPT-10	15.2	163.6	51	3.2	80.0	47.2	47.2	51.4	new tailings	mud rotary	7.285	3.013	7.285	3.013	South Side
DH-00-11	SPT-11	16.7	158.6	55	3.5	80.0	48.4	48.4	54.9	new tailings	mud rotary	7.510	3.106	7.510	3.106	South Side
DH-00-11	SPT-12	18.2	153.6	20	3.8	80.0	20.1	20.1	18.0	new tailings	mud rotary	3.371	1.394	2.985	1.235	South Side
DH-02-06	SPT-1	1.8	177.4	6	0.4	85.0	15.9	15.9	14.7	new tailings	mud rotary	2.539	1.050	2.356	0.974	South Side
DH-02-06	SPT-2	3.3	172.4	6	0.7	87.0	13.5	13.5	12.9	new tailings	mud rotary	2.207	0.913	2.118	0.876	South Side
DH-02-06	SPT-3	4.8	167.4	6	1.0	85.0	12.5	12.5	11.8	new tailings	mud rotary	2.100	0.869	2.007	0.830	South Side
DH-02-06	SPT-4	6.3	162.4	6	1.3	85.0	12.3	12.3	11.5	new tailings	mud rotary	1.957	0.810	1.853	0.766	South Side
DH-02-06	SPT-5	7.8	157.4	10	1.6	85.0	15.9	15.9	14.4	new tailings	mud rotary	2.374	0.982	2.169	0.897	South Side
DH-02-06	SPT-6	9.4	152.4	8	1.9	85.0	13.1	13.1	12.0	new tailings	mud rotary	1.880	0.778	1.741	0.720	South Side
DH-02-06	SPT-7	10.6	148.3	34	2.0	85.0	69.5		73.4	new tailings	mud rotary	6.392	2.644	6.392	2.644	South Side
DH-05-11	SPT-1	3.6	201.5	58	0.8	65.0	93.5		72.0	new tails	mud rotary	7.606	3.146	7.606	3.146	South Side
DH-05-11	SPT-2	4.4	199.0	2	0.9	65.0	7.9	7.9	8.1	new tails	mud rotary	1.461	0.604	1.483	0.613	South Side
DH-05-11	SPT-3	5.9	194.0	50	1.2	65.0	68.5		61.5	new tails	mud rotary	7.413	3.066	7.413	3.066	South Side
DH-05-11	SPT-4	7.6	188.4	36	1.6	63.8	50.0		46.8	new tails	mud rotary	7.055	2.918	7.055	2.918	South Side
DH-05-11	SPT-5	9.4	182.5	41	2.0	65.0	51.2		50.3	new tails	mud rotary	6.833	2.826	6.833	2.826	South Side
DH-05-11	SPT-6	10.6	178.7	0	2.2	75.0	5.0	5.0	5.6	new tails	mud rotary	0.990	0.409	1.048	0.433	South Side
DH-05-11	SPT-7	12.0	174.0	42	2.5	86.4	49.1	49.1	50.8	new tails	mud rotary	6.952	2.875	6.952	2.875	South Side
DH-05-11	SPT-8	13.8	168.2	16	2.9	85.0	20.7	20.7	18.6	new tails	mud rotary	3.186	1.318	2.825	1.169	South Side
DH-05-11	SPT-9	15.2	163.7	34	3.2	85.0	36.8	36.8	36.8	new tails	mud rotary	7.275	3.009	7.275	3.009	South Side
DH-05-11	SPT-10	16.7	158.7	46	3.5	81.5	46.0	46.0	51.1	new tails	mud rotary	7.502	3.103	7.502	3.103	South Side
BH97-2	SPT-3	4.5	150.5	13	0.8	80.0	15.5	15.5	14.4	old tailings	mud rotary	2.209	0.914	2.072	0.857	South Side
BH97-2	SPT-4	6.0	145.5	33	0.8	80.0	34.4	34.4	29.6	old tailings	mud rotary	5.850	2.419	5.199	2.150	South Side
BH97-2	SPT-5	7.8	139.5	29	0.9	80.0	30.6	30.6	26.7	old tailings	mud rotary	5.111	2.114	3.373	1.395	South Side
DH-00-06	SPT-11	17.0	154.3	23	3.4	80.0	15.3	15.3	21.3	old tailings	mud rotary	3.828	1.583	3.393	1.403	South Side
DH-00-06	SPT-12	18.5	149.3	6	3.6	80.0	3.9	3.9	8.8	old tailings	mud rotary	1.625	0.672	1.518	0.628	South Side
DH-00-06	SPT-13	20.0	144.3	19	3.7	80.0	12.1	12.1	17.4	old tailings	mud rotary	3.139	1.298	2.780	1.150	South Side
DH-00-06	SPT-14	21.6	139.3	61	3.9	80.0	38.1	38.1	61.3	old tailings	mud rotary	7.687	3.179	7.687	3.179	South Side
DH-00-11	SPT-13	19.7	148.6	52	4.0	80.0	43.3	43.3	48.6	old tailings	mud rotary	7.941	3.284	7.941	3.284	South Side
DH-00-11	SPT-14	21.3	143.6	47	4.1	80.0	39.0	39.0	42.0	old tailings	mud rotary	8.110	3.354	8.110	3.354	South Side
DH-05-11	SPT-11	18.3	153.4	42	3.8	85.0	40.8	40.8	44.1	old tailings	mud rotary	7.802	3.227	7.802	3.227	South Side
DH-05-11	SPT-12	19.8	148.4	38	4.1	85.0	36.2	36.2	37.7	old tailings	mud rotary	8.140	3.367	8.140	3.367	South Side
DH-05-11	SPT-13	21.4	143.4	23	4.3	91.1	23.5	23.5	21.6	old tailings	mud rotary	4.428	1.831	3.940	1.630	South Side
DH-05-11	SPT-14	22.9	138.4	40	4.4	85.0	36.7	36.7	39.0	old tailings	mud rotary	8.579	3.548	8.579	3.548	South Side
DH-05-15	SPT-1	1.6	195.3	9	0.3	85.0	23.7	23.7	21.1	new tails	mud rotary	4.023	1.664	3.443	1.424	Southeast Corner
DH-05-15	SPT-2	3.1	190.3	13	0.6	85.0	26.5	26.5	23.2	new tails	mud rotary	4.932	2.040	3.948	1.633	Southeast Corner
DH-05-15	SPT-3	4.6	185.2	14	1.0	85.0	25.1	25.1	22.3	new tails	mud rotary	4.502	1.862	3.785	1.566	Southeast Corner
DH-05-16	SPT-1	1.3	201.3	13	0.3	85.0	32.1	32.1	28.1	new tailings	mud rotary	7.468	3.089	5.558	2.299	Southeast Corner
DH-05-16	SPT-2	2.9	196.1	6	0.6	85.0	14.6	14.6	13.9	new tailings	mud rotary	2.367	0.979	2.257	0.933	Southeast Corner
DH-05-16	SPT-3	4.4	191.1	6	0.9	85.0	13.8	13.8	13.0	new tailings	mud rotary	2.275	0.941	2.154	0.891	Southeast Corner
DH-05-17	SPT-1	3.1	195.8	25	0.6	85.0	46.4	46.4	37.8	new tailings	mud rotary	7.574	3.132	7.574	3.132	Southeast Corner
DH-05-17	SPT-2	4.6	190.8	13	1.0	85.0	23.7	23.7	21.2	new tailings	mud rotary	4.116	1.702	3.540	1.464	Southeast Corner

Table 7.1 Tailings – SPT Reduction and Liquefaction Assessment Summary (cont'd)

Drill hole	SPT Name	Middle Depth (m)	Elevation (ft)	Field N	Effective Stress (tsf)	% Fines (used for correction)	$(N_1)_{60\text{-}cs}$ (NCEER)	$(N_1)_{60\text{-}cs}$ (NCEER) <50 only	$(N_1)_{60\text{-}cs}$ (Id&B)	Stability Soil Unit	Drill Rig	FOS - DBE (NCEER)	FOS - MDE (NCEER)	FOS - DBE (Id&B)	FOS - MDE (Id&B)	Location
DH-05-18	SPT-1	1.3	198.4	3	0.3	85.0	11.2	11.2	10.7	new tails	mud rotary	1.856	0.768	1.787	0.739	Southeast Corner
DH-05-18	SPT-2	2.8	193.3	13	0.6	85.0	26.0	26.0	22.8	new tails	mud rotary	4.741	1.961	3.839	1.588	Southeast Corner
DH-05-18	SPT-3	4.3	188.4	27	0.9	85.0	78.5		64.4	new tails	mud rotary	7.649	3.163	7.649	3.163	Southeast Corner
DH-05-20	SPT-1	1.3	196.8	18	0.3	85.0	42.5	42.5	35.8	new tails	mud rotary	7.467	3.088	7.467	3.088	Southeast Corner
DH-05-20	SPT-2	2.8	191.6	18	0.6	85.0	34.1	34.1	28.8	new tails	mud rotary	7.559	3.126	6.040	2.498	Southeast Corner
DH-05-20	SPT-3	4.4	186.5	0	0.9	85.0	5.0	5.0	5.5	new tails	mud rotary	1.103	0.456	1.164	0.481	Southeast Corner
DH-05-20	SPT-4	4.8	185.3	23	1.0	85.0	91.9		77.0	new tails	mud rotary	7.755	3.207	7.755	3.207	Southeast Corner
DH-00-12	SPT-1	1.0	161.8	2	0.2	80.0	8.7	8.7	8.6	new tailings	hollow stem auger	1.514	0.626	1.505	0.623	West Buttress
DH-00-12	SPT-2	1.8	159.3	7	0.4	80.0	17.7	17.7	16.3	new tailings	hollow stem auger	2.831	1.171	2.592	1.072	West Buttress
DH-00-12	SPT-3	2.5	156.8	7	0.5	80.0	15.6	15.6	14.8	new tailings	hollow stem auger	2.510	1.038	2.380	0.984	West Buttress
DH-00-12	SPT-4	3.3	154.3	5	0.7	80.0	12.1	12.1	11.7	new tailings	hollow stem auger	2.003	0.829	1.953	0.808	West Buttress
DH-00-12	SPT-5	4.0	151.8	14	0.8	80.0	24.9	24.9	22.1	new tailings	hollow stem auger	4.045	1.673	3.398	1.405	West Buttress
DH-00-12	SPT-6	4.8	149.3	14	0.9	80.0	23.9	23.9	21.3	new tailings	hollow stem auger	3.554	1.470	3.044	1.259	West Buttress
DH-00-12	SPT-7	5.6	146.8	100	0.9	80.0	133.9		107.6	new tailings	hollow stem auger	6.226	2.575	6.226	2.575	West Buttress
DH-00-13	SPT-1	1.0	161.8	6	0.2	80.0	16.0	16.0	14.7	new tailings	hollow stem auger	2.540	1.050	2.344	0.969	West Buttress
DH-00-13	SPT-2	1.8	159.3	5	0.4	80.0	14.1	14.1	13.2	new tailings	hollow stem auger	2.265	0.937	2.134	0.883	West Buttress
DH-00-13	SPT-3	2.5	156.8	8	0.5	80.0	17.1	17.1	16.0	new tailings	hollow stem auger	2.751	1.138	2.566	1.061	West Buttress
DH-00-13	SPT-4	3.3	154.3	4	0.7	80.0	10.7	10.7	10.5	new tailings	hollow stem auger	1.807	0.747	1.789	0.740	West Buttress
DH-00-13	SPT-5	4.0	151.8	15	0.8	80.0	26.3	26.3	23.2	new tailings	hollow stem auger	4.473	1.850	3.635	1.503	West Buttress
DH-00-13	SPT-6	4.8	149.3	15	0.9	80.0	25.2	25.2	22.4	new tailings	hollow stem auger	3.883	1.606	3.246	1.342	West Buttress
DH-00-13	SPT-7	5.6	146.8	10	0.9	80.0	17.9	17.9	16.4	new tailings	hollow stem auger	2.373	0.982	2.171	0.898	West Buttress
DH-05-10	SPT-1	3.0	204.3	22	0.6	85.0	39.8	39.8	33.0	new tailings	mud rotary	7.566	3.129	7.566	3.129	West Buttress
DH-05-10	SPT-2	3.8	201.7	14	0.8	85.0	26.0	26.0	23.0	new tailings	mud rotary	4.771	1.973	3.903	1.614	West Buttress
DH-05-10	SPT-3	5.3	196.5	19	1.1	85.0	30.4	30.4	26.9	new tailings	mud rotary	7.571	3.131	5.096	2.108	West Buttress
DH-05-10	SPT-4	6.8	191.7	12	1.4	83.8	20.9	20.9	18.9	new tailings	mud rotary	3.272	1.353	2.907	1.202	West Buttress
DH-05-10	SPT-5	8.3	186.7	10	1.8	85.0	17.0	17.0	15.4	new tailings	mud rotary	2.507	1.037	2.272	0.940	West Buttress
DH-05-10	SPT-6	10.8	178.7	8	2.3	80.0	13.9	13.9	12.5	new tailings	mud rotary	2.048	0.847	1.873	0.775	West Buttress
DH-05-10	SPT-7	12.3	173.6	12	2.5	70.8	17.5	17.5	15.7	new tailings	mud rotary	2.572	1.064	2.308	0.954	West Buttress
DH-05-10	SPT-8	13.8	168.7	10	2.7	75.0	15.1	15.1	13.5	new tailings	mud rotary	2.179	0.901	1.967	0.814	West Buttress
DH-05-10	SPT-9	15.3	163.7	41	2.9	75.0	45.4	45.4	47.5	new tailings	mud rotary	6.690	2.767	6.690	2.767	West Buttress
DH-05-13	SPT-1	1.7	152.8	14	0.3	85.0	34.1	34.1	28.2	new tailings	mud rotary	7.490	3.098	5.659	2.340	West Buttress
DH-05-13	SPT-2	3.1	148.2	16	0.6	88.7	31.6	31.6	27.0	new tailings	mud rotary	7.572	3.132	5.144	2.128	West Buttress
DH-05-13	SPT-3	4.6	143.3	23	1.0	85.0	38.3	38.3	33.1	new tailings	mud rotary	7.662	3.169	7.662	3.169	West Buttress
DH-05-12	SPT-1	1.6	152.7	13	0.3	88.5	32.1	32.1	27.0	new tails	mud rotary	7.488	3.097	5.067	2.096	West Buttress
DH-05-10	SPT-10	16.9	158.6	16	3.0	77.8	20.3	20.3	18.3	old tailings	mud rotary	2.940	1.216	2.611	1.080	West Buttress
DH-05-10	SPT-11	18.4	153.7	19	3.2	80.0	22.7	22.7	20.7	old tailings	mud rotary	3.425	1.417	3.033	1.254	West Buttress
DH-05-10	SPT-12	20.0	148.4	9	3.3	84.0	13.2	13.2	11.7	old tailings	mud rotary	1.972	0.816	1.779	0.736	West Buttress
DH-05-10	SPT-13	21.5	143.5	15	3.5	85.0	18.4	18.4	16.4	old tailings	mud rotary	2.792	1.155	2.474	1.023	West Buttress

		(N1)60-cs				FOS - MDE (NCEER)			
Location	Tailings	*Average*	*Min*	*Max*	*Count*	*Average*	*Min*	*Max*	*Count*
East Side	Old	-	-	-	-	-	-	-	-
	New	23.1	14.1	40.9	6.0	1.6	0.8	2.9	6.0
Old Tailings Pile	Old	16.2	2.9	49.3	198.0	1.1	0.3	3.2	198.0
	New	23.5	9.9	40.5	11.0	1.7	0.7	3.2	11.0
South Side	Old	28.4	3.9	43.3	13.0	2.4	0.7	3.5	13.0
	New	18.9	2.1	49.1	34.0	1.5	0.4	3.1	34.0
Southeast Corner	Old	-	-	-	-	-	-	-	-
	New	25.0	5.0	46.4	13.0	1.9	0.5	3.1	13.0
West Buttress	Old	18.7	13.2	22.7	4.0	1.2	0.8	1.4	4.0
	New	22.8	8.7	45.4	26.0	1.7	0.6	3.2	26.0

Table 7.2 Summary of Tailings Liquefaction Assessment by Location

		Excluding $(N_1)_{60\text{-cs}} > 50$			
Location	**New or Old Tailings**	**Average $(N_1)_{60\text{-cs}}$**	**Average FOS (MDE)**	**No. of SPT's Conducted**	**Percentage of SPT's Indicating Liquefaction Under MDE (%)**
East Side	New	23.1	1.8	6	29
Old Tailings Pile	New	23.5	1.9	11	25
	Old	16.2	1.2	198	66
South Side	New	18.9	1.7	34	44
	Old	28.4	2.4	13	15
West Buttress	New	22.8	1.7	26	41
	Old	18.7	1.2	4	25
Southeast Corner	New	25.0	2.1	13	27

Summary plots showing $(N_1)_{60\text{-cs}}$ vs. elevation and depth are on Figures 7.1 to 7.4. The FOS under the MDE vs. $(N_1)_{60\text{-cs}}$ is plotted on Figure 7.5. The figures also show that, depending on stress condition, $(N_1)_{60\text{-cs}}$ values from 16 to 21 result in a factor of safety (FOS) under the MDE of 1.1.

FOS under DBE and MDE are plotted against elevation in Figures 7.6 and 7.7. In general, there is minimal liquefaction potential under the DBE. However, a significant number of SPT tests indicate liquefaction under the MDE. About 43% of new tailings tests and 66% of old tailings tests indicate liquefaction potential.

Moisture content is plotted against $(N_1)_{60\text{-cs}}$ in Figure 7.8. No apparent correlation between the two parameters can be made for the tailings.

It should be noted that the liquefaction assessment for each borehole is based on the as-drilled elevation conditions, and does not take the ultimate tailings pile overburden stresses into consideration, as discussed for the shallow sand and gravel layer.

III-8. CONCLUSIONS

The shallow sand and gravel layer and tailings materials were assessed for their potential to liquefy during an MDE and DBE earthquake. The Youd et al (2001) method based on SPT data was used.

In general, there is minimal liquefaction potential for either the sand or tailings under the DBE.

The shallow sand and gravel layer beneath the Northeast Expansion has the potential to liquefy under the MDE (the average FOS is less than 1.1 in this region). It is planned to excavate out this layer (KC 2004).

Liquefaction of tailings under MDE is more debatable, as the average SPT in the new tailings is just above the level required for a safety factor of 1.1, whereas an average for the old tailings is below that required for a safety factor of 1.1. Consequently, large scale liquefaction is likely for old tailings and possible for the new tailings. Even if the new tailings does not liquefy, saturated zones in the pile could experience softening since the average FOS against liquefaction over a large area of the pile is less than 1.4.

REFERENCES

Cone Tec (1997). Field Report – Cone Penetration Testing and SPT Energy Calibration at Green Creek Mine. November 5, 1997.

Idriss, I. M. and Boulanger, R. W. (2004). Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes. 11th International Conf. on Soil Dynamics and Earthquake Engineering and 3rd International Conf. on Earthquake Geotechnical Engineering, Berkeley, CA, Jan. 7-9, 2004.

Klohn Crippen (1998). Seismic Hazard Assessment at Greens Creek Mine Site, Admiralty Island, Alaska, August.

Klohn Crippen (2004). Kennecott Greens Creek Mine Stage 2 Tailings Facility Expansion – Design Overview for Forest Service Submission (Final). April.

Liao, S. and Whitman, R. V. (1986). Overburden Correction Factors for SPT in Sand. ASCE Journal of Geotechnical Engineering, Vol. 112, No. 3, pp373-377.

Robert Miner Dynamic Testing Inc. (2005). Standard Penetration Test Energy Measurements, Drill Hole DH05-08, March 21, 22, & 23, 2005, CME 850 Drill Rig, CME Auto Hammer, Cascade Drilling, Greens Creek Mine, Admiralty Island, Alaska, April 2, 2005.

Seed, H.B., Tokimatsu, K., Harder, L. F. and Chung, R. M. (1985). The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, ASCE Journal of Geotechnical Engineering, Vol. 111, No. 12, pp1425-1445.

Seed, R.B., K.O. Cetin, R.E.S. Moss, A. Kammerer, J. Wu, J.M. Pestana and M.F. Riemer, (2001) Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation, Fourth International Conference and Symposium on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper SPL-2, San Diego, California, March 26-28, 2001.

Youd, T.L., Idriss, I.M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson III, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., Stoke II, K. H. (2001). 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.

FIGURES

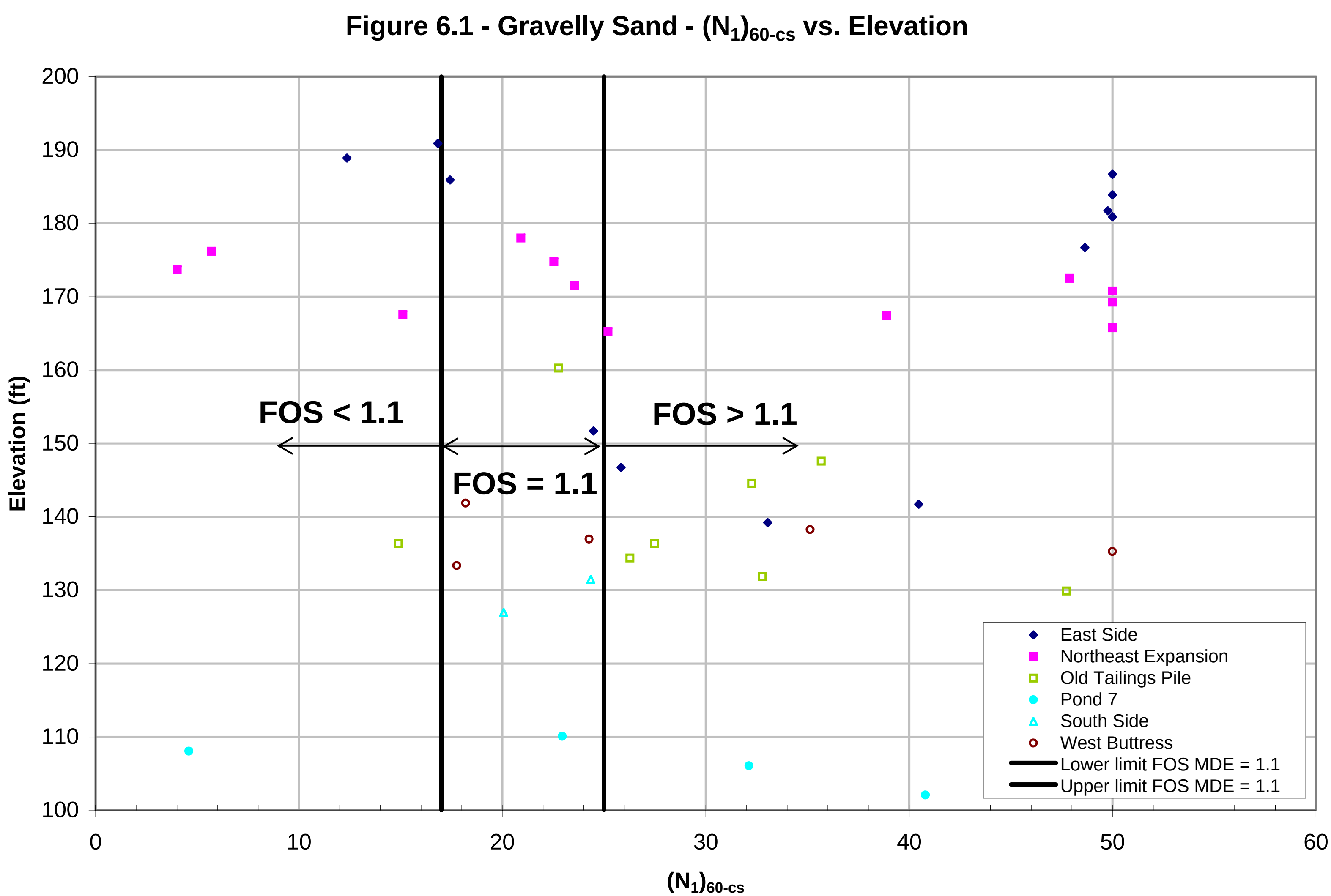


Figure 6.1 - Gravelly Sand - $(N_1)_{60\text{-}cs}$ vs. Elevation
Elevation (ft)
200
190
180
170
160
150
140
130
120
110
100
0
10
20
30
40
50
60
$(N_1)_{60\text{-}cs}$
FOS < 1.1
FOS = 1.1
FOS > 1.1
East Side
Northeast Expansion
Old Tailings Pile
Pond 7
South Side
West Buttress
Lower limit FOS MDE = 1.1
Upper limit FOS MDE = 1.1

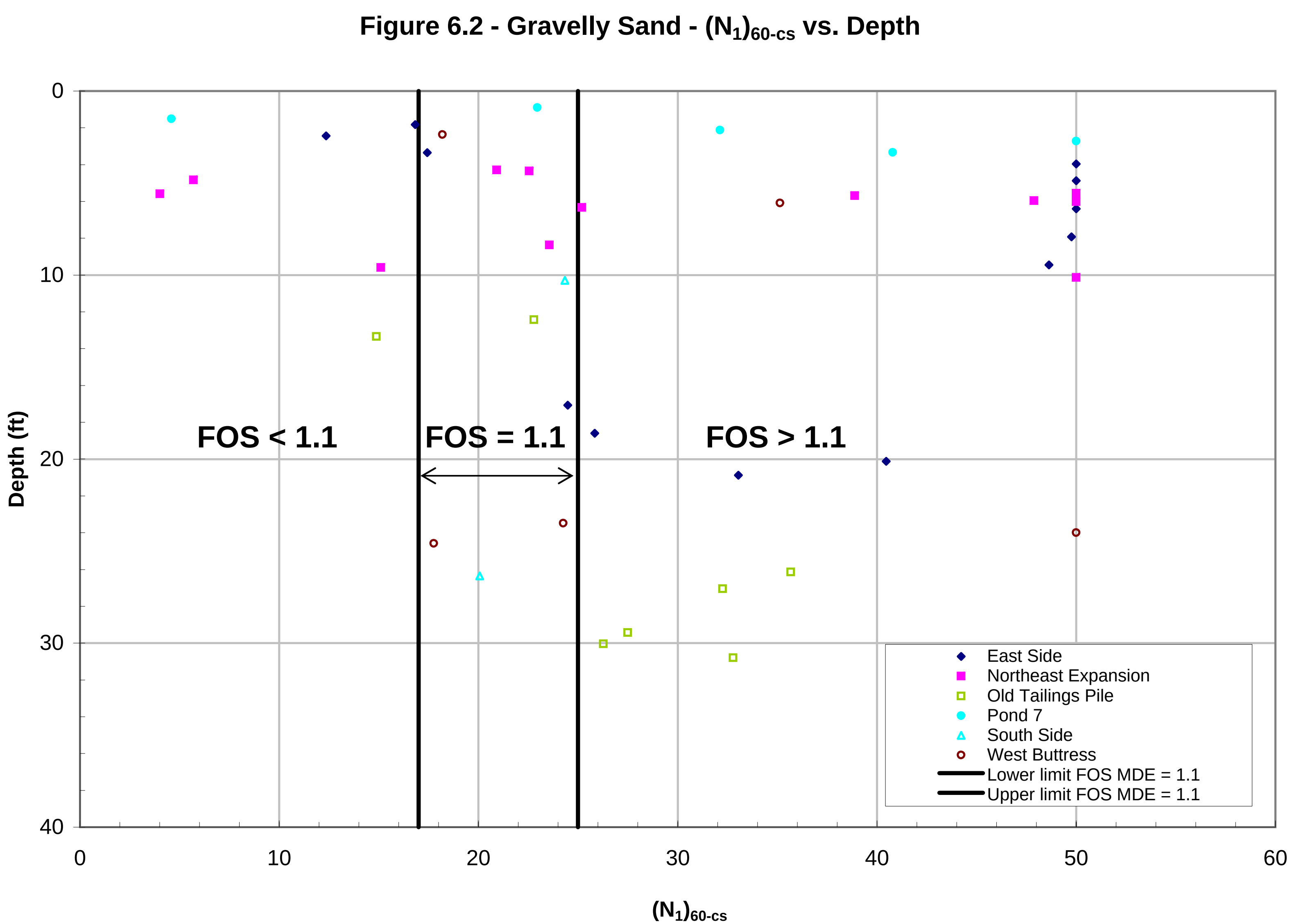
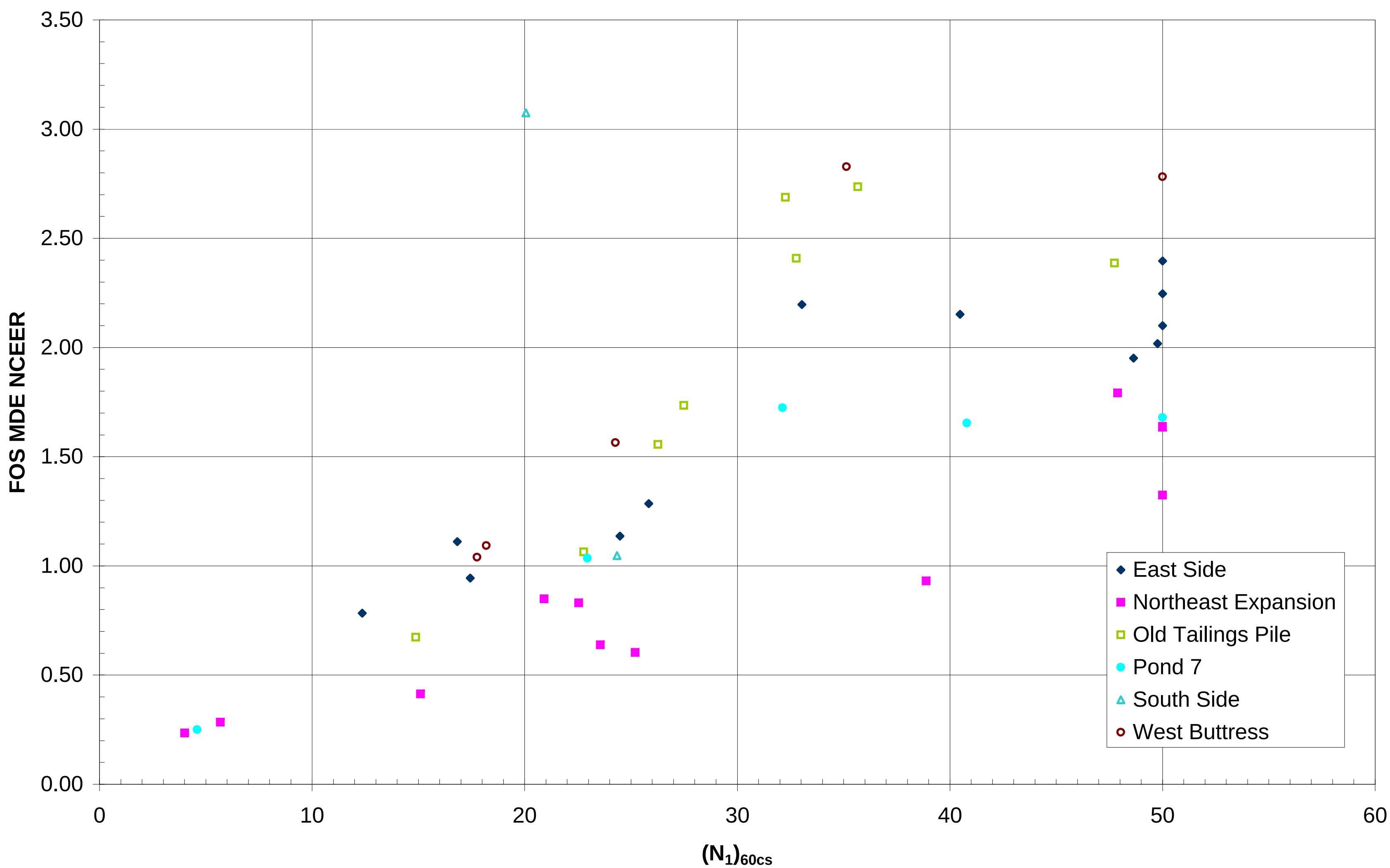


Figure 6.2 - Gravelly Sand - $(N_1)_{60\text{-}cs}$ vs. Depth
Depth (ft)
0
10
20
30
40
0
10
20
30
40
50
60
$(N_1)_{60\text{-}cs}$
FOS < 1.1
FOS = 1.1
FOS > 1.1
East Side
Northeast Expansion
Old Tailings Pile
Pond 7
South Side
West Buttress
Lower limit FOS MDE = 1.1
Upper limit FOS MDE = 1.1

Figure 6.3 - $(N_1)_{60cs}$ vs. FOS (MDE) for Gravelly Sand



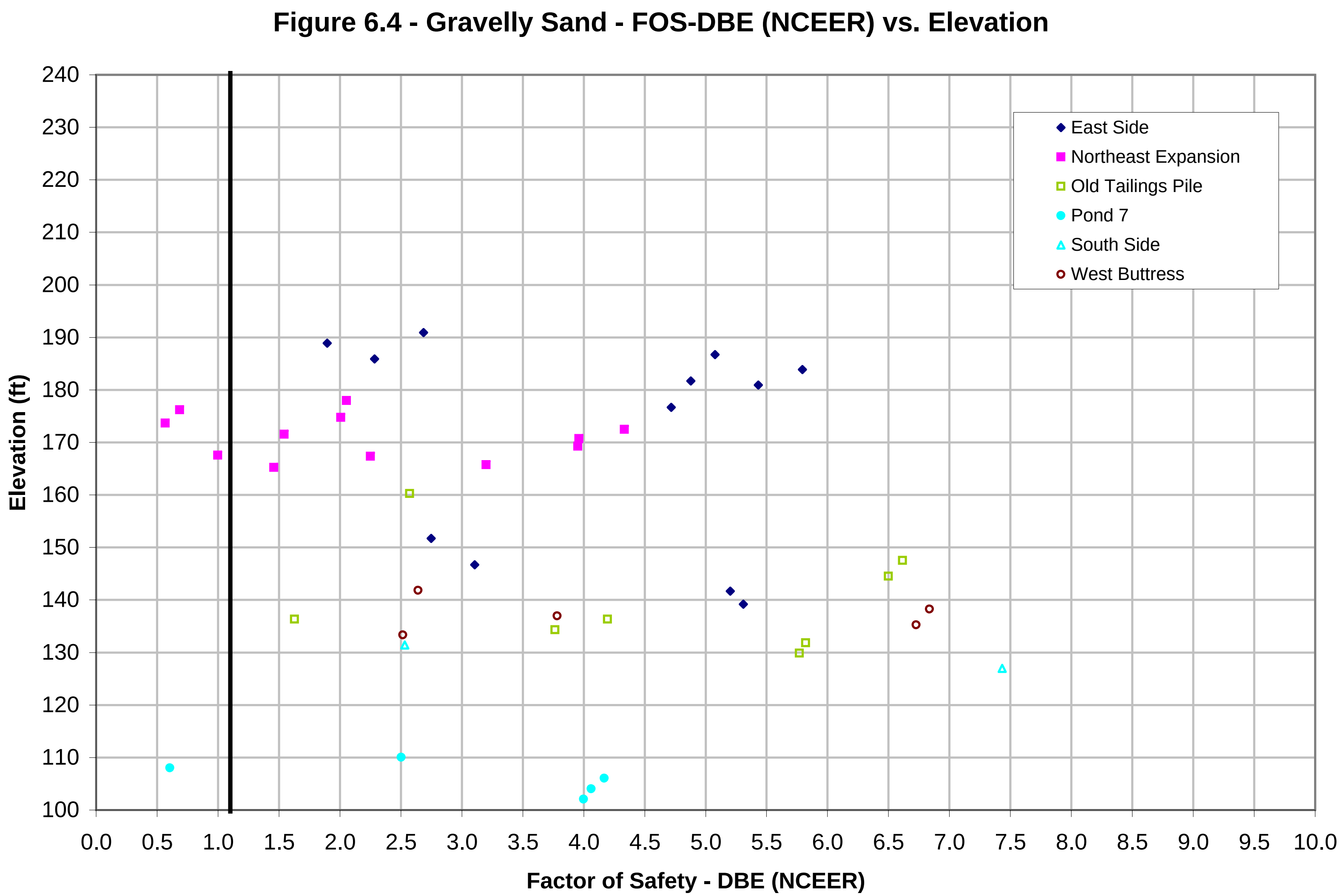


Figure 6.4 - Gravelly Sand - FOS-DBE (NCEER) vs. Elevation
Elevation (ft)
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
Factor of Safety - DBE (NCEER)
East Side
Northeast Expansion
Old Tailings Pile
Pond 7
South Side
West Buttress

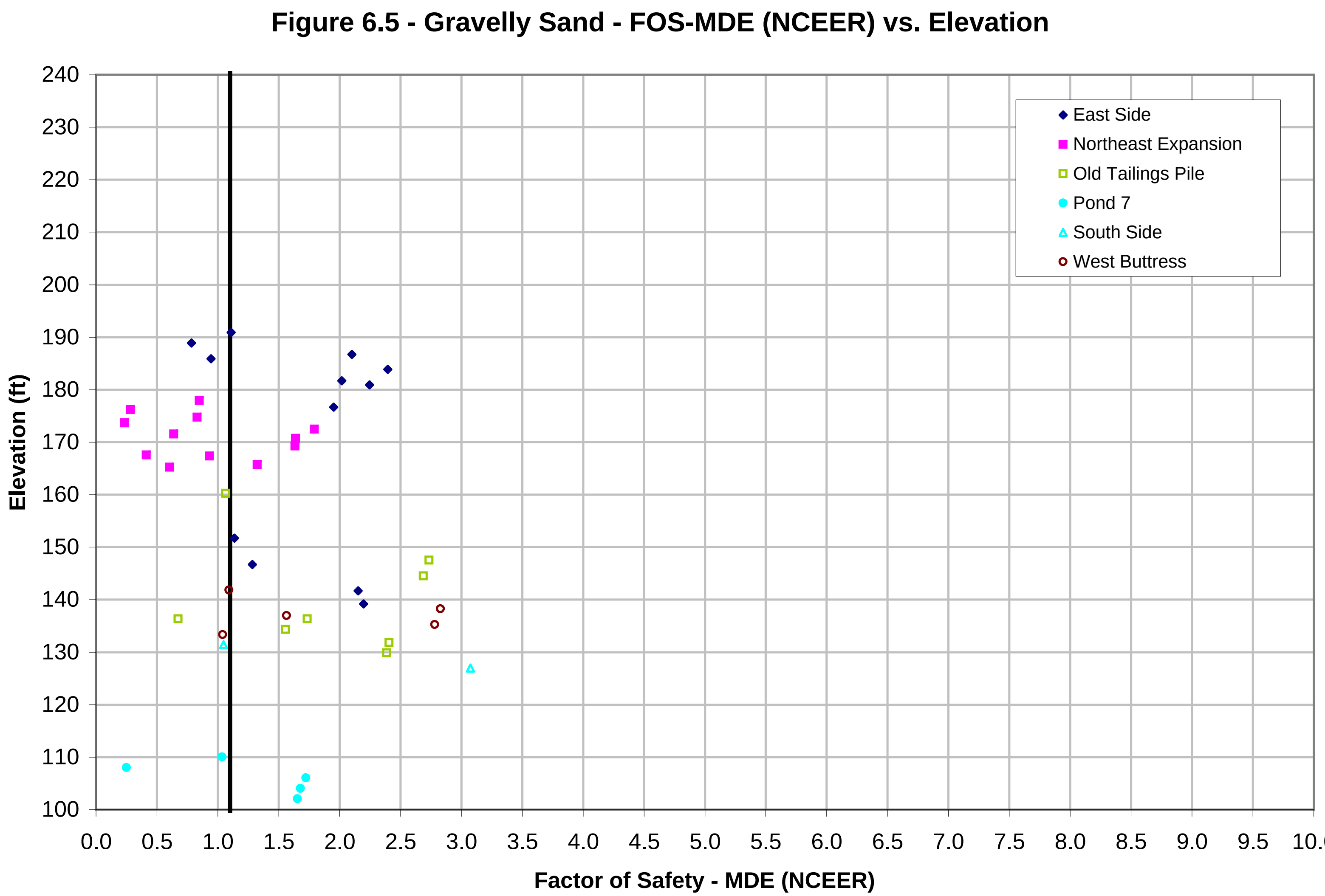


Figure 6.5 - Gravelly Sand - FOS-MDE (NCEER) vs. Elevation
Elevation (ft)
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
Factor of Safety - MDE (NCEER)
East Side
Northeast Expansion
Old Tailings Pile
Pond 7
South Side
West Buttress

Figure 7.1 - New Tailings - $(N_1)_{60\text{-cs}}$ vs. Elevation

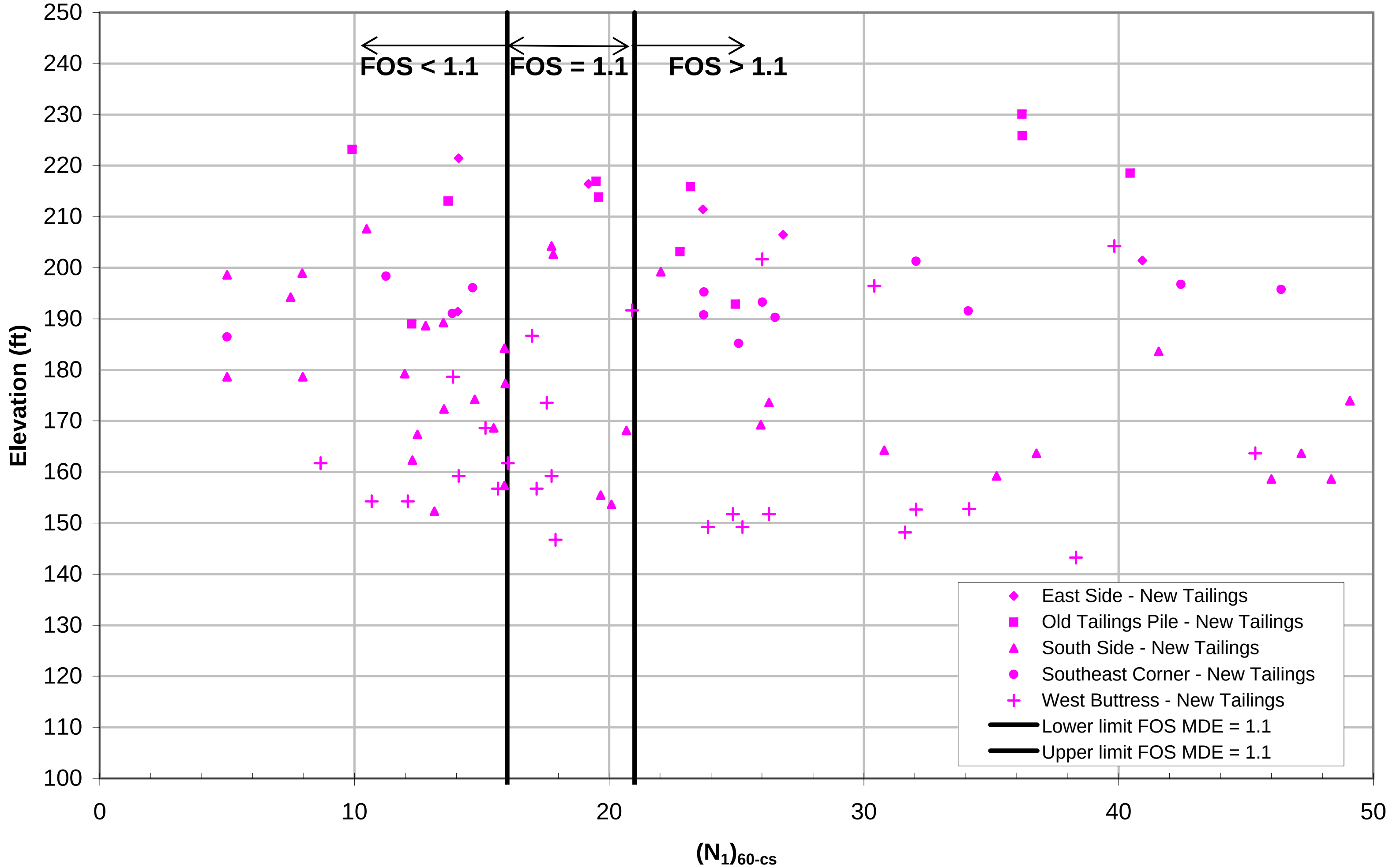
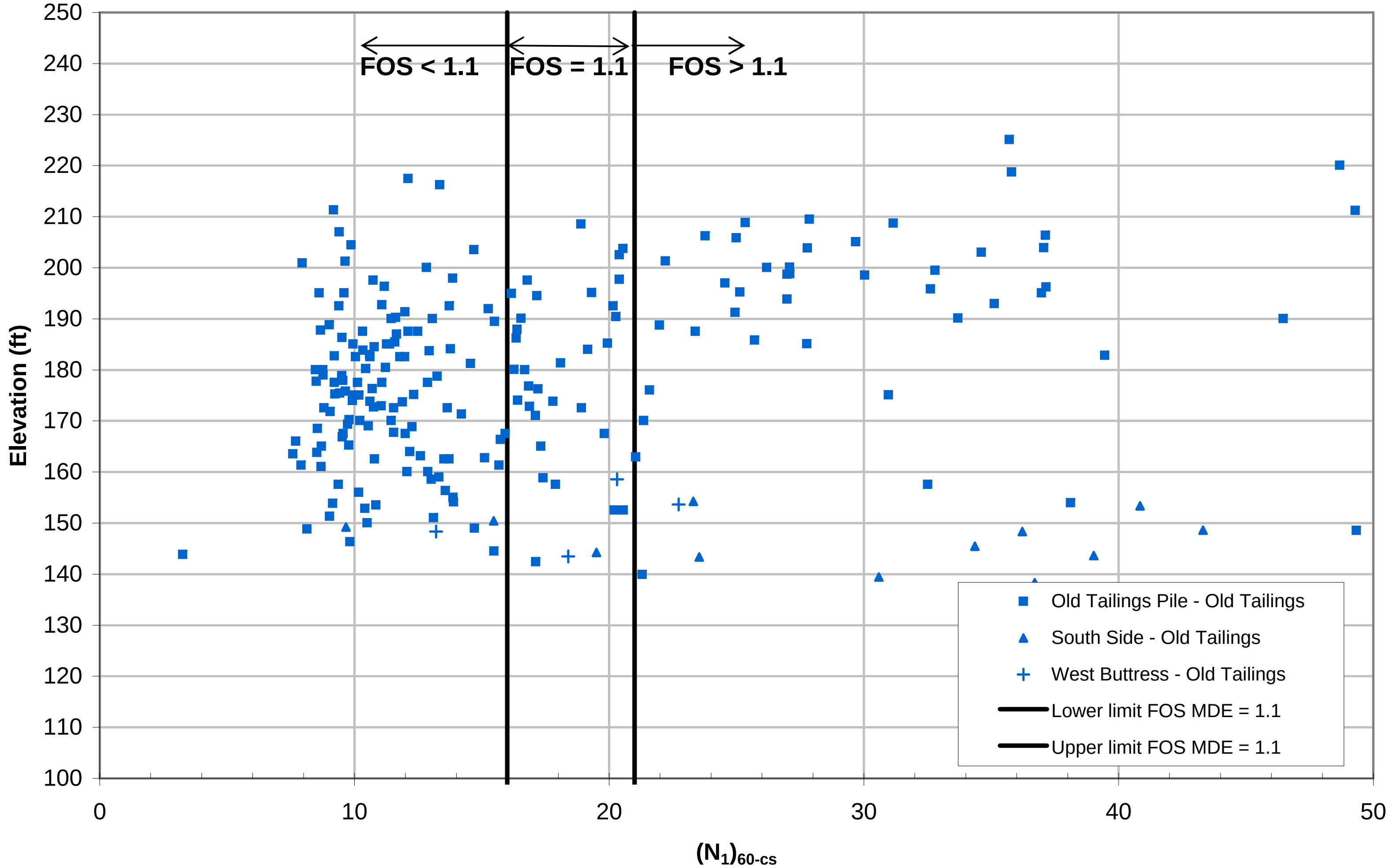


Figure 7.2 - Old Tailings - $(N_1)_{60\text{-}cs}$ vs. Elevation



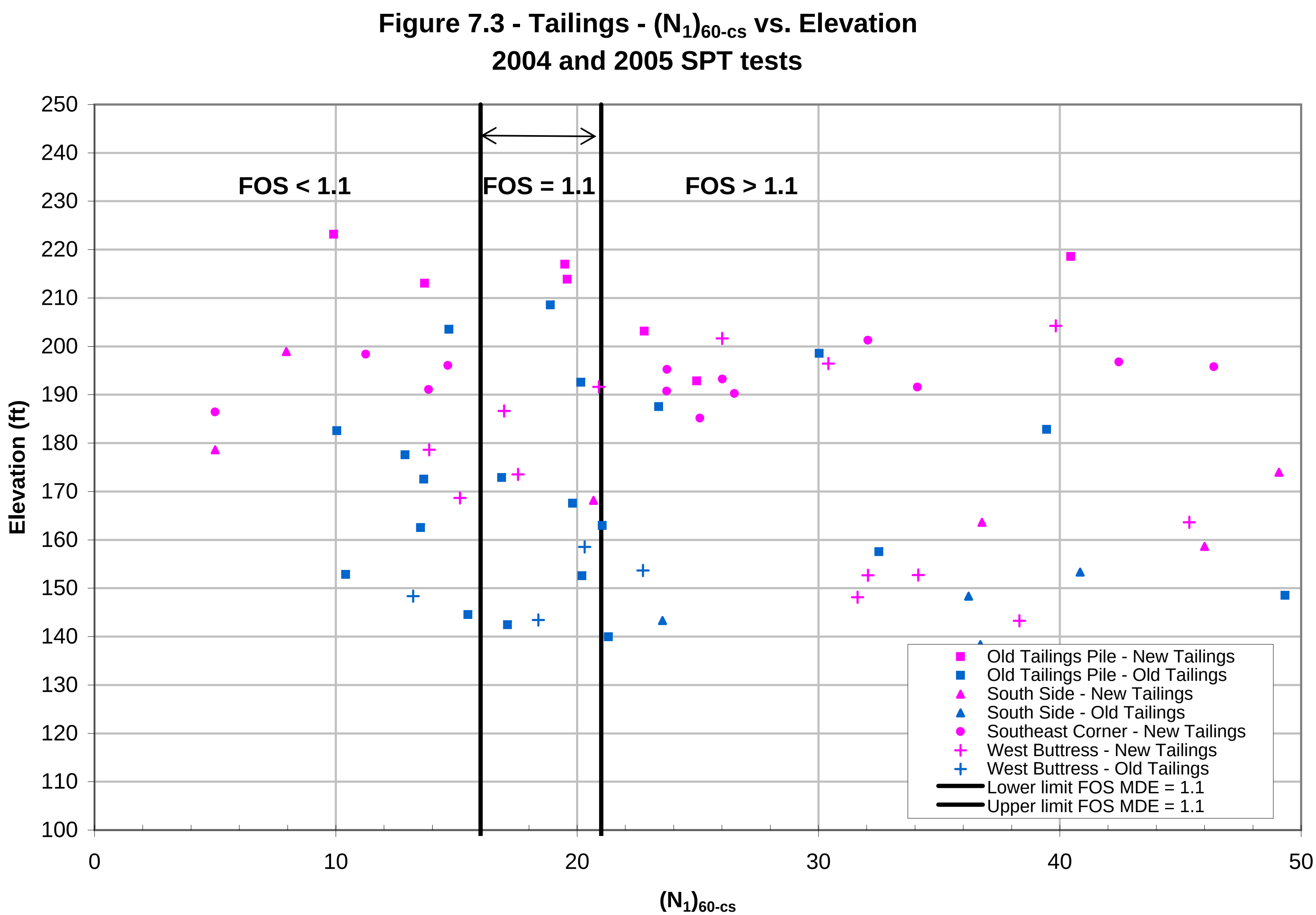


Figure 7.3 - Tailings - $(N_1)_{60\text{-cs}}$ vs. Elevation
2004 and 2005 SPT tests
FOS < 1.1
FOS = 1.1
FOS > 1.1
Elevation (ft)
$(N_1)_{60\text{-cs}}$
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
0
10
20
30
40
50
Old Tailings Pile - New Tailings
Old Tailings Pile - Old Tailings
South Side - New Tailings
South Side - Old Tailings
Southeast Corner - New Tailings
West Buttress - New Tailings
West Buttress - Old Tailings
Lower limit FOS MDE = 1.1
Upper limit FOS MDE = 1.1

Figure 7.4 - Tailings - $(N_1)_{60\text{-}cs}$ vs. Depth

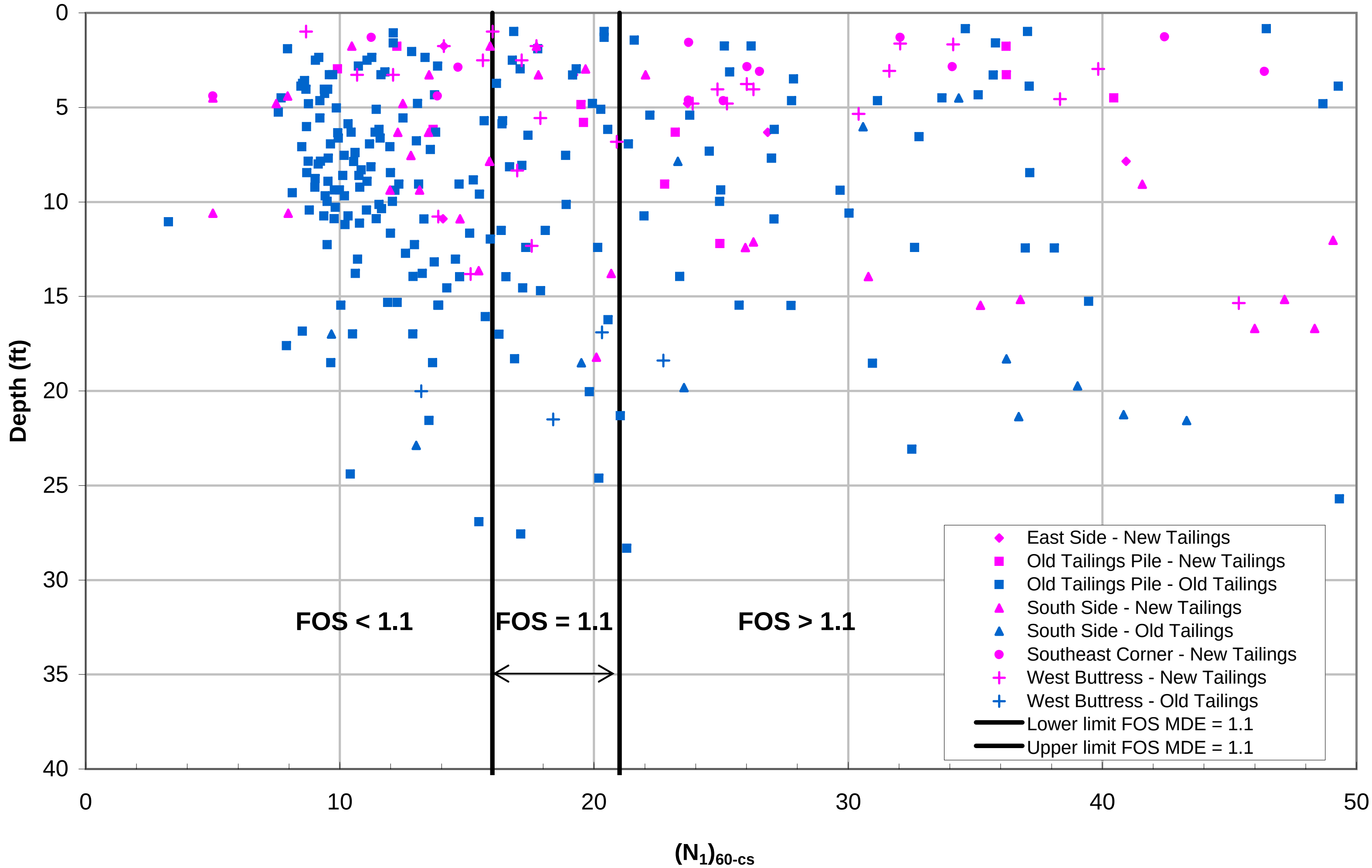


Figure 7.5 - Tailings - $(N_1)_{60\text{-}cs}$ vs. FOS MDE

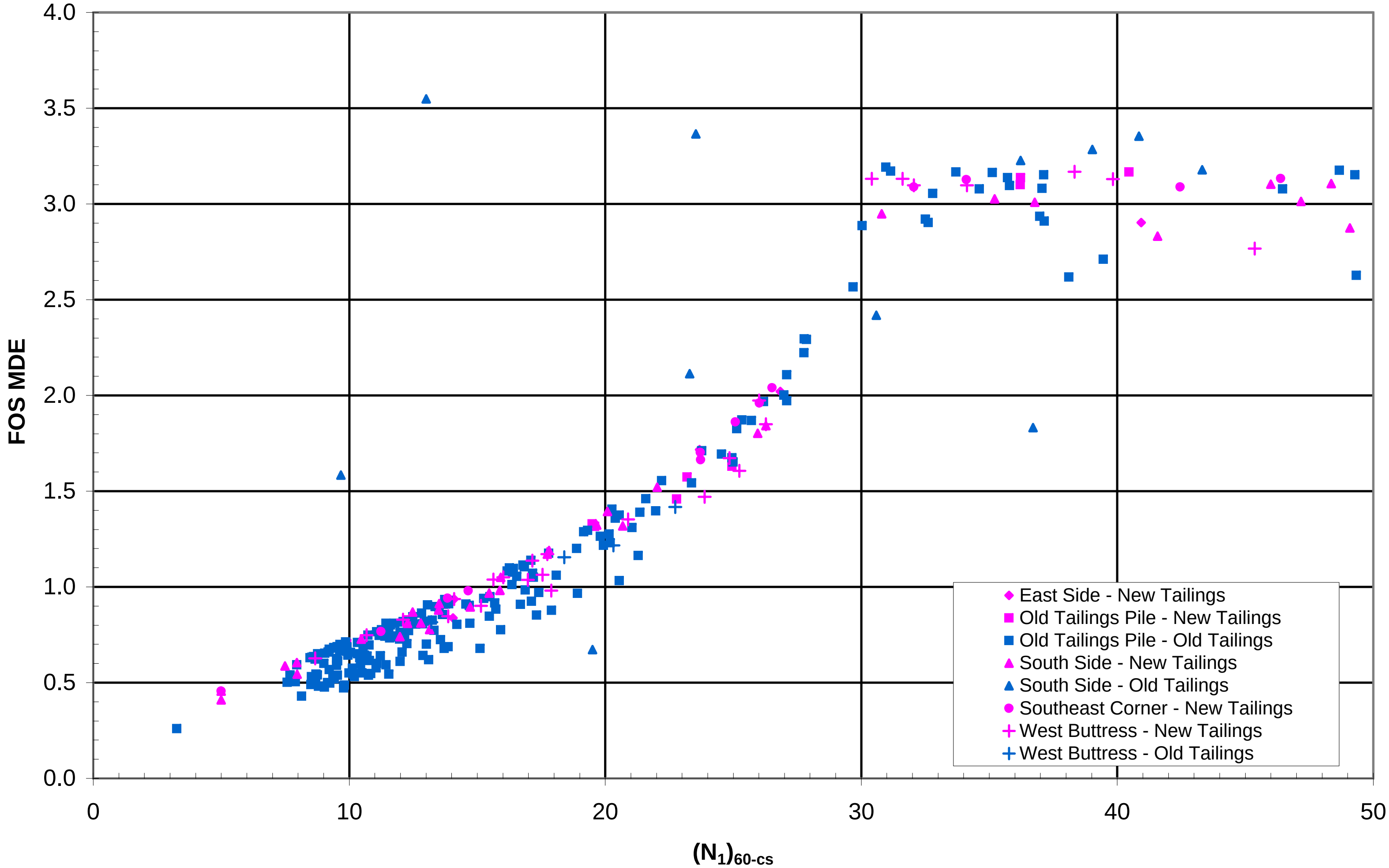
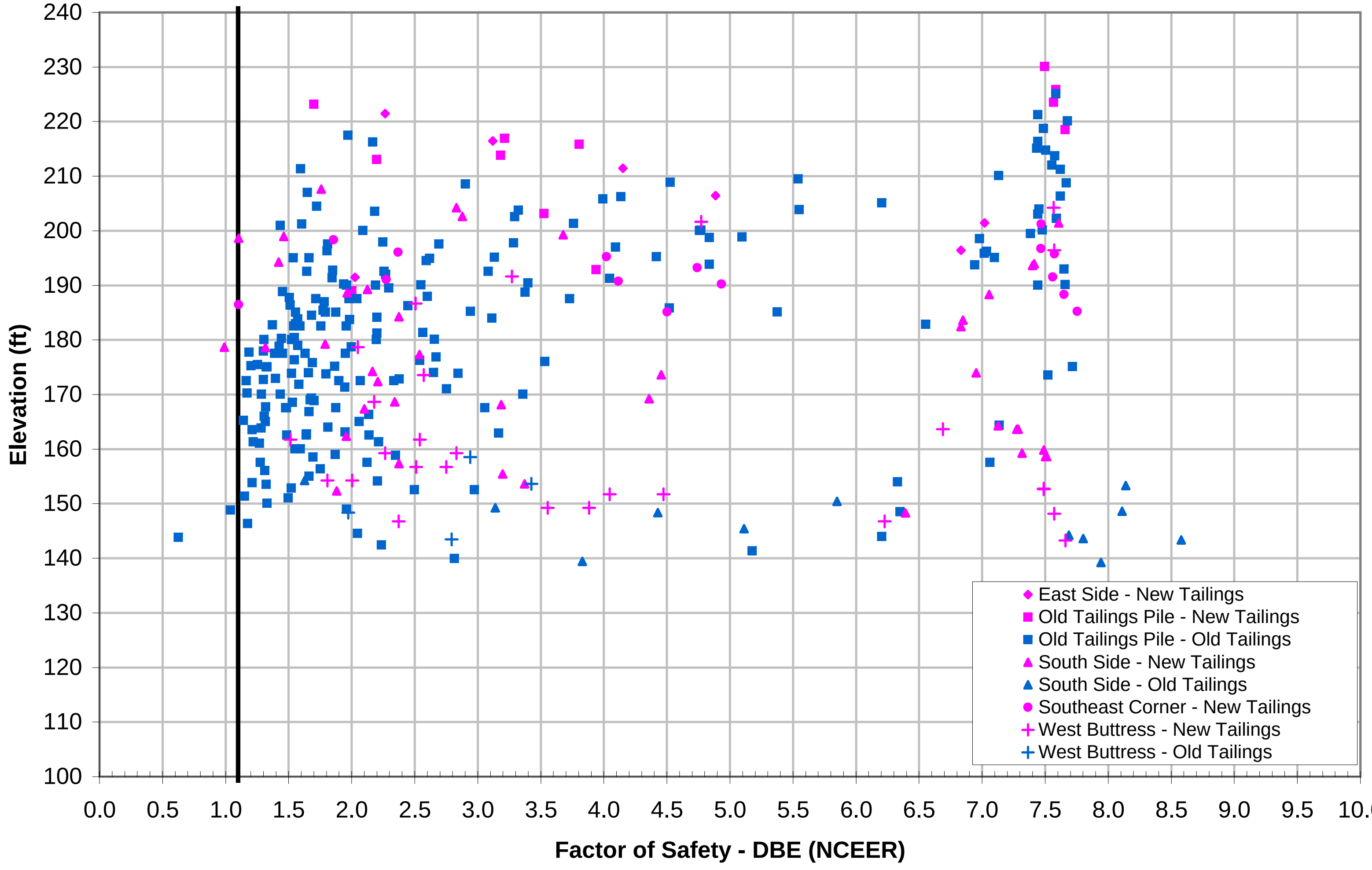
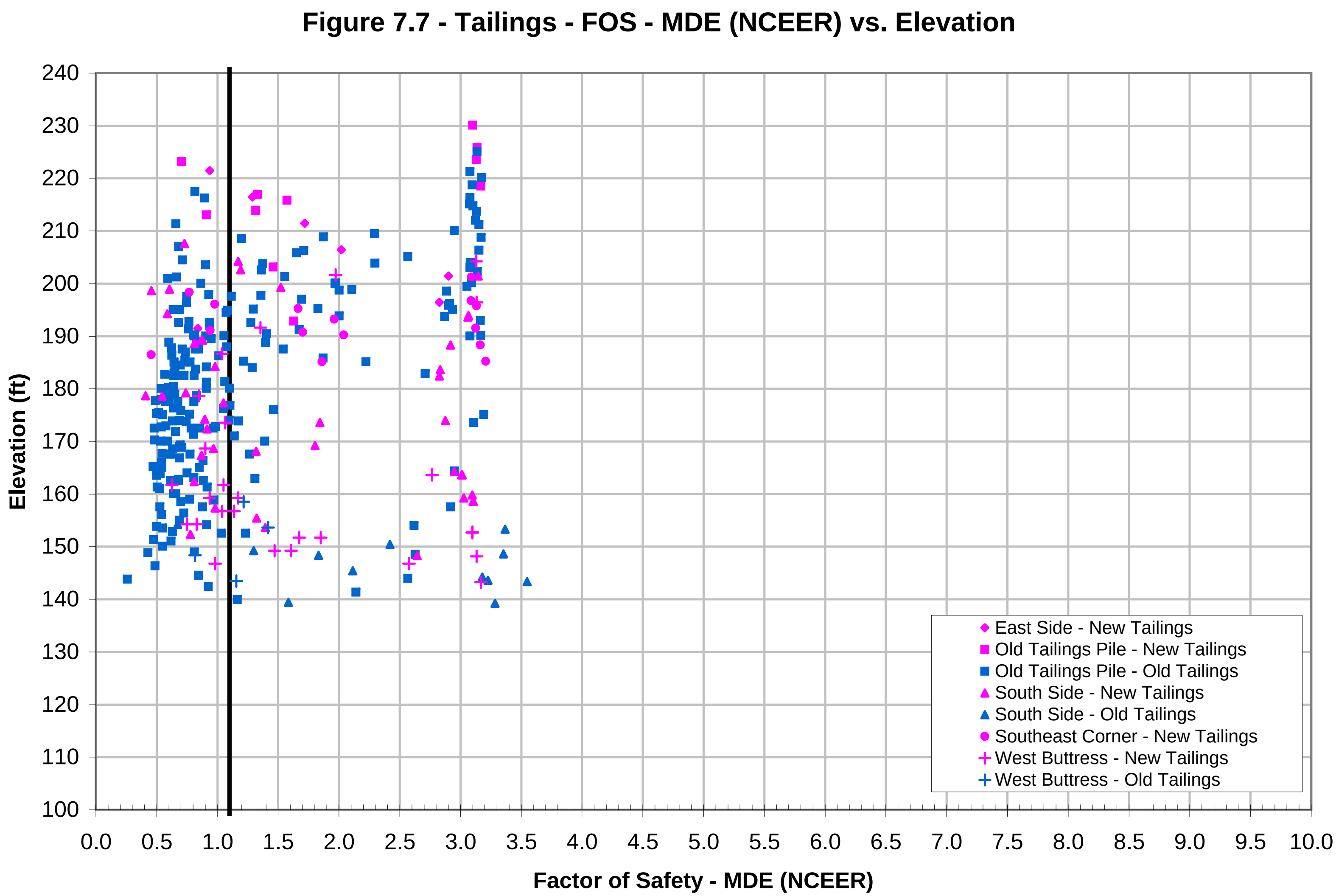


Figure 7.6 - Tailings - FOS - DBE (NCEER) vs. Elevation



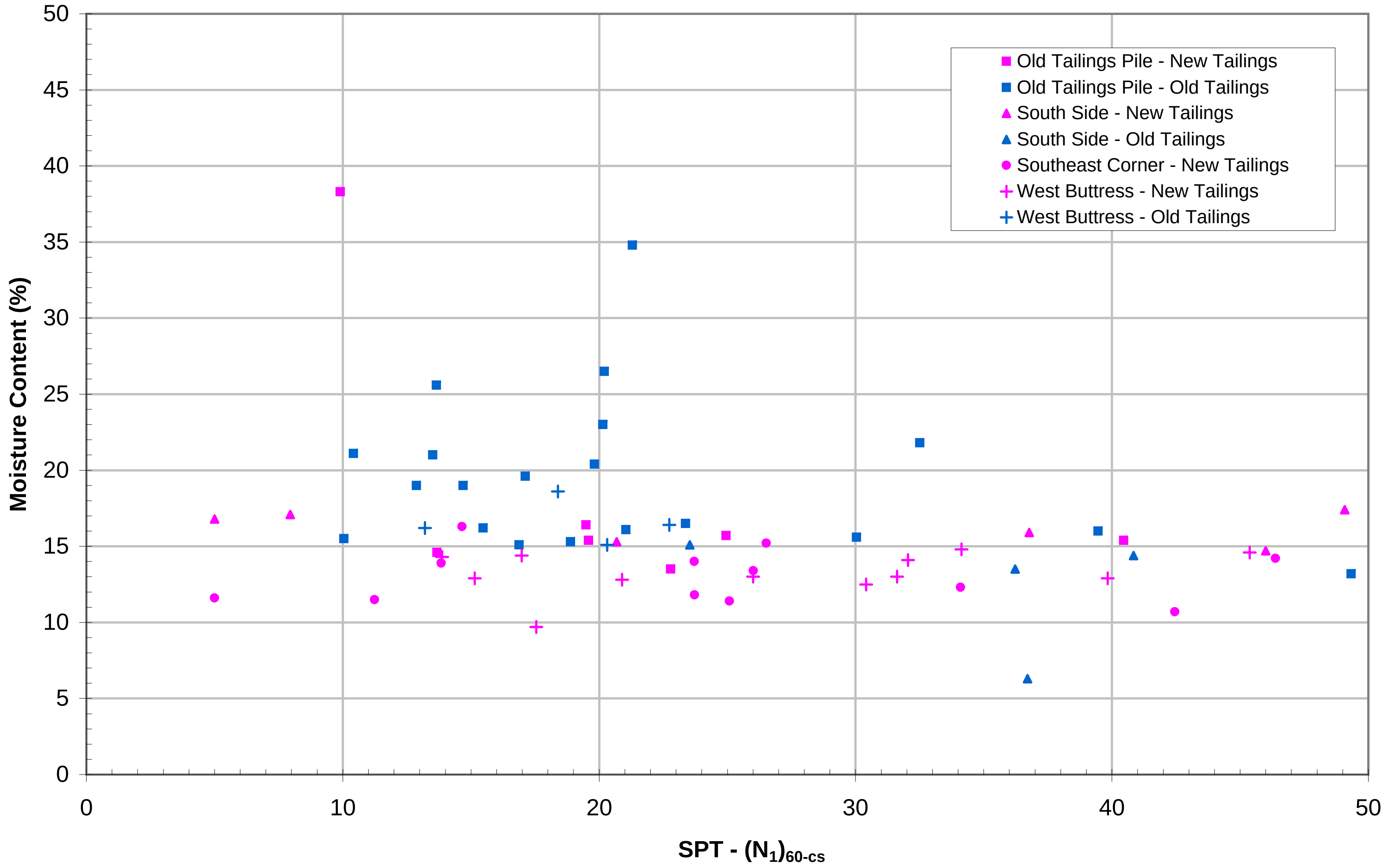
KENNECOTT GREENS CREEK MINING COMPANY
Stage 2 Tailings Expansion Overall Stability Update
Appendix III - Liquefaction Assessment - SPT



spt-all summary-060123.xls [Fig7.7 tailings-FOS-MDE(NCEER)]
M07802A41.420\SPT Data - All

KENNECOTT GREENS CREEK MINING COMPANY
Stage 2 Tailings Expansion Overall Stability Update
Appendix III - Liquefaction Assessment - SPT

Figure 7.8 - Tailings Moisture Content vs. $(N_1)_{60\text{-}cs}$



spt-all summary-060123.xls [Fig 7.8moisture-spt-tails chart]
M07802A41.420\SPT Data - All

APPENDIX IV

Liquefaction Assessment - CPT

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(continued)

IV-1. INTRODUCTION

An assessment of liquefaction susceptibility of tailings at the Greens Creek Mine was made under the Maximum Design Earthquake (MDE) loading using CPT data. This work was completed to check the conclusion from a report in 1997 by Klohn Crippen following a CPT test program and to assess whether CPT and SPT gave a similar estimate of liquefaction susceptibility. CPT interpretation methods have evolved in recent years and methods recommended in Youd et al. (2001) were mainly followed in this assessment.

In general, CPT data can either be analyzed directly to compare with liquefaction potential based on normalized tip resistance or can be converted to SPT equivalent and assessed based on SPT methods as discussed in Appendix III. Both approaches were adopted in this Appendix. To improve reliance on the CPT to SPT conversion nearby pairs of CPT and SPT holes were compared. A test hole pair on the old tailings pile (CPT97-16 and BH97-3) was mainly used, with a second check being made on hole pair CPT97-14 and DH02-08.

IV-2. DESIGN EARTHQUAKE

The liquefaction evaluation was carried out for the Maximum Design Earthquake (MDE). Klohn Crippen completed a detailed probabilistic and deterministic seismic hazard analyses for the Greens Creek mine site in 1998 (Klohn Crippen, 1998). Based on these analyses, the MDE is a magnitude 7 (M7.0) earthquake with Peak Firm Ground Acceleration (PGA) of 0.3 g.

IV-3. PIEZOMETRIC CONDITIONS

The following data were reviewed to estimate the piezometric surface in the tailings pile at the time of the testing of CPT97-16 and BH97-3:

- Pore pressure dissipation data from CPT97-16 and other CPTs in the vicinity of test hole CPT97-16;
- Historical piezometric data in the vicinity of test holes CPT97-16 and BH97-3 including PZ-41 to 45 and PZ-46 to 51; and
- Drill hole logs at DH02-04 and DH-18.

The estimated piezometric surface at the time of the testing of CPT97-16 and BH97-3 was taken as El. 176 ft, which is about 29 ft below the ground level at the time of drilling.

The liquefaction susceptibility of tailings was evaluated for this piezometric level and no downward gradient was considered in the evaluation (i.e. a hydrostatic condition was assumed).

IV-4. LIQUEFACTION POTENTIAL

Seismic liquefaction assessment was carried out in general accordance with the procedures recommended in Youd et al. (2001). In this method, the earthquake induced Cyclic Stress Ratios (CSR) are compared to the Cyclic Resistance Ratios (CRR) to determine whether the liquefaction will be triggered or not during the design earthquake.

IV-4.1 Cyclic Stress Ratio (CSR)

Seed's simplified method was used to determine the Cyclic Stress Ratio (CSR) induced by the earthquake. The peak ground acceleration (PGA) under MDE is taken as 0.3 g in the liquefaction evaluation and this value was applied at the surface of the tailings pile in all the analyses presented in this report.

The CSR is expressed as:

$$CSR = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\sigma_{vo}')\ r_d$$

in which a_{max} is the peak horizontal acceleration at the ground surface, g is the acceleration of gravity , σ_{vo} and σ_{vo}' are total and effective vertical stresses, respectively, and r_d is the stress reduction coefficient. The variation of r_d with depth proposed by Liao and Whitman (1986) and recommended in Youd et al. (2001) was used.

IV-4.2 Cyclic Resistance Ratio (CRR)

IV-4.2.1 Cyclic Resistance Ratio (CRR) for Reference Conditions

The CRRs are determined using both the SPT and CPT data and the liquefaction evaluation charts in Youd et al. (2001). These charts, which are reproduced in Figures 4.1 and 4.2, were derived from sites with known SPT blow counts or CPT data that have or have not liquefied during earthquakes. The SPT based chart was originally developed by Seed et al. (1985) and modified by Youd et al. (2001). The CPT based chart was originally developed by Robertson and Wride (1998) and adopted by Youd et al. (2001).

IV-4.2.2 Correction Factors (K_m, K_σ and K_α) for CRR

The CRRs in the SPT and CPT based liquefaction charts in Youd et al. (2001) are applicable for magnitude M7.5 earthquake and for an overburden stress of 1 tsf. Therefore, the CRRs obtained from these charts should be corrected for the design earthquake magnitude, M7.0 and for the location specific overburden pressure. The earthquake magnitude correction factor, K_m, and overburden correction factor, K_σ, recommended in Youd et al. (2001) were used. In the estimation of $K_{\sigma,}$ the relative density, D_r of the tailings was taken as 50%. The correction factor for the initial static shear stress, K_α was assumed to be 1.0 as in common practice.

IV-5. EVALUATION OF LIQUEFACTION USING SPT DATA

The liquefaction potential was evaluated using the measured SPT blow counts at BH97-3. In addition, the CPT tip resistance at CPT97-16 was converted into equivalent SPT blow counts and used in the evaluation.

IV-5.1 Estimation of $(N_1)_{60cs}$ from Measured SPT Blow Counts

The measured SPT blow count N_m are corrected to obtain $(N_1)_{60}$ as follows (Youd et al., 2001):

$$(N_1)_{60} = N_m\, C_N C_E C_B C_R C_S$$

in which C_N is the factor to normalize N_m to an effective overburden stress of 1 tsf, C_E is the correction for hammer energy ratio (ER), C_B is the correction factor for borehole diameter, C_R is the correction factor for rod length and C_S is the correction factor for samplers with or without liners.

The corrected $(N_1)_{60}$ values are then corrected for the effect of fines content to obtain the clean sand corrected SPT blow count $(N_1)_{60cs}$ as follows (Youd et al. (2001):

$$(N_1)_{60cs} = \alpha+\beta\ (N_1)_{60}$$

in which α and β are coefficients depending on the fines content (FC). For FC greater than or equal to 35%, the values of α and β are 5 and 1.2, respectively.

IV-5.1.1 Correction Factor for Overburden Stress, C_N

Since SPT blow count value increases with increasing overburden stress, overburden correction factor, C_N is applied to normalize the measured SPT blow count to an effective overburden stress of approximately 1 tsf. The factor, C_N was estimated using the equation proposed by Liao and Whitman (1986) and recommended in Youd et al. (2001).

IV-5.1.2 Correction for Hammer Energy Ratio (ER), C_E

An important factor in the interpretation of SPT blow counts is the energy transferred to SPT sampler from the falling hammer. An energy ratio (ER) of 60% is considered as the reference value for energy corrections and the correction factor for energy, C_E is defined as C_E =ER/60. The energy ratio delivered to the sampler depends on several factors including the type of hammer, hammer weight, drop height, lifting mechanism, size of the sampler etc.

The SPT measurements at BH97-3 were carried out using a safety hammer and rope and cathead system. The quoted drop height of the hammer was 18 inches. No energy measurements were carried out at this borehole. However, at a nearby borehole BH97-2, energy measurements were carried which showed that the average energy delivered to the sampler during testing was about 35% (Cone Tec, 1997). Thus, the same average energy of 35% was assumed for the SPT measurements taken at borehole BH97-3 and the correction factor, C_E was estimated for this energy ratio.

IV-5.1.3 Correction Factors, C_B, C_R, and C_S

The recommendations given in Youd et al. (2001) were closely followed to estimate the correction factors, C_B, C_R, and C_S.

IV-5.1.4 Correction Factors for Fines Content (FC), α and β

The apparent fines content as defined by Robertson and Wride (1998) were estimated from the CPT data at CPT97-16. Grain size analysis of SPT samples at BH97-3 was unavailable. However, grain size analysis of tailings showed that the fines content are generally very high (>80%). Typical grain size curves of tailings are shown in Figure 5.1 (Klohn Crippen, 2003).

IV-5.2 Estimation of $(N_1)_{60cs}$ from Measured CPT Data

CPT tip resistance at CPT97-16 was converted into equivalent SPT blow counts ($(N_1)_{60cs}$) and used in the liquefaction resistance evaluation. The methods proposed by the following were used for conversion:

- Jeffries and Davies (1993): They related the q_c/N_{60} (q_c is the cone tip resistance and N_{60} is the SPT blow count with 60% energy ratio) ratio to the soil classification index, I_c. I_c is a function of stress normalized cone tip resistance, Q, stress normalized friction ratio, F and pore pressure ratio, B_q.

- Robertson et al. (1983): They related q_c/N_{55} (N_{55} is the SPT blow count with 55 energy ratio) to the mean grain size, D_{50} (0.02 mm as per Figure 5.1).

- Stark and Olson (1995): They related q_c/N_{60} ratio to the mean grain size, D_{50} (0.02 mm as per Figure 5.1). The Stark and Olson's proposed relationship agreed with those by Seed and De Alba (1986) and Kulhawy and Mayne (1990) for D_{50} in the range from about 0.02 mm to 0.07 mm.

IV-5.2.1 Liquefaction Potential Based on SPT Blow Counts at BH97-3 and CPT97-16

In hole BH97-3 the pre 1993 (old) tailings surface elevation was about El. 190 ft, the elevation at time of drilling was El. 204.9 ft. Consequently the top 15 ft or so represent new tailings compacted to current standards. Inspection of Figure 5.2 shows the new tailings, plus about 3 ft or so below the old tailings surface to be much denser than old tailings.

Figure 5.2 shows the SPT $(N_1)_{60cs}$ profiles estimated from both measured SPT N and converted N values from CPT tip resistance. The $(N_1)_{60cs}$ corresponding to the earthquake induced CSR for MDE with FOS of 1 and 1.1 were back calculated. These profiles are also shown in Figure 5.2. The estimated water surface location at the time of testing is also shown in Figure 5.2. The key results are summarized below:

- The converted SPT blow counts from the measured CPT data by the three methods agree well below a depth of about 20 ft. From 8 ft to 20 ft depth, the Jeffries and Davies (1993) CPT conversion method under estimated the blow counts compare to the other two methods. Above about 20 ft depth, corresponding to the new tailings and the old running surface on top of the old tailings, the $(N_1)_{60\text{-}cs}$ exceed the liquefaction requirements.

- The $(N_1)_{60cs}$ estimated from both SPT data at BH97-3 and CPT data at CPT97-16 agree very well.

- The $(N_1)_{60cs}$ required to have a FOS of 1 and 1.1 (demand) below 20 ft depth and below the water table are much higher than the $(N_1)_{60cs}$ estimated from the measured values (resistance), indicating potential liquefaction. However, due to the high fines content of the tailings, its liquefaction susceptibility was also screened using the criteria for fine grained soils as discussed below.

IV-6. DIRECT EVALUATION OF LIQUEFACTION USING CPT DATA

The liquefaction potential of tailings was evaluated directly using the measured CPT data at CPT 97-16 (i.e. without first converting to SPT). The method outlined in Youd et al. (2001) was used to determine, $q_{c1Ncs,}$ the clean-sand cone penetration resistance normalized approximately to 1 tsf overburden stress.

Figure 6.1 shows the estimated q_{c1Ncs} profile with depth. It also shows the soil behavior type index, I_c, apparent fines content (interpreted from the cone response), FC, and the soil type as defined by Robertson and Wride (1998).

The q_{c1Ncs} corresponding to the earthquake induced CSR for MDE with FOS of 1 and 1.1 were back calculated and shown in Figure 6.1. The key results are summarized below.

- The apparent fines content shows that the tailings below 20 ft depth is 100% fines. However, the apparent fines content up to 20 ft depth was less than 40%.
- The soil behavior type index, I_C of greater than 2.6 below 20 ft depth suggests that the tailings is “clayey”, or at least behaving as a clayey soil.
- q_{c1Ncs} profiles suggest that, except for some thin layers, the tailings below 20 ft is unlikely to liquefy as it is “too-clay rich “ to liquefy according to Robertson and Wride (1998).

Figure 6.2 shows the CPT97-16 data on the CPT-based soil behavior type chart proposed by Robertson (1990). Also shown on the figure are the lines of constant I_C values. Most tailings data deeper than 20 ft fall in the region classified as “Clays” and “Silt Mixtures” and the corresponding I_C values was greater than 2.60. Majority of these data are in the region classified as “Clays” and the corresponding I_C values was greater than 2.95.

Youd et al. (2001) suggest that for soils that are classified as “clayey” according to the CPT based method soil sample should be retrieved and tested to confirm the soil type and liquefaction resistance. They also suggest that the criteria such as Chinese criteria (which is used to evaluate the liquefaction susceptibility of fine grained soils) can be used as screening tools.

IV-7. ANALYSIS OF HOLES CPT97-14 AND DH02-08

Using similar methods to that described for holes CPT97-16 and BH97-3, a comparison of SPT data from holes CPT97-14 and DH02-08 was made. The methods and the results are as follows:

- In DH02-08, the energy ratio was assumed as 60% and the depth to the water table was taken as 29.9 ft based on the borehole log (Water level on Sept. 30, 2002). The surface elevation of this borehole is 194.8 ft.
- In CPT97-14, the surface elevation was taken as 180.9 ft and the depth to the water table was taken as 27.5 ft based on the porewater pressure dissipation data.
- CSR was computed for conditions during CPT97-14.
- CPT97-14 consists of old tailings throughout; and DH02-08 has all old tailings, except for the topmost 6 ft.

The results are summarized in Figures 7.1 and 7.2. The results were similar to those for hole pair CPT97-16 and BH97-3 in that the SPT data indicate liquefaction susceptibility while the direct CPT analysis indicates that the material is too clayey to be susceptible to liquefaction.

IV-8. SUMMARY AND CONCLUSIONS

The criteria for liquefaction based on SPT $(N_1)_{60cs}$ was satisfied for most of the submerged tailings analyzed. However, the criteria based on CPT q_{c1Ncs} was not satisfied and the high values of soil behavior type index, I_C greater than 2.6 suggested that the tailings may be too clayey-rich to liquefy. Note that Youd et al. (2001) suggested that the soils classified as “clayey” according to the CPT based method should be tested in the lab to confirm the soil type and liquefaction resistance. Laboratory testing was undertaken and is discussed in the main report.

REFERENCE

Andrews, D. C. A. and Martin, G. R. (2000). Criteria for Liquefaction of Silty Soils. 12th World Conference in Earthquake Engineering, Paper No. 312.

Bray, J.D., Sancio, R.B., Riemer, M.F. and Durgunoglu, T. (2004). Liquefaction Susceptibility of Fine-Grained Soils. 11th International Conf. on Soil Dynamics and Earthquake Engineering and 3rd International Conf. on Earthquake Geotechnical Engineering, Berkeley, CA, Jan. 7-9, 2004.

Cone Tec (1997). Field Report – Cone Penetration Testing and SPT Energy Calibration at Green Creek Mine. November 5, 1997.

Idriss, I. M. and Boulanger, R. W. (2004). Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes. 11th International Conf. on Soil Dynamics and Earthquake Engineering and 3rd International Conf. on Earthquake Geotechnical Engineering, Berkeley, CA, Jan. 7-9, 2004.

Idriss, I.M. and Sun, J.I. (1992). User's Manual for SHAKE91. Center for Geotechnical Modeling, Dept. of Civil and Environ. Eng., University of California, Davis, CA.

Jeffries, M. G. and Davies, M. P. (1993). Use of CPTs to Estimate Equivalent SPT N_{60}. Geotechnical Testing Journal, Vol. 16, No. 4, pp. 458-468.

Klohn Crippen (2003). Kennecott Greens Creek Mine 2002 Geotechnical Investigation Field Report. April 3, 2003.

Klohn Crippen (1998). Seismic Hazard Assessment at Greens Creek Mine Site, Admiralty Island, Alaska, August.

Koester, J. P. (1992). The Influence of Test Procedure on Correlation of Atterberg Limits with Liquefaction in Fine Grained Soils, Geotechnical Testing Journal, Vo. 15, No. 4, pp. 352-361.

Kulhawy, F. H. and Mayne, P. W. (1990). Manual on Estimating Soil Properties for Foundation Design, Electric Power Res. Inst. EL-6800; Prof. 1493-6, Electric Power Res. Inst., Palo Alto, Calif., 2-38.

Liao, S. and Whitman, R. V. (1986). Overburden Correction Factors for SPT in Sand. ASCE Journal of Geotechnical Engineering, Vol. 112, No. 3, pp373-377.

Lysmer, J.; Udaka, T.; Tsai, C.-F.; Seed, H. B. (1975). FLUSH: : a computer program for approximate 3-D analysis of soil-structure interaction problems. UCB/EERC-75/30, Berkeley: Earthquake Engineering Research Center, University of California, Nov. 1975, 139 pages.

Robertson , P. K., Campanella, R. G. and Wightman, A. (1983). SPT-CPT Correlations, ASCE Journal of Geotechnical Engineering, Vo. 109, No. 11, pp. 1449-1459.

Robertson, P. K. (1990). Soil Classification Using CPT, Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 151-158.

Robertson, P. K., and Wride, C. E. (1998). Evaluation of Cyclic Liquefaction Potential Using the Cone Penetration Test, Canadian Geotechnical Journal, Vol. 35, No. 3, pp. 442-459.

Schnabel, P.B., Lysmer, J., and Seed, H.B., (1972). SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. Report No. EERC 72-12, Earthquake Engineering Research Centre, University of California, Berkeley, Calif., Feb.

Seed, H.B., Tokimatsu, K., Harder, L. F. and Chung, R. M. (1985). The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, ASCE Journal of Geotechnical Engineering, Vol. 111, No. 12, pp1425-1445.

Seed, H. B. and De Alba, (1986). Use of SPT and CPT tests for Evaluating the Liquefaction Resistance of Sand, Proc. Insitu ’86, ASCE Spec. Conf. On Use of Insitu Testing in Geotech Engrg., ASCE Special Publication No. 6.

Stark, T. M. and Olson, S. M. (1995). Liquefaction Resistance Using CPT and Field Case Histories, ASCE Journal of Geotechnical Engineering, Vo. 121, No. 12, pp. 856-869.

Youd, T.L., Idriss, I.M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson III, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., Stoke II, K. H. (2001). 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.

FIGURES

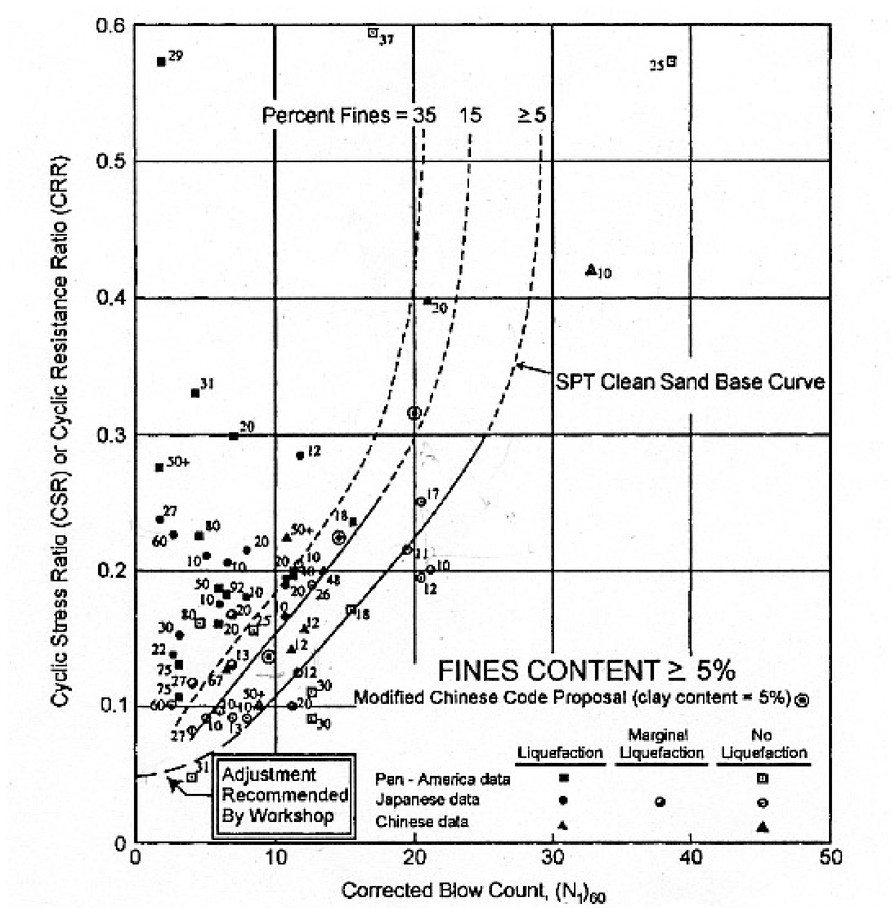


Figure 4.1 SPT Based Liquefaction Resistance Chart (Youd et al., 2001)

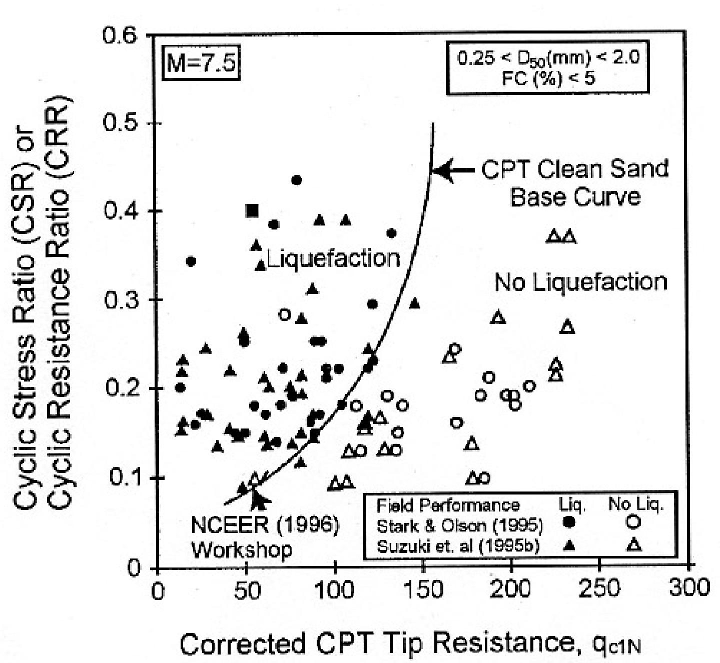


Figure 4.2 CPT Based Liquefaction Resistance Chart (Youd et al., 2001)

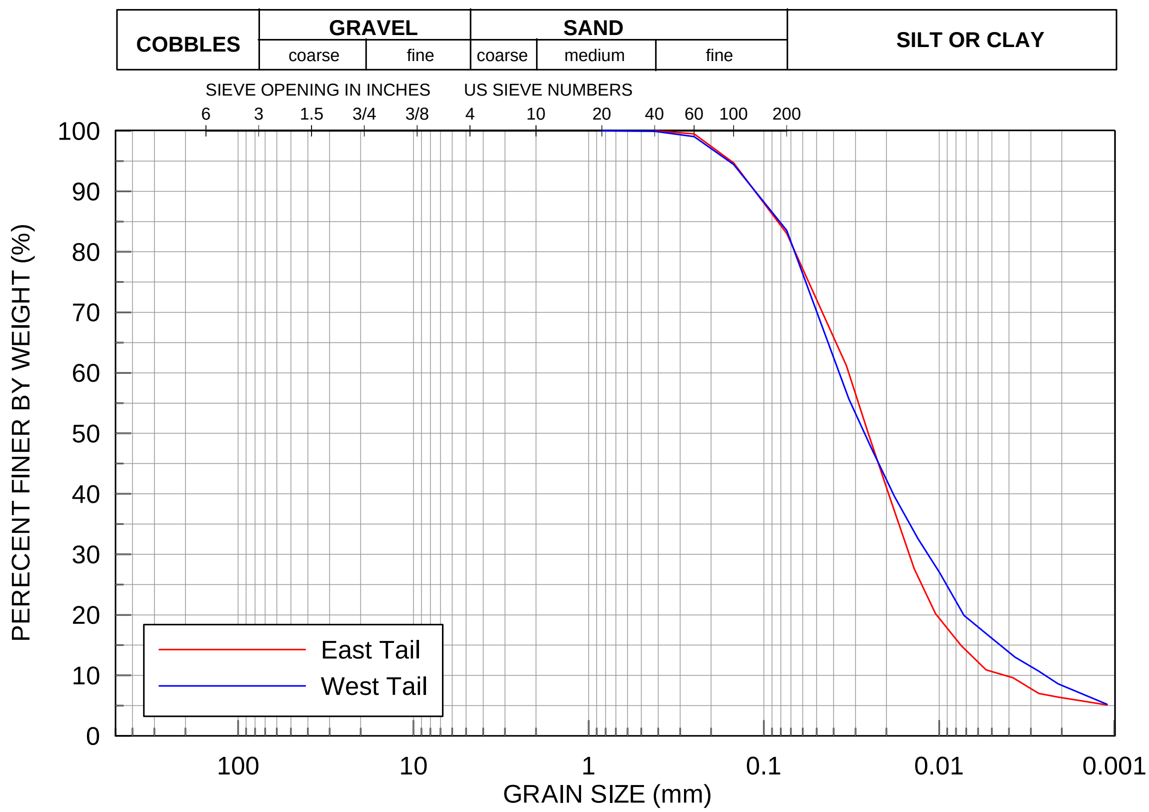


Figure 5.1 Typical Grain Size Curves for Tailings (KC 2003)

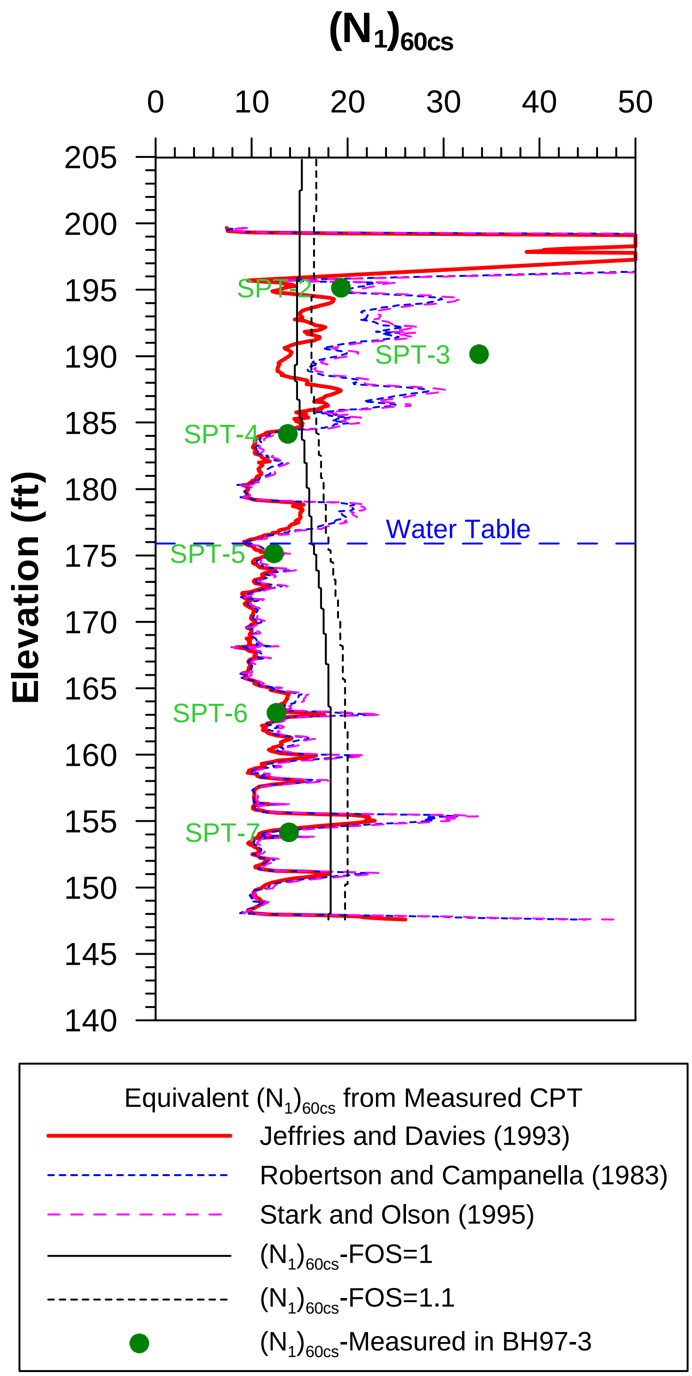


Figure 5.2 Liquefaction of Old Tailings Pile Using SPT Based Method (CPT97-16 and BH97-3)

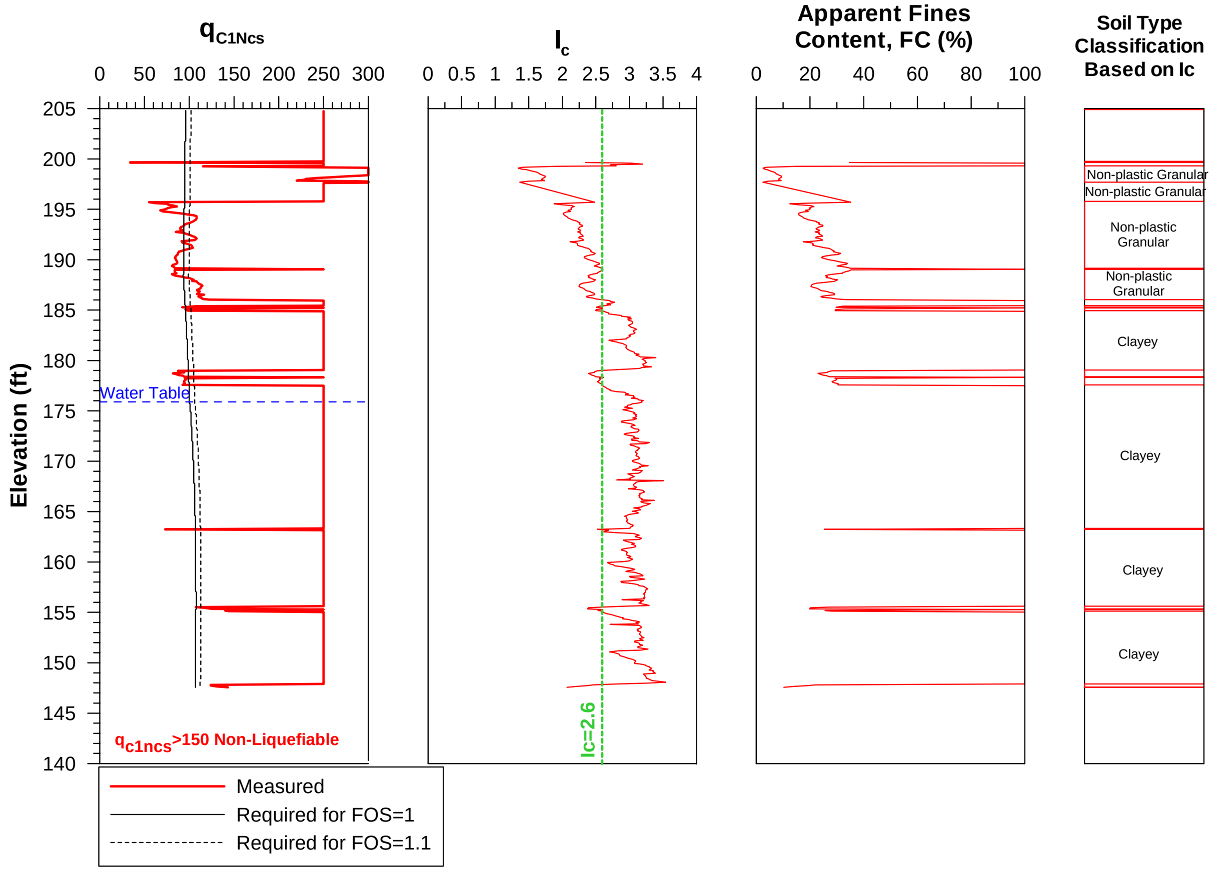
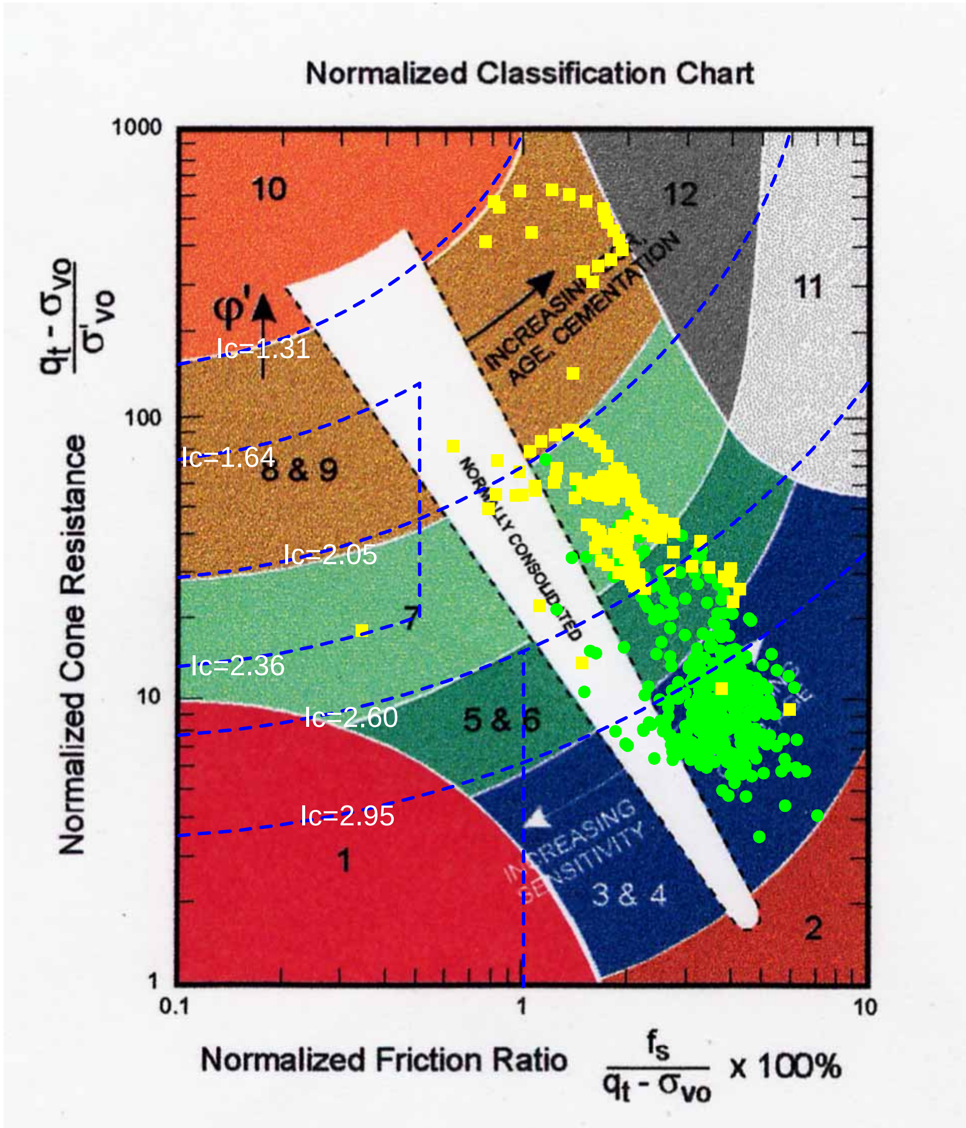


Figure 6.1 Liquefaction of Old Tailings Pile Using CPT Based Method (CPT97-16)



Zone	Soil Behaviour Type
1	Sensitive, fine grained
2	Organic soils – peats
3	Clays – silty clay to clay
4	Silt mixtures - clayey silt to silty clay
5	Sand mixtures – silty sand to sandy silt
6	Sands – clean sand to silty sand
7	Gravelly sand to dense sand
8	Very stiff sand to clayey sand
9	Very stiff, fine grained

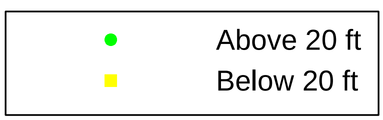


Figure 6.2 Classification of Tailings Data from CPT97-16

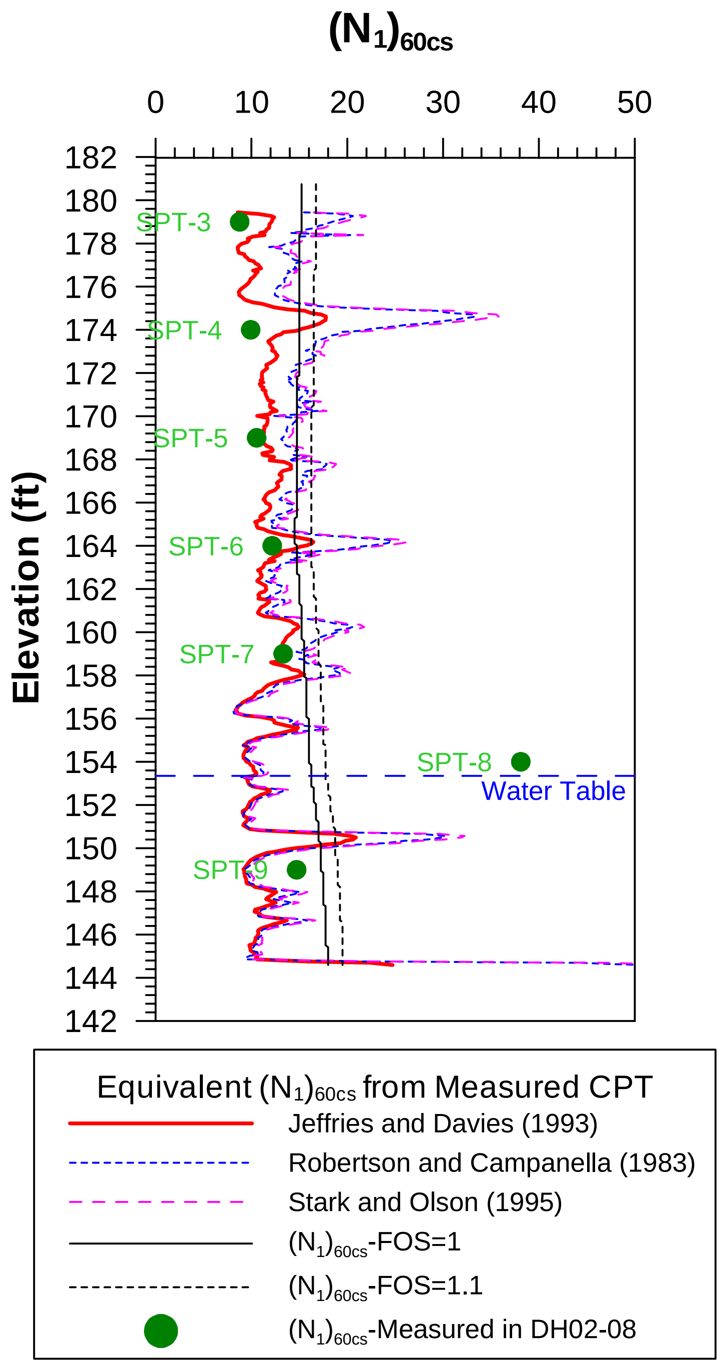


Figure 7.1 SPT Based Method of Liquefaction Evaluation at CPT97-14 and DH02-08

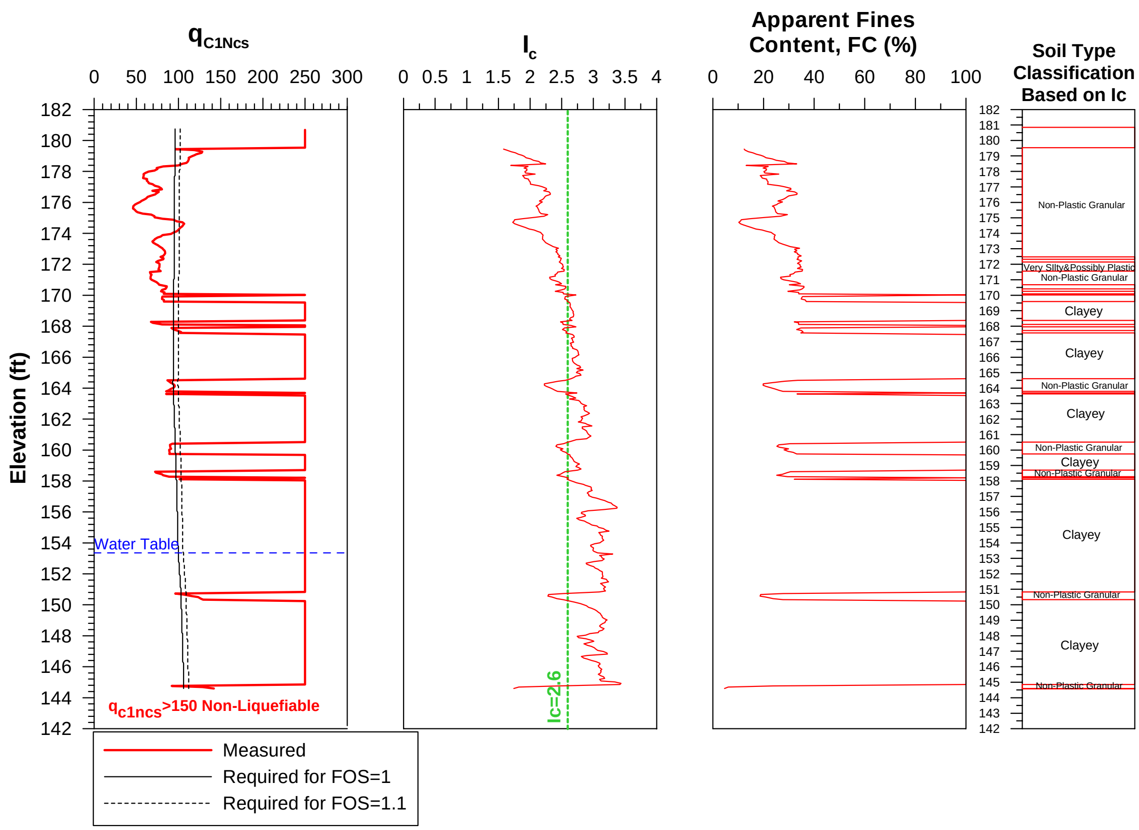


Figure 7.2 CPT Based Method of Liquefaction Evaluation at CPT97-14

APPENDIX V

Drill Hole Logs

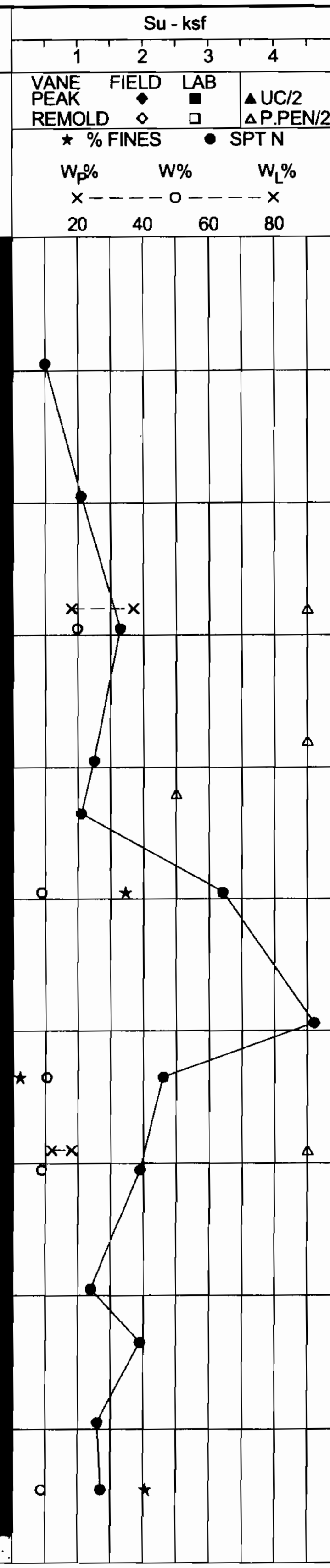
DH-05-06	DH-02-07
DH-05-07	DH-02-08
DH-05-08	DH-02-10
DH-05-09	DH-01-01
DH-05-10	DH-01-02
DH-05-11	DH-01-03
DH-05-12	DH-01-04
DH-05-13	DH-01-11
DH-05-14	DH-00-04
DH-05-15	DH-00-05
DH-05-16	DH-00-06
DH-05-17	DH-00-11
DH-05-18	DH-00-12
DH-05-20	DH-00-13
DH-04-01	BH-97-01
DH-04-02	BH-97-02
DH-04-03	BH-97-03
DH-04-04	TA-1
DH-04-05	TA-2
DH-04-06	TA-3
DH-04-07	TA-4
DH-04-08	TA-5
DH-04-09	TB-1
DH-04-11	TB-2
DH-02-04	TB-3
DH-02-05	TB-4
DH-02-06	TB-5

TEST HOLE LOG

STARTED: 3/19/2005 **FINISHED:** 3/19/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 114.2
COORDINATES (ft): N 51910.85 E 39336.46

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	DESCRIPTION OF MATERIALS
	10,6,4	0	SPT-1	-no recovery
6.0 / 108.2				CLAY (CI) Some silt, some gravel, medium plasticity, hard to stiff, gravel subangular, grey, no odour, moist, contorted laminae less than 0.1 inch thick, gravel to 0.75 inch diameter.
10	6,11,10	2.5	SPT-2	
	6,11,22	8	SPT-3	Trace to some silt, hard, grey, no odour, slightly moist, very thin laminae to less than 0.05 inch thick.
20	12,11,14	10	SPT-4	Trace to some silt, hard, grey, no odour, moist, -subrounded stone in bottom third of section -soft and damp clay below stone
	2,9,12	13	SPT-5	Trace silt, trace fine sand, firm to soft, grey, moist, thin laminae 0.05 to 0.1 inch thick, increasing sand in lower portion of sample.
22.0 / 92.2				SAND (SP) Some silt and clay, compact, grey, no odour, moist, thin laminae 0.05 to 0.1 inch thick; interlayered with: -silty CLAY (CL),medium plasticity, firm, grey, no odour, moist. -angular GRAVEL (GP), broken, green and black, up to 1 in diameter. -SAND (SW), some silt and clay, well graded, compact, some subangular sand to 0.1 inch diameter, grey, moist. - stone in sampler tip with disked stone in split spoon.
	38,38,26	6	SPT-6	
30	11,42,50	2	SPT-7	Some sand, medium grained, tawny, moist.
	9,26,20	8	SPT-8	Some gravel, trace to some silt, coarse grained, angular to subangular, wet.
	18,20,19	13	SPT-9	Trace to some silt, coarse grained, angular to subangular, wet.
35.0 / 79.2				SILT (ML) Some clay, trace to some sand and fine gravel, low plasticity, hard, moist, interlayered with:
40	8,13,11	0	SPT-10	-some gravel, possible slough
	10,28,11	4	SPT-11	-interbedded with GRAVEL (GP), trace to some sand, quartz and chlorite clasts, subangular to rounded. Some clay, some sand, medium to low plasticity, soft to firm, grey; no odour, damp.
42.5 / 71.7	17,12,14	1.5	SPT-12	SAND (SP) Some gravel, one rounded gravel about 1 inch diameter, sand subangular to angular, hard, wet. (slough?)
	3,10,17	6	SPT-13	Some silt to silty, some gravel, compact, gravel subrounded to rounded, grey, no odour, wet.
48.0 / 66.2				PHYLLITE (Bedrock) -drilling difficult at 48-49ft
50				

INSTRUMENT DETAILS



Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** DH05-06-PZ

TEST HOLE LOG

STARTED: 3/19/2005 **FINISHED:** 3/19/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 114.2
COORDINATES (ft): N 51910.85 E 39336.46

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					-phyllite cobble and gravel in top of barrel Run 1: 48 - 51 ft. Recovery 66%, RQD 60% Run 2: 51 - 56 ft Recovery 100% RQD 60%	
					56.0 58.2	
60					End of Hole at 56.0 ft	
70						
80						
90						
100						

Su - ksf: 1 2 3 4
VANE PEAK / REMOLD — FIELD ◆ ◇ — LAB ■ □ — ▲ UC/2, △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
20 40 60 80

End of Hole at 56.0 ft

Drill Notes:
1) Drill hole terminated at 56ft
2) Rock is graphitic phyllite.

Well Installation Notes:
1) Hole flushed with freshwater.
2) Installed 1 inch diameter standpipe peizometer (Schedule 80 PVC), 5 ft long No. 10 slotted screen.
3) Bottom of screen tip at 54 ft 4 inches. Stickup is 16 inches.
4) Clean 10/20 sand to 49 ft, medium bentonite chips to surface.

-water elevation in piezometer 104.95 ft (Apr-29-05)

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** DH05-06-PZ

TEST HOLE LOG

STARTED: 3/20/05 **FINISHED:** 3/20/05
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 144.4
COORDINATES (ft): N 52178.77 E 39753.98

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Depth / Elev.	DESCRIPTION OF MATERIALS
0					PEAT (PT) Some gravel, fibrous, wet.
	3,4,2	0	SPT-1		
				6.0 / 138.4	- harder drilling at 6 ft CLAY (CL) Some fine sand, some gravel, low plasticity, hard, dark grey, no odour, moist to dry, very thin laminae to less than 0.05 inch thick, gravel rounded to 1 inch diameter.
10	15,23,28	5	SPT-2		
				14.0 / 130.4	GRAVEL (GP) Sandy, some silt and clay, poorly graded, gravel to 1.2 inch diameter, subrounded to angular, some angular course sand, very dense, grey, moist, gravel supported in sand matrix. One disked granite pebble in tip, one subangular green gravel clast in sampler 1 inch diameter.
	22,38,51	9	SPT-3		
20	8,50	2	SPT-4		
				23.0 / 121.4	- hard layer at 23ft PHYLLITE (Bedrock) - Rock is chloritic phyllite Run 1: 23 - 27 ft. Recovery 89%, RQD 89% Run 2: 27 - 32 ft Recovery 100% RQD 100%
30				32.0 / 112.4	
40					
50					

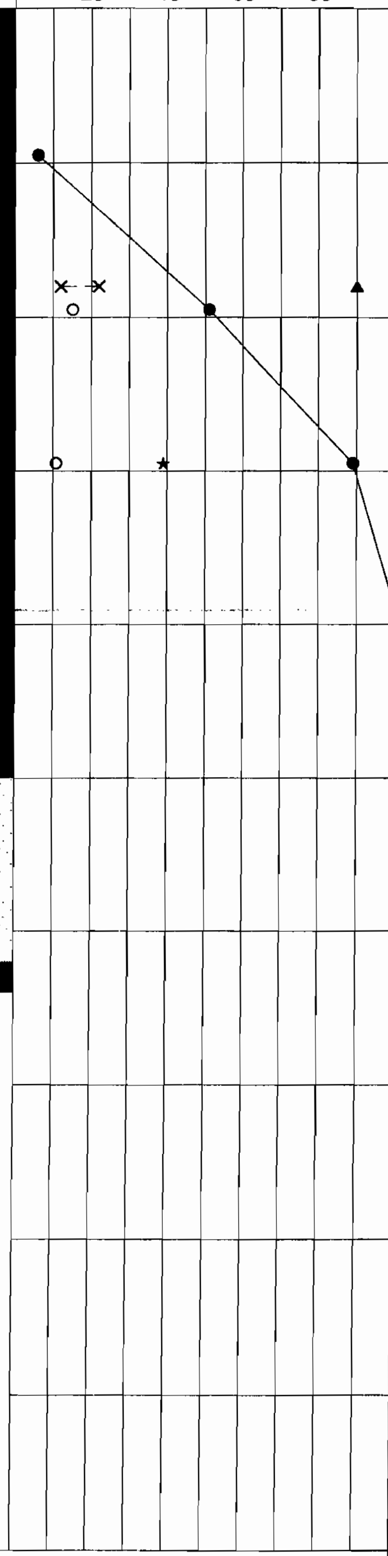
End of Hole at 32.0 ft

Drill Notes:
1) Drill hole terminated at 32 ft.
2) Rock is chloritic phyllite

Well Installation Notes:
1) Hole flushed with freshwater.
2) Installed 2 inch diameter standpipe piezometer (Schedule 40 PVC), 5 ft long slotted No.20 screen, and 30 ft of threaded PVC pipe.
3) Bottom of screen tip at 31 ft. Stickup is 20 inches.
4) Clean 10/20 sand to 25ft, medium bentonite chips to surface.

INSTRUMENT DETAILS

Su - ksf: 1 2 3 4
VANE PEAK: FIELD ◆ LAB ■ ▲ UC/2
REMOLD: ◇ □ △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
20 40 60 80



KLOHN CRIPPEN

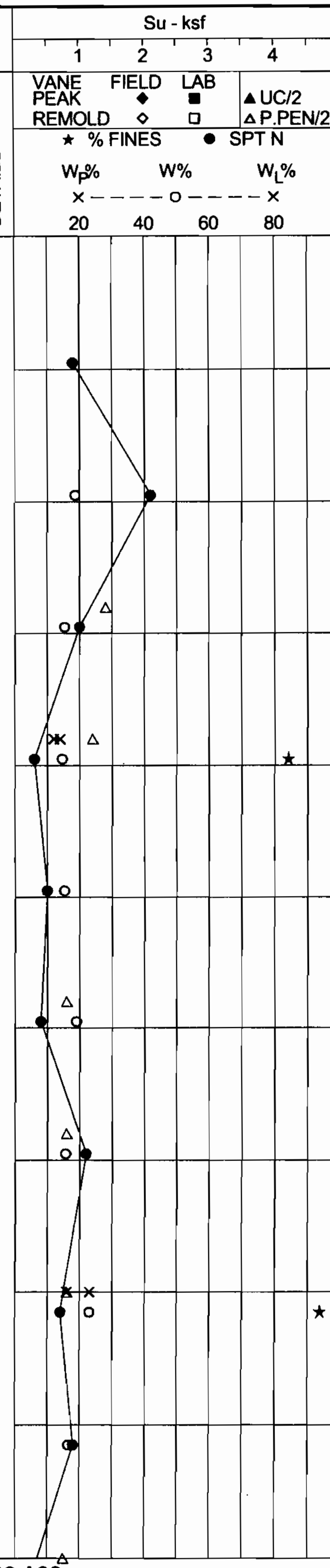
PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** DH05-07-PZ

TEST HOLE LOG

STARTED: 3/21/2005 **FINISHED:** 3/23/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 233.3
COORDINATES (ft): N 53252.76 E 39975.57

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	DESCRIPTION OF MATERIALS
				GRAVEL (GP) Up to 1 inch diameter, subangular to angular, moist.
	7,11,7	1.5	SPT-1	
6.5 / 226.8				SILT (ML) And gravel, low plasticity, soft, dark grey, angular gravel, moist, slightly dilatant. (Tailings)
10	20,24,18		SPT-2	
10.5 / 222.8				SILT (ML) Low plasticity, uniform, stiff, dark grey, moist, slightly dilatant. (Tailings)
	8,9,11	15	SPT-3	
20	6,4,2	10.5	SPT-4	- damp
	3,5,5	11	SPT-5	- soft (less than 1 TSF on pocket penetrometer), damp
30	5,4,4	10	SPT-6	- trace gravel, firm, gravel is angular with particles to 0.5 inches in diameter in top 1.5 inches of spoon.
	6,11,11	7.5	SPT-7	- moist, firm - some crushed argillite in top of spoon
40	6,7,7	12	SPT-8	- trace to some clay (ML-CL), firm, damp, not dilatant.
	5,9,9	12	SPT-9	- some clay, trace fine sand, stiff, moist, very slow dilatancy.
50				

INSTRUMENT DETAILS



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PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 1 **OF** 3 **HOLE NO.:** DH05-08

KLOHN CRIPPEN

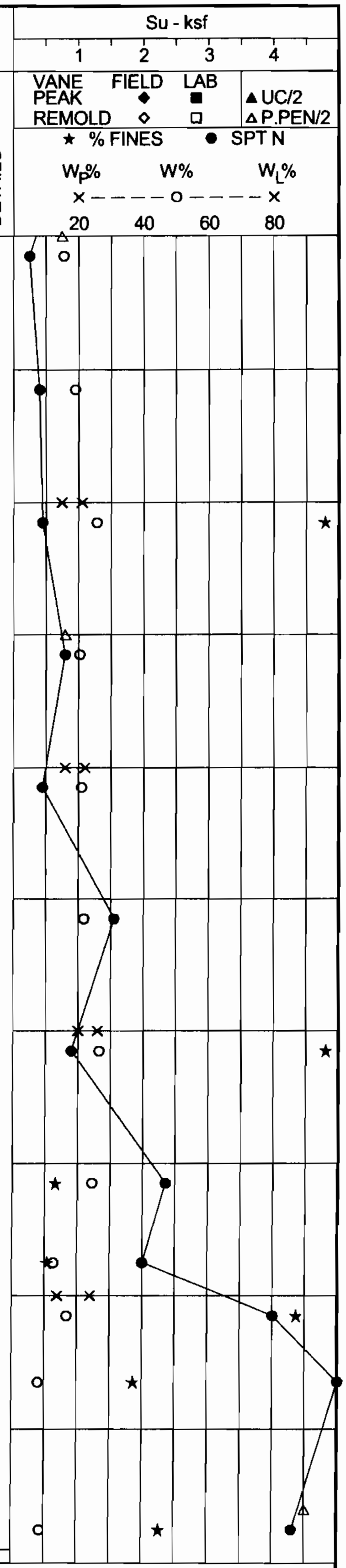
TEST_KC TEST HOLE IMP 2005DH-050601JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

STARTED: 3/21/2005 **FINISHED:** 3/23/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 233.3
COORDINATES (ft): N 53252.76 E 39975.57

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Elev.	DESCRIPTION OF MATERIALS
	3,2,3	14	SPT-10		- some clay, trace fine sand, less than 0.75 TSF on pocket penetrometer, damp, not dilatant.
	5,4,4	13	SPT-11		- some clay, trace fine sand, less than 0.75 TSF on pocket penetrometer, not dilatant.
60	7,5,4	13.5	SPT-12		- trace of clay, trace fine sand, less than 0.75 TSF on pocket penetrometer, moist, not dilatant.
	4,5,11	8	SPT-13		- trace of clay, trace fine sand, firm, damp, slightly dilatant, occasional angular argillite chip to 5/8 inches in diameter.
70	6,5,4		SPT-14		- some clay, trace fine sand, less than 0.75 TSF on pocket penetrometer, moist, slightly dilatant.
	4,12,19	12	SPT-15		- trace of clay, trace fine sand, less than 0.75 TSF on pocket penetrometer, moist, not dilatant.
80	7,9,9	13.5	SPT-16		- trace to some clay, trace fine sand, less than 0.75 TSF on pocket penetrometer, moist to damp, not dilatant.
84.0				149.3	PEAT (PT) Firm, orangy brown, amorphous to fibrous
	12,17,30	12	SPT-17		
86.0				147.3	SAND (SW) Gravelly, trace of silt, well graded, compact, angular, dark brown, rock 1.25 inches in diameter stuck in tip.
88.0	18,22,18	6	SPT-18	145.3	SAND (SP) Gravelly, trace of silt, poorly graded, medium grained, uniform, compact, greenish-grey, wet, gravel angular, broken from penetration. Broken gravel clast in tip of spoon.
90	18,30,50	11	SPT-19		
90.0				143.3	
92.0	50	3	SPT-20	141.3	SILT (ML) Some fine sand, trace of clay, low plasticity, very hard, dark grey, slightly moist, contorted sand lenses to 0.1 inch thick, very thin silt laminae (<<0.05 inch), occasional gravel clasts.
95.5				137.8	SAND (SP) Some silt and clay, trace of fine gravel, poorly graded, very hard, greenish-grey, dry to slightly moist.
	18,36,50	7.5	SPT-21		SILT (ML) Some gravel, trace of sand and clay, low plasticity, very hard, greenish-grey, slightly moist, gravel angular to
99.5 / 100					

INSTRUMENT DETAILS



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End

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 2 **OF** 3 **HOLE NO.:** DH05-08

TEST_KC TEST HOLE IMP 2005DH-050801JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

STARTED: 3/21/2005	**FINISHED:** 3/23/2005
DRILL METHOD: Mud Rotary	
GROUND ELEV. (ft): 233.3	
COORDINATES (ft): N 53252.76	E 39975.57

Su - ksf: 1 2 3 4

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

★ % FINES ● SPT N

$W_P\%$ W% $W_L\%$

× ---- o ---- ×

20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					133.8 subangular up to 0.75 inches in diameter.	
					End of Hole at 99.5 ft	
110						
120						
130						
140						
150						

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38	
PROJECT: 2005 Geotechnical Investigation	
LOCATION: Tailings Facility Expansion	
LOGGED BY: KE	**CHECKED BY:**
SHEET 3 **OF** 3	**HOLE NO.:** DH05-08

TEST_KC TEST HOLE IMP 2005DH-050601JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

STARTED: 5/1/2005 **FINISHED:** 5/10/2005

DRILL METHOD: Mud Rotary

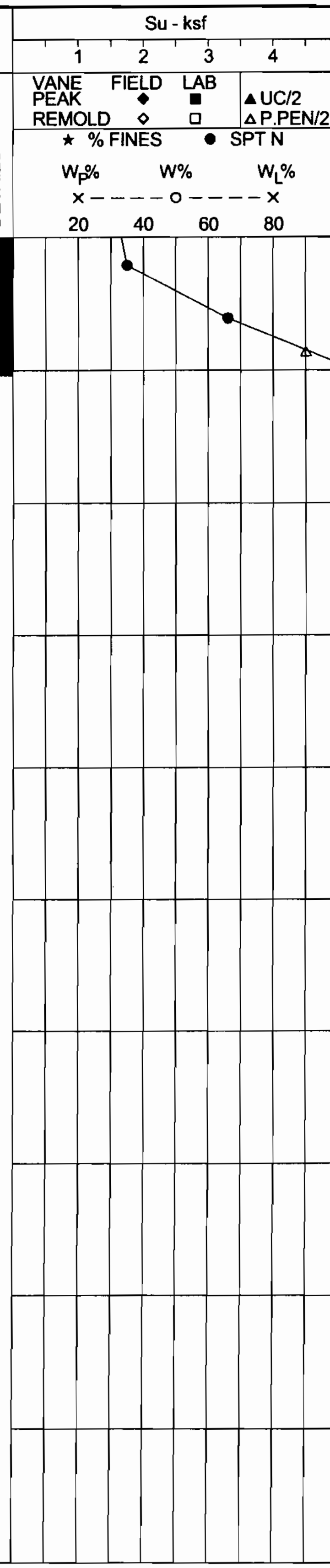
GROUND ELEV. (ft): 232.9

COORDINATES (ft): N 53077.19 E 39934.15

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	14,24,11	4.5	SPT-17		sub-angular gravel, strong odour of decaying vegetal organics.	
	9,25,41	0	SPT-18		SAND (SP) Trace to some silt, trace organics, poorly-graded, compact to dense, medium to fine grained, angular to sub-rounded sand, brown to grey, moist, no bedding, uniform, some roots. -broken granitic gravel clast in sampler tip. (SPT 17) -rough drilling on 100.5 to 101.5 ft. -some angular rock fragments in sand catcher. (SPT 18)	
	12,50	5	SPT-19	103.5 / 129.4	CLAY (CI) Silty to some silt, trace sand, intermediate plasticity, hard, grey with faint green mottling, moist to dry, fine laminae <0.05 inches thick. (Marine)	
				105.2 / 127.7	End of Hole at 105.2 ft	
110						
120						
130						
140						
150						

Well Installation Notes:
1) Hole flushed with fresh water to about 65 ft depth, pulled surface casing and hole sloughed to -50 ft
2) Placed 2x50lb Oglebay/Norton 10/20 sand to -47.75 ft in drill hole
3) Inserted vibrating wire piezometer tip and added 2x50lb Oglebay/Norton 10/20 sand to approx. -46.5 ft.
4) Placed 8x50 lb bags of Cetco "Puregold" medium bentonite chips to surface.
5) Inserted 1 inch diameter schedule 40 pvc pipe to -10 ft and cut off with 5 ft of stick up to hang piezometer cable on (above the ground).

Piezometer Notes:
1) SINCO Model 52611020, no. 80219 (50 psi c/w 500 ft cable) at depth of 47.75 ft.
-Reading about 0.5 hours after installation: 3007.5 Hz; 7.9 C (calculated water elevation is 197.35 ft - April 29/05)



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2005 Geotechnical Investigation

LOCATION: Tailings Facility Expansion

LOGGED BY: KAE **CHECKED BY:**

SHEET 3 **OF** 3 **HOLE NO.:** DH05-09-PZ

TEST_KC TEST HOLE IMP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

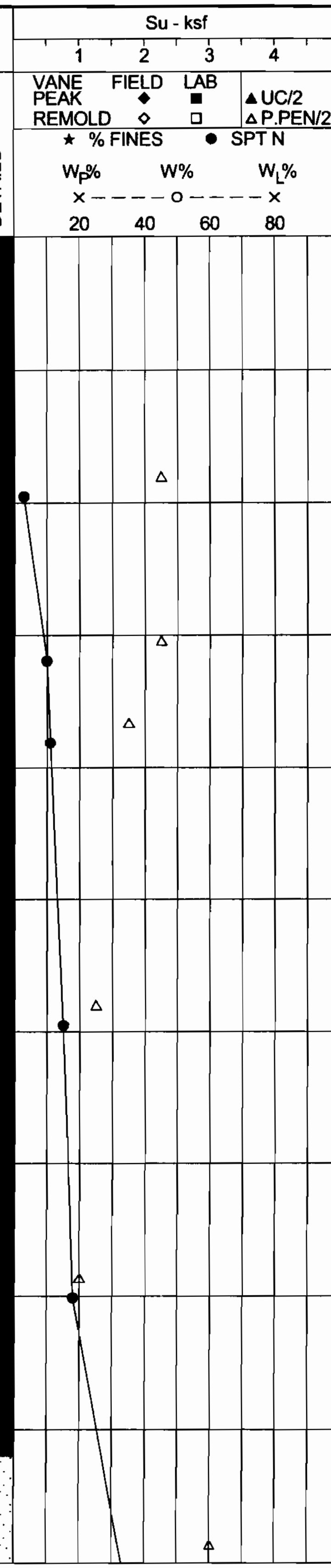
STARTED: 5/1/2005 **FINISHED:** 5/10/2005

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 232.9

COORDINATES (ft): N 53077.19 E 39934.15

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
10	2,2,1	12	SPT-1		SILT (ML) Some fine sand, trace to some clay, low plasticity, very stiff, dark grey, chemical odour, moist, no bedding, uniform, not dilatant, pearly lustre from fine graphite. (Tailings)	
	2,3,7	8.5	SPT-2		Some fine sand, trace clay, low plasticity, very stiff, dark grey, moist, no bedding, uniform, very slow dilatancy, higher sand content than previous SPT. (Tailings)	
20	7,8,3	3.5	SPT-3		Some fine sand, trace clay, low plasticity, stiff, dark grey, chemical odour, moist, no bedding, uniform, very slow dilatancy. (Tailings)	
30	3,7,8	9	SPT-4		Some fine sand, trace clay, low plasticity, stiff, dark grey, chemical odour, moist, no bedding, uniform, not dilatant. (Tailings) GRAVEL (GP) in sampler tip; poorly graded, dark grey, angular. (Argillite Road Fill) -rough drilling on 32ft to 34 ft	
40	3,4,14	12	SPT-5		Some fine sand to sandy, trace clay, low plasticity, firm, dark grey, chemical odour, moist to wet, no bedding, uniform, not dilatant. (Tailings)	
					-rough drilling at 46 ft	
50					Some to trace fine sand, trace clay, low plasticity, very stiff	



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KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2005 Geotechnical Investigation

LOCATION: Tailings Facility Expansion

LOGGED BY: KAE **CHECKED BY:**

SHEET 1 **OF** 3 **HOLE NO.:** DH05-09-PZ

TEST HOLE LOG

Su - ksf: 1, 2, 3, 4

STARTED: 5/1/2005 **FINISHED:** 5/10/2005

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 232.9

COORDINATES (ft): N 53077.19 E 39934.15

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL		DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	13,15,18	9.5	SPT-6			dark grey, chemical odour, dry to moist, no bedding, uniform, not dilatant. (Tailings)	
60	4,5,7	9.5	SPT-7			Some fine sand, trace clay, low plasticity, firm, dark grey, chemical odour, moist, no bedding, uniform, not dilatant. (Tailings)	
70	7,9,8	12	SPT-8			Some fine sand, trace clay, trace gravel, low to low-intermediate plasticity, stiff, dark grey, no odour, moist, no bedding, uniform, very slow dilatancy, gravel angular up to 1 inch diameter. (Tailings)	
80	3,3,3	14	SPT-9			Some fine sand, trace clay, low plasticity, firm, dark grey, moist, no bedding, uniform, very slow dilatancy. (Tailings)	
	50	1	SPT-10			-gravelly at 83.7 ft Broken chloritic phyllite in tip of sampler; slightly weathered; some drill mud and loose tailings in sampler.(Road Fill?) -drilled through rock to 85.8 ft	
	3,8,4	11	SPT-11			Some fine sand to sandy, trace clay, low plasticity, firm, dark grey, moist, very slow dilatancy, fine pyrite laminae 0.1 inches thick. (Tailings)	
90	5,7,7	9.5	SPT-12			Some fine sand to sandy, trace to some clay, low plasticity, firm, dark grey, no odour, moist, no bedding, uniform. (Tailings)	
	3,9,10	15.5	SPT-13		92.9 140.0	Some fine sand, some clay, low to low-intermediate plasticity, firm, dark grey, moist to damp, no bedding, uniform. (Tailings)	
	5,7,16	17	SPT-14			PEAT (Pt)	
					95.8 137.1	Some sand, trace to some silt, amorphous, some fibres, firm, moist, black to reddish brown, earthy odour.	
	12,14,19	9	SPT-15			SAND (SW)	
	13,15,16	9.5	SPT-16		97.8 135.1	Some gravel, trace to some silt, trace to some organics, well-graded, compact to dense, up to 3/8 inch diameter, angular to sub-rounded sand, sub-rounded to	
100							

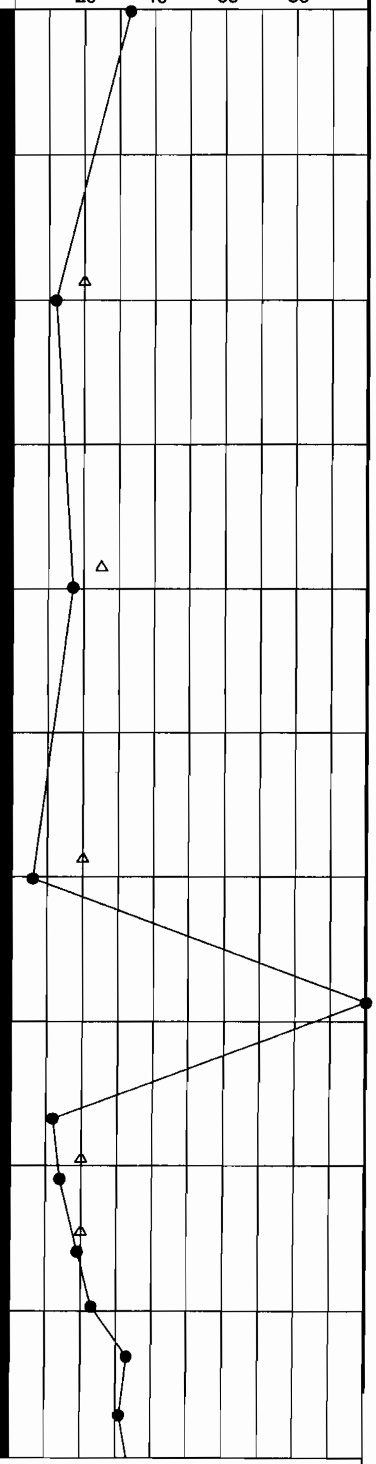
Continued Next Page

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

★ % FINES ● SPT N

W_P% W% W_L%

20 40 60 80



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2005 Geotechnical Investigation

LOCATION: Tailings Facility Expansion

LOGGED BY: KAE **CHECKED BY:**

SHEET 2 **OF** 3 **HOLE NO.:** DH05-09-PZ

TEST HOLE LOG

STARTED: 5/12/2005 **FINISHED:** 5/13/2005

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 214.0

COORDINATES (ft): N 53076.4 E 39805.4

Su - ksf: 1 2 3 4

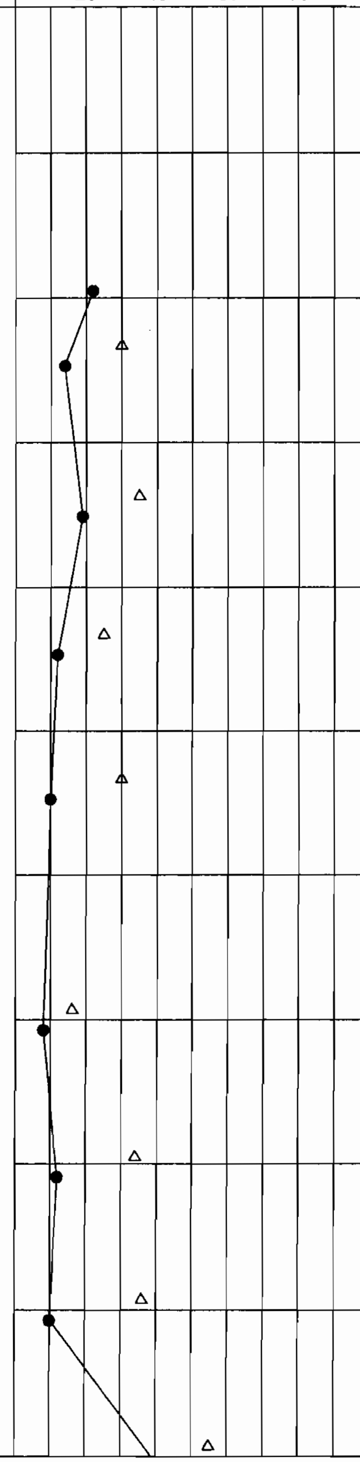
VANE / FIELD / LAB: PEAK ◆ ■; REMOLD ◇ □; ▲ UC/2; △ P.PEN/2

★ % 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
10	8,11,11	12	SPT1		SILT (ML) Trace to some fine sand, trace to some clay, low plasticity, dark grey, no odour, moist to dry, no bedding, uniform, not dilatant. (Tailings)	
	4,7,7	11.5	SPT2		Some fine sand, trace clay, low plasticity, stiff, dark grey, no odour, moist, no bedding, uniform, pearly lustre from fine graphite. (Tailings)	
	7,8,11		SPT3		Some fine sand to sandy, trace clay, low plasticity, stiff, dark grey, no odour, moist to dry, no bedding, uniform, very slow dilatancy. (Tailings)	
20	4,5,7	11.5	SPT4		Some fine sand, trace clay, low plasticity, stiff, dark grey, no odour, moist to dry, no bedding, uniform, not dilatant. (Tailings)	
	5,5,5	11	SPT5		Some fine sand, trace clay, low plasticity, stiff, dark grey, no odour, moist to dry, no bedding, uniform, not dilatant. (Tailings)	
30	2,4,4	14	SPT6		Some fine sand, trace to some clay, low plasticity, firm, dark grey, no odour, moist, no bedding, uniform, slow dilatancy. (Tailings)	
40	5,7,5	10	SPT7		Some fine sand, trace to some clay, low to low-intermediate plasticity, stiff, dark grey, no odour, moist, no bedding, uniform, very slow dilatancy. (Tailings) GRAVEL (GP) approx. 3.5 inches of angular gravel in middle of sample (Road Fill).	
	6,4,6	14	SPT8		Trace to some fine sand, trace to some clay, low to low-intermediate plasticity, stiff, dark grey, no odour, moist, no bedding, uniform, not dilatant. (Tailings)	
50						



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KLOHN CRIPPEN

PROJECT NO.: PM7802 A38	
PROJECT: 2005 Geotechnical Investigation	
LOCATION: Tailings Facility Expansion	
LOGGED BY: KAE	**CHECKED BY:**
SHEET 1 **OF** 2	**HOLE NO.:** DH05-10

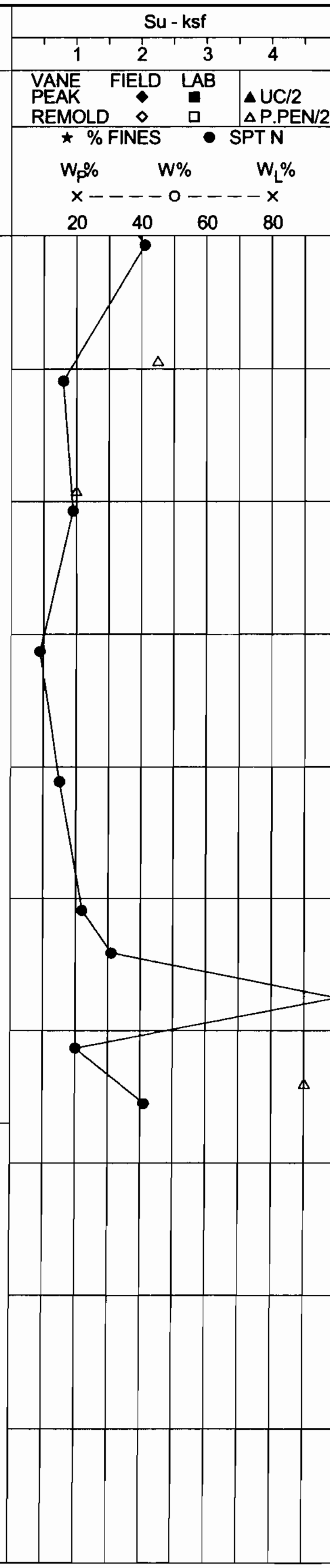
TEST_KC TEST HOLE IMP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

STARTED: 5/12/2005 **FINISHED:** 5/13/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 214.0
COORDINATES (ft): N 53076.4 E 39805.4

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Depth / Elev.	DESCRIPTION OF MATERIALS
	20,19,22	8	SPT9		Trace to some fine sand, some clay, low to low-intermediate plasticity, very stiff, dark grey, no odour, moist, visually above average graphite content. (Tailings) Upper approx. 2 inches of SPT10 is sand and angular gravel road fill; recovered small scrap of woven geotextile (Road Fill)
	4,8,8		SPT10		Trace to some fine sand, some clay, low to low-intermediate plasticity, very stiff, dark grey, no odour, moist to dry, no bedding, uniform, not dilatant. (Tailings) -rough drilling at about 48 to 49 ft, lost circulation, circulated thick mud and continued drilling.
				59.6 / 154.4	CLAY (CI)
60	11,8,11	9	SPT11		Silty, trace fine to medium sand, intermediate plasticity, firm, dark grey, no odour, moist, not dilatant. (Tailings) -upper 4 inches and lowest inch are angular gravel clasts (argillite or graphitic phyllite).
	3,4,5	13	SPT12		Silty, trace fine sand, trace fine gravel, intermediate plasticity, soft, dark grey, no odour, moist, not dilatant, no bedding, uniform, pocket penetrometer < 1 TSF. (Tailings)
70	2,4,11		SPT13		Silty, trace fine sand?, intermediate plasticity, soft, dark grey, no odour, moist, not dilatant, no bedding, uniform, pocket penetrometer < 1 TSF. (Tailings) -disked phyllite fragment in tip of sampler. -rough drilling on 72 ft to 73 ft.
				73.0 / 141.0	PEAT (Pt) Amorphous, some fibres, some wood, firm, reddish-brown, earthy odour.
	7,10,12	18	SPT14	75.5 / 138.5	SAND (SW)
	18,18,13	9	SPT15		Some silt, trace to some gravel, well-graded, compact, brown, moist, no odour, sand rounded to sub-angular, gravel sub-rounded to sub-angular, infrequent thin angular rock chips.
	31,50	1.5	SPT16		
				79.9 / 134.1	Gravelly, trace silt, well-graded, compact to dense, brown, moist, sand rounded to angular, gravel sub-rounded to sub-angular up to 1 inch diameter.
80	10,13,7	14	SPT17	82.0 / 132.0	Gravelly to some gravel, trace silt, well-graded, compact, sand rounded to sub-angular, gravel sub-rounded to sub-angular up to 3/8 inch diameter. -very rough drilling on 78 to 79 ft.
	4,16,25	14	SPT18	83.5 / 130.5	SAND (SP) Trace to some silt, some clay, poorly-graded, dense, medium to fine grained, sand rounded to sub-rounded, grey, wet, no bedding, uniform, many shells in middle of interval. (Marine Sand)
					CLAY (CI or CH) Silty, trace fine sand, intermediate to high plasticity, hard, grey, no odour, moist to dry, fine laminae <0.05 inches thick, infrequent shells. (Marine Clay)
90					
100					

INSTRUMENT DETAILS



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KAE **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** DH05-10

TEST HOLE LOG

STARTED: 5/14/2005 **FINISHED:** 5/15/2005

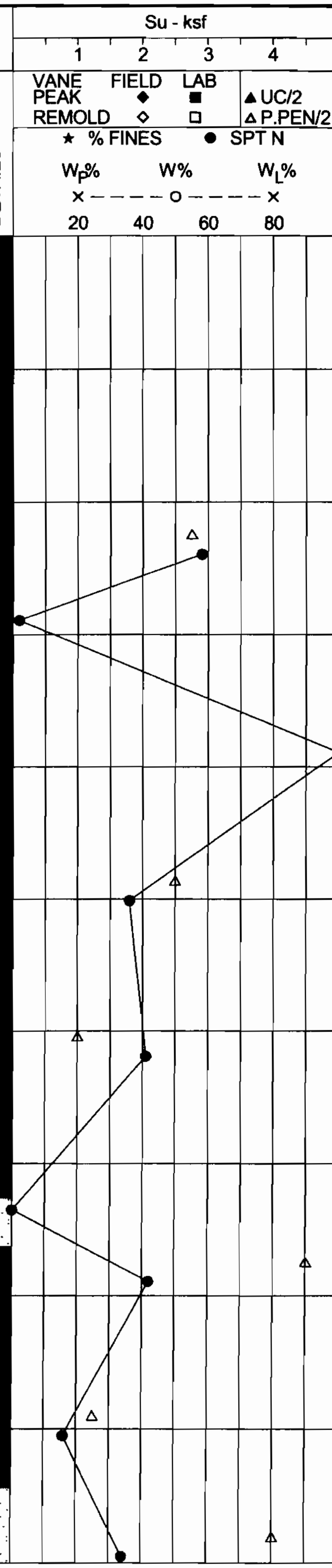
DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 213.4

COORDINATES (ft): N 52719.08 E 40012.53

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS
10					SILT (ML) Trace fine sand, trace clay, low plasticity, very stiff, dark grey, no odour, moist, no bedding, uniform, not dilatant. (Tailings) -4 inches gravel (GP) in lower portion of sampler, angular.
	7,23,35	12	SPT1		
	2,1,1		SPT2		Trace to some fine sand, trace clay, low plasticity, very soft, dark grey, no odour, wet, no bedding, uniform, slow dilatancy, pocket penetrometer less than 1 TSF. (Tailings)
20	50	3	SPT3		GRAVEL (GP), some sand, trace silt, very dense, angular , black, wet, tailings in pores (Road rock/ argillite)
23.1 / 190.3	20,18,18	8.5	SPT4		CLAY (CL) Silty, trace fine sand, intermediate plasticity, very stiff, dark grey, no odour, moist to dry, no bedding, not dilatant. -upper portion of sample is angular gravel (GP) to 1 inch diameter, some quartz and phyllite clasts.
28.0 / 185.4					SILT (ML) Trace to some clay, trace fine sand, low plasticity, firm, dark grey, no odour, moist to dry, no bedding, uniform, slow dilatancy, one angular gravel clast 1/2 inch diameter. (Tailings) -disked phyllite clast in tip of sampler.
30	12,28,13	8.5	SPT5		
					- drill hit void at 34 ft, lost circulation, drilled to 35 ft, put on sampler but spoon dropped under self-weight to 38.7 ft.
	0	0	SPT6		
40	6,12,30		SPT7		Some clay, trace fine sand, low to low-intermediate plasticity, stiff to hard, dark grey, no odour, dry, no bedding, uniform, very slow dilatancy. (Tailings)
	2,5,11	15	SPT8		Some clay, trace fine sand, low plasticity, stiff, dark grey, no odour, moist, no bedding, uniform, very slow dilatancy. (Tailings)
50	14,17,17				Some clay, trace fine sand, low plasticity, stiff to very stiff

INSTRUMENT DETAILS



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KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2005 Geotechnical Investigation

LOCATION: Tailings Facility Expansion

LOGGED BY: KAE **CHECKED BY:**

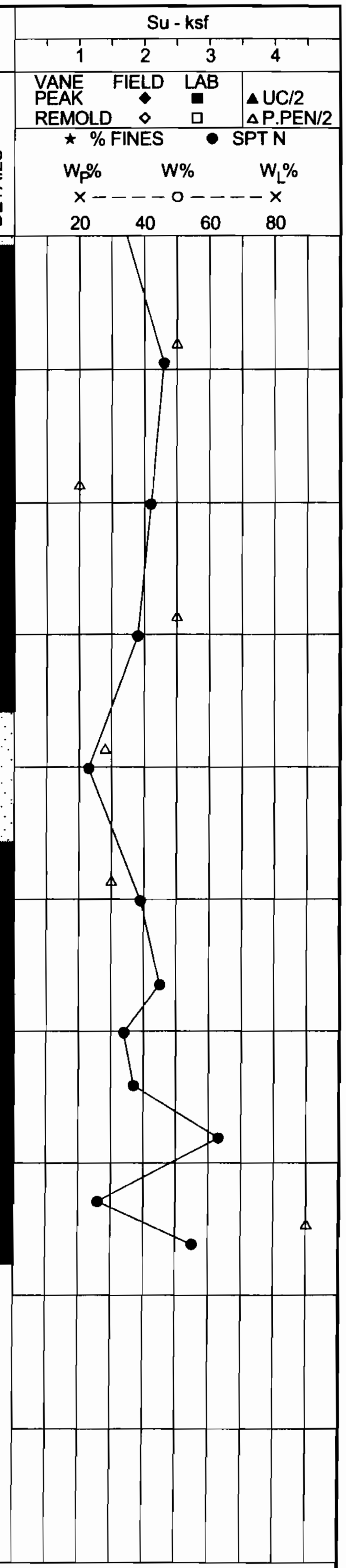
SHEET 1 **OF** 3 **HOLE NO.:** DH05-11-PZ

TEST HOLE LOG

STARTED: 5/14/2005 **FINISHED:** 5/15/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 213.4
COORDINATES (ft): N 52719.08 E 40012.53

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Depth / Elev.	DESCRIPTION OF MATERIALS
		12	SPT9		dark grey, no odour, dry, no bedding, uniform, not dilatant. (Tailings)
				52.0 / 161.4	CLAY (CI) Silty, trace fine sand, intermediate plasticity, firm to very stiff, dark grey, no odour, dry to moist, no bedding, uniform, not dilatant.
	22,24,22	10	SPT10		
60	5,17,25	12	SPT11		-one angular gravel fragment 3/8 inches in diameter in middle of SPT11. (Tailings)
				64.3 / 149.1	SILT (ML) Some clay, trace to some fine sand, low to low-intermediate plasticity, very stiff, dark grey, no odour, dry to moist, no bedding, uniform, very slow dilatancy. (Tailings)
	12,18,20	15.5	SPT12		
70	11,12,11	12.5	SPT13		Some clay, trace fine sand, low to low-intermediate plasticity, stiff, dark grey, no odour, moist, no bedding, uniform, very slow dilatancy. (Tailings)
	8,15,25	15	SPT14		Some clay, trace fine sand, low plasticity, stiff, dark grey, no odour, moist to dry, no bedding, uniform, not dilatant, one angular gravel clast 1/4 inches in diameter in sample. (Tailings)
				75.8 / 137.6	SAND (SW) Some gravel, trace silt, very dense, sand rounded to sub-angular, gravel sub-rounded to sub-angular, grey to tawny, no odour. -lower 3.5 inches Gravel (GP), some sand, trace silt, compact, gravel subangular to angular, no odour, black to dark grey, wet (Road Fill ?)
	40,24,21	10	SPT15		
				79.0 / 134.4	SAND (SP) Some gravel, trace silt, poorly graded coarse, dense, sand angular to sub-rounded, wet, no odour, grey to black with some quarts clasts, gravel sub-angular to sub-rounded to 0.75 inches diameter. -occasional clasts of weathered green rock, rusty stained, friable, dry.
80	15,17,17	7	SPT16		
	9,17,20	10.5	SPT17		
				82.6 / 130.8	PEAT (PT) Trace to some fine gravel, some wood, amorphous, some fibrous, firm, moist, reddish-brown, thin silty sand laminae to 0.1 inches thick, gravel angular to 0.75 inches in diameter. 2 pieces non-woven geotextile.
	19,26,37	14	SPT18	84.2 / 129.2	SAND (SP) Some organics, trace fine gravel, trace silt, poorly graded, very dense, medium to fine grained, sand rounded to subrounded, brown, wet, no odour.
				128.6	SAND (SM) Some silt, trace clay, poorly graded, medium to fine grained, grey, wet, one angular clast to 1 inch diameter, shells in lower portion of sample.
	9,10,16	16.5	SPT19		-some to trace silt, trace fine gravel, well graded, very dense, medium to fine grained, sand angular to rounded, abundant shells, grey, wet, no odour. (Marine Sand)
	18,19,36	12.5	SPT20	88.0 / 125.4 / 124.6	End of Hole at 88.8 ft
90					
100					

Continued Next Page



PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KAE **CHECKED BY:**
SHEET 2 **OF** 3 **HOLE NO.:** DH05-11-PZ

KLOHN CRIPPEN

TEST HOLE LOG

STARTED: 5/14/2005	**FINISHED:** 5/15/2005
DRILL METHOD: Mud Rotary	
GROUND ELEV. (ft): 213.4	
COORDINATES (ft): N 52719.08	E 40012.53

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

Su - ksf: 1 2 3 4

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

★ % FINES ● SPT N

W_P% W% W_L%

× — — — — o — — — — ×

20 40 60 80

Depth: 110, 120, 130, 140, 150

DESCRIPTION OF MATERIALS

CLAY (CI)
Silty, trace fine sand, intermediate plasticity, hard, grey, no odour, dry, fine laminae less than 0.05 inches thick, not dilatant.

Well Installation Notes:
1) Hole flushed with fresh water to about
2) Inserted Cetco "Puregold" chips to -72.8 ft.
3) Placed 25lb Oglebay/Norton 10/20 sand to -71.3 ft in drill hole
4) Inserted vibrating wire piezometer tip and added 2x50lb Oglebay/Norton 10/20 sand to approximately -67.9 ft.
5) Placed Cetco "Puregold" bentonite chips to -50.3 ft.
6) Placed Oglebay/Norton 10/20 sand to -49.1 ft in drill hole
7) Inserted vibrating wire piezometer tip and added 2x50lb Oglebay/Norton 10/20 sand to approximately -47.2 ft.
8) Placed Cetco "Puregold" bentonite chips to -38.1 ft.
9) Placed Oglebay/Norton 10/20 sand to -37.2 ft in drill hole
10) Inserted vibrating wire piezometer tip and added 2x50lb Oglebay/Norton 10/20 sand to approximately -36.3 ft.
11) Placed Cetco "Puregold" bentonite chips to -15.7 ft.

Piezometer Notes:
1) SINCO Model 52611020, no. 80218 at depth of 71.3 ft.
-water elevation = 148.84 ft (16-May-2005)
2) SINCO Model 52611020, no. 80212 at depth of 49.1 ft.
-water elevation = 172.07 ft (16-May-2005)
3) SINCO Model 52611020, no. 80209 at depth of 37.2 ft.
-water elevation = 182.80 ft (16-May-2005)

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38	
PROJECT: 2005 Geotechnical Investigation	
LOCATION: Tailings Facility Expansion	
LOGGED BY: KAE	**CHECKED BY:**
SHEET 3 **OF** 3	**HOLE NO.:** DH05-11-PZ

TEST HOLE LOG

STARTED: 5/16/2005 **FINISHED:** 5/16/2005

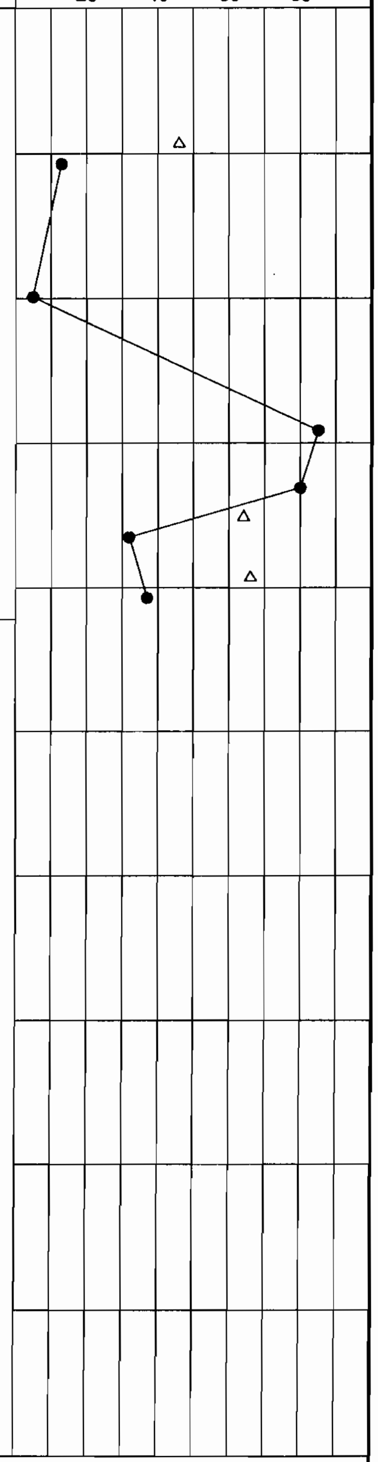
DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 158.0

COORDINATES (ft): N 53106.5 E 39629.32

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	DEPTH / ELEV.	DESCRIPTION OF MATERIALS
					SILT (ML) Some clay, trace fine sand, low to intermediate plasticity, very stiff, dark grey, no odour, dry, very slow dilatancy, no bedding, uniform. (Tailings)
	6,6,7	16	SPT1		SILT (ML) Some clay, trace fine sand, low plasticity, less than 1 TSF on pocket penetrometer, dark grey, no odour, dry, very slow dilatancy, no bedding, uniform. (Tailings)
10	4,3,2	9.5	SPT2	10.7 / 147.3	CLAYEY GRAVEL (GC) Road fill, angular argillite to 1 inch in diameter, clay high plasticity, gravel black, clay grey, wet, very soft.
				14.1 / 143.9	CLAYEY GRAVEL (GC) Some sand, trace to some silt, gravel angular to 3/8 inch diameter, dark grey, wet, clay, high plasticity, very soft, grey, wet. (Road Fill)
	19,35,50	13.5	SPT3	15.7 / 142.3	SAND (SW) Well graded, very dense, rounded to sub-angular to 0.1 inch diameter, grey to brown, moist, single fine sand nodule 0.5 inch diameter, single band of black sand 0.5 inches thick, vaguely sweet odour. (glycol?)
	28,43,37	8.5	SPT4	17.4 / 140.6	
	6,12,20	10	SPT5	140.0	GRAVEL (GS) Some sand to sandy, trace silt, gap-graded, dense, subangular to angular to 1 inch diameter, grey and orange bands, sand angular to rounded, coarse to fine sand, some weathered granite clasts. (Fill?)
20	8,14,23	10	SPT6	21.1 / 136.9	CLAYEY GRAVEL (GC) Some to trace silt, some sand, gravel angular to subangular, dark grey, wet. (slough)
					CLAY (CH) Silty, high plasticity, very stiff, grey, no odour, moist to dry, not dilatant, fine laminae less than 0.05 inches thick.
					End of Hole at 21.1 ft
30					
40					
50					

INSTRUMENT DETAILS



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2005 Geotechnical Investigation

LOCATION: Tailings Facility Expansion

LOGGED BY: KAE **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** DH05-12

TEST_KC TEST HOLE mP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

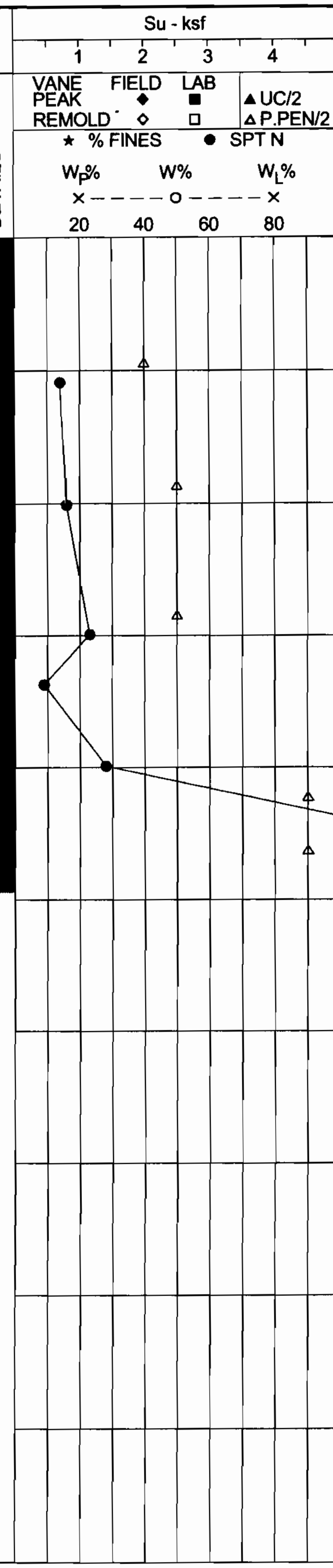
TEST HOLE LOG

STARTED: 5/17/2005 **FINISHED:** 5/17/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 158.2
COORDINATES (ft): N 52850.83 E 39761.15

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	7,8,6	7	SPT1		SILT (ML) Some clay, trace fine sand, stiff, dark grey, no odour, moist to dry, very slow dilatancy, no bedding, uniform. (Tailings)	
7.8 / 150.4					CLAY (CI) Silty, trace fine sand, intermediate plasticity, very stiff, dark grey, no odour, moist to dry, not dilatant, no bedding, uniform. (Tailings) -Rock at 8.5 ft.	
10	13,8,8	14	SPT2			
12.5 / 145.7					SILT (ML) Some clay, trace fine sand, low plasticity, very stiff, dark grey, no odour, dry, very slow dilatancy, no bedding, uniform. (Tailings)	
	14,14,9	14.5	SPT3			
15.9 / 142.3	7,2,7	6	SPT4		GRAVEL (GP) Trace to some sand, trace silt, poorly graded, compact to loose, gravel angular to approximately 1 inch diameter, black to dark grey, wet. (Road rock/ argillite)	
18.4 / 139.8					SAND (SW) Trace gravel, trace silt, compact, well-graded sand rounded to sub-rounded, many quartz grains, single piece non-woven geotextile, orange to beige, wet, some rusty stains.	
20	9,11,17	7.5	SPT5			
20.0 / 138.2; 20.9 / 137.3	33,50	7	SPT6		GRAVEL (GW) Some sand, well-graded, dense, gravel angular to sub-angular, sand angular to sub-rounded, grey, no odour, wet.	
	50	4	SPT7			
24.7 / 133.5					CLAY (CL) Silty, sandy, low plasticity, very stiff, grey with green mottling, no odour, dry, some angular gravel fragments. -sandy, some silt, low plasticity, very stiff, grey with faint green mottling, no odour, dry, not dilatant, contorted laminae less than 0.05 inches thick, sand coarse to fine, sand to 0.2 inches average diameter, sub-rounded to sub-angular.	
					End of Hole at 24.7 ft	
30						
40						
50						

Well Installation Notes:

1) Placed 2.5 bags of Cetco "Puregold" medium bentonite chips to surface. No instrument inserted.



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KAE **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** DH05-13

TEST_KC TEST HOLE imP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

STARTED: 5/19/2005 **FINISHED:** 5/19/2005

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 153.0

COORDINATES (ft): N 52443.03 E 39783.79

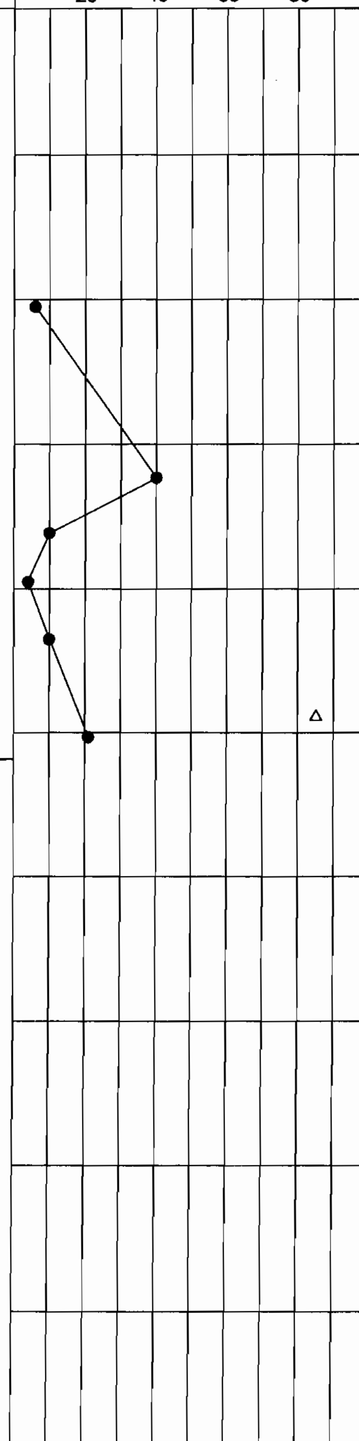
Su - ksf: 1 2 3 4

VANE PEAK / REMOLD — FIELD ◆ / ◇ — LAB ■ / □ — ▲ UC/2 — △ 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
0				Gravel	GRAVEL (GP) Loose, wet (Fill) -lower 3 inches is CLAY (CI), silty, gravelly, some sand to sandy, intermediate plasticity, soft, no odour, moist, no structure, gravel sub-angular to sub-rounded to 3/8 inches in diameter. (Fill)	
10	2,3,3	5	SPT1	Gravel		
	16,18,22	9.5	SPT2	Gravel / Peat	GRAVEL (GP) Some sand to sandy, trace silt, dense, poorly graded, angular to sub-angular, grey to black, wet, sand medium to fine grained (Fill) -softer at 17ft	
16.0 / 137.0					PEAT (PT) Fibrous, some to trace silt, some rotting wood, firm, reddish-brown to brick, earthy odour, moist.	
	1,3,7	14	SPT3	Peat		
20	1,2,2	12	SPT4	Peat		
	1,9,1	8	SPT5	Peat		
24.0 / 129.0	3,9,12	16	SPT6	Clay	CLAY (CH) Silty, high plasticity, hard, grey, no odour, moist, bedding laminae to 0.05mm thick, not dilatant.	
25.9 / 127.1					End of Hole at 25.9 ft	
30						
40						
50						



TEST_KC TEST HOLE IMP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2005 Geotechnical Investigation

LOCATION: Tailings Facility Expansion

LOGGED BY: **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** DH05-14

TEST HOLE LOG

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	STARTED: 5/18/2005 FINISHED: 5/18/2005 DRILL METHOD: Mud Rotary GROUND ELEV. (ft): 200.4 COORDINATES (ft): N 52857.09 E 40367.8 DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					SILT (ML) Some clay, trace to some sand, low plasticity, stiff, dark grey, no odour, dry, slow dilatancy, no bedding, uniform. (Tailings)	
5	3,5,4	6.5	SPT1			
10	4,5,8	12.5	SPT2		-some clay, trace to some sand, low plasticity, stiff, dark grey, no odour, dry, slow dilatancy, no bedding, uniform. (Tailings)	
15	6,9,5	14	SPT3		-some clay, some sand, low plasticity, stiff, grey, no odour, dry, no dilatancy, no bedding, uniform. (Tailings) 16.0 184.4	

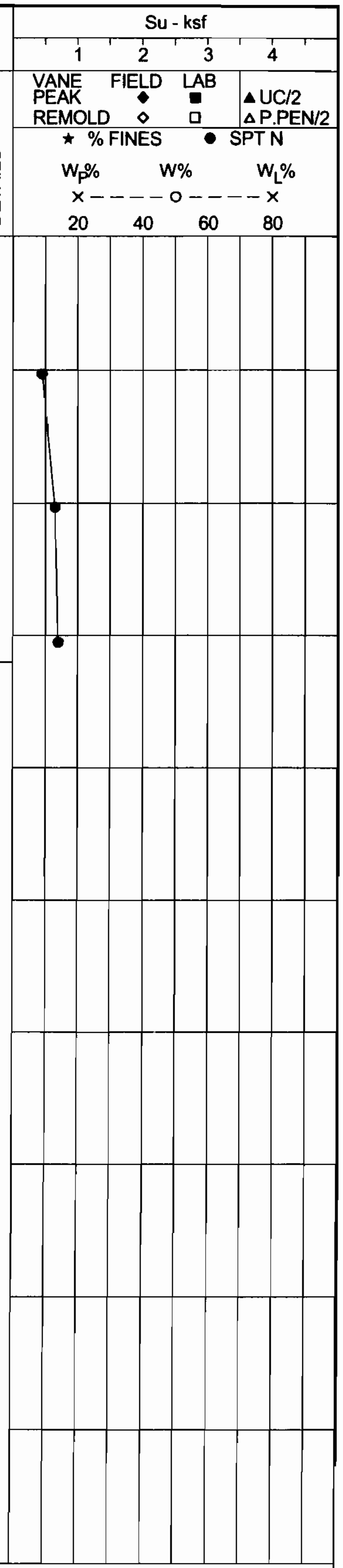
End of Hole at 16.0 ft

Piezometer Installation Notes:

1) Placed 10/20 silica sand to -15.3 ft
2) Inserted SINCO vibrating wire piezometer tip No. 80210 at -15.3 ft
3) Placed 10/20 silica sand to -14.5 ft.
4) Placed Cetco "Puregold" medium bentonite chips to surface.

-water elevation (calculated) on 18-May-2005 = 185.09 ft

Depth scale: 10, 20, 30, 40, 50



PROJECT NO.: PM7802 A38	
PROJECT: 2005 Geotechnical Investigation	
LOCATION: Tailings Facility Expansion	
LOGGED BY: KAE	CHECKED BY:
SHEET 1 OF 1	HOLE NO.: DH05-15

KLOHN CRIPPEN

TEST HOLE LOG

STARTED: 5/18/2005 **FINISHED:** 5/18/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 205.5
COORDINATES (ft): N 52876.8 E 40491.62

Su - ksf: 1, 2, 3, 4
VANE PEAK / REMOLD; FIELD ◆ ◇; LAB ■ □; ▲ UC/2; △ 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
					SILT (ML) Some clay, trace to some fine sand, low plasticity, stiff, dark grey, no odour, dry, no bedding, uniform. (Tailings)	
	6,6,7	12.5	SPT1			
10	1,2,4	16.5	SPT2		Some clay, some sand, low plasticity, firm, dark grey, no odour, dry to moist, slow dilatancy, uniform. (Tailings)	
	3,2,4	9	SPT3		Some sand, some clay, low plasticity, stiff, dark grey, no odour, moist, very slow dilatancy, no bedding, uniform. (Tailings) -sample is notably cold, no ice visible in sample	
15.2 / 190.3						
20					End of Hole at 15.2 ft Note: KGCMC lysimeter installed in hole; tip at -17.2 ft	
30						
40						
50						

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KAE **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** DH05-16

TEST HOLE LOG

STARTED: 5/18/2005 **FINISHED:** 5/18/2005

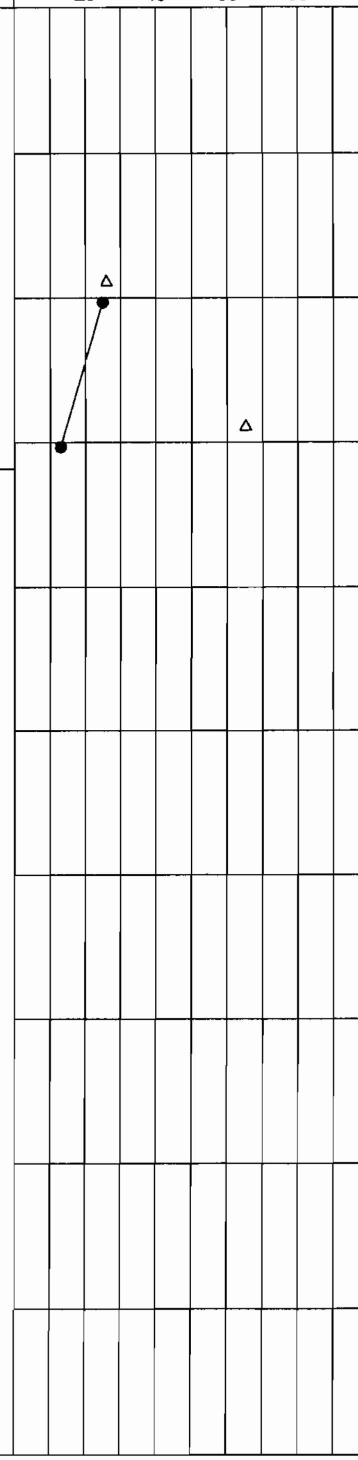
DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 205.9

COORDINATES (ft): N 52879.96 E 40485.43

Legend: Su - ksf (1, 2, 3, 4); VANE PEAK / REMOLD — FIELD ◆ / ◇, LAB ■ / □; ▲ UC/2, △ 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
0					SILT (ML) Some clay, trace to some sand, low plasticity, stiff, dark grey, no odour, moist, very slow dilatancy, no bedding, uniform. (Tailings)	
10	2,10,15	9	SPT1		Some clay, some sand, low plasticity, very stiff, dark grey, no odour, moist, no bedding, uniform. (Tailings) -a few barley grains from test pad	
	4,4,9	13	SPT2		15.9 190.0	
20					End of Hole at 15.9 ft Note: KGCMC lysimeter installed in hole.	
30						
40						
50						



KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2005 Geotechnical Investigation

LOCATION: Tailings Facility Expansion

LOGGED BY: KAE **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** DH05-17

TEST_KC TEST HOLE IMP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

STARTED: 5/18/2005 **FINISHED:** 5/18/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 202.6
COORDINATES (ft): N 52872.12 E 40410.31

Su - ksf: 1, 2, 3, 4
VANE / FIELD / LAB: PEAK ◆ ■; REMOLD ◇ □; ▲ UC/2; △ 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
					SILT (ML) Some clay, trace to some sand, low plasticity, soft, dark grey, no odour, dry, slow dilatancy, no bedding, uniform. (Tailings) Some clay, trace to some fine sand, low plasticity, hard, dark grey, no odour, dry, very slow dilatancy, no bedding, uniform. (Tailings) Some clay, some fine sand, low plasticity, hard, dark grey, no odour, moist, very slow to slow dilatancy, no bedding, uniform. (Tailings)	
	2,1,2	3	SPT1			
10	3,6,7	11.5	SPT2			
	4,12,15	12	SPT3		14.9 187.7	
20					End of Hole at 14.9 ft Note: KGCMC lysimeter installed in hole.	
30						
40						
50						

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KAE **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** DH05-18

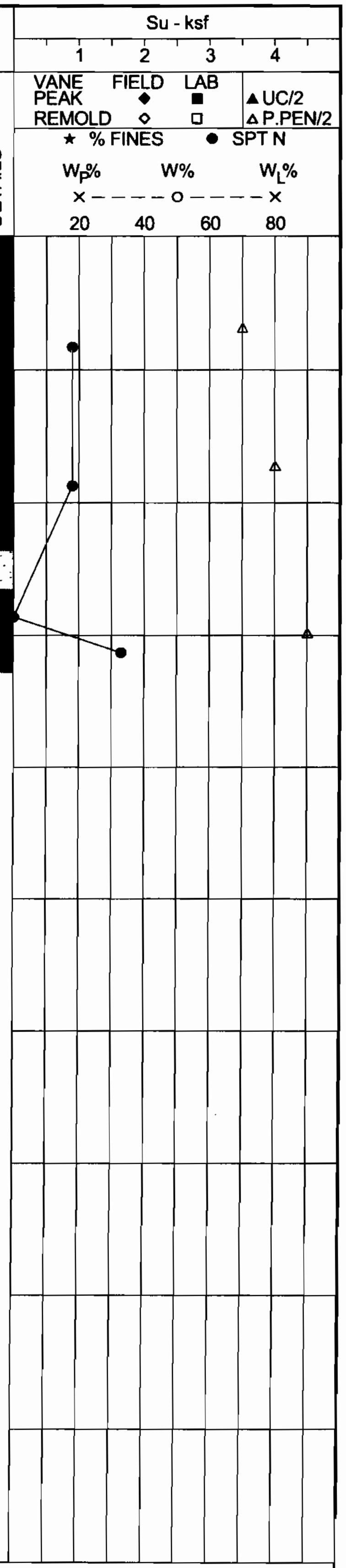
TEST_KC TEST HOLE IMP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

TEST HOLE LOG

STARTED: 5/18/2005 **FINISHED:** 5/18/2005
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 200.9
COORDINATES (ft): N 52753.55 E 40526.11

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	DESCRIPTION OF MATERIALS
				SILT (ML) Some clay, some sand, low plasticity, very stiff, dark grey, no odour, dry, slow dilatancy, no bedding. (Tailings)
	6,9,9	11	SPT1	
10	5,8,10	14	SPT2	Some clay, trace to some sand, low plasticity, very stiff to hard, dark grey, no odour, dry, very slow dilatancy, no bedding, uniform. (Tailings)
				-soft at 12.7 ft
				Some clay, some sand, low plasticity, very soft, dark grey, no odour, wet, no bedding. (Tailings)
	0	7	SPT3	Some sand (sandy?), some clay, low plasticity, hard, dark grey, no odour, dry, slow dilatancy, no bedding, uniform. (Tailings)
	7,12,11	13.5	SPT4	
16.4 / 184.5				
20				
30				
40				
50				

INSTRUMENT DETAILS



End of Hole at 16.4 ft

Well Installation Notes:

1) Hole slough to 13.25 ft
2) Placed 2x50lb Oglebay/Norton 10/20 sand to -12.75 ft in drill hole
3) Inserted vibrating wire piezometer tip and added 2x50lb Oglebay/Norton 10/20 sand to approx. -11.8 ft
4) Placed Cetco "Puregold" medium bentonite chips to surface.

Piezometer Notes:
1) SINCO Model 52611020, no. 80211 (50 psi c/w 500 ft cable) at depth of -12.75 ft.
-Reading in air: 3095.6 Hz, 29.50 C
-Reading approximately 1 hour after installation: 3096.0 Hz; 19.2 C

-water elevation (calculated) = 189.12 ft (18-May-2005)

TEST_KC TEST HOLE IMP 2005DH2-050601JM.GPJ KC_DATA.GDT 6/3/05

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38
PROJECT: 2005 Geotechnical Investigation
LOCATION: Tailings Facility Expansion
LOGGED BY: KAE **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** DH05-20-PZ

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.
	12,21,9	6	SPT-1
10	1,<1,<1	7	SPT-2
	4,7,6	2	SPT-3
	22,27,28	12	SPT-4
20	26,45,43	12	SPT-5
	21,34,48	14	SPT-6
30	24,36,41	16	SPT-7
	16,28,30	13	SPT-8
40	3,11,18		SPT-9
	26,31,36		SPT-10
50	50/3"		~~SPT-11~~

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80

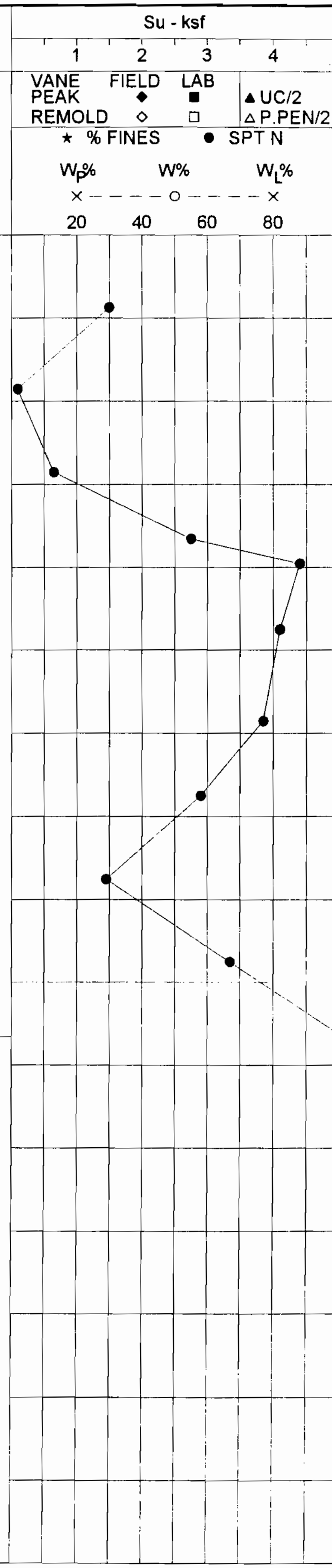
DESCRIPTION OF MATERIALS

Depth / Elev.	Description
	GRAVEL (GP) Sandy, some to trace silt, poorly graded, angular, firm to dense, moist to wet, grey and black.
4.9 / 184.1	PEAT (PT) Amorphous, soft, odour of decayed vegetal, trace brown grass, wet, dark brown.
12.0 / 177.0	GRAVEL (GP) Poorly graded, angular, loose, some quartz clasts 3 mm to 8 mm diameter, wet, dark grey and black. Single angular pebble 25 mm diameter stuck in spoon tip. Gravel appears to be argillite road fill slough. Interpretation of interval not conclusive.
16.0 / 173.0	SAND (SP) Gravelly, some to trace silt, poorly graded, gravel angular with some quartz clasts, dense, shell fragments. Sand coarsening in lower part of interval, more uniform gradation, less gravel, particles sub-rounded to rounded, moist, grey. (Marine) Weathered phyllite stone at 20 ft 3 in; boulder weathered, talcy.
22.0 / 167.0	SAND (SP) Silty, uniform gradation, medium to fine grained, sub-rounded to sub-angular, firm to dense, occasional irregular silty nodules to 20 mm diameter, moist, grey. Sand uniformly fine grained, silty, possible trace clay? Sand uniformly graded, medium grain size, dense to firm. Sand, silty; uniformly graded, medium grain size, rounded to sub-rounded, firm to loose, moist to wet. Sand, silty, trace clay, uniformly graded, rounded to sub-rounded, firm to dense, moist to wet. 2 chloritic angular rock fragments to 25 mm diameter.
45.7 / 143.3	CLAY (CL) Silty, sandy, trace gravel, poorly graded, gravel sub-angular, very dense, dry, dark grey with pale green mottling.
48.3 / 140.7	

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 in. inside diameter; sand catcher used; no sleeve.
3) Hole flushed with freshwater and backfilled with 0.375 in bentonite chips (200 lbs, approx. 2.75 cubic feet).

INSTRUMENT DETAILS



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.:** DH04-01

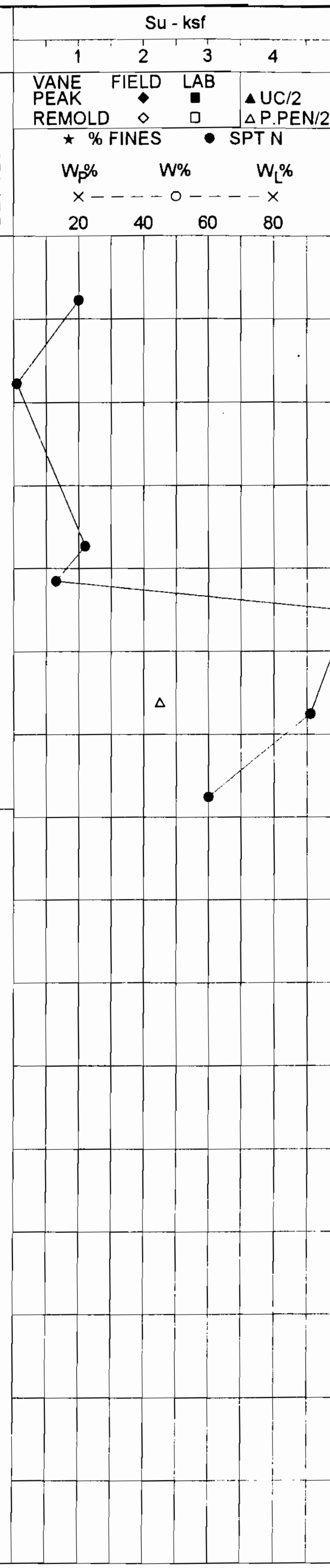
TEST HOLE LOG

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.	Depth / Elev.	DESCRIPTION OF MATERIALS
					GRAVEL (GP) Sandy, poorly graded, angular, compact, wet, no odour, dark grey and black. (Argillite Rock Fill)
	9,8,12	5	SPT-1	5.0 / 181.0	PEAT (PT) Amorphous, odour of decayed vegetal. Occasional twigs and decayed wood chunks, soft, wet, orangey brown.
10	1		SPT-2		
	9,9,13	6	SPT-3	17.9 / 168.1	GRAVEL (GP) Sandy, trace silt, poorly graded, rounded to sub-angular clasts to 12 mm diameter, compact, no odour, wet, dark grey and black.
20	8,7,6	5	SPT-4	22.0 / 164.0	In lower part interval: decreasing sand content, some angular quartz clasts, loose.
	24,32		SPT-5		CLAY (CI-CL) Silty, some sand, some gravel, medium to low plasticity, gravel angular to 10 mm diameter, very stiff, moist, blue-grey.
	40,50/5"		SPT-6	27.0 / 159.0	Gravelly from 25.4 ft to about 27 ft.
30	41,41,50	3	SPT-7		CLAY (CI) Trace silt, medium plasticity, very stiff, no odour, no bedding, moist, medium grey.
	13,24,36	12	SPT-8	34.5 / 151.5	Trace sand, indistinct contorted laminae to 1 mm thick.

End of Hole at 34.5 ft

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 in. inside diameter; sand catcher used; no sleeve.
3) Hole flushed with freshwater and backfilled with 0.375 in bentonite chips (200 lbs, approx. 2.75 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 ...H.GPJ KC_DATA.GDT 1/19/05

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

Su - ksf: 1 2 3 4

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

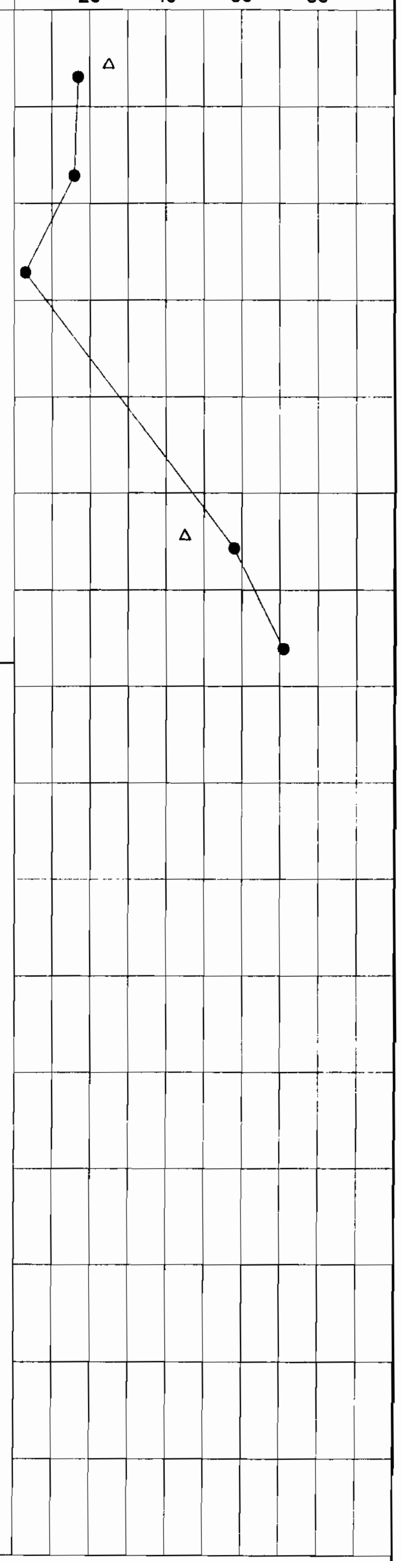
★ % FINES ● SPT N

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
0.0–1.0 (189.0)					SAND (SP) some silt, uniform gradation, loose, moist, beige. (bedding sand fill)		
1.0–12.0 (178.0)	5,8,9	14	SPT-1		SILT (ML) Gravelly, some sand, some clay; poorly graded, firm, gravel angular, black, to 5 mm diameter, no odour, moist, grey-green. (general fill or spoil with argillite)		
	6,8,8	12	SPT-2		Angular argillite clasts to 25 mm diameter.		
12.0–27.1 (162.9)	1,1,2	16	SPT-3		PEAT (PT) fibrous, odour of decayed vegetal, soft, wet, orangey brown.		
27.1–33.8 (156.2)	17,27,31	10	SPT-4		SILT(ML-CL) Some clay, low plasticity, very stiff, no odour, no bedding, moist, blue-grey.		
	22,33,38	16	SPT-5				

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80



End of Hole at 33.8 ft

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 in. inside diameter; sand catcher used; no sleeve.
3) Hole flushed with freshwater and backfilled with 0.375 in bentonite chips (200 lbs, approx. 2.75 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

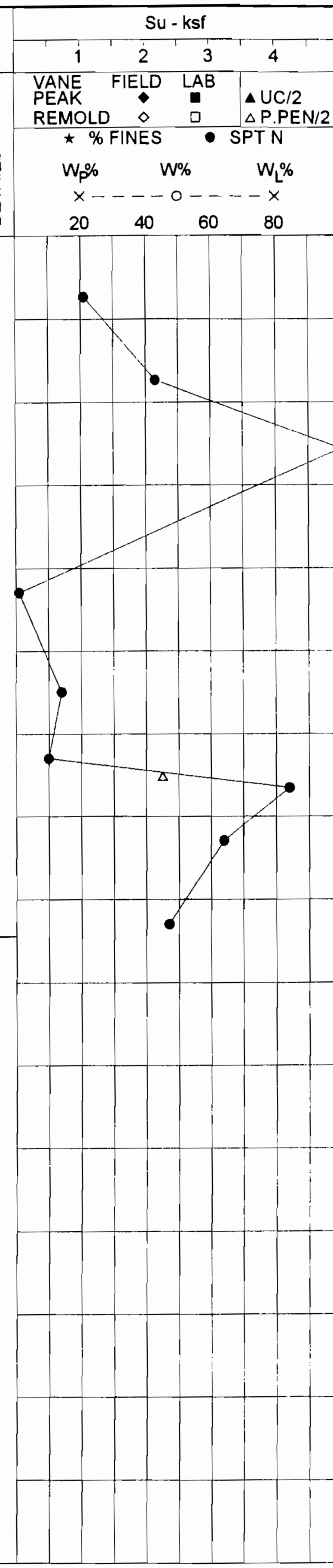
STARTED: 12/2/2004 **FINISHED:** 12/4/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 199.0
COORDINATES (ft): N 53835 E 40019

SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.
4,3,18	11	SPT-1
6,11,32	12	SPT-2
50/2"	3	~~SPT-3~~
0,0,1	2	SPT-4
8,8,6	11	SPT-5
14,4,6	10	SPT-6
5,32,52	11	SPT-7
14,23,41	16	SPT-8
18,21,26	17	SPT-9

DEPTH / ELEV.	DESCRIPTION OF MATERIALS
	SAND (SP) Silty, some gravel, trace clay, poorly graded, mottled, soft, gravel angular, moist to damp, brown and grey. (random fill) Suspected stone on final interval.
5.0 / 194.0	PEAT (PT) Fibrous, soft, odour of decayed vegetal, wet, dark brown.
7.5 / 191.5	GRAVEL (GP) Sandy, poorly graded, angular, dense, black. (argillite fill)
12.7 / [illegible] / 186.1	CLAY (CI) Sandy, silty, some gravel; medium to low plasticity, sand and gravel angular, no bedding, no odour, moist, blue-grey. (fill)
16.3 / 182.7	Argillite boulder – cored section. Siliceous veins at approx. 30 degrees to core axis; occasional quartz blebs to 1 cm diameter, disseminated sulfide mineralization on parting fabric, dark grey to black. (fill) PEAT (PT) Amorphous, soft, slight odour of decayed vegetal, wet, reddish brown.
26.7 / 172.3	SAND (SW) some silt, some gravel; well graded, loose, grey, wet, rapid dilatancy, gravel angular to sub-angular, black, to 8 mm diameter. interbedded with,
33.0 / 166.0	GRAVEL (GP) Sandy, angular to sub-rounded, loose, wet. About half of sample is peat (likely slough).
35.7 / 163.3	SILT (ML) Clayey, trace fine sand, low plasticity, very stiff, laminae to 2 mm thick; no odour, moist, blue-grey. SILT (ML) some clay; very stiff; low plasticity, blue-grey; moist; laminae 1 mm to 2 mm thick; no odour. Fine sand laminae to 2 mm thick in lower portion of interval. interbedded with, SAND (SP) some to trace silt; poorly graded (uniform); medium to fine grained; grey; moist to wet; firm to dense. Silty veins to 18 mm thick.
42.2 / 156.8	

End of Hole at 42.2 ft

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 in long split-spoon, 1.5 in. inside diameter; sand catcher used; no sleeve.
3) Hole flushed with freshwater and backfilled with 0.375 in bentonite chips (200 lbs, approx. 2.75 cubic feet).



DEPTH (feet): 10, 20, 30, 40, 50, 60, 70, 80

INSTRUMENT DETAILS

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.:** DH04-04

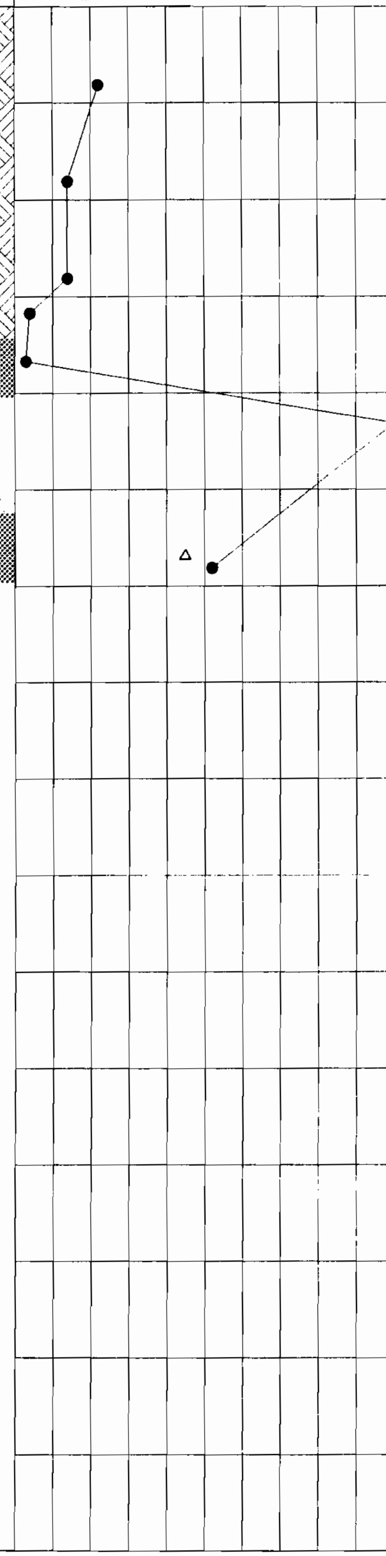
TEST HOLE LOG

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

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Depth / Elev.	DESCRIPTION OF MATERIALS
					GRAVEL (GP) Sandy, trace silt, trace clay, poorly graded, compact, gravel angular to sub-angular, occasional white quartz clasts, wet, dark grey to black. (fill)
	13,11,11	4	SPT-1		
				8.3 / 183.7	
10	32,9,5	7	SPT-2	183.2	PEAT (PT) Amorphous, odour of decayed vegetal, wet, soft, reddish brown.
				12.0 / 180.0	SAND (SPG) Gravelly, poorly graded, sand medium to coarse grained, loose, wet, some quartz clasts, gravel angular, green and grey, grey.
	7,8,6	6	SPT-3		
	2,2,2	4	SPT-4		SAND (SWG) Gravelly, trace to some silt, well graded, loose, gravel angular to sub-rounded; no odour, wet, brownish grey.
	1,1,2	5	SPT-5		
20				20.0 / 172.0	SAND (SP) Silty, poorly graded (uniform), very dense, faint bedding 1 mm to 3 mm thick, moist, grey. Compact and wet in lower portion of interval.
	28,59,60/4.5"	15	SPT-7		
	18,26,26	18	SPT-8	29.2 / 162.8	
30				162.2	SILT (ML) Some clay, some fine sand, very stiff, low plasticity, moist, no bedding, blue grey.

INSTRUMENT DETAILS

Su - ksf: 1 2 3 4
VANE / FIELD / LAB: PEAK ◆ ■; REMOLD ◇ □; ▲ UC/2; △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
20 40 60 80



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 in. 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.
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.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80

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.:** DH04-05-PZ

TEST HOLE LOG

STARTED: 12/5/2004 **FINISHED:** 12/7/2004

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 141.0

COORDINATES (ft): N 51888 E 39866

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.
	6,4,5		SPT-1
	5,6,14	14	SPT-2
	10,12,18	14	SPT-3
10	24,19,25		SPT-4
	50/1.5"		~~SPT-5~~
	14,40/refusal		SPT-6
20			
30			
40			
50			
60			
70			
80			

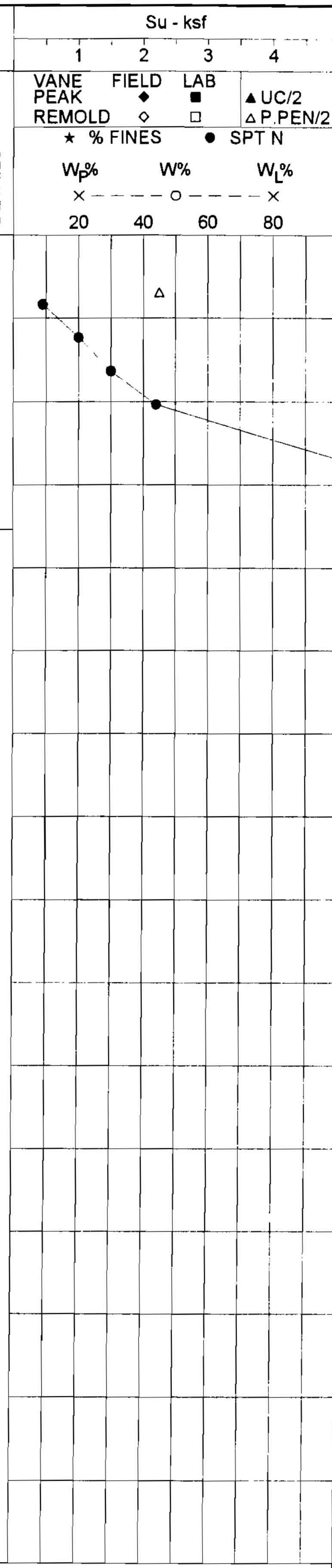
DESCRIPTION OF MATERIALS

Depth	Elev.	Description
1.0	140.0	GRAVEL (GP) poorly graded, coarse, angular, dry, grey. (crushed rock fill)
5.4	135.6	SAND (SC) Clayey, some silt, some gravel, well graded, firm, moist.. Gravel angular up to 4 mm diameter. 2 inch thick interval of loose wet sand in about the middle of the sample, greenish grey.
9.4	131.6	SILT (ML) Clayey, trace fine sand, low plasticity, stiff to very stiff, moist, indistinct laminae to 2 mm thick. grading to,
11.8	129.2	CLAY (CI) Silty, trace to some fine sand, trace fine gravel, gravel angular to 3 mm diameter, medium to low plasticity, very stiff, indistinct laminae 1 mm to 2 mm thick, blue grey.
17.7	123.3	SAND (SW) Some silt, some gravel, well graded, gravel sub-rounded to sub-angular up to 12 mm diameter, dense, moist, no odour, grey.
		BEDROCK Phyllite, some talc on folia, dark green. Spoon tip broke off in hole, random grey clay, green gravel and bentonite mud in spoon.

End of Hole at 17.7 ft

Drill Notes:
1) Drill hole terminated at 17.7 ft in bedrock.
2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 in long split-spoon, 1.5 in. inside diameter; sand catcher used; no sleeve. HQ coring rods with triple tube coring barrel.
3) Lost circulation at 12.3 ft. Placed 50 lb. bentonite chips in hole, hydrated them, and reamed the hole to seal open zone. 4)Cored interval down to about 14.5 ft. Coring apparatus malfunctioning. Switch back to tricone and bore to 17.7 ft (rate about 18 in/hr).

INSTRUMENT DETAILS



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.:** DH04-06

TEST_KC TEST HOLE IMP ...H.GPJ KC_DATA.GDT 1/19/05

TEST HOLE LOG

STARTED: 12/7/2004 **FINISHED:** 12/8/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 112.0
COORDINATES (ft): N 51705 E 39294

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Depth/Elev.	DESCRIPTION OF MATERIALS
				2.0 / 110.0	PEAT (PT) Log Mat and Peat
	2,6,9	12	SPT-1	5.0 / 107.0	CLAY (CI) Sandy, silty, trace fine gravel, medium plasticity, soft, no bedding, gravel angular to subangular, moist, dark grey. Occasional root holes, occasional green nodules to 3 mm diameter.
	38,50/2.5"	7	SPT-2	6.4 / 105.6	SILT (ML) Sandy, some clay, trace gravel, low plasticity, very stiff, dry to slightly moist, no odour; gravel angular up to 5 mm diameter, grey.
10	50/1.5"	1	~~SPT-3~~ RUN 1 ~~RUN 2~~ ~~RUN 3~~ RUN 4 RUN 5	17.8 / 94.2	BEDROCK Argillite, slaty; siliceous veins; disseminated PyX. Run 1: 8.3 – 10.4 Recovery 100%, RQD=90%. Run 2: 10.4 – 10.6 ft Recovery 0%, RQD 0%. Run 3: 10.6 – 11.0 ft Recovery 75%, RQD=0%. Run 4: 11.0 – 11.6 ft Recovery 100%, RQD=100% Run 5: 11.6 – 16.6 Recovery 100%, RQD=100%

Su - ksf: 1 2 3 4
VANE PEAK / REMOLD — FIELD ◆ ◇ — LAB ■ □ — ▲ UC/2 — △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
20 40 60 80

INSTRUMENT DETAILS

End of Hole at 17.8 ft

Drill Notes:
1) Drill hole terminated at 16.6 ft in bedrock.
2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 in long split-spoon, 1.5 in. inside diameter, sand catcher used; no sleeve. HQ coring rods with triple tube coring barrel.
3) Much more firm at 6.8 ft. Rough drilling on 7.8 ft to 8.2 ft.
4) Difficult to advance core barrel. Problems with set up.

Well Installation Notes:
1) Hole flushed with freshwater.
2) Installed 1 in. diameter standpipe piezometer, 5 ft long No. 10 slotted screen, 20 ft threaded Schedule 80 PVC pipe. 3) Bottom of screen tip at 16.5 ft below collar; 2.7 ft stickup.
4) Static water level in drill hole during installation at 3.2 ft below collar.
5) Clean silica sand to approx. 9.9 ft below collar. 65 lb. 0.375 in. bentonite chips in annulus from top of sand to collar. 6) Minimum 2.7 ft of artesian head observed 2 hours with very low flow rate.

Depth scale (feet): 10, 20, 30, 40, 50, 60, 70, 80

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.:** DH04-07-PZ

TEST HOLE LOG

STARTED: 12/8/2004 **FINISHED:** 12/9/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 113.0
COORDINATES (ft): N 51916 E 39330

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.
	9,8,7	9	SPT-1
	4,2,1	3	SPT-2
	5,10,10	11	SPT-3
	4,19,15	1	SPT-4
10	4,5,20	13	SPT-5
	9,45,50/4.5"	9	SPT-6
	19,44,24	7	SPT-7
20	11,22,27	9	SPT-8
		3	SPT-9
			RUN 1
30			RUN 2
			RUN 3
40	6,14,10/4"	4	SPT-10
			RUN 4
	6,42,50/4.5"	6	SPT-11
50			
60			
70			
80			

DESCRIPTION OF MATERIALS

GRAVEL (GW)
Sandy, trace silt, trace clay, well graded, loose, gravel angular to sub-rounded, wet; grey-brown and black.

6.7 / 106.3

SAND (SW)
Some gravel, some clay, some to trace silt, well graded, firm, occasional dark grey to black silty lenses to 10 mm thick shell fragments, gravel angular to 6 mm diameter, moist, grey.

10.0 / 103.0
11.2 / 101.8

GRAVEL (GP)
Trace sand, trace silt, poorly graded, loose, gravel angular to sub-rounded, wet, grey.

15.0 / 98.0

SILT (ML)
Clayey, some fine sand, trace fine gravel, low plasticity, stiff, shell fragments, slightly moist, grey. (marine)

20.0 / 93.0

CLAY (CI)
Some to trace silt, medium plasticity, stiff, indistinct thin bedding laminae to 1 mm thick, no odour, occasional silty lenses to 3 mm thick, moist, grey.

24.0 / 89.0

SILT (ML)
Sandy, some clay, some gravel, low plasticity, very stiff, gravel angular to sub-angular to 10 mm diameter, moist, grey.

27.6 / 85.4

GRAVEL (GW)
Some sand, trace silt, well graded, loose, wet, grey.
Rough drilling from 24 ft to 24.8 ft.

31.2 / 81.8

BOULDER
Phyllite conglomerate (likely a dropstone). Intersected 2 in. thick fault gouge, 20 degrees to core axis, at 29.2 ft. Smooth, no slickensides. Gouge is clay, sandy, silty, medium plasticity, very dense, grey, moist; (CL-CS).

Run 1: 27.6 to 30.3 ft. Recovery = 85%, RQD = 65%.

Run 2: 30.3 to 35.3 ft. Recovery = 18%, RQD = 10%.

38.0 / 75.0

CLAY (CI)
Sandy, silty, medium plasticity, wet, grey, gravel sub-angular to sub-rounded, green to black.

Run 3: 35.3 to 40.3 ft. Recovery = 14%, RQD = 0%.

43.0 / 70.0

SILT (ML)
Some fine sand, trace clay, trace fine gravel, low plasticity, stiff to firm, no odour, no bedding, gravel angular, moist, grey.

Run 4: 40.3 to 45.3 ft. Recovery = 16%, RQD = 0%.

46.8 / 66.2

SAND (SM)
Silty, clayey, some gravel, soft; moist to wet; grey; gravel sub-angular to rounded to 15 mm diameter. Two boulder fragments in core barrel on 40.3 ft to 45.3 ft; different lithology (dropstones?).

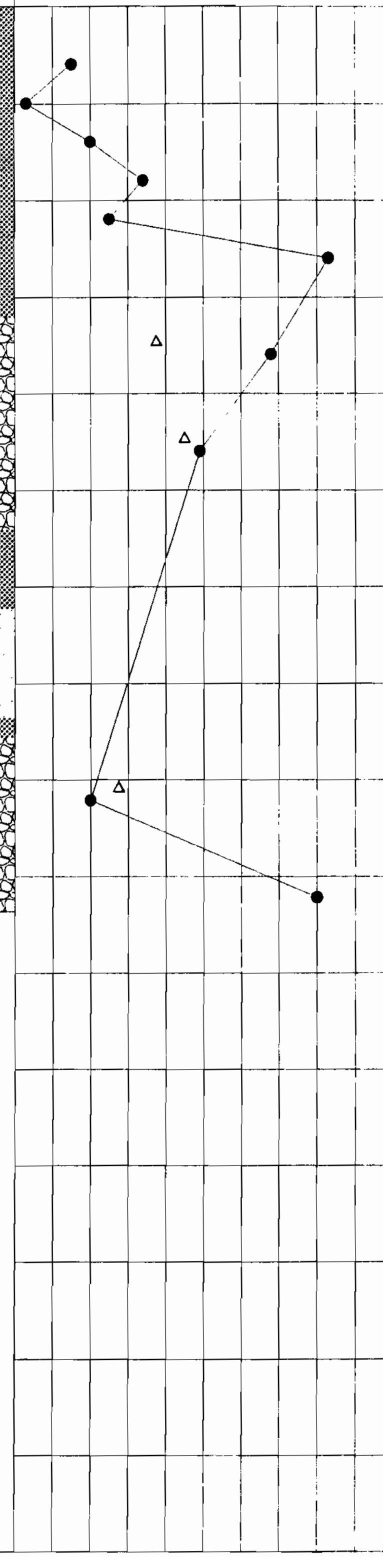
End of Hole at 46.8 ft

Drill Notes:
1) Drill hole terminated at 46.8 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 in. inside diameter; sand catcher used; no sleeve. HQ Coring rods with triple tube coring barrel.

Continued Next Page

INSTRUMENT DETAILS

Su - ksf: 1 2 3 4
VANE: FIELD ◆ LAB ■ (PEAK); REMOLD ◇ □
▲ UC/2 △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
20 40 60 80



PROJECT NO.: PM7802 A38
PROJECT: 2004 Geotechnical Investigation
LOCATION: Tailings Facilty Expansion
LOGGED BY: KE **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** DH04-08-PZ

KLOHN CRIPPEN

TEST HOLE LOG

STARTED: 12/8/2004	**FINISHED:** 12/9/2004
DRILL METHOD: Mud Rotary	
GROUND ELEV. (ft): 113.0	
COORDINATES (ft): N 51916	E 39330

Su - ksf: 1 2 3 4

VANE PEAK: FIELD ◆ LAB ■ ▲ 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

90

100

110

120

130

140

150

160

Well Installation Notes:
1) Hole flushed with freshwater. 2) Hole sloughed - put 25 lbs. (about 0.35 cu ft) bentonite chips over slough in bottom of hole. Installed 1 in. diameter standpipe piezometer; 5 ft long No. 10 slotted screen; 40 ft threaded Schedule 80 PVC pipe. Bottom of screen tip at 36.8 ft below collar; ___ ft stickup.
2) Clean silica sand to approx. 31.1 ft below collar. 0.375 in. bentonite chips in annulus from top of sand to about 27 ft below collar.
3) Drill hole necked at 16 ft below collar; filled to surface with bentonite chips.

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.:** DH04-08-PZ

TEST_KC TEST HOLE IMP ⁻H GPJ KC_DATA GDT 1/19/05

TEST HOLE LOG

STARTED: 12/10/2004 **FINISHED:** 12/10/2004
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 130.0
COORDINATES (ft): N 52246 E 39420

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	Elev.	DESCRIPTION OF MATERIALS
1.0				129.0	ROCK FILL
	<1,<1,<1	0	SPT-1		PEAT (PT) Amorphous, soft, odour of decayed vegetal, wet, dark brown. Advanced spoon with drill head to hard layer at 8.4 ft; recovered peat.
8.4	33,55,51	18	SPT-2	121.6	PHYLLITE (Bedrock) Foliated, dark green, occasional siliceous veins, upper 2 ft weathered, friable, joints at about 30 degrees and 60 degrees to core axis, joint spacing about 15 in. Some folia/joints infilled with, CLAY, trace silt; high plasticity; light grey; damp; soft (talcy?); up to 0.5 ft thick.
			RUN 1		Run 1: 10.6 to 16.5 ft. Recovery = 68%, RQD = 35%.
			RUN 2		Run 2: 16.5 to 18.8 ft. Recovery = 100%, RQD = 33%.
20.6			RUN 3	109.4	Run 3: 18.8 to 20.6 ft. Recovery = 100%, RQD = 45%. Rough drilling from 10.6 ft to 11.8 ft. Switch to core barrel.

INSTRUMENT DETAILS

Su - ksf: 1 2 3 4
VANE PEAK: FIELD ◆ LAB ■; REMOLD: FIELD ◇ LAB □; ▲ UC/2; △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%: 20 40 60 80

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80

End of Hole at 20.6 ft

Drill Notes:
1) Drill hole terminated at 20.6 ft in
2) Hole drilled with NW rods using tricone bit with bentonite mud. SPT performed with AWJ rods; 18 in long split-spoon, 1.5 in. inside diameter; sand catcher used; no sleeve. HQ Coring rods with triple tube coring barrel.

Well Installation Notes:
1) Hole flushed with freshwater.
2) Installed 1 in. diameter standpipe piezometer; 5 ft long No. 10 slotted screen; 20 ft threaded Schedule 80 PVC pipe. 3) Bottom of screen tip at 16.7 ft below collar; 3.1 ft stickup.
4) Clean silica sand to approx. 11 ft below collar. 75 lb. 0.375 in. bentonite chips in annulus from top of sand to collar.

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.:** DH04-09-PZ

TEST HOLE LOG

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

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 144.0

COORDINATES (ft): N 52183 E 39765

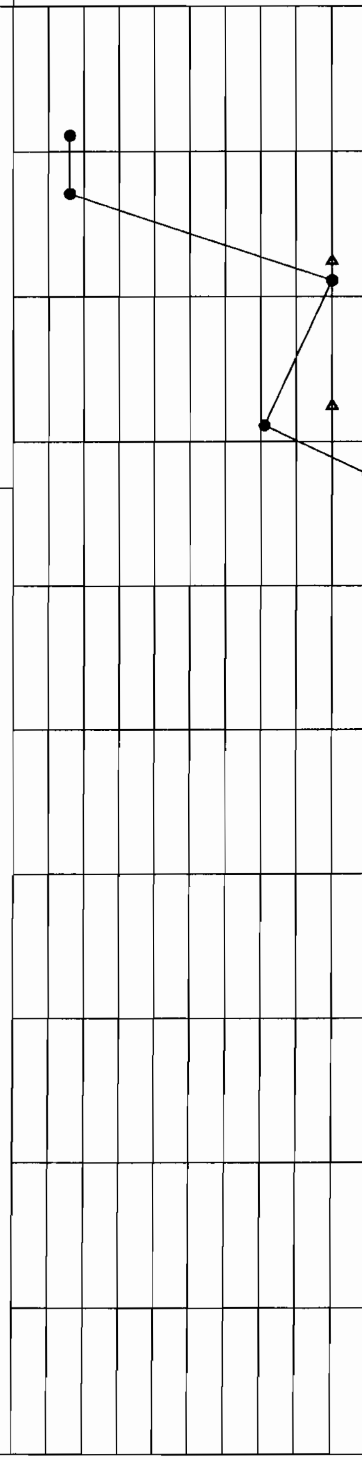
Su - ksf: 1, 2, 3, 4

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

★ % FINES ● SPT N

$W_P\%$ $W\%$ $W_L\%$ (X ---- O ---- X): 20, 40, 60, 80

DEPTH (feet)	SPT BLOWS PER 6"	RECOVERY (in.)	SAMPLE No.	SYMBOL	Depth / Elev.	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
						PEAT (PT) Brown, wet, amorphous, soft.	
	4,6,10	6	SPT-1		5.0		
					139.0	SAND (SM) Silty, clayey, some gravel, low plasticity, soft, clasts to 0.6 inch diameter, subrounded to subangular, greenish-grey, moist, weathered light green stone in tip of spoon.	
	1,4,12	8	SPT-2		8.0		
					136.0	SILT (ML) Clayey to some clay, some sand, trace gravel, low plasticity, very stiff, clasts to 0.3 inch diameter, gravel subangular to subrounded blue-grey, slightly moist.	
10	19,48,42	16	SPT-3		12.0		
					132.0	SAND (SM) Silty, trace clay, trace gravel, coarse sand, very dense, clasts to 0.2 inch diameter, subangular to subrounded grey, slightly moist.	
	32,50,50/2.5"	14.5	SPT-4		15.0		
					129.0	SAND (SW) Gravelly, trace to some silt, very dense, clasts to 0.8 inch diameter, gravel subangular to subrounded, moist, no odour, grey, no bedding. Slough/mud in top of sampler.	
	4,50/5"	5	SPT-5		16.6 / 127.4		
20							
30							
40							
50							



End of Hole at 16.6 ft

Drill Notes:
1) Drill hole terminated at 20.6 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. HQ Coring rods with triple tube coring barrel.

KLOHN CRIPPEN

PROJECT NO.: PM7802 A38

PROJECT: 2004 Geotechnical Investigation

LOCATION: Tailings Facilty Expansion

LOGGED BY: KE **CHECKED BY:**

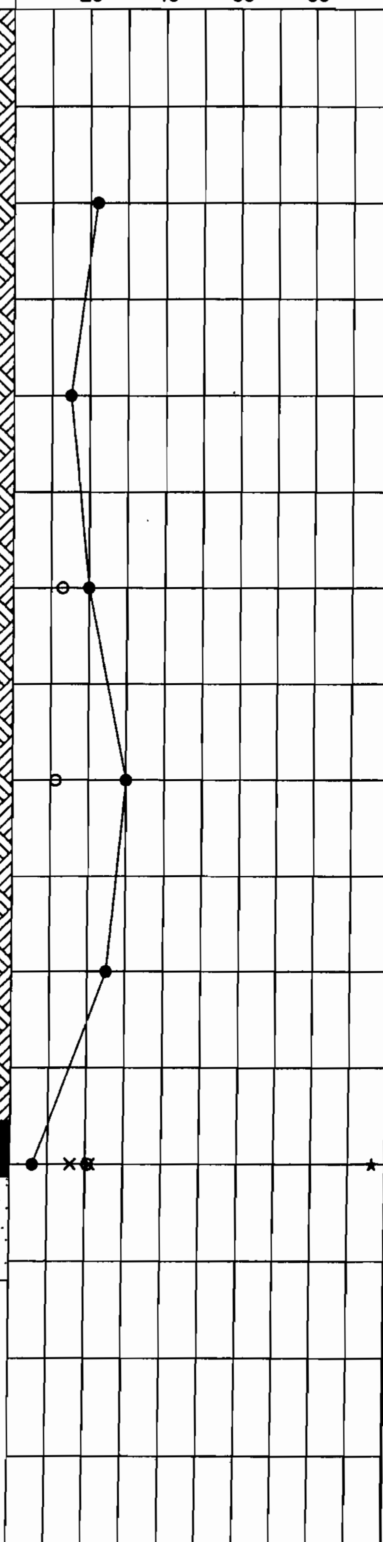
SHEET 1 **OF** 1 **HOLE NO.:** DH04-11

TEST HOLE LOG

STARTED: 9/24/2002 **FINISHED:** 9/24/2002
DRILL METHOD: Hollow Stem Auger
GROUND ELEV. (ft): 236.6
COORDINATES (ft): N 53465 E 39948

Su - ksf: 1 2 3 4
VANE / FIELD / LAB: PEAK ◆ ■ ; REMOLD ◇ □ ; ▲ UC/2 ; △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%: × – – – o – – – ×
20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	DESCRIPTION OF MATERIALS
0				SILT (ML) some fine sand gradation, moderately dense, low to no plasticity, pyrite crystals, dry, grey-black (tailings)
10	17,12,10		SPT-1	
				trace gravel to 1", angular
20	9,8,7		SPT-2	
30	5,9,11		SPT-3	
40	13,14,16		SPT-4	trace gravel to 1", angular
50	6,11,14		SPT-5	
60	1,2,4		SPT-6	loose
66.0 / 170.6				End of Hole at 66.0 ft



Drill Notes:
1) Drill hole terminated at 66 ft to refusal - possibly bedrock.
2) Drill hole completed using Hollow Stem Augers.

Well Installation Notes:
1) 5 ft screen 10 slot 2" schedule 40 PVC from 66 ft.

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: PM7802 29
PROJECT: 2002 Geotechnical Investigation
LOCATION: Existing Tailings Facility
LOGGED BY: CH **CHECKED BY:**
SHEET 1 OF 2 **HOLE NO.:** DH-02-04

TEST HOLE LOG

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS	Su - ksf

STARTED: 9/24/2002 FINISHED: 9/24/2002

DRILL METHOD: Hollow Stem Auger

GROUND ELEV. (ft): 236.6

COORDINATES (ft): N 53465 E 39948

Su - ksf: 1 2 3 4

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

★ % FINES ● SPT N

W_P% W% W_L%

× — — — — o — — — — ×

20 40 60 80

Depth scale: 90, 100, 110, 120, 130, 140, 150, 160

to 61 ft.
2) 10/20 filter sand from 66 ft to 60.6 ft.
3) 3/8" medium bentonite chips from 60.6 ft to 57.7 ft.
4) Quik grout from 57.7 ft to surface.
5) Well stick up 3 ft.
6) Water level on Sept 30, 2002 was 56.24 ft.

KLOHN CRIPPEN

PROJECT NO.: PM7802 29	
PROJECT: 2002 Geotechnical Investigation	
LOCATION: Existing Tailings Facility	
LOGGED BY: CH	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-02-04

TEST HOLE LOG

STARTED: 9/25/2002 FINISHED: 9/25/2002

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 227.2

COORDINATES (ft): N 53390 E 40402

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS
0					SILT (ML) some fine sand (pyrite crystals), loose to dense, non-plastic, dry to moist, grey-brown (Tailings).
5	3,2,3		SPT-1		
10	6,5,5		SPT-2		
15	6,6,9		SPT-3		
20	5,7,11		SPT-4		
25	4,17,16		SPT-5		At 24.5 ft: Trace of gravel to 1 in. (waste rock fill).
30	50/5"		SPT-6		At 29.5 ft: Silty Gravel seam, some fine sand, angular, dense, dark grey (waste rock fill).
35	3,2,3		SPT-7		35.3 / 191.9 At 34.5 ft: Trace of 1 in. gravel (waste rock fill). End of Hole at 35.3 ft

Depth scale (feet): 10, 20, 30, 40, 50, 60, 70

INSTRUMENT DETAILS

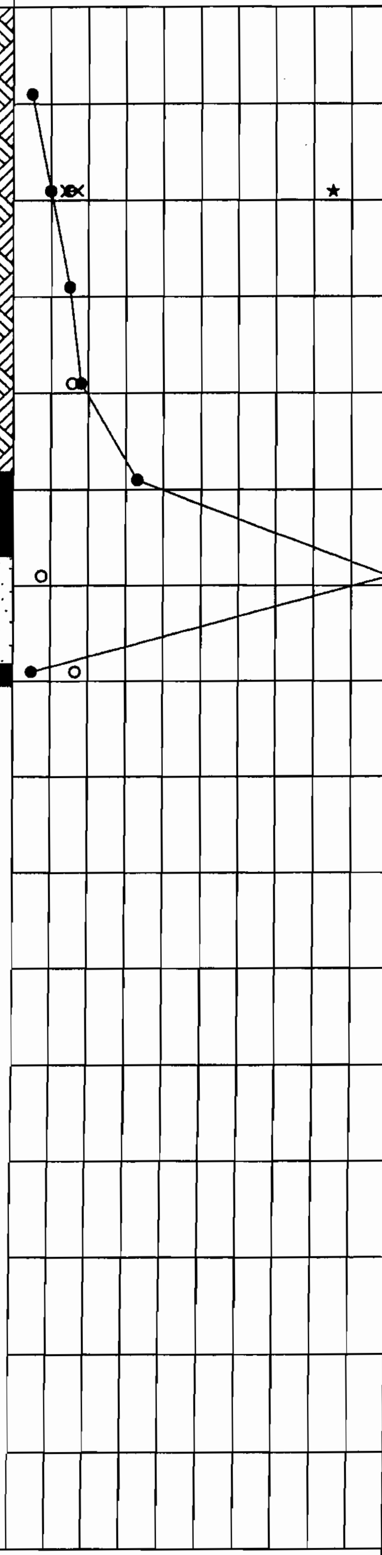
Su - ksf: 1 2 3 4

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

★ % FINES ● SPT N

W_P% W% W_L%

20 40 60 80



Drill Notes:
1) Drill hole terminated at 35.3 ft in tailings.
2) Drill hole completed using mud rotary.

Well Installation Notes:
1) Quik grout from 24.0 ft to surface.
2) 3/8" medium bentonite chips from 24.0 ft to 28.5 ft.
3) 10/20 filter sand from 28.5 ft to 34.0 ft.
4) 5 ft long, 2 in. diameter, schedule 40 PVC slotted screen, 0.1 slot, 34.0 ft to 29.0 ft.
5) 2 in. diameter, schedule 40 PVC pipe, 29.0 ft to surface.
6) 3/8" medium bentonite chips 34.0 ft to 35.3 ft.
7) Well stick up
8) Water level on Sept 30, 2002 was 33.55 ft.

KLOHN CRIPPEN

PROJECT NO.: PM7802 29

PROJECT: 2002 Geotechnical Investigation

LOCATION: Existing Tailings Facility

LOGGED BY: CH CHECKED BY:

SHEET 1 OF 1 HOLE NO.: DH-02-05

TEST HOLE LOG

STARTED: 9/26/2002 FINISHED: 9/26/2002

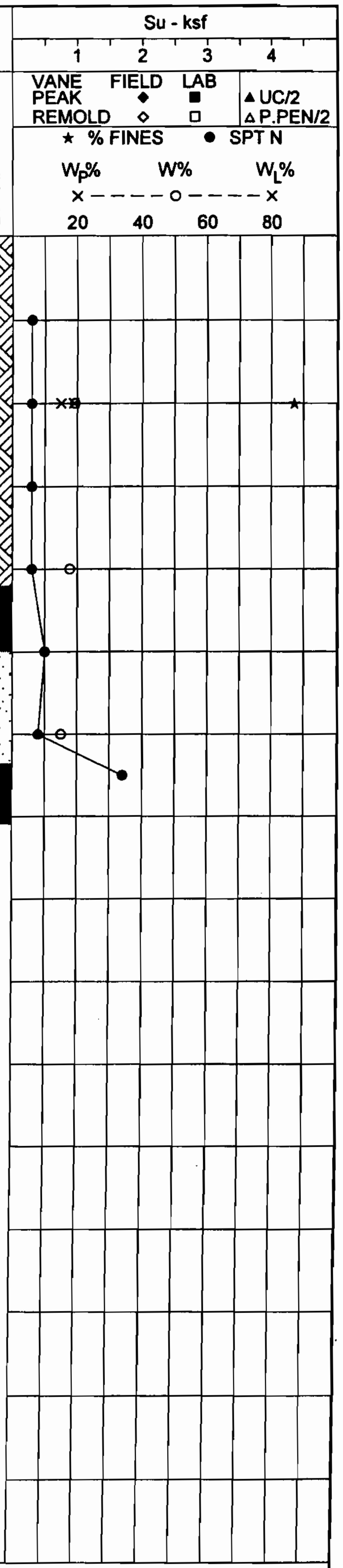
DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 183.1

COORDINATES (ft): N 52995 E 39735

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS
					SILT (ML) some fine sand (pyrite crystals), loose, non-plastic, dry to moist, grey-brown (Tailings).
	3,3,3		SPT-1		
10	2,2,4		SPT-2		
	1,2,4		SPT-3		
					At 18.0 ft: Driller observed 1.0 ft thick coarse seam - possibly waste rock fill - not sampled.
20	0,1,5		SPT-4		
					At 22.0 ft: Driller observed 0.5 ft thick coarse seam - possibly waste rock fill - not sampled.
	5,6,4		SPT-5		
					At 29.0 ft: Driller observed 0.5 ft thick coarse seam - possibly waste rock fill - not sampled.
30	2,4,4		SPT-6		
	5,11,23		SPT-7		
35.5 / 147.6					Trace of fine to medium sand at end of split spoon - possibly blanket drain material.
					End of Hole at 35.5 ft

INSTRUMENT DETAILS



Drill Notes:
1) Drill hole terminated at 35.3 ft in tailings.
2) Drill hole completed using mud rotary.
3) Drill hole logged by Environmental Design Engineering of Sheridan Wyoming.

Well Installation Notes:
1) Monitoring well installed under Supervision of Environmental Design Engineering.
2) Quik grout from 21.0 ft to surface.
3) 3/8" medium bentonite chips from 25.0 ft to 21.0 ft.
4) 10/20 filter sand from 31.8 ft to 25.0 ft.
5) 5 ft long, 2 in. diameter, schedule 40 PVC slotted screen, 0.1 slot, 31.8 ft to 26.3 ft.
6) 2 in. diameter, schedule 40 PVC pipe, 26.3 ft to surface.
7) 3/8" medium bentonite chips 35.5 ft to 31.8 ft.
8) Well stick up 2.1 ft.
9) Water level on Oct 7, 2002 was 28.27 ft.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 0

KLOHN CRIPPEN

PROJECT NO.: PM7802 29

PROJECT: 2002 Geotechnical Investigation

LOCATION: Existing Tailings Facility

LOGGED BY: EDE CHECKED BY:

SHEET 1 OF 1 HOLE NO.: DH-02-06

TEST HOLE LOG

STARTED: 9/26/2002 FINISHED: 9/27/2002

DRILL METHOD: Hollow Stem Auger / Mud Rotary/HQ Core

GROUND ELEV. (ft): 143.4

COORDINATES (ft): N 52356 E 40354

Su - ksf: 1 2 3 4

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

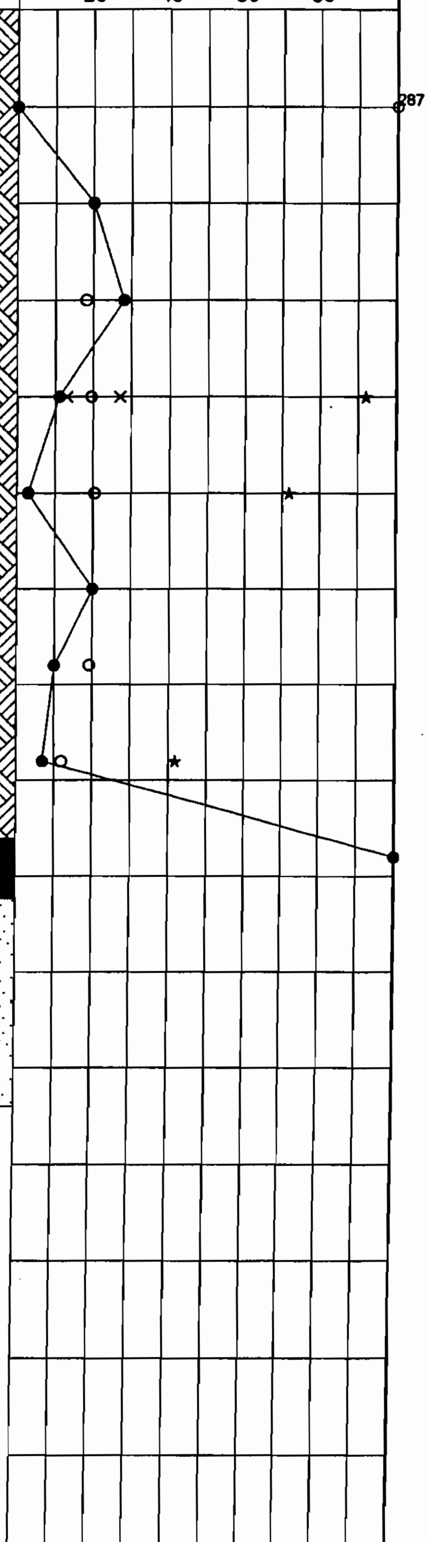
★ % FINES ● SPT N

W_P% W% W_L%

20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	Depth / Elev.	DESCRIPTION OF MATERIALS
						PEAT (PT) fibrous, wood fragments, wet, dark brown.
	0,0,0		SPT-1			
10	3,8,12		SPT-2		10.0 133.4	SILT (ML) Some fine to medium sand, some plastic clay, compact, moist, grey.
	6,14,14		SPT-3			
20	2,3,8		SPT-4			
	1,1,2		SPT-5		25.0 118.4	SAND (SM) Fine to medium sand, silty, trace plastic clay, loose, moist, grey.
30	2,9,11		SPT-6		30.0 113.4	CLAY (CLG) gravelly, some fine sand, gravel to 1", rounded, medium plastic, dense, moist, grey. (Glacial Till)
	5,4,6		SPT-7			mariposite
40	3,3,4		SPT-8			soft
	50/5"		SPT-9		44.0 99.4	BEDROCK Phyllite, coarse grained, silicieous, fractured, pyrite crystals, quartz viening, grey. Run 1: 49 ft to 51 ft. Recovery = 54%, RQD = 0%. Run 2: 52 ft to 57 ft. Recovery = 85%, RQD = 35%.
50			RUN 1			
			RUN 2		57.0 86.4	
60						End of Hole at 57.0 ft
70						
80						

INSTRUMENT DETAILS



End of Hole at 57.0 ft

Drill Notes:
1) Drill hole terminated in bedrock.
2) Hollow Stem Auger from 0 to 30 feet, Mud Rotary from 30 to 49 feet, HQ Core from 49 to 57 feet.
3) Drill hole started making water at 33 feet.

Well Installation Notes:
1) 57 to 46.3 feet 10/20 filter sand.
2) 56.5 to 51.5 feet 2" schedule 40 PVC 10 slot screen.
3) 46.3 to 43 feet 3/8 inch medium bentonite chips.
4) 43 to surface Quik grout.
5) Well stick up 2.1 ft.
6) This instrument is flowing.

KLOHN CRIPPEN

PROJECT NO.: PM7802 29	
PROJECT: 2002 Geotechnical Investigation	
LOCATION: East of Pond 6	
LOGGED BY: CH	CHECKED BY:
SHEET 1 OF 1	HOLE NO.: DH-02-07

TEST HOLE LOG

STARTED: 9/26/2002 **FINISHED:** 9/28/2002

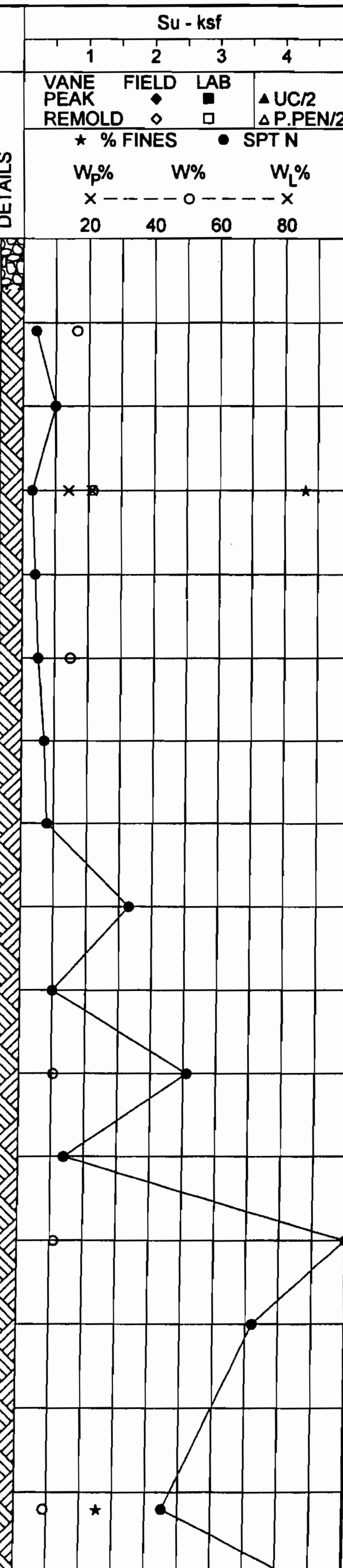
DRILL METHOD: Mud Rotary/HQ Core

GROUND ELEV. (ft): 194.8

COORDINATES (ft): N 53150 E 39848

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	DEPTH / ELEV.	DESCRIPTION OF MATERIALS
					SILT (ML) some fine sand (pyrite crystals), loose to dense, non-plastic, dry to moist, grey-brown (Tailings).
	4,2,2		SPT-1		
10	3,2,8		SPT-2		
	1,1,2		SPT-3		
20	0,1,3		SPT-4		
	2,2,3		SPT-5		
30	3,3,4		SPT-6		
	3,2,6		SPT-7		
40	7,7,26		SPT-8		
	4,4,6		SPT-9	47.0 / 147.8	SAND AND GRAVEL (SWG) Fine to coarse sand, trace silt, (tailings), angular, compact, wet, grey, with argillite to 1 in. (Waste rock fill).
50	9,15,36		SPT-10	53.0 / 141.8	SAND (SP) Fine sand, trace to some silt, trace fine gravel, trace medium to coarse sand, subrounded, compact, slow dilatancy, moist, brown, occasional pyrite crystal, occasional organic inclusion, roots.
	7,7,7		SPT-11		At 56.5 ft, changing to gravelly sand - not sampled.
60	38,50/3"		SPT-12	59.5 / 135.3	SILT (ML) Some fine sand, trace medium to coarse sand, trace gravel (>1.5 in.), subrounded to subangular, very dense, low to non plastic, dry, grey (Glacial Till). Occasional Clay inclusion, medium plasticity, grey, thickness and spacing varies.
	13,21,51		SPT-13		
70			RUN 1		
	15,17,28		SPT-14		
80					

INSTRUMENT DETAILS



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KLOHN CRIPPEN

PROJECT NO.: PM7802 29

PROJECT: 2002 Geotechnical Investigation

LOCATION: Existing Tailings Facility

LOGGED BY: EDE/JP **CHECKED BY:**

SHEET 1 OF 2 **HOLE NO.:** DH-02-08

TEST HOLE LOG

STARTED: 9/26/2002	FINISHED: 9/28/2002
DRILL METHOD: Mud Rotary/HQ Core	
GROUND ELEV. (ft): 194.8	
COORDINATES (ft): N 53150	E 39848

Su - ksf: 1 2 3 4

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

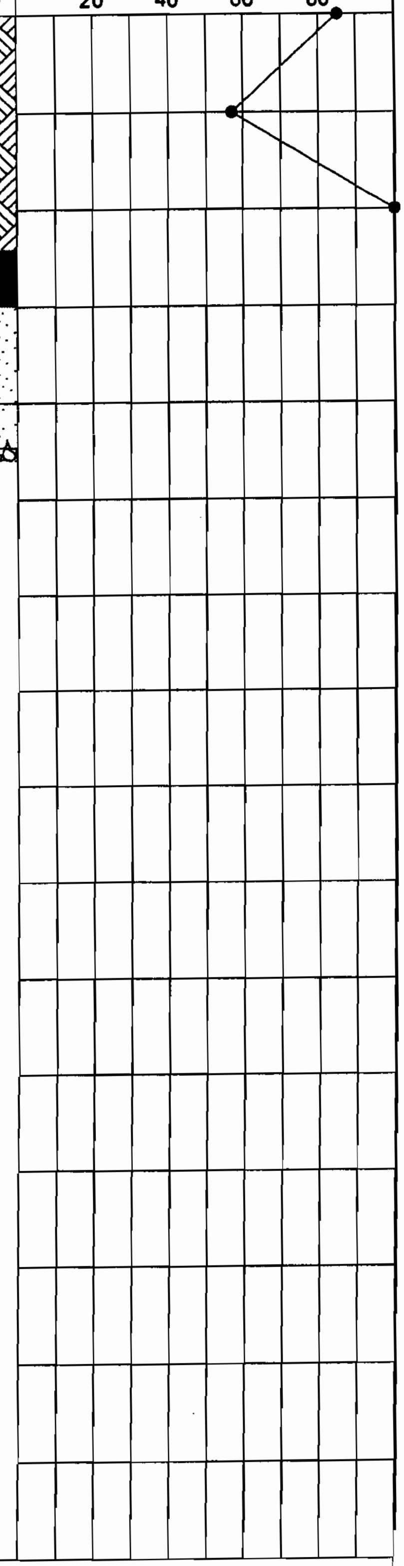
★ % FINES ● SPT N

W_p% W% W_L%

× – – – – o – – – – ×

20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	Elev./Depth	DESCRIPTION OF MATERIALS
	27,39,46		SPT-15			At 80.0 ft: Changing to gravelly silt, some sand, fine to coarse gradation, subangular, very dense, wet, grey, pyrite inclusions in gravel (Glacial Till).
	6,22,35		SPT-16			
					88.0 / 106.8	At 87.5 ft to 88.0 ft: 6 in. sand seam - not sampled. BOULDERS
90	50/3"		SPT-17			
			RUN 2		91.5 / 103.3	BEDROCK RUN 2: 90.0 ft to 91.5 ft; REC = 88%; RQD = 0%. Gravel fragments, changing to heavily broken graphitic argillite, with pyrite crystal inclusions at tip.
			RUN 3			RUN 3: 91.5 ft to 93.5 ft; REC = 96%; RQD = 0%. Broken sericitic argillite, with quartz stringers, and pyrite inclusions transitioning to broken quartz, with occasional pyrite inclusion.
			RUN 4			RUN 4: 93.5 ft to 98.0 ft; REC = 91%; RQD = 54%; Graphitic argillite, with quartz and pyrite inclusions, transitioning to chloritized phyllite, with polished faces.
100			RUN 5		103.0 / 91.8	RUN 5: 98.0 ft to 103.0 ft; REC = 100%; RQD = 48%; Chloritized phyllite transitioning to graphitic argillite, with carbonate inclusions.
110						End of Hole at 103.0 feet
120						
130						
140						
150						
160						



Drill Notes:
1) Drill hole terminated at 103.0 ft in bedrock.
2) Drill hole observed to be caving at 50.0 ft, 59.0 ft, 75.0 ft, 82.0 ft, and 87.0 ft.
3) Drill hole completed using combination of mud rotary (tricone bit) and HQ coring methods.
4) HW casing installed from surface to 89.0 ft.
5) Drill hole completed under supervision of Environmental Design Engineering of Sheridan Wyoming from Surface to 50.0 ft.
6) Drill fluid lost at 51.5 ft and 86.5 ft.

Well Installation Notes:
1) Tailings backfill from surface to 3.0 ft.
2) Quik grout from 3.0 ft to 92.0 ft.
3) 3/8" medium bentonite chips from 92.0 ft to 95.0 ft.
4) 10/20 filter sand from 95.0 ft to 102.0 ft.
5) 5 ft long, 2 in. diameter, 10 slot size, schedule 40 PVC pipe from 97.0 ft to 102.0 ft.
6) 2 in. diameter, schedule 40 PVC pipe, 97.0 ft to surface.
7) Hole slough from 102.0 ft to 103.0 ft.
8) Drill hole took approximately 400 gallons of grout during completion. Grout believed to be taken into formation at depth of 51.5 ft.
9) Well stick up 2.25 ft.
10) Water level on Sept 30, 2002 was 29.9 ft.

KLOHN CRIPPEN

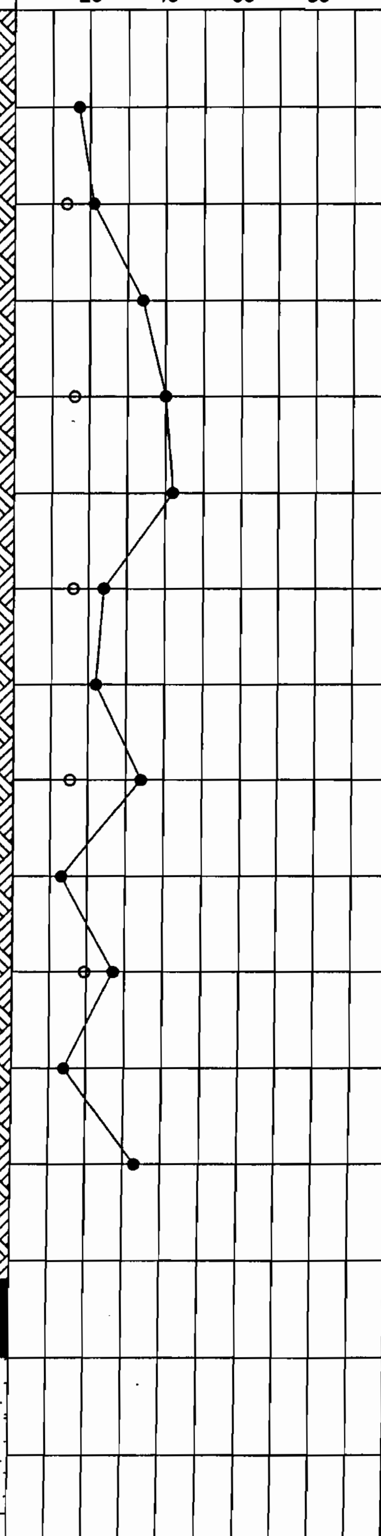
PROJECT NO.: PM7802 29	
PROJECT: 2002 Geotechnical Investigation	
LOCATION: Existing Tailings Facility	
LOGGED BY: EDE/JP	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-02-08

TEST HOLE LOG

STARTED: 9/30/2002 **FINISHED:** 10/2/2002
DRILL METHOD: Mud Rotary
GROUND ELEV. (ft): 235.9
COORDINATES (ft): N 53452 E 39946

Su - ksf: 1 2 3 4
VANE: FIELD / LAB; PEAK ◆ ■; REMOLD ◇ □; ▲ UC/2; △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
× – – – – o – – – – ×
20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT	DETAILS
					SILT (ML) trace fine sand, compact, damp, grey-brown with sandy gravel, angular, pyrite crystals. (tailings/waste rock fill)		
	3,4,13		SPT-1				
10	14,10,11		SPT-2		10.0 / 225.9 SILT (ML) Some fine sand, occasional gravel, angular,compact, moist, grey-brown. (tailings)		
	6,16,18		SPT-3		15.0 / 220.9 SAND AND GRAVEL (GW-ML) Some silt, trace to some fine sand, angular, dense, moist, grey-brown. (waste rock fill/tailings)		
20	13,18,22		SPT-4		20.0 / 215.9 SAND AND GRAVEL (SG) Fine to coarse sand, some silt, angular, dense, wet, grey. (waste rock fill)		
	14,21,21		SPT-5		25.0 / 210.9 SILT (ML) Some fine sand, trace gravel, dense, moist, grey-brown. (tailings)		
30	9,10,14		SPT-6		30.0 / 205.9 SILT (ML) some fine sand, occasional gravel, compact, non-plastic, moist, grey-black. (tailings)		
	8,9,13		SPT-7				
40	16,18,16		SPT-8				
	10,7,6		SPT-9		45.0 / 190.9 SILT (ML) some fine sand, compact, non-plastic, moist, pyrite crystals, grey. (tailings)		
50	10,15,12		SPT-10				
	4,6,8		SPT-11				
60	6,13,10/3"		SPT-12		60.0 / 175.9 SILT (ML) and PEAT (PT) Some fine sand, occasional gravel, wet, non-plastic, grey to organics, tree remnants, brown. (tailings/peat)		
					62.0 / 173.9 BEDROCK Phyllite, sericitic, pyrite crystals, fractured, oxidized, quartz viens.		
70							
					73.0 / 162.9 BEDROCK graphitic argillite, oxidized pyrite inclusions, fractured, quartz viens.		
80					78.0 / 157.9 End of Hole at 78.0 ft		



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KLOHN CRIPPEN

PROJECT NO.: PM7802 29
PROJECT: 2002 Geotechnical Investigation
LOCATION: Existing Tailings Facility
LOGGED BY: JP **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** DH-02-10

KC_TEST_HOLE-IMP 2002DH _DATA.GDT 6/10/03

TEST HOLE LOG

STARTED: 9/30/2002	FINISHED: 10/2/2002
DRILL METHOD: Mud Rotary	
GROUND ELEV. (ft): 235.9	
COORDINATES (ft): N 53452	E 39946

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS	Su - ksf: 1 2 3 4; VANE PEAK FIELD ◆ LAB ■; REMOLD ◇ □; ▲ UC/2 △ P.PEN/2; ★ % FINES ● SPT N; W_P% W% W_L% ×----o----×; 20 40 60 80
90					Drill Notes: 1) Drill hole terminated at 78 ft in bedrock. 2) Drill hole cased with HW casing from surface to 65 ft. 3) Mud rotary from 0 to 65 ft, HQ core from 65 to 78 ft. 4) Rig chatter at 21 ft, 23 to 24 ft, 24.5 to 25 ft, 27 to 28 ft. Well Installation Notes: 1) 5 ft 10 slot screen schedule 40 2" PVC from 78 to 73 ft. 2) 10/20 filter sand from 78 to 70 ft. 3) 3/8" medium bentonite chips from 70 to 65.9 ft. 4) Quik grout from 65.9 ft to surface. 5) Well drilled out and grouted on October 3, 2002.		
100							
110							
120							
130							
140							
150							
160							

KLOHN CRIPPEN

PROJECT NO.: PM7802 29	
PROJECT: 2002 Geotechnical Investigation	
LOCATION: Existing Tailings Facility	
LOGGED BY: JP	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-02-10

TEST HOLE LOG

STARTED: 2/4/2001 FINISHED: 2/5/2001

DRILL METHOD: Mud Rotary, HS Augers, NQ Core

GROUND ELEV. (ft): 115.1

COORDINATES (ft): N 52242.24 E 39217.76

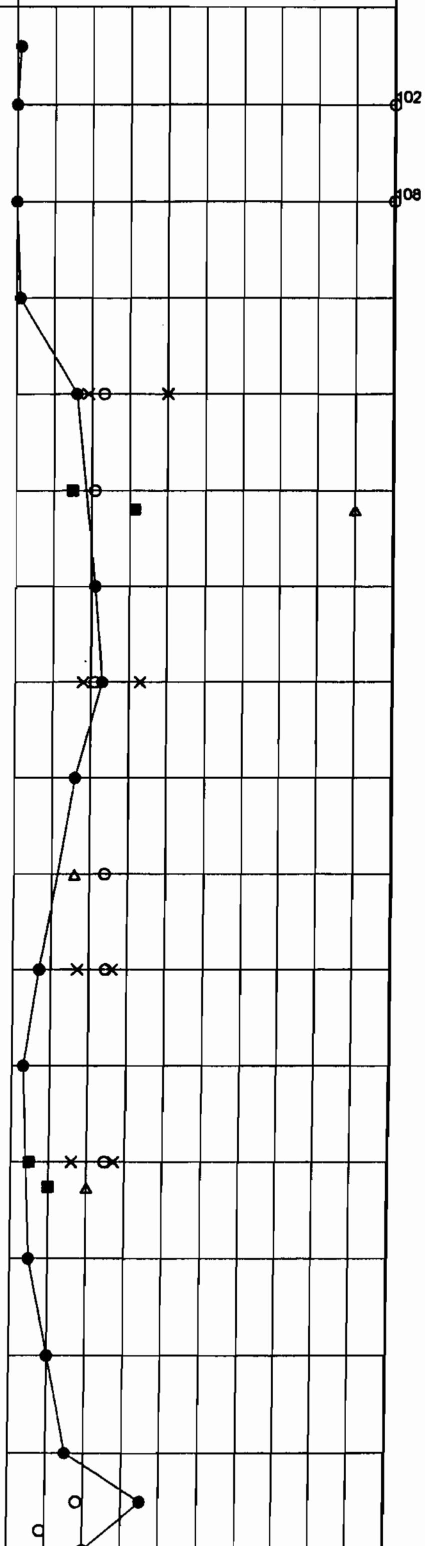
Su - ksf: 1, 2, 3, 4

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

★ % FINES ● SPT N

W_P% W% W_L%: × – – – – o – – – – ×; 20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	DESCRIPTION OF MATERIALS
	1/15", 1		SPT-1	PEAT (PT) Amorphous, slightly fibrous, root inclusions, wet, brown to purple brown, slight organic odor, compressible.
	WT/18"		SPT-2	
10	WT/18"		SPT-3	
	1/15",1		SPT-4	
18.0 / 97.1				CLAY (CI) Organic inclusions (wood), medium plasticity, firm to stiff, grey.
20	3,7,9		SPT-5	
			SHELBY-1	
26.0 / 89.1				SANDY GRAVEL (GM) Fine to coarse gradation sand, trace silt, subrounded, wet, brown (alluvial).
28.0 / 87.1				CLAY (CI) Trace fine to coarse gradation sand, trace fine gravel, low to medium plasticity, firm to stiff, grey to grey-blue.
30	5,9,12		SPT-6	
	7,10,13		SPT-7	At 35.0 ft, randomly spaced silt seams.
40	5,7,9		SPT-8	Decreasing sand/gravel content at 40.0 ft.
			SHELBY-2	At 45.0 ft consistency of clay observed to be decreasing to soft.
50	3,3,4		SPT-9	At 50.0 ft, randomly spaced silt seams (1/8 inch thick). Some fine sand.
	0,0,3		SPT-10	At 55.0 ft medium to high plasticity, very soft to soft.
60			SHELBY-3	At 60.0 ft medium plasticity, very soft to soft.
	WT/4", 1/4", 5		SPT-11	At 65.0 ft, firm to soft, 1/8 inch thick silt and sandy silt seams randomly spaced throughout.
70.0 / 45.1				SILT (ML) Some fine to medium gradation sand, trace fine gravel, non-plastic, loose to compact, wet, grey-black to grey, slow dilatancy.
70	4,4,6		SPT-12	
72.0 / 43.1				SAND (SW) Fine to coarse gradation sand, some gravel, trace silt, subangular to subrounded, compact, wet, grey-black (alluvial).
	10,7,8		SPT-13	
78.3 / 36.8	13,23,12		SPT-14	
80				



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KLOHN CRIPPEN

PROJECT NO.: PM7802 20

PROJECT: 2001 Geotechnical Drill Program

LOCATION: 600 ft West of Pond 6

LOGGED BY: JP CHECKED BY:

SHEET 1 OF 2 HOLE NO.: DH-01-01

KC_TEST_HOLE-IMP WESTTI KC_DATA.GDT 8/10/03

TEST HOLE LOG

STARTED: 2/4/2001	FINISHED: 2/5/2001
DRILL METHOD: Mud Rotary, HS Augers, NQ Core	
GROUND ELEV. (ft): 115.1	
COORDINATES (ft): N 52242.24	E 39217.76

Su - ksf: 1, 2, 3, 4

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

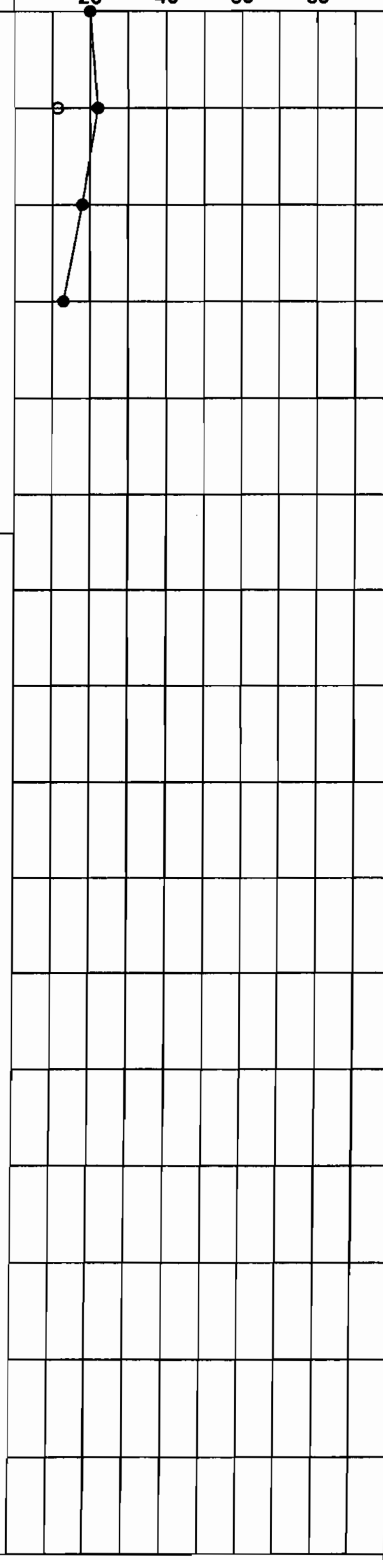
★ % FINES ● SPT N

W_P% W% W_L%

× – – – – ○ – – – – ×

20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	18,9,11		SPT-15		SILT (ML) Sandy, to some fine to coarse gradation sand, some gravel, subrounded to subangular, low plasticity, compact, grey-blue (glacial till). At 80.0 ft, some medium gradation sand. At 85.0 ft, fine sand seams, randomly spaced throughout with random thickness.	
	6,4,18		SPT-16			
90	18,11,7		SPT-17		At 95.0 ft, changing to SAND AND SILT, fine gradation sand, trace medium to coarse gradation sand, trace fine gravel, subangular to subrounded, compact, wet, grey, slow dilatancy (glacial till).	
	13,10,3		SPT-18		100.0	
100			RUN 1 ~~RUN 2~~ RUN 3		15.1 BEDROCK Phyllite, chloritized, with talc infilling, sericitic. Run 1: 100.0 ft - 101.5 ft; Rec = 56%; RQD = 0%. Fractures at 60 degrees to axis. Run 2: 101.5 ft - 102.0 ft; Rec = 100%; RQD = 0%. 107.0 Run 3: 102.0 ft - 107.0 ft; Rec = 100%; RQD = 95%. 8.1	
110					End of Hole at 107.0 ft	
120						
130						
140						
150						
160						



Drill Notes:

1) Drill hole terminated in bedrock at a depth of 107.0 ft.
2) Drill hole grouted to surface with bentonite grout upon completion.
3) Soft drilling to 18.0 ft.
4) Hard drilling at 26.0 - 28.0 ft, 70.0 ft, 87.5 ft, 92.0 ft.
5) Hollow stem augers to 55.0 ft then switch to mud rotary. Use water as fluid from 55.0 ft to 72.0 ft. Use drilling mud (EZ Mud) from 72.0 ft to 107.0 ft.
6) Hole noted to be caving between 77.0 ft and 78.0 ft.
7) Hole caving from above when coring from 100.0 ft to 107.0 ft.

KLOHN CRIPPEN

PROJECT NO.: PM7802 20	
PROJECT: 2001 Geotechnical Drill Program	
LOCATION: 600 ft West of Pond 6	
LOGGED BY: JP	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-01-01

TEST HOLE LOG

STARTED: 2/6/2001 FINISHED: 2/6/2001

DRILL METHOD: Mud Rotary, HS Augers, NQ Core

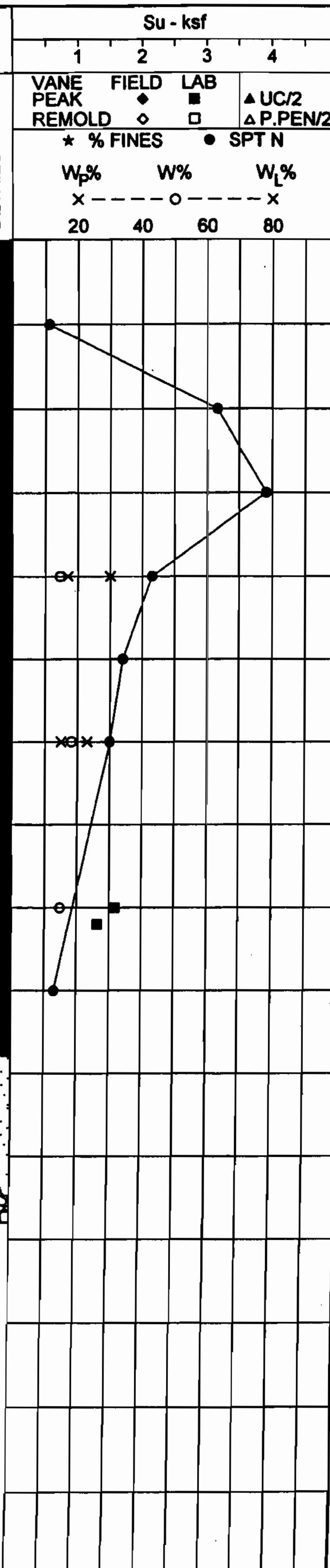
GROUND ELEV. (ft): 104.7

COORDINATES (ft): N 52052.59 E 39067.33

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.
	WT/3",1/6",6,5		SPT-1
10	15,34,29		SPT-2
	17,28,50		SPT-3
20	12,19,24		SPT-4
	10,16,18		SPT-5
30	11,14,16		SPT-6
			SHELBY-1
40			SHELBY-2
	4,5,8		SPT-7
50			RUN 1
			RUN 2
60			
70			
80			

Depth / Elev.	DESCRIPTION OF MATERIALS
	PEAT (PT) Amorphous, slightly fibrous, wood inclusions, wet, brown, dark-brown, slight organic odor, compressible.
10.7 / 94.0	SILTY SAND (SM) To SAND AND SILT (SP-SM), fine gradation sand, trace medium gradation sand, trace coarse gradation sand, trace fine gravel, subrounded to angular, very dense, moist to wet, grey, shell fragments (marine sand). At 11.5 ft, 1 inch silt seam.
15.0 / 89.7	CLAY (CI) Trace fine to coarse gradation sand, trace fine gravel, low plasticity near 15.0 ft, medium plasticity to 40.0 ft then high plasticity, hard, grey-blue, homogenous structure, shell fragments to 25.0 ft (marine clay). At 25.0 ft, sand content and shell fragments decreasing. At 30.0 ft, low plasticity, firm to stiff consistency, clay becoming softer. Sample from Shelby Tube 1 at 35 ft not retrieved, and lost in hole.
41.0 / 63.7	SILT (ML) Some to trace fine gradation sand, trace medium to coarse sand, trace gravel to 3 inch, medium to high plasticity, compact, wet, grey, slow dilatancy (glacial till).
47.5 / 57.2	BEDROCK Phyllite, sericitic, with 1.5 ft quartz seam at 56.5 ft, changing to soapstone, chloritized, with talcy infilling at 58.5 ft. Run 1: 49.0 ft to 54.0 ft; Rec = 100%; RQD = 100%; Fractures at 30 degrees to 45 degrees, tight, infilled with silt, R3 strength. Run 2: 54.0 ft to 59.0 ft; Rec = 100%; RQD = 85%; Fractures at 75 degrees, 45 degrees. Quartz seam, with dolomitic seams, R4 strength, open fracture at 57.0 ft. Soapstone, crumbles easily, R1 strength.
59.0 / 45.7	End of Hole at 59.0 ft

INSTRUMENT DETAILS



Drill Notes:
1) Drill hole hole terminated in bedrock at a depth of 59.0 ft.
2) Monitoring well installed in hole upon completion under direction of Environmental Design Engineering.
3) Hollow stem augers to 15.0 ft; mud rotary (with EZ-Mud and quik gel) 15.0 ft to 49.0 ft; NQ core 49.0 ft to 59.0 ft.
4) Drilling becoming easier at 25.0 ft, clay becoming softer.
5) Hard drilling at 41.0 ft and 47.5 ft.

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: PM7802 20

PROJECT: 2001 Geotechnical Drill Program

LOCATION: 800 Southwest of Pond 6

LOGGED BY: JP CHECKED BY:

SHEET 1 OF 2 HOLE NO.: DH-01-02

TEST HOLE LOG

STARTED: 2/6/2001	FINISHED: 2/6/2001
DRILL METHOD: Mud Rotary, HS Augers, NQ Core	
GROUND ELEV. (ft): 104.7	
COORDINATES (ft): N 52052.59	E 39067.33

Su - ksf: 1 2 3 4

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

★ % FINES ● SPT N

W_P% W% W_L%

20 40 60 80

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

Monitoring Well Installation Notes:
1) Drill hole flushed with water prior to well installation.
2) Slough from 59.0 ft to 57.0 ft.
3) 5 ft well screen, 10 slot size, schedule 40 PVC, 2 inch diameter from 57.0 ft to 52.0 ft.
4) 2 inch diameter, schedule 40 PVC pipe from 52.0 ft to surface.
5) 20/40 silica sand from 52.0 ft to 49.0 ft.
6) Bentonite grout from 49.0 ft to surface.

90

100

110

120

130

140

150

160

KLOHN CRIPPEN

PROJECT NO.: PM7802 20	
PROJECT: 2001 Geotechnical Drill Program	
LOCATION: 800 Southwest of Pond 6	
LOGGED BY: JP	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-01-02

KC_TEST_HOLE-IMP WESTTI KC_DATA.GDT 6/10/03

TEST HOLE LOG

STARTED: 2/7/2001 **FINISHED:** 2/8/2001
DRILL METHOD: Mud Rotary, HS Augers, NQ Core
GROUND ELEV. (ft): 108.9
COORDINATES (ft): N 52607.95 E 39090.4

Su - ksf: 1 2 3 4
VANE PEAK / REMOLD — FIELD ◆ ◇ — LAB ■ □ — ▲ UC/2 — △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%: 20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.
	WT/18"		SPT-1
10	5,1/11",1		SPT-2
	3,1/10",1		SPT-3
20	13,24,29		SPT-4
	52/5"		SPT-5
30	50,52,50/4"		SPT-6
			RUN 1
40			RUN 2
	34,47,43		SPT-7
50	13,26,34		SPT-8
	25,22,23		SPT-9
60	18,9,19		SPT-10
	16,14,14		SPT-11
70	19,31,37		SPT-12
	21,23,33		SPT-13
80			

DESCRIPTION OF MATERIALS

PEAT (PT)
Amorphous, slightly fibrous, root inclusions, wood fragments, wet, purple-brown, compressible.

14.3 / 94.6 / 94.4
SAND (SP)
Fine gradation sand, some medium gradation sand, trace coarse gradation sand, trace gravel, some silt, subrounded, loose, wet, brown-grey, slow dilatancy (marine sand).

CLAY (CI)
Trace to some silt, trace fine to medium gradation sand, organic inclusions near 14.5 ft, medium to high plasticity, firm to hard, grey, homogeneous structure at 19.0 ft.

22.0 / 86.9
SILT (ML)
Some sand to sandy, fine gradation sand, trace medium gradation sand, trace coarse gradation sand, trace gravel, subrounded to subangular, non-plastic, dense to very dense, wet near 22 ft, grey.
At 32.0 ft, cobbles or boulders evident by hard layers and rig chatter.

At 38.0 ft, decreasing sand content.

At 44.2 ft, 2 inch thick fine gradation sand seam, some silt, trace medium to coarse gradation sand, damp to moist.

At 49.0 ft, 5 inch seam SAND (SP), fine gradation sand, some medium gradation sand, trace coarse sand, some silt, subrounded, moist, dense.

54.3 / 54.6 / 53.4
SAND (SP)
Medium to fine gradation sand, trace coarse gradation sand, trace gravel, trace silt, angular to subrounded, compact to dense, dry to moist, grey.

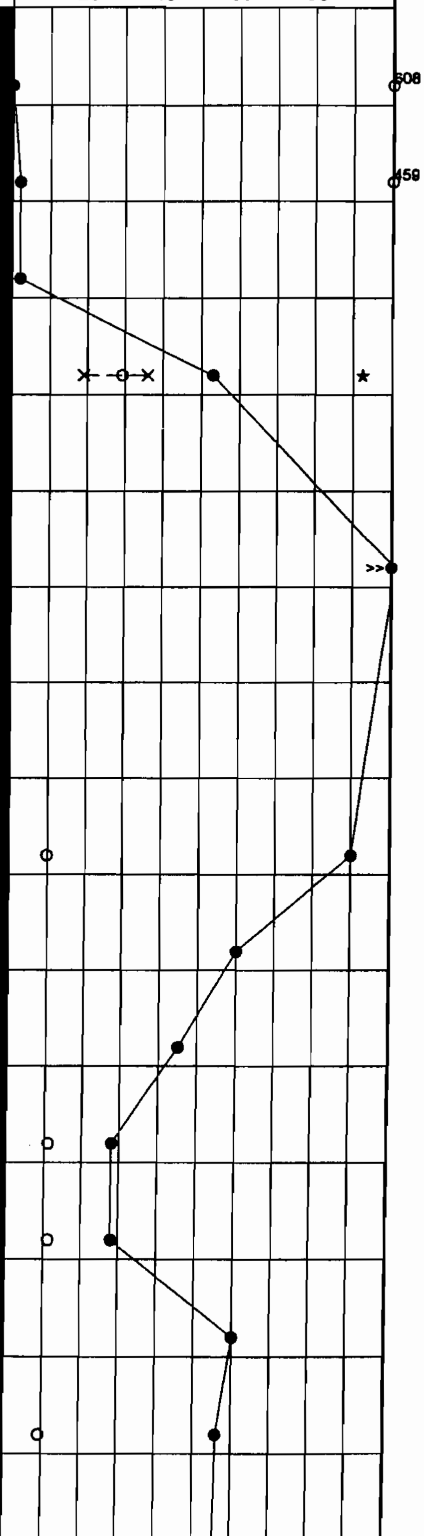
SILT (ML)
Some fine gradation sand, to SAND AND SILT, compact, wet, grey, slow dilatancy, with randomly spaced (1/8 inch) sand seams throughout, moist to wet, grey-black.

64.0 / 44.9
SILTY SAND (SM)
Fine to medium gradation sand, trace coarse gradation sand, trace to some gravel, subrounded to subangular, compact, wet, grey, slow dilatancy (glacial till). At 64.5 ft, 3 inch SILT (ML) seam, some fine gradation sand, non-plastic, firm, grey.

70.2 / 38.8 / 38.4
SAND (SP)
Fine to medium gradation sand, trace silt, wet, grey.

SILT (ML)
Some fine to coarse gradation sand, grading to SILT AND SAND near 84.0 ft, trace gravel, non-plastic to low plasticity, dense to very dense, wet near 80.0 ft.

INSTRUMENT DETAILS



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KLOHN CRIPPEN

PROJECT NO.: PM7802 20
PROJECT: 2001 Geotechnical Drill Program
LOCATION: Southwest corner of proposed expansion
LOGGED BY: JP **CHECKED BY:**
SHEET 1 OF 2 **HOLE NO.:** DH-01-03

TEST HOLE LOG

STARTED: 2/7/2001 **FINISHED:** 2/8/2001
DRILL METHOD: Mud Rotary, HS Augers, NQ Core
GROUND ELEV. (ft): 108.9
COORDINATES (ft): N 52607.95 E 39090.4

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	54,54/4'		SPT-14		grey, slow dilatancy (glacial till).	
	31,25,30		SPT-15		84.2 / 84.7 / 85.4 — CLAY (CL) Trace fine to coarse gradation sand, low plasticity, firm, grey.	
					86.0 / 22.9 — SILT (ML) Dry, grey, homogeneous structure.	
90			RUN 3		BEDROCK Phyllite, slightly chloritized, talcy, R2 to R3 strength. Run 3: 86.0 ft - 91.5 ft; Rec = 93%; RQD% = 86%. Fractures at 60 to 80 degrees to axis, tight with no evidence of weathering.	
			RUN 4		96.5 / 12.4 — Run 4: 91.5 ft - 96.5 ft; Rec = 100%; RQD% = 100%.	
					End of Hole at 96.5 ft	
100						
110						
120						
130						
140						
150						
160						

Su - ksf: 1 2 3 4
VANE PEAK / REMOLD; FIELD ◆ ◇; LAB ■ □; ▲ UC/2; △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
20 40 60 80

Drill Notes:
1) Drill hole terminated in bedrock at a depth of 96.5 ft.
2) Monitoring well installed in hole upon completion under direction of Environmental Design Engineering.
3) Hollow Stem Augers to 19.0 ft; Mud Rotary 19.0 ft to 86.0 ft (Using EZ-Mud and Quik Gel); NQ Core 86.0 ft to 96.5 ft.
4) Hard drilling at 32.0 ft to 24.0 ft, 45.0 ft, 48.5 ft, 53.5 ft, 66.5 ft, and 86.0 ft.

Monitoring Well Installation Notes:
1) Hole flushed with water prior to well installation.
2) 5 ft well screen, 10 slot size, schedule 40 PVC from 96.0 ft to 91.0 ft.
3) 2 inch diameter schedule 40 PVC pipe from 91.0 ft to surface.
4) 20/40 silica sand from 91.0 ft to 87.0 ft.
5) Bentonite grout from 87.0 ft to surface.

KLOHN CRIPPEN

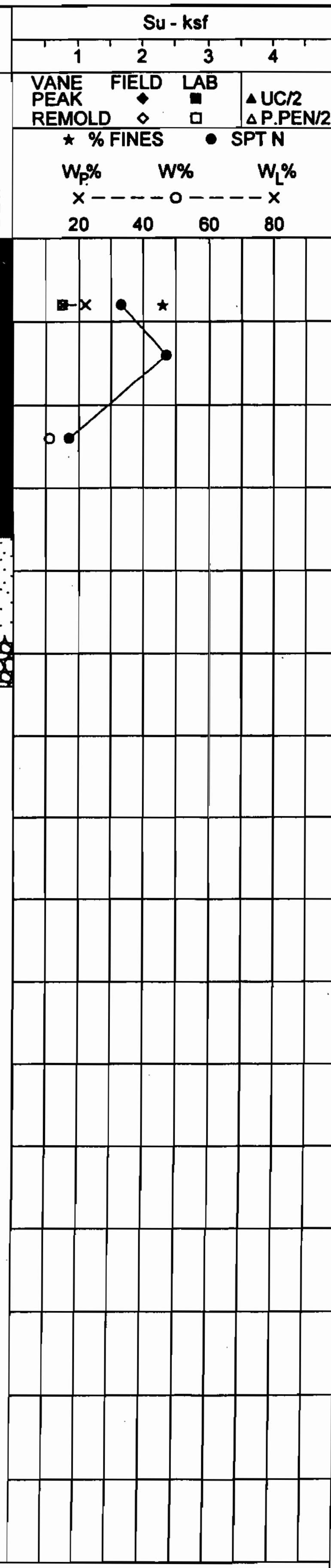
PROJECT NO.: PM7802 20
PROJECT: 2001 Geotechnical Drill Program
LOCATION: Southwest corner of proposed expansion
LOGGED BY: JP **CHECKED BY:**
SHEET 2 OF 2 **HOLE NO.:** DH-01-03

TEST HOLE LOG

STARTED: 2/8/2001 FINISHED: 2/8/2001
DRILL METHOD: HS Augers, HQ Core
GROUND ELEV. (ft): 134.1
COORDINATES (ft): N 53087.25 E 39497.48

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	Depth / Elev.	DESCRIPTION OF MATERIALS
			GRAB 1	2.0 / 132.1	PEAT (PT) Amorphous, woody, root and wood inclusions, wet, brown-black, compressible.
			GRAB 2	4.0 / 130.1	SILTY SAND (SM) Fine to coarse gradation sand, some gravel, subangular to subrounded, brown, wet.
	3,16,17		SPT-1	5.5 / 128.6	SILTY CLAY (CL-ML) Some fine to coarse gradation sand, trace fine gravel, low plasticity, subrounded, compact, moist, grey (glacial till). Transition to underlying silt not known.
	11,21,26		SPT-2		
10	6,7,10		SPT-3	17.0 / 117.1	SILT (ML) Some sand, to SAND AND SILT, fine to medium gradation sand, trace coarse gradation sand, some gravel, low plasticity, subrounded, compact to dense, grey-blue (glacial till).
	35/0"		SPT-4		
20			RUN 1	27.0 / 107.1	BEDROCK Sericitic Phyllite, slightly chloritized, with graphitic seams to 22.0 ft, becoming more graphitic 22.0 ft to 27.0 ft, R2 - R3 strength, pyrite crystals throughout, talcy infilling in fractures. 8.5 inch quartz seam at 23.8 ft. RUN 1: 17.0 ft to 22.0 ft; Rec = 98%; RQD%=58%. RUN 2: 22.0 ft to 27.0 ft; Rec = 97%; RQD%=60%.
			RUN 2		
30					End of Hole at 27.0 ft
40					
50					
60					
70					
80					

INSTRUMENT DETAILS



Drill Notes:
1) Drill hole terminated in bedrock at a depth of 27.0 ft.
2) Drill hole advanced in 2.5 ft excavation to allow drill rig to be level. Samples of peat and silty sand obtained from bank of excavation adjacent to drill hole.
3) Monitoring well installed in drill hole under direction of Environmental Design Engineering.
4) Hollow stem augers to 17.0 ft; NQ Core to 27.0 ft.
5) Hard drilling at 9.0 ft, 17.0 ft.

Monitoring Well Installation Notes:
1) Hole flushed with water prior to installation.
2) Hole caved from 24.0 ft to 27.0 ft.
3) 5 ft long screen, 2 inch diameter, schedule 40 PVC pipe, 10 slot size, from 24.0 ft to 19.0 ft.
4) Schedule 40 solid PVC pipe, 2 inch diameter, 19.0 ft to surface.
5) 20/40 silica sand from 24.0 ft to 18.0 ft.
6) Medium bentonite chips from 18.0 ft to surface.

KLOHN CRIPPEN

PROJECT NO.: PM7802 20
PROJECT: 2001 Geotechnical Drill Program
LOCATION: West of West Buttress, South Location
LOGGED BY: JP CHECKED BY:
SHEET 1 OF 1 HOLE NO.: DH-01-04

TEST HOLE LOG

STARTED: 3/3/2001 FINISHED: 3/4/2001

DRILL METHOD: HS Augers, NQ Core

GROUND ELEV. (ft): 79.0

COORDINATES (ft): N 53100.09 E 39163.79

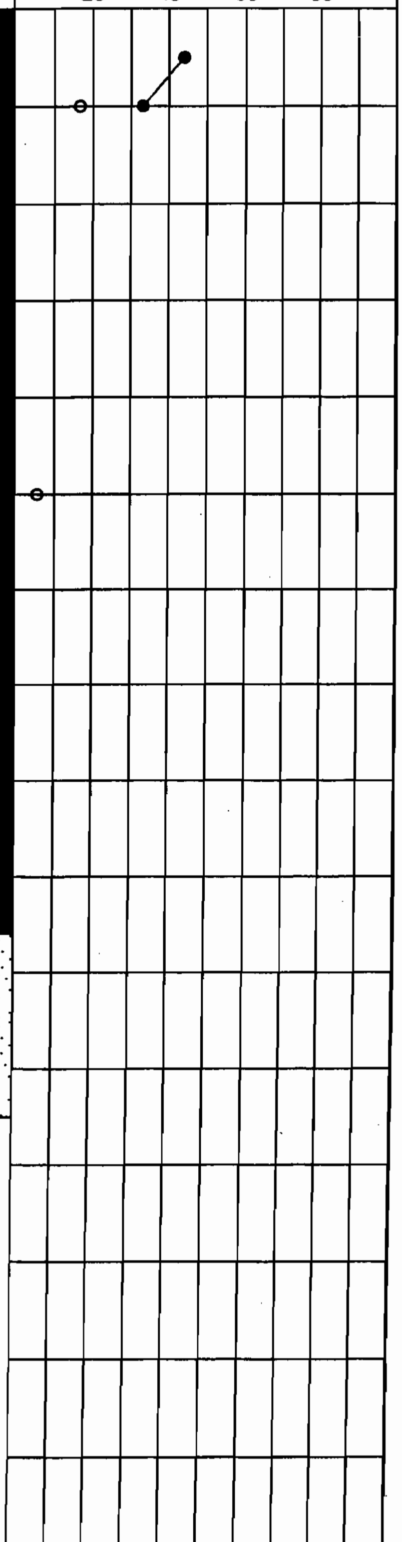
Su - ksf: 1 2 3 4

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"	SAMPLE TYPE	SAMPLE No.	Elev./Depth	DESCRIPTION OF MATERIALS
0					PEAT (PT) Amorphous, woody, wet, brown, organic odor.
	15,24,20		SPT-1	2.5 / 76.5	SILTY SAND (SM) Fine to coarse gradation sand (predominantly fine), trace to some gravel, angular to subrounded, compact to very dense, wet, grey, shell fragments.
	1,10,23		SPT-2		
10	50/2"		SPT-3		At 10.0 ft, becoming moist.
	100/2"		SPT-4		
20	100/1"		SPT-5	22.0 / 57.0	At 20.0 ft changing to sandy silt, fine to coarse gradation sand, some gravel, subrounded very dense, low plasticity, dry, grey, shell fragments. At 21.0 ft, transitioning to SILT (ML), trace fine gradation sand, dry, brown (observed from drill cuttings).
	100/2"		SPT-6		SAND (SP) Fine gradation sand, some silt to SAND AND SILT, very dense, damp, light brown, light grey at 35.0 ft with higher silt content.
30	100/2"		SPT-7		
	95/3"		SPT-8	36.0 / 43.0	SAND (SP) Fine gradation sand, some medium to coarse gradation sand, some silt, some gravel, subrounded, very dense, dry to moist, grey (glacial till).
40	100/1"		SPT-9		At 40.0 ft, changing to SANDY SILT, fine to medium gradation sand, trace coarse gradation sand, angular, very dense, wet, grey-brown.
	100/2"		SPT-10	47.5 / 31.5	At 45.0 ft, CLAY (CI), some sand, fine to coarse gradation, trace gravel, medium plasticity, subrounded to subangular, very dense, hard, grey (glacial till).
50			RUN 1		BEDROCK Graphitic Phyllite, slaty, quartz banding, R2 strength, fractures along planes and friable, black with pyrite crystals. Quartz seam from 47.5 ft to 49.4 ft, and 56.5 ft to 56.8 ft, carbonate inclusion.
			RUN 2		
			RUN 3	57.5 / 21.5	Run 1: 48.0 ft to 52.0 ft; Rec = 75%; RQD% = 35%. Run 2: 52.0 ft to 55.0 ft; Rec = 100%; RQD% = 39%. End of Hole at 57.5 ft. Run 3: 55.0 ft to 57.5 ft; Rec = 100%; RQD% = 73%.
60					
70					
80					



Drill Notes:
1) Drill hole terminated in bedrock at a depth of 57.5 ft.
2) Helicopter used to mobilize drill rig to site.
3) Hollow stem augers to 47.5 ft. Augers removed then hole cased with HQ casing to 50.0 ft. NQ core from 48.0 ft to 57.5 ft.
4) Rig chatter at 3.5 ft to 4.0 ft, 8.0 ft to 10.0 ft, 13.5 ft, 36.0 ft to 38.0 ft, 41.0 ft to 44.0 ft, 45.0 ft to 47.5 ft.
5) 20.0 ft of cave in hole after removing augers and prior to advancing HQ casing.
6) Monitoring well installed upon completion under direction of Environmental Design Engineering.

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: PM7802 20

PROJECT: 2001 Geotechnical Drill Program

LOCATION: Former DH-C, West of Existing Tailings

LOGGED BY: JP CHECKED BY:

SHEET 1 OF 2 HOLE NO.: DH-01-11

TEST HOLE LOG

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	STARTED: 3/3/2001 FINISHED: 3/4/2001 DRILL METHOD: HS Augers, NQ Core GROUND ELEV. (ft): 79.0 COORDINATES (ft): N 53100.09 E 39163.79 DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS	Su - ksf: 1 2 3 4 VANE PEAK: FIELD ◆ LAB ■; REMOLD: FIELD ◇ LAB □; ▲ UC/2, △ P.PEN/2 ★ % FINES ● SPT N W_p% W% W_L% 20 40 60 80
90					Monitoring Well Installation Notes: 1) Schedule 40 PVC screen, 10 slot size, 2 inch diameter, from 57.5 ft to 52.5 ft. 2) 20/40 silica sand from 57.5 ft to 48.0 ft. 3) 2 inch diameter solid PVC pipe, schedule 40 from 52.5 ft to surface. 4) Grout from 48.0 ft to surface.		
100							
110							
120							
130							
140							
150							
160							

KLOHN CRIPPEN

PROJECT NO.: PM7802 20

PROJECT: 2001 Geotechnical Drill Program

LOCATION: Former DH-C, West of Existing Tailings

LOGGED BY: JP CHECKED BY:

SHEET 2 OF 2 HOLE NO.: DH-01-11

KC_TEST_HOLE-IMP WESTT KC_DATA.GDT 6/10/03

TEST HOLE LOG

STARTED: 6/17/2000 **FINISHED:** 6/17/2000

DRILL METHOD: Hollow Stem Auger

GROUND ELEV. (ft): 196.9

COORDINATES (ft): N 53566.92 E 40451.79

Su - ksf: 1 2 3 4

VANE: PEAK (FIELD ◆, LAB ■); REMOLD (FIELD ◇, LAB □); ▲UC/2; △P.PEN/2

★ % FINES ● SPT N

W_P% W% W_L%

× – – – – o – – – – ×

20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DEPTH / ELEV.	DESCRIPTION OF MATERIALS
					2.5 / 194.4	SAND (SP) Fine gradation, some medium to coarse gradation sand, peat at bottom (Fill).
	3,3,8		SPT-1			
	2,5,6		SPT-2		7.5 / 189.4	SAND (SP) Fine gradation sand, some medium to coarse, trace to some silt, gravel to 1.5 ", organic inclusions, subangular to subrounded, wet, brown.
	7,5,4		SPT-3			
10	4,8,5		SPT-4		12.5 / 184.4	SANDY GRAVEL (GWS) To gravelly sand, fine to coarse gradation sand, gravel to 1.5 ", trace silt, subangular to subrounded, compact, lightly cemented, wet, grey to orange brown. Rapid dilatancy at 10 ft.
	8,21,27		SPT-5			
	23,26,33		SPT-6		17.5 / 179.4	SAND (SW) Fine to coarse gradation sand, trace to some gravel, trace to some silt, subrounded to subangular, dense, no to slow dilatancy, wet, grey. Silt seam at 15.25 ft, some sand, medium plasticity.
	30,16,25		SPT-7			
20	14,21,37		SPT-8		24.0 / 172.9	SILT (MI) Trace fine gradation sand, clay inclusions throughout, medium plasticity, very stiff, grey. Increasing gravel content near 20 ft.
	14,25,27		SPT-9		26.5 / 170.4	GRAVELLY SAND (SPG) To sandy gravel (fine to medium gradation sand), trace silt, angular to subrounded, dense to very dense, low to medium dilatancy, wet, grey. End of Hole at 26.5 ft
30						
40						
50						
60						
70						
80						

INSTRUMENT DETAILS

Drilling Notes:

1) Drill hole terminated at 26.5 ft with grinding refusal to augers.
2) Drill hole sealed to surface with medium bentonite chips upon completion.
3) Rig chatter at 7.5 ft, 10 ft and 24 ft.

KLOHN CRIPPEN

PROJECT NO.: PM7802 17

PROJECT: Waste Site 23 and E Geotechnical Drilling

LOCATION: East Tailings

LOGGED BY: JP **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** DH-00-04

TEST HOLE LOG

STARTED: 6/17/2000 **FINISHED:** 6/17/2000

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 207.7

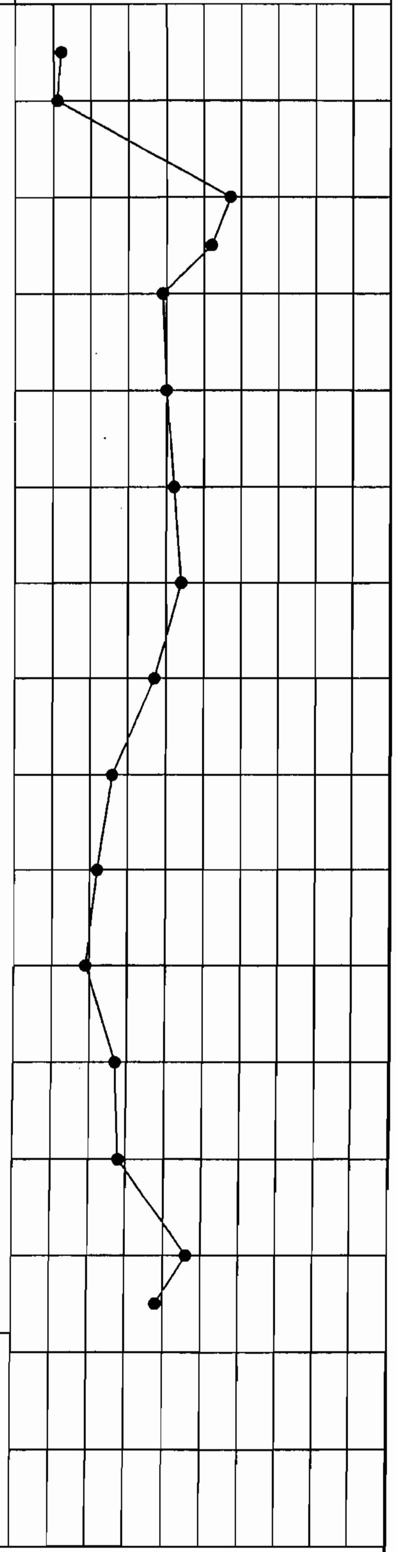
COORDINATES (ft): N 53290.26 E 40557.51

Su - ksf: 1, 2, 3, 4
VANE PEAK: FIELD ◆ LAB ■ ▲ UC/2
REMOLD: FIELD ◇ LAB □ △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.
	5,6,6		SPT-1
	7,8,3		SPT-2
	28,45,50/3"		SPT-3
10	14,18,39		SPT-4
	20,27,25		SPT-5
	13,17,22		SPT-6
20	10,18,22		SPT-7
	14,15,27		SPT-8
30	17,20,24		SPT-9
	8,12,25		SPT-10
40	9,12,14		SPT-11
	7,10,12		SPT-12
50	8,9,10		SPT-13
	25,12,15		SPT-14
60	16,11,17		SPT-15
	13,16,30		SPT-16
	29,13,25		SPT-17
70			
80			

Depth / Elev. (ft)	DESCRIPTION OF MATERIALS
0.0 / 207.7	SAND (SP) Fine to medium gradation sand, trace to some silt, trace coarse sand, gravel, subangular to subrounded, compact, wet, grey overlying thin layer of peat (Fill).
2.5 / 205.2	SAND (SP) Fine to medium gradation sand, trace silt, trace coarse gradation sand, subangular to subrounded, compact, wet, grey. Very dense, moist at 7.5 ft.
10.0 / 197.7	SILT (MI) Occasional clay inclusion, pyrite crystals, medium plasticity, firm to stiff, dry, grey with occasional fine sand seams (1/10 "), trace medium sand, dry, grey-brown. Sand seams more closely spaced, damp, brown at 12.5 ft. At 15 ft, sand seam thickness increasing, changing to fine sand, some silt.
17.5 / 190.2	GRAVELLY SAND (SPG) Fine to medium gradation sand, trace silt, subrounded to subangular, compact to dense, wet, brown with oxidized seams. Rapid dilatancy, with 1 " silty sand seam, medium plasticity, brown at 20 ft. Organic seam at 26.5 ft, Peat (PT), amorphous, brown. Decreasing gravel content, rapid dilatancy, wet, brown at 30 ft.
35.7 / 172.0	SANDY SILT (MLS) Clay inclusions, trace coarse gradation sand to fine gravel, subangular, wet, grey.
40.0 / 167.7	SILTY SAND (SM) Fine gradation sand, trace to some medium to coarse gradation sand, gravel, angular to subrounded, pyrite crystals throughout, compact, medium plasticity, wet, grey (Glacial Till).
50.2 / 157.5	SAND (SW) Fine to medium gradation sand, trace to some coarse gradation sand, fine gravel, subrounded to subangular, compact, medium dilatancy, wet.
55.0 / 152.7	GRAVELLY SAND TO SAND AND GRAVEL (SG) Fine to coarse gradation sand, trace silt, angular to subrounded, compact, medium dilatancy, wet, grey.
60.0 / 147.7	SAND (SP) Fine gradation sand, some silt, trace medium to coarse gradation sand, fine gravel, subangular to subrounded, compact to dense, dry, grey. Pyrite crystals, shell fragments, dense, rapid dilatancy at 65 ft. Siltier near base at 67 ft.
69.0 / 138.7	End of Hole at 69.0 ft

INSTRUMENT DETAILS



Drilling Notes:

1) Drill hole terminated at a total depth of 69 ft due to grinding refusal of augers.
2) Hole grouted to surface upon completion.

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: PM7802 17
PROJECT: Waste Site 23 and E Geotechnical Drilling
LOCATION: East Tailings
LOGGED BY: JP **CHECKED BY:**
SHEET 1 OF 2 **HOLE NO.:** DH-00-05

TEST HOLE LOG

STARTED: 6/17/2000	FINISHED: 6/17/2000
DRILL METHOD: Mud Rotary	
GROUND ELEV. (ft): 207.7	
COORDINATES (ft): N 53290.26	E 40557.51

Su - ksf: 1 2 3 4

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

★ % FINES ● SPT N

W_P% W% W_L%

× – – – – ○ – – – – ×

20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					3) Rig chatter at 10 ft, 17.5 ft, 32 ft, 37 ft, 43 ft, 47 ft, and 69 ft. 4) Heaving sands encountered at 35 ft.	
90						
100						
110						
120						
130						
140						
150						
160						

KLOHN CRIPPEN

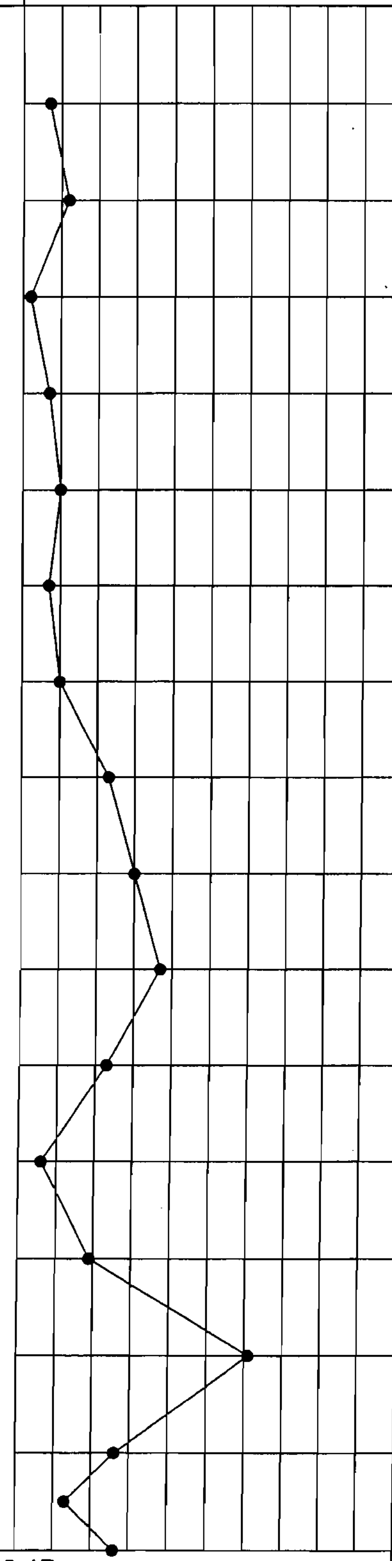
PROJECT NO.: PM7802 17	
PROJECT: Waste Site 23 and E Geotechnical Drilling	
LOCATION: East Tailings	
LOGGED BY: JP	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-00-05

TEST HOLE LOG

STARTED: 6/18/2000	FINISHED: 6/18/2000
DRILL METHOD: Mud Rotary	
GROUND ELEV. (ft): 210.0	
COORDINATES (ft): N 52690.79	E 40110.22

Su - ksf: 1, 2, 3, 4
VANE PEAK: FIELD ◆, LAB ■; REMOLD: FIELD ◇, LAB □; ▲ UC/2; △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%: 20, 40, 60, 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					SILT (ML) Trace to some fine gradation sand, trace clay throughout, loose to compact, low plasticity to non-plastic, pyrite crystals throughout, dry, grey-black (tailings fill).	
	6,4,3		SPT-1			
10	11,6,6		SPT-2			
	1,1,1		SPT-3		Very loose at 15 ft.	
20	5,4,3		SPT-4		Soft consistency at 20 ft.	
	2,3,7		SPT-5			
30	5,4,3		SPT-6			
	2,4,6		SPT-7			
40	10,12,11		SPT-8		Gravel fragments at 40 ft.	
	13,14,16		SPT-9		Medium gradation sand to gravel fragments at 45 ft.	
50	14,15,22		SPT-10			
	10,9,14		SPT-11			
60	3,2,4		SPT-12		Slow dilatancy at 60 ft.	
	8,8,11		SPT-13		Trace fine gravel at 65 ft.	
70	8,12,49		SPT-14		71.0 / 139.0 SILTY SAND (SM) Fine gradation sand, trace medium to coarse gradation sand, gravel, angular, very dense, grey (waste rock fill).	
	5,10,16		SPT-15		75.3 / [illegible] / 134.2 PEAT (PT) Amorphous, woody, organic odour, wet, brown, compressed.	
	7,6,7		SPT-16			
80						



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KLOHN CRIPPEN

PROJECT NO.: PM7802 17	
PROJECT: Waste Site 23 and E Geotechnical Drilling	
LOCATION: South Expansion	
LOGGED BY: JP	CHECKED BY:
SHEET 1 OF 2	HOLE NO.: DH-00-06

TEST HOLE LOG

STARTED: 6/18/2000	FINISHED: 6/18/2000
DRILL METHOD: Mud Rotary	
GROUND ELEV. (ft): 210.0	
COORDINATES (ft): N 52690.79	E 40110.22

Su - ksf: 1, 2, 3, 4

VANE PEAK: FIELD ◆ LAB ■ ▲ UC/2
REMOLD: ◇ □ △ P.PEN/2
★ % FINES ● SPT N
W_P% W% W_L%
× – – – – o – – – – ×
20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	ELEV./DEPTH	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	9,11,15		SPT-17			SILTY SAND (SM) Fine to medium gradation sand, to some silt, trace to some coarse gradation sand, fine gravel, angular to subangular, compact, slow to medium dilatancy, wet, brown with oxidized zones, lightly cemented, shell fragments throughout.	
	3,4,9		SPT-18				
	6,13,21		SPT-19		85.0 / 125.0	SILTY CLAY (CL-ML) Trace fine to coarse gradation sand, fine gravel, subrounded to subangular, medium plasticity, dense, stiff consistency, grey, shell fragments throughout, occasional fine sand seam.	
90	10,16,23		SPT-20				
	8,11,16		SPT-21		96.5 / 113.5	End of Hole at 96.5 ft	
100							
110							
120							
130							
140							
150							
160							

Drilling Notes:

1) Drill hole terminated at a total depth of 96.5 ft.
2) Hole grouted to surface upon completion.
3) Hollow stem augers to 10 ft, then switch to mud rotary.
4) Drill change at 69 ft, 78 ft, and 87.5 ft.
5) Hole observed to be making water at 70 ft.

KLOHN CRIPPEN

PROJECT NO.: PM7802 17	
PROJECT: Waste Site 23 and E Geotechnical Drilling	
LOCATION: South Expansion	
LOGGED BY: JP	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-00-06

TEST HOLE LOG

STARTED: 6/24/2000 **FINISHED:** 6/24/2000

DRILL METHOD: Mud Rotary

GROUND ELEV. (ft): 213.4

COORDINATES (ft): N 52789.47 E 39989.68

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					SILT (ML) Trace fine gradation sand, clay inclusions, non-plastic to low plasticity, very loose to compact, dry to wet at 14 ft, grey-brown with metallic sheen, pyrite crystals, (tailings fill).	
	6,2,1		SPT-1			
10	3,3,6		SPT-2			
	WT/18"		SPT-3		At 14 ft, clay seam, medium to high plasticity, soft consistency.	
20	47,50/5'		SPT-4		19.0 / 194.4 SILTY SAND (SM) Gravel to 1.5 ", very dense, wet, grey, mixed with tailings (waste rock fill).	
	3,4,3		SPT-5		24.0 / 189.4 SILT (ML) Trace fine gradation sand, non plastic, very loose to compact, dry, grey-brown (tailings fill).	
30	7,15,21		SPT-6			
	1,2,1		SPT-7			
40	16,13,10		SPT-8			
	4,7,5		SPT-9			
50	17,23,28		SPT-10			
	23,26,29		SPT-11			
60	8,6,14		SPT-12			
	5,23,29		SPT-13			
70	13,22,25		SPT-14			
	14,12,14		SPT-15		74.0 / 139.4 SANDY SILT (MLS) Fine to coarse gradation sand, gravel to 1.5 ", organic inclusions, angular, compact to dense, damp to wet, greenish white, chloritic fragments, talcy (waste rock	
80						

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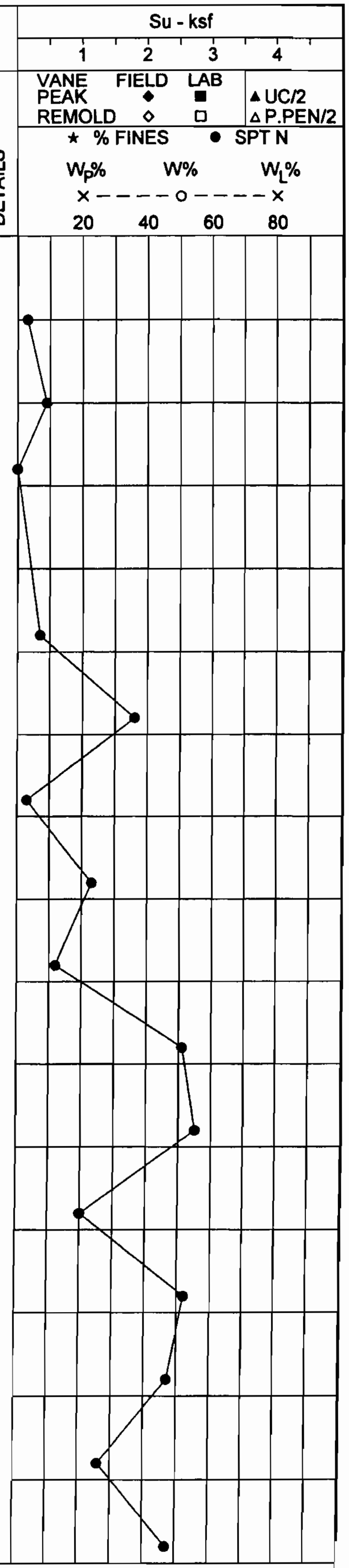


KLOHN CRIPPEN

PROJECT NO.: PM7802 17

PROJECT: Waste Site 23 and E Geotechnical Drilling

LOCATION: South Expansion

LOGGED BY: JP **CHECKED BY:**

SHEET 1 OF 2 **HOLE NO.:** DH-00-11

TEST HOLE LOG

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	STARTED: 6/24/2000 FINISHED: 6/24/2000 DRILL METHOD: Mud Rotary GROUND ELEV. (ft): 213.4 COORDINATES (ft): N 52789.47 E 39989.68 DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS	Su - ksf (1, 2, 3, 4) VANE PEAK / REMOLD: FIELD ◆ / ◇; LAB ■ / □; ▲ UC/2; △ P.PEN/2 ★ % FINES ● SPT N W_P% W% W_L% 20 40 60 80
	10,10,37		SPT-16		132.9 fill). End of Hole at 80.5 ft		
90							
100							
110							
120							
130							
140							
150							
160							

Drilling Notes:

1) Drill hole terminated a total depth of 80.5 ft in waste rock fill.
2) Hole grouted to surface upon completion.
3) Hollow stem augers to 10 ft, then switch to mud rotary.
4) Rig chatter at 21 ft, 24 ft, 34.5 ft, 37 ft, 74 ft, 76 ft, and 77.5 ft.
5) Lost circulation at 74 ft.

KLOHN CRIPPEN

PROJECT NO.: PM7802 17	
PROJECT: Waste Site 23 and E Geotechnical Drilling	
LOCATION: South Expansion	
LOGGED BY: JP	CHECKED BY:
SHEET 2 OF 2	HOLE NO.: DH-00-11

KC_TEST_HOLE-IMP KGCM.G DATA.GDT 8/10/03

TEST HOLE LOG

STARTED: 6/25/2000 **FINISHED:** 6/25/2000

DRILL METHOD: Hollow Stem Auger

GROUND ELEV. (ft):

COORDINATES (ft):

Su - ksf: 1 2 3 4

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

★ % FINES ● SPT N

W_P% W% W_L%: 20 40 60 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0 – 15.0					SILT (ML) Trace fine gradation sand, very loose to compact, low plasticity, dry, grey-brown with metallic sheen, pyrite crystals (tailings).	
	3,1,1		SPT-1			
	6,3,4		SPT-2			
	4,3,4		SPT-3			
10	3,3,2		SPT-4			
	7,8,6		SPT-5			
15.0 – 20.0	8,4,10		SPT-6		SAND AND SILT (SW-SM) To sand, some silt, some gravel size particles to 1.5 ", dry to wet, green-brown, chloritic rock (waste rock fill).	
	50/4"		SPT-7			
20.0 – 21.5	,1/10",		SPT-8		PEAT (PT) Amorphous, woody, wet, brown.	
21.5					End of Hole at 21.5 ft	

Depth scale: 10, 20, 30, 40, 50, 60, 70, ,0

Drilling Notes:

1) Drill hole terminated at a total depth of 21.5 ft in peat unit.
2) Drill hole backfilled with medium bentonite chips upon completion.

KLOHN CRIPPEN

PROJECT NO.: PM7802 17

PROJECT: Waste Site 23 and E Geotechnical Drilling

LOCATION: West Buttress

LOGGED BY: JP **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** DH-00-12

TEST HOLE LOG

STARTED: 6/25/2000 FINISHED: 6/25/2000

DRILL METHOD: Hollow Stem Auger

GROUND ELEV. (ft):

COORDINATES (ft):

Su - ksf: 1, 2, 3, 4

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

★ % FINES ● SPT N

W_P% W% W_L%: 20, 40, 60, 80

DEPTH (feet)	SPT BLOWS PER 6"	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
	4,3,3		SPT-1		SILT (ML) Trace to some fine gradation sand, loose to compact, dry to wet (17.5 ft), grey-black with metallic sheen, pyrite crystals (tailings).	
	2,2,3		SPT-2			
	5,4,4		SPT-3			
10	2,2,2		SPT-4			
	2,6,9		SPT-5			
	5,8,7		SPT-6			
18.5	2,1,9		SPT-7		SAND (SW) Fine to coarse gradation sand, some gravel, trace silt, compact, dry, brown (fill).	
19.0					End of Hole at 19.0 ft	

Drilling Notes:

1) Drill hole terminated at a total depth of 19 ft in sand and gravel unit.
2) Drill hole grouted to surface upon completion.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80

DATA.GDT 6/10/03

KC_TEST_HOLE-IMP KGCM.G

KLOHN CRIPPEN

PROJECT NO.: PM7802 17

PROJECT: Waste Site 23 and E Geotechnical Drilling

LOCATION: West Buttress

LOGGED BY: JP CHECKED BY:

SHEET 1 OF 1 HOLE NO.: DH-00-13

TEST HOLE LOG

STARTED: 10/20/97 FINISHED: 10/20/97

DRILLING METHOD: Hollow Stem Auger

GROUND ELEV(ft): 149.6

COORDINATES(ft): N 53,111 E 39,653

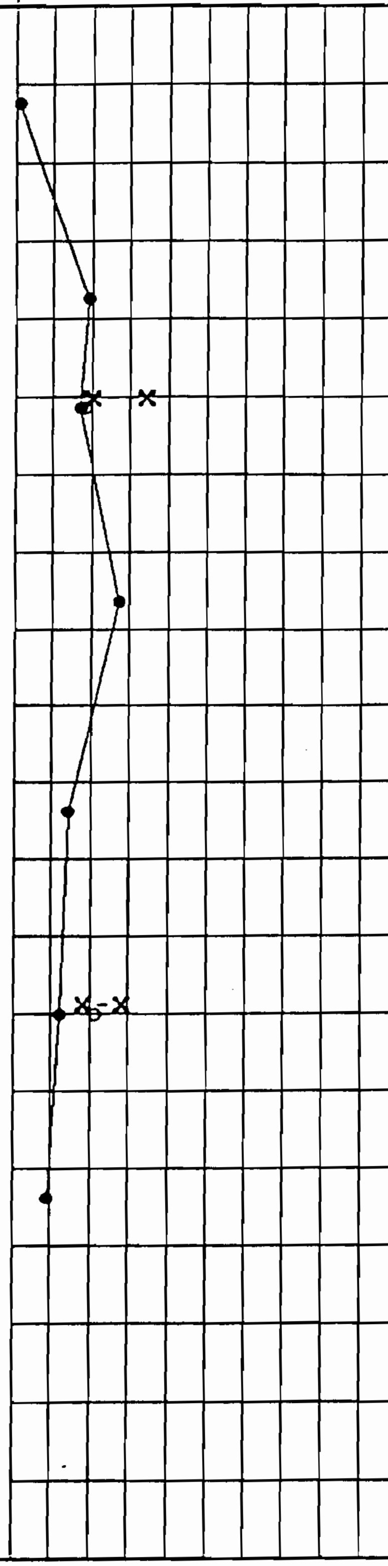
DEPTH (ft)	SPT BLOWS PER 6 In	OTHER TESTS	SAMPLE TYPE AND NUMBER	DESCRIPTION OF MATERIALS	PIEZOMETER DETAILS
0–3.0	0, 1, 0, 1		SS 1	Peat(PT) - dark brown, moist, woody to coarse fibrous, trace coarse sand, trace gravel, moderate organic odour, with roots and woody fragments up to 1.5" length, appears disturbed [possible FILL]. Layer of Moss (original ground surface ?) at 3 ft. depth.	
3.0–7.0				PEAT (PT) Dark brown to black, moist to wet, soft, brown, fine fibrous to amorphous, with thin (less than 0.5") layer of non-plastic silt (Marl Deposit) moderate methane odour.	
7.0–10.0	3, 9, 10, 8		SS 2	SAND and GRAVEL(SM-GM), silty, compact, dark grey-brown, wet, slightly organic	
10.0	5, 6, 11, 19		SS 3	SILT-CLAY [CL-ML], trace fine gravel, low plasticity, very stiff to stiff, light grey, moist, homogeneous.	
15	7, 10, 17, 22		SS 4		
20	3, 6, 8, 10		SS 5	Firm to Stiff below 20 feet depth.	
26	4, 6, 6, 7		SS 6		
30	3, 4, 5, 10		SS 7		
40				...Cont'd	

Su - ksf: 1 2 3 4

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

W_P % W % W_L %

10 30 50 70 90



FORM: IM01 93/01/26

PROJECT NO.: BH97-1

PROJECT: KGCM - Tailings Investigation

LOCATION: Admiralty Island, Alaska

LOGGED BY: Lisa Brown CHECKED BY:

SHEET 1 OF 2 HOLE NO.: BH97-1

TEST HOLE LOG

STARTED:10/20/97 FINISHED:10/20/97

DRILLING METHOD: Hollow Stem Auger

GROUND ELEV(ft): 149.6

COORDINATES(ft): N 53,111 E 39,653

Su - ksf: 1 2 3 4

VANE: FIELD, LAB — PEAK ◆ ■, REMOLD ◇ □; ▲ UC/2; △ P.PEN/2; ● SPT N

W_P % W% W_L % — 10 30 50 70 90

DEPTH(ft)	SPT BLOWS PER 6 In	OTHER TESTS	SAMPLE TYPE AND NUMBER	SYMBOL	DESCRIPTION OF MATERIALS	PIEZOMETER DETAILS
	4 6 7 13		SS 8		41.7	
42					END OF HOLE at 41.7 ft.	
44						
46						
48						
50						
52						
54						
56						
58						
60						
62						
64						
66						
68						
70						
72						
74						
76						
78						
80						

PROJECT NO.: BH97-1

PROJECT: KGCM - Tailings Investigation

LOCATION: Admiralty Island, Alaska

LOGGED BY: Lisa Brown CHECKED BY:

SHEET 2 OF 2 HOLE NO.: BH97-1

FORM: IM01 93/01/26

TEST HOLE LOG

STARTED: 10/23/97 FINISHED: 10/24/97

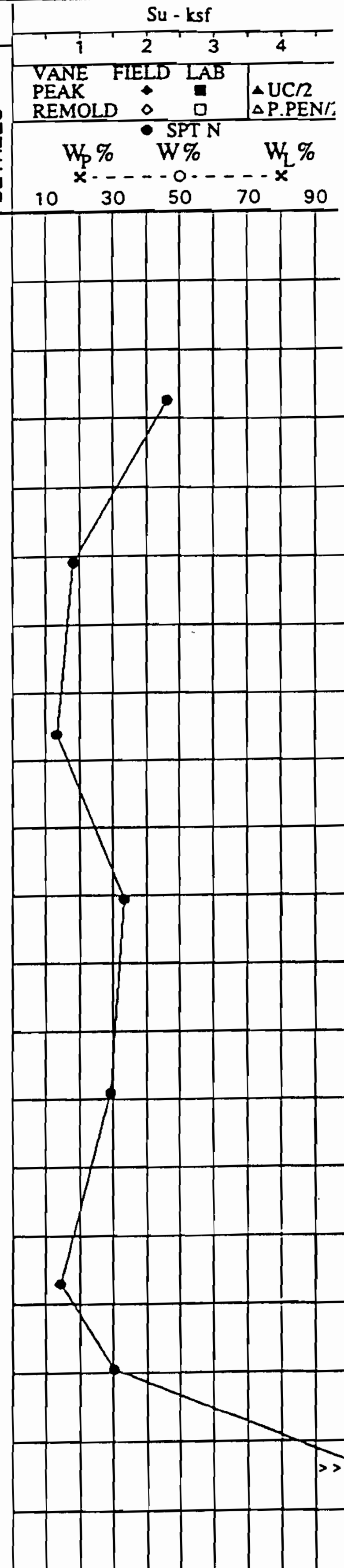
DRILLING METHOD: Mud Rotary w/ circ.

GROUND ELEV(ft): 165.2

COORDINATES(ft): N 52,786 E 39,940

DEPTH (ft)	SPT BLOWS PER 6 in	OTHER TESTS	SAMPLE TYPE AND NUMBER	DESCRIPTION OF MATERIALS
0–5				Compact to dense, dry to moist, dark grey pyritic tailings. Sample breaks in 0.2-inch "books".
5–7	18, 21, 25, 26	GSD	SPT1	Trace gravel and geomembrane encountered between 5 ft. and 10 ft.
9.5–11.5	7, 7, 11, 16	GSD	SPT2	Compact, moist, dark grey-black pyritic tailings. Structureless.
14.5–16.5	1, 3, 10, 17	GSD	SPT3	Becoming moist wet below 14 ft.
19.5–21.5	10, 15, 18, 22	GSD	SPT4	Minor gravel encountered at 19 ft.
24.9				
25–27	17, 17, 12, 11		SPT5	SAND and GRAVEL [GM-SM], silty, some gravel, compact, sub-angular, grey, dry to moist [possible fill]
30.0				PEAT [PT], dark brown, wet, fibrous, firm.
30.5–32.5	4, 6, 8, 8		SPT6	
33–35	4, 15, 15, 26		SPT7	
34.2				SAND [SW-SM], coarse, trace fine gravel, dense, grey, wet, with occasional rootlets.
36–38	20, 34, 71, 133		SPT8	
37.0				CLAY [CL], silty, low plasticity, very stiff to hard, light grey, moist, homogeneous.
38.3				No sample recovered (SPT-9). Inferred very dense sand and gravel layer. ...Cont'd

PIEZOMETER DETAILS



PROJECT NO.:	PM7802-03-03
PROJECT:	KGCM - Tailings Investigation
LOCATION:	Admiralty Island, Alaska
LOGGED BY:	Lisa Brown
CHECKED BY:	
SHEET 1 OF 2	HOLE NO.: BH97-2

FORM: IM01 93/01/26

TEST HOLE LOG

STARTED: 10/23/97	FINISHED: 10/24/97
DRILLING METHOD:	Mud Rotary w/ circ.
GROUND ELEV(ft):	165.2
COORDINATES(ft):	N 52,786 E 39,940

Su - ksf: 1 2 3 4

VANE FIELD LAB — PEAK ◆ ■ — REMOLD ◇ □ — ▲UC/2 — △P.PEN/
● SPT N
W_P % W% W_L %
10 30 50 70 90

DEPTH(ft)	SPT BLOWS PER 6 In	OTHER TESTS	SAMPLE TYPE AND NUMBER	SYMBOL	DESCRIPTION OF MATERIALS	PIEZOMETER DETAILS
40	218/6"		SPT9			
45					Losing drilling mud at 45 ft.	
47.2					Artesian water pressure observed when rods pulled out of hole at completion. Base heave of approx. 2 ft.	
					END OF HOLE 47.2 ft.	

Depth scale: 42, 44, 46, 48, 50, 52, 54, 56, 58, 60, 62, 64, 66, 68, 70, 72, 74, 76, 78, 80

FORM: IM01 93/01/26

PROJECT NO.:	PM7802-03-03
PROJECT:	KGCM - Tailings Investigation
LOCATION:	Admiralty Island, Alaska
LOGGED BY:	Lisa Brown CHECKED BY:
SHEET 2 OF 2	HOLE NO.: BH97-2

TEST HOLE LOG

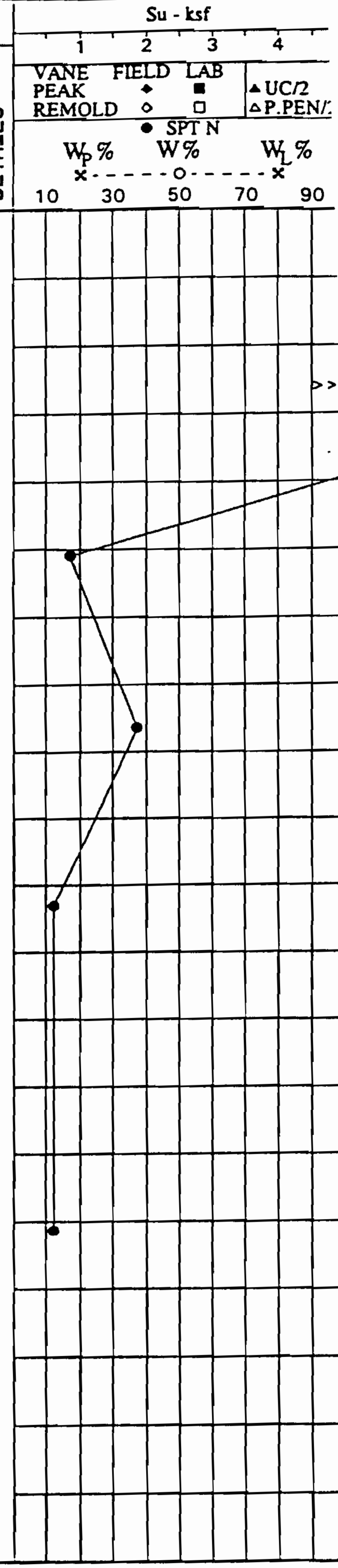
STARTED: 10/24/97 FINISHED: 10/25/97

DRILLING METHOD: Mud Rotary w/ circ.

GROUND ELEV(ft): 204.9

COORDINATES(ft): N 53,204 E 39,986

DEPTH(ft)	SPT BLOWS PER 6 In	OTHER TESTS	SAMPLE TYPE AND NUMBER	DESCRIPTION OF MATERIALS
0				0-3 ft. Dark grey, dry, 3" minus waste rock at surface.
3				3-5 ft. Dry, light grey silt-sized tailings and gravel.
4–6	24 68 127 67		SPT1	Dense, dry to moist, dark grey fine grained pyritic tailings, trace gravel. Layering in 0.2-inch books.
9–11	5 6 11 13		SPT2	Compact, moist, dark grey-black tailings, trace gravel below 9 ft.
14–16	14 18 19 20		SPT3	Moist to wet below 14 ft.
20–22	5 4 8 10		SPT4	Loose to compact below 20 ft.
29–31	5 5 7 10		SPT5	
40				...Cont'd



PIEZOMETER DETAILS

PROJECT NO.:	PM7802-03-03
PROJECT:	KGCM - Tailings Investigation
LOCATION:	Admiralty Island, Alaska
LOGGED BY:	Lisa Brown CHECKED BY:
SHEET 1 OF 2	HOLE NO.: BH97-3

FORM: IM01 93/01/26

TEST HOLE LOG

STARTED: 10/24/97 FINISHED: 10/25/97

DRILLING METHOD: Mud Rotary w/ circ.

GROUND ELEV(ft): 204.9

COORDINATES(ft): N 53,204 E 39,986

DEPTH (ft)	SPT BLOWS PER 6 In	OTHER TESTS	SAMPLE TYPE AND NUMBER	DESCRIPTION OF MATERIALS
40–42	5, 5, 9, 12		SPT6	40-45 ft. Lost mud during drilling.
48				Driller notes: Drilling more difficult below 48 ft.
50–52	12, 10, 8, 13		SPT7	
58.0				PEAT[PT], dark brown, moist to wet, fibrous.
60–62	7, 10, 23, 89		SPT8	
61.2				SAND AND GRAVEL[SG-SM], silty, fine sand sizes, dense, dark grey, moist.
64.3	116, 312		SPT9	SILT[ML], low plasticity, hard, grey, with occasional fine sand laminations.
65.3				END OF HOLE 65.25 ft.

PIEZOMETER DETAILS

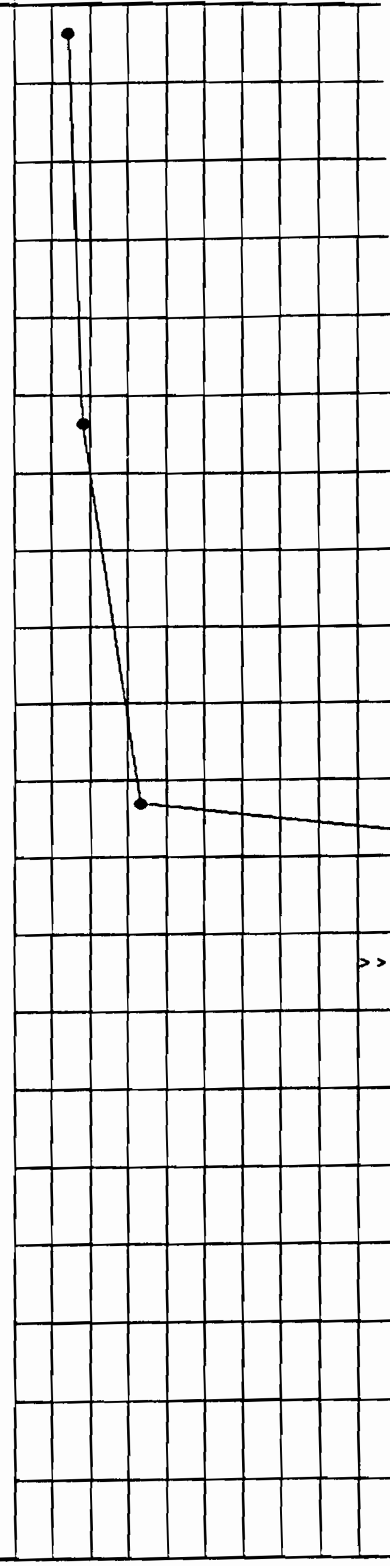
Su - ksf: 1 2 3 4

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

● SPT N

W_P % — W% — W_L %

10 30 50 70 90



FORM: IM01 93/01/26

PROJECT NO.:	PM7802-03-03
PROJECT:	KGCM - Tailings Investigation
LOCATION:	Admiralty Island, Alaska
LOGGED BY:	Lisa Brown CHECKED BY:
SHEET 2 OF 2	HOLE NO.: BH97-3

88

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TA-1
DATE: 8/9/94

DRILLER: Wink International Geotech. Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORHTING	EASTING	ELEVATION
53061.7781	40113.6550	180.8

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
0							cap rock	
1								
2								
3			8				tailings	Very fine-grained, dark grey-brn, pyritic tailings. Mod.-vig. fizz.
		1.9	8					Breaks when sqeezed. Damp, not wet. Only a few pieces of
4			6					capping gravel (siliceous rock and sericitic chlorite).
			6	GC94TA1				
5				3-5				
				6.5-8.5				
6				bags A&B				
			8	2201651				Five inches of tailings above 16" of gravel. Mostly msp and pyritic
7		1.7	20				gravel, msp	cr. Minor mfp and mfb. Tails breaks when squeezed. Fairly dry.
			25				cr, mfp, mfb	Vig. fizz. Minor fizz on gravel.
8			18					
9								
			12					Same tailings as above. Vig. fizz. Clast of msr at 9.5'. Tails
10		1.9	8	GC94TA1			tails w/ msr	breaks when squeezed.
			5	9-11				
11			6	11.5-13.5				
				bags A&B				
12			3	2201652				All tailings. Mod fizz. Moisture content increases significantly by
		1.7	2				tailings	the bottom of the sample interval. Top is dry and breaks when
13			2					squeezed. Bottom is very soft and beads when squeezed.
			2					
14								
			1					Tailings. Mod fizz. Beads-breaks when squeezed. About as
15		2.2	1	GC94TA1				moist as above (cuts easily). One clast of hard impermeable
			2	14-16			tails w/ dry	tailings (<2") in diameter. Mod fizz. Odd that it didn't hydrate like
16			2	16.5-18.5			clast	the surrounding material.
				bags A&B				
17			1	2201653				Tailings. Same as above. Soft, beads when squeezed.
		1.9	1					
18			2					
			3					
19							tailings	
			1					Same as above.
20		2.0	2					
								Continued on next page.

HOLE: TA1 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20			2	GC94TA1				
			3	19-21				
21				21.5-23.5				
			1	bags A&B			tails w/ msp	Same as above plus a couple of pieces of <1" msp. Top is
22		1.9	4	2201654				more moist (beads) than the bottom (beads-breaks).
			5					
23			5					
24								
			2					Same very fine pyritic tailings but with rare clasts of sp. One is
		2.0	3	GC94TA1			tails w/ sp	about 2" and oxidized to a red-brn color in spots (slight fizz).
25			3	24-26				Tails has mod. fizz, is soft and moist (breaks when squeezed).
			13	26.5-28.5				
26				29-30				
			3	bags A&B	▽			
27		1.9	3	2201655			tails w/ sp	Tailings. Mod. fizz, dryer, breaks. One clast of sp (<1").
			4					
28			3					
29								
			2					One foot of moist tailings (breaks/beads) sits on 2mm felt. Below
30		1.7	4	GC94TA1			2mm felt	felt is about four inches of oxidized (FeOx and clay) pyritic sp
			6	30-31			gravel	gravel that sits atop another layer of felt. The felt sits on methane
31			5	one bag			2mm felt	smelling wet bark, organics and wood. Last 2" is decomposed
								roots and dirt. Separated out tailings and bagged it with the
32							organics	composite above.
33								
								All med grey, very dense clay with some silt and sand. Thinly
34			18	GC94TA1			clay with	layered (<1cm). Cuts hard. Rare quartz pebble.
			35	33.5-35			silt and sand	
35			30	one bag				Note: Mod. amount of water within the organics above the clay.
	EOH							Auger fills with about six feet of water during the last sampling.
36								Might not have reached equilibration hight by the end of sampling.

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TA-2
DATE: 8/8-9/94

DRILLER: Wink International Geotech. Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53211.8854	40115.8212	192.8

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
0								
							cap rock	
1							pyritic sr	
2								
			15					Mod. to vig. fizz in tails and gravel cap. Six inches of <1" pyritic
3		1.8	18				tailings	siliceous rock. Then dry (crumbly), very fine-grained, dk gry-brn,
			17				rare sr, sp	pyritic tailings. Breaks when cut. <2" gravel randomly spaced
			13	GC94TA2			rare arg	throughout hole (~5%). One larger clast of white siliceous rock
4				2-4				Also rare clasts of sp and arg. Low moisture content.
			3	4.5-6.5				
5		1.6	3	bags A&B			tailings	Vig. fizz. Almost all tailings. Some rare <1" pyritic sr gravel.
			3	2201657				More moisture. Upper portion breaks. Lower portion slices.
6			7					
7								
			1					All moist tailings. Mod. to vig. fizz. Not wet (slices easily).
8		1.7	3				tailings	Dk gry-brn very fine-grained. Breaks when squeezed.
			3					
			3	GC94TA2				
9				7-9				
			3	9.5-11.5			tailings	Same as above. Mod. to vig. fizz, soft, moist tailings.
10		1.9	3	bags A&B				Beads-breaks when squeezed.
			4	2201658				
11			3					
12								
			1					All moist, dk gry-brn, very fine, pyritic tailings. Mod. fizz. Beads
13		1.7	2				tailings	when squeezed. Moistest yet in hole.
			2					
			1	GC94TA2				
14				12-14				
			2	14.5-16.5				A bit less moisture (beads). Gets dryer towards the bottom.
15		2.0	2	bags A&B			tailings	Mod. fizz. Dk gry-brn.
16			3	2201659				
			4					
17								
			8				tailings	Piece of 2.5" pytitic sr clogged sampler. Only recover 6" of same
18		0.5	7				sr	tailings as above (beads). Gravel in sample is likely fragmented
			8					sr.
			6	GC94TA2				
19				17-17.5				
20			2	19.5-20.5				
				bags A&B				
				2201660				Continued on next page.

HOLE: TA2 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20		1.1	3				tailings	Clast of 2" mfb clogs sampler. Sample is about 80% tailings and
21			5				gravel	20% gravel consisting of mixed <1" msr, msp, mfp, mfb. Tailings
			4					beads when squeezed.
22								
			3				tailings	Only recovered 1' again. Sampler clogged by a clast of sp.
23		1.0	7				gravel	Sample also contains about 50% gravel consisting of mfb, mfp,
			14					and sp. The tailings is moist (beads) and has slight to mod fizz.
24			9	GC94TA2				
				22-24				
25			3	24.5-26.5			tailings	Mod. fizz. Moist (beads), very fine, pyritic tailings. Rare clasts of
		1.7	3	bags A&B			rare pysr	pyritic sr.
26			3	2201661				
			4					
27								
			1					Same dk gry-brn, very fine, pyritic tailings. Moist (beads-breaks).
28		1.9	3				tailings	Slight to mod. fizz. No gravel.
			2					
29			4	GC94TA2				
				27-29				Stopped drilling at 29.5' 6:30pm 8/8/94.
30		8/9	3	29.5-31.5				Same as above. 6.5 feet of water accumulated in the well over-
		1.5	3	bags A&B			tailings	night. Probably from gravel layer at 22'. Ponded water makes top
31			5	2201682				of sample runny. The rest is moist (beads-breaks).
			6					
32								
			2					Same as above. Mod. fizz. Dryer (breaks). Less ponded water.
33		1.9	5				tailings	
			5					
34			6	GC94TA2				
				32-34				
35			3	34.5-36.5				Same as above. Mod. fizz. Moist (beads-breaks). Includes a
		2.2	3	bags A&B			qz-py sand	4" layer of med-fine angluar quartz and coarse pyrite sand.
36			3	2201663				Likely storm wash. Less fizz in sand. Clayey (finer) below.
			6					
37								
			1	GC94TA2			clayey tails	1.5 feet of very fine, clayey, pyritic tailings (finer than most tails).
38		2.2	2	37-39			sticks, org.	Moist (beads). Then 4" of tailings-sticks-muskeg mix above 1.5"
			4	bags A&B			organics	of anoxic smelling (more like sulfur or fuel than methane) muskeg.
39			8	2201664			sand-gravel	Muskeg is dark brown, decomposed and compacted.
								Then 2" of med tan-brn clayey sand above 2" of gravely sand.
40								Clasts of weathered biotite granite (gneiss?) and oxidized
								clastic rock (sandstone or tuff).
41								
42		1.2	18	GC94TA2			clay w/sand	Very compacted, dense clay with some sand layers. Clay is
			44	41.5-42.5			clayey sand	med. gry w/ brn patches. Rare rock clasts (<1" qz hornblende
43	EOH		refusal	one bag				granodiorite?) Sand (in <3mm layers) imparts brown coloring.
								Rare black organics.

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TA-3
DATE: 8/13-14/94

DRILLER: Wink International Geotech. Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORHTING	EASTING	ELEVATION
53361.2982	40117.2040	201.0

Legend
gravel; tailings; organics; clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

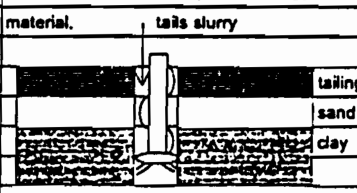
depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
0								
							gravel	
1							cap rock	
2								
			8					Mix of gravel and tailings. Dk gry-brn. Damp (breaks when sqeezed)
3		1.7	8				tailings w/	Not really crumbly (soft). Vig. fizz. Gravel is <3" pysp and siliceous
			5	GC94TA3			pysp and sr	rock (sr). Tailings is very fine-grained and pyritic.
4			5	2.5-4.5				
				5-7				
5			20	bags A&B			tailings w/	Dk gry-brn, very fine-grained, pyritic tailings with gravel. Drilled
		1.5	9	2201665			gravel	through a boulder of very pyritic sr. Then into more damp (breaks)
6			8					tailings.
			9					
7								
			3				tailings	Soft, moist (beads), brn-gry, homog. tailings.
8		1.6	2					
			4	GC94TA3				
9			3	7.5-9.5				
				10-12				
10			2	bags A&B			tailings	Moist (beads), brn-gry tailings with <5% <1" gravel (silic. slaty arg
		1.7	3	2201666			ssa	with <1mm euhed py).
11			4					
			4					
12								
			1					Very moist (beads), dk brn-gry tailings. Vig. fizz. Then 2" of
13		1.8	2				tailings	graphitic sr above 2" of dryer (breaks) tails.
			2	GC94TA3				
14			3	12.5-14.5				
				15-17				
15			6	bags A&B				Damp, dryer (crumbly), homog , less brown tailings. Vig. fizz.
		2.0	8	2201667			tailings	
16			9					
			10					
17								
			2					Moister (beads), homog. tails. Vig. fizz.
18		2.0	2				tailings	
			3	GC94TA3				
19			3	17.5-19.5				
				20-22				
20				bags A&B				
				2201668				Continued on next page.

HOLE: TA3 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20			2					Moist (beads-breaks), homog., dk brn-gry tailings. Vig. fizz.
		2.0	3				tailings	
21			3					
			3					
22								
			1					Very moist (beads easily) with saturated film, very soft, homog.,
23		2.1	2				tialings	dk brn-gry tailing.
			2	GC94TA3				
24			3	22.5-24.5				
				25-27				
25			1	bags A&B				Very moist (beads) with slight film. Dryer towards bottom.
		2.1	2	2201669			tailings	Vig. to mod. fizz.
26			3					
			4					
27								
			2					Moist (beads), very fine-grained, homog., dk gry-brn tailings.
28		2.0	3				tailings	Vig. fizz.
			4	GC94TA3				
29			4	27.5-29.5				
				30-32	▽			
30			2	bags A&B				A bit dryer esp. near the bottom (breaks). Still moist however.
		1.7	3	2201670			tailings	Less brown.
31			3					
			4					Two feet of water pond in hole overnight.
32								
		8/14	2					First foot is very soupy. Then it firms up to moist (beads), dk brn-gry,
33		2.2	3				tailings	homog. tailings. Mod. fizz.
			5	GC94TA3				
34			6	32.5-34.5				
				35-37				
35			2	37.5-38.5				Moist (beads), soft, dk brn-gry, homog. tailings. Mod. fizz.
		2.0	4	bags A&B			tailings	
36			2	2201671				
			5					
37								
			3				tails w/ sp	Reach base of pile. 1.4 feet of dk gry tailings with one inclusion of
38		1.8	3				black tails	2" qz-sp which is relatively unoxidized. At the contact with the
			10				organics	organics there is a <1" black layer within the tailings. Tailings are
39			refusal				sticks	dryer (breaks). The organics are dk red-brn and smell like methane.
								Organics include sticks, limbs and <1mm black spherical seeds.
40								
								Continued on next page.

HOLE: TA3 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
40								
41			7	GC94TA3				1.5 feet of dk red-brn, sandy gravel with <2" rounded clasts of many
		1.8	13	40.5-42.5			gravel	compositions. No fizz. Smells like methane. Clasts of sp. tuff,
42			23	one bag				chloritic rock. Last four inches are very flakey, dk gry phyllite which
			28					is mod. silicic. No fizz. Gravel is very wet, almost saturated.
43				GC94TA3				
			8	43-44			sand	One foot of gravelly coarse sand <3/4". Very wet, saturated,
44		2.1	12	one bag				heterolithic and immature. Sand is angular and gravel is more
			20	GC94TA3			clay	rounded. Sand sits on one ft. of med. gry, very dense, sandy clay.
45			50	44-45				Material is layered (clay-sandy clay-clay). Varves? The sand is
				one bag				very fine and the clay contains black organics which look a lot like
46								grass. Sand content increases towards the bottom. Contact
								between grav. sand and clay is sharp.
47								
48								
			15					Laminated med. grey clay (photo) with many <.5mm layers of v. fine
49		2.0	30	GC94TA3			lam. clay	sand. Layering is 1-20 mm scale.
			40	48-50				
50			52	one bag				Well installation: Put down five feet of slotted PVC and 50 feet of
								PVC riser to a depth of 53 feet. Cap is 33 inches above ground.
51								
								Note: Some tailings slurry got to the bottom of the hole via the augers.
52								This will influence the well data unless flushing can remove the
								material.
53	EOH							
54								
55								



WELL LEGEND

BACKFILL ∇ WATER LEVEL
BENTONITE
.020 SILICA SAND
SLOTTED PVC
UNSLOTTED PVC

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TA4
DATE: 8/12-13/94

DRILLER: Wink International Geotech. Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53513.3366	40119.4758	206.8

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
								Well top is 5.0 feet above ground level.
0								
							cap rock	
1								
2								
								Note: Had meeting with USFS so John Morris sampled from 0-16 feet.
3								
			5				tailings	Very fine-grained, pyritic tailings with some gravel. Moist.
4		1.4	8				rare gravel	Dk gry-black color.
			5	GC94TA4				
5			4	3.5-5.5				
				6-8				
6			2	bags A&B				Very fine-grained, pyritic tailings. Less gravel more moisture.
		1.8	4	2201673			tailings	Dk brn-gry color.
7			3	2201832				
			3					
8								
			2					Same as above. More gravel.
9		2.1	3				tailings	
			3	GC94TA4			some gravel	
10			2	8.5-10.5				
				11-13				
11			2	bags A&B			tailings	Very fine-grained, pyritic tailings. Dryer than above.
		1.7	2	2201674			rare gravel	
12			2					
			4					
13								
			4				tailings	Very fine-grained, homog., pyritic tailings. Dryer than above.
14		1.6	5					No gravel.
			5	GC94TA4				
15			7	13.5-15.5				
				16-18				
16			3	bags A&B				Moist (beads-breaks), dk brn-gry, soft, homog. tailings. Vig. fizz.
		1.6	5	2201675			tailings	
17			3					
			3					
18								
			2				tailings	Moist (beads), dk brn-gry, very fine-grained, pyritic tailings. Very
19		1.7	3					soft. Mod. fizz.
			4					
20								

HOLE: TA4 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20			4	GC94TA4				
				18.5-20.5				
21			3	21-23			tailings	Moist (beads-breaks), dk gry-brn, very fine-grained, pyritic tailings.
		1.8	3	bags A&B				Mod. fizz.
22			3	2201676				
			4					
23								
			2				tailings	Same as above. Moist (beads-breaks). Bottom is slightly dryer.
24		2.0	3					Mod. fizz. Rare fine gravel.
			4	GC94TA4				
25			5	23.5-25.5				
				26-28	∇			
26			3	bags A&B			sat. tailings	First four inches are very moist (bead, wet film). A clasts of qz-
		1.7	6	2201677			qz-chl rock	chlorite rock sits above much dryer (breaks) tailings.
27			9				dryer tails.	
			9					
28								
			4					Moist (beads), dk gry-brn, pyritic tailings. Vig. fizz.
29		1.8	4				tailings	
			4	GC94TA4				
30			4	28.5-30.5				
				31-33				
31			2	bags A&B				Moist (beads-breaks), dk gry-brn, pyritic tailings. Mod. fizz.
		2.0	3	2201678			tailings	
32			4					
			4					
33								
			2					Moist (beads), very soft, dk brn-gry, relatively oxidized tailings.
34		2.1	2				tailings	Mod. fizz.
			3				oxidized	
35			2					
				GC94TA4				
36			3	33.5-35.5				Moist (bead-breaks), less brown, pyritic tailings. Mod. fizz.
		2.1	3	36-38			tailings	
37			4	38.5-40				
			4	bags A&B				
38				2201679				
			2				tails, grav.	Two inches of coarse, heterolithic sand above 8" of moist (beads).
39		1.5	4					dk gry-brn tails with some gravel atop very coarse, rounded, <1"
			11				gravel	river gravel. Includes qz; granite; biotite schist. One clast of silic.
40			refusal					graphitic, sericite schist. Last three inches are tails-gravel mix.
		8/13						Water rose to 26.2' overnight.
41			10				grav., tails	Six inches of gravel-tailings mix. The gravel is crushed rock not
		1.9	11				felt	river gravel as above. Piece of 2mm felt sits atop 1.5' of red-brn,
42			16				organics	compacted, moist (not soggy) peat.
			16				wood	
43	EOH							Install well: Driller said slotted PVC extended to 43' when actually
								it only went to 41'. That only leaves ~ 2' of screening within the gravel
								Slotted PVC from 40' to 35'. 40 feet of PVC riser from 35' to 5' above
								the surface.

WELL LEGEND
- BACKFILL
- ∇ WATER LEVEL
- BENTONITE
- .020 SILICA SAND
- SLOTTED PVC
- UNSLOTTED PVC

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TA-5
DATE: 8/5-6/94

DRILLER: Wink International Geotech. Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53662.9010	40124.0838	207.2

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
0							cap rock	Cap rock consisting of pyritic siliceous rock, sericitic phyllite,
							pysr, sp,	and argillite.
1							arg	
								Moist, dk grey, very fine-grained, pyritic tailings. Vig. fizz w/ 10% HCl
2								Several <1cm aggregates. Material is damp.
			13					Tailings is slightly lighter colored at 2.5-3.0 feet. More fizz. Only
3		1.8	14				tailings	slight fizz and more moisture (beads when squeezed) from
			15					3.0-4.5 feet. Almost saturated.
4			10	GC94TA5				
				2.5-4.5				
5				5.5-7.5				
			2	bags A&B				Much darker. Dk gry-black, very pyritic, very fine-grained tailings.
6		1.5	1	2201681			tailings	May contain some concentrate. Moist (beads)
			2				conc.?	
7			4					
8								
			2					Poor recovery sampler spring malfunction.
9		0.5	3				tailings	Very fine-grained pyritic tailings.
			7					
10			11	GC94TA5				
				8.5-10.5				
11				11.5-13.5				
			3	bags A&B				Same as above. Moist, dk gry, very fine-grained, pyritic tailings.
12		1.5	4	2201682			tailings	Mod. fizz. Less moisture than 5-10 foot interval.
			9					
13			15					
			8					Very much dryer (crumbly) tailings. Mod. fizz.
14		1.8	15				tailings	
			20					
15			18	GC94TA5				
				13.5-15.5				
16			5	16-18			tailings	Same as above.
		1.8	10	bags A&B				
17			9	2201683				
			9	2201833				
18								
			4					More moist. Easier to cut with knife. Very fine-grained, pyritic
19		1.8	6				tailings	tailings. Very rare rock chip of siliceous rock.
			9				rare sr	
20								

Continued on next page.

HOLE: TA5 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20			10	GC94TA5				
				18.5-20.5				
21			4	21-23				More moisture. Cuts easily. Not saturated. Slight fizz.
		1.6	4	bags A&B				Same very fine-grained, pyritic tailings.
22			4	2201684				
			6					
23								
			1					Poor recovery. Even more moisture (cuts easily). Less fizz than
24		1.2	4				Tailings	above. Some <3cm angular clasts of graphitic sp or silicic arg with
			3				spgr	<2mm euhed. py.
25			3	GC94TA5				
				23.5-25.5				
26			2	26-28				Slightly dryer than above (cuts easily). Rare <2" clasts of gravel.
		1.8	4	bags A&B			tailings	Slight fizz.
27			4	2201685				
			6					
28								
			2					Same as above.
29		2.0	2				tailings	
			4					
30			6	GC94TA5				
				28.5-30.5				
31			2	31-33				Same dk gry, very fine-grained, moist (cuts easily), pyritic tailings.
		2.0	3	bags A&B			tailings	Slight fizz. Slightly more moisture.
32			3	2201686				
			5					
33								
			2					Same as above. Slight to mod. fizz. Same moisture (cuts easily).
34		2.0	3					
			5				tailings	
35			5	GC94TA5				
				33.5-35.5				
36			2	36-37			organics	Reach well compacted, moist (not soggy), red-brn peat. Abundant
		2.2	3	bags A&B				roots and fine organics. Methane smell.
37			5	2201687				
			5					
38				GC94TA5				Red-brn <3/8 inch roots and organics. One large piece of a log with
			3	37-38			organics	bark attached. Moister than above (more compacted and
39		2.2	4	38.5-40.5			log	decomposed). Not soggy. Cuts easier than material above. Doesn't
			6	bags A&B				have near the methane smell as the last sample.
40								
								Continued on next page.

HOLE: TA5 continued								
depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
40			10				organics	
41								
42								
43			4	GC94TA5				Two inches of red-brn woody peat with roots and bark. The rest
		2.0	9	43-45				of the hole is dense, med. gry, moist (not soggy) clay. Clay contains
44			16	bags A&B			clay with	< 3mm layers of med. gry sand. ~60% subrounded qz. Sand is light
			26				sand layers	grey, fine-grained and contains mafic minerals and rock fragments
45							clayey sand	in addition to qz. No fizz. Looks pretty impermeable.
46								
47								
48			40	GC94TA5			clayey sand	Very compact, crumbly, clayey sand. Med. gry, fine to very fine.
		1.8	81	48-49.5				Damp not wet. Dryest material in the hole. Vig. fizz.
49			90	bags A&B			sandy clay	The hole ends in med-dk gry sandy clay. Very fine and very dense.
50	EOH		refusal					Thin <1mm sand layers containing predom. qz. Vig. fizz.

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TB-1
DATE: 8/16-17/94

DRILLER: Wink International Geotech, Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53064.7881	39952.3759	180.1

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

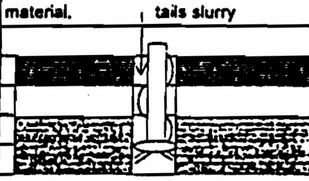
depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
				GC94TB1				This hole is on a cat cut bench so the ground surface is about a foot
				+1-0				lower than the original pile surface. The pile has 0-2 feet of dk gry
0				one bag			sp and	cap rock above 6-12 inches of red-brown, oxidized tailings. Below
				2201834			tailings	the oxidation layer the tailings is dk gry-brn.
1								
2								
			4					Dk gry-brn, damp (breaks), very fine-grained, pyritic tailings. About
3		1.8	4				tailings	1" of sericitic phyllite capping material. Vig. fizz. Relatively homog.
			6	GC94TB1				
4			5	2.5-4.5				
				5-7				
5			3	bags A&B				Dk gry-brn, damp, compact (breaks), homog. tailings. Vig.-mod. fizz.
		2.0	4	2201688			tailings	
6			7					
			8					
7								
			2					Dk brn-gry, much more moist (beads, no film), homog. tailings.
8		1.8	2				tailings	Mod. fizz.
			2	GC94TB1				
9			1	7.5-9.5				
				10-12				
10			1	bags A&B				Dk gry-brn, very moist (beads, film), homog. tailings. Mod. fizz.
		2.1	3	2201689				
11			2				tailings	
			1					
12								
			2				tailings	Dk gry-brn, moist (beads, minor film), pyritic tailings. Contains a <1"
13		1.7	2				some gravel	clast of siliceous slaty argillite with qz veining. Sample contains less
			3	GC94TB1				than 2% gravel.
14			8	12.5-14.5				
				15-15.5				
15		0.5	refusal	bags A&B			tails + grav.	Five inches of sticky tailings and gravel above large boulder of
				2201690			ssa	siliceous slaty argillite with quartz veins. Looks like Pit 5 rock.
16								
17								
18								
			4					Dk brn-gry tailings. Top of hole is very moist (beads, film). Grades
19		1.8	5	GC94TB1			tailings	downward into dryer (breaks) tailings with mod. fizz.
			8	18-20				
			16	20.5-22.5				
20				bags A&B				
				2201691				Continued on next page.

HOLE: TB1 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20								
			4					Dk gry-brn (less brown), homog. tailings. Top 6" is moist (beads,
21		2.0	6				tailings	film) grades into much dryer tails (breaks). Vig. fizz.
			8					
22			8					
23			2					Dk gry, less oxidized, homog. tailings. Moist (beads). Vig. fizz.
		1.7	5		▽		tailings	
24			5					
			6	GC94TB1				
25				23-25				
			1	25.5-27.5			tails, gravel	Top one foot is very moist (beads, cuts easily). Bottom foot contains
26		2.0	2	bags A&B				some gravel. Dk gry with minor brown. Mod. to vig. fizz.
			3	2201692				
27			4				tailings	
28			1					Dk gry with some brn, moist (beads), homog. tailings. Gets moister
		2.1	2				tailings	towards the bottom of the sample.
29			3					
			3	GC94TB1				
30				28-30				
			1	30.5-32.5				Dk gry with minor brn, very moist (beads, no film), homog. tailings.
31		2.2	2	bags A&B			tailings	Mod. fizz.
			2	2201693				
32			2					
33			1					Very moist (beads, slight film), dk gry-brn, homog. tailings. Mod. fizz.
		2.1	3				tailings	
34			3					
			4	GC94TB1				
35				33-35				
			1	35.5-37.5			pysp, ma	Even more moist (beads, heavy film, deforms under own weight),
36		1.9	2	bags A&B			tailings	dk gry-brn tailings. Liberates water when left standing. Last six
			3	2201694			saturated	inches is very pyritic (some electrum?). Vig. fizz. Top one foot
37			4					contains clasts of very pyritic sp and massive argillite.
38			15	GC94TB1			silt	Four inches of very wet, dk brn-gry tailings above one inch of dk brn-
		1.0	29	38-39			gravel	blk laminated silt. Silt sits on gravel containing several large chunks of
39				bags A&B				siliceous slaty argillite (hard, w/ qz along foliation, <1% <1mm euhed
40								pyrite). Very wet sample. Water rose to 24' when we punctured the
								gravel (finger drain). Water has bad anoxic smell (like TA4 and
								french drain. Xanthate etc? Water pH=7.1, cond.=3850 uMHO/cm
								Continued on next page.

HOLE: TB1 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
40								
			8	GC94TB1			tails slurry	Six inches of tailings slurry (probably contam. from above). Sits on
41		1.7	7	41-42.5			felt	ralatively dry muskeg (organics). Piece of 2mm felt prob. dragged
			9	bags A&B			organics	down by spoon covers organics. Organics are dk brn-red, containing
42			20					large chunks of wood, twigs and seeds.
43								
			5	GC94TB1				Five inches of decomposed, moist (almost soggy), dk brn peat.
		2.1	7	43-45			sandy grav.	Peat sits on 1.5' of sandy gravel. Gravel is poorly sorted, immature
44			12	bags A&B				(rounded to subangular <1" pebbles), mixed with organics and
			8				peat layer	coarse sand. The overall color is red-brn. No fizz. Oxidized. Clasts
45								are qz, silic. volcanics, granite, qz-py-chlorite, siltstone or altered
46								aphanitic basalt, and silic. slaty argillite.
47								
48							med. sand,	
			5	GC94TB1			pebbles	One foot of saturated, dk brn, pebbly, med. sand. Mixed with auger
49			14	48-50				cuttings from above. This presents a problem with trying to put a
			27	bags A&B			clay	screened interval beneath the pile. The relocated tailings will have
50			50					an effect on the water below the pile. The sand layer sits above
	EOH							very dense, compacted clay with minor v. fine sand (<.5mm lenses)
51								and black organics (grass). Clay contains silvery flakes (mica?)
								Note: Some tailings slurry got to the bottom of the hole via the augers.
52								This will influence the well data unless flushing can remove the
								material.
53								
54								
55								



WELL LEGEND
- BACKFILL ∇ WATER LEVEL
- BENTONITE
- .020 SILICA SAND
- SLOTTED PVC
- UNSLOTTED PVC

Well installation: To a depth of 50'. Installed:
1- 6" plug, 1- 5' screen, 1- 5' riser, 4- 10' risers, 1- 2" cap

Filled augers with water to try to keep slurry out as we pulled augers up.

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TB-2
DATE: 8/14-15/94

DRILLER: Wink International Geotech. Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53215.0309	39965.2206	205.8

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
								Note: Well head only stands 1.5' above ground.
0								
1								
2			12				gravel, tails	2' to 3' is a mix of gravel, cobbles and tailings. Tailings is dk brn-gry,
		1.9	13				sa, sp, sr,	damp (breaks, crumbly) and has mod. fizz in 10% HCl. Gravel is slaty
3			12				gz	argillite and sericitic phyllite. Qz and siliceous rock are less abundant.
			16	GC94TB2				
4				2-4				
				5-7				
5			11	bags A&B				Dryer (crumbly), dk gry (not brn) tails with vig. fizz. Rare <1" clasts
		2.0	11	2201695			tailings	of sp and pyritic slaty arg.
6			7	2201835			sp, pysa	
			6					
7								
			4					Damp (crumlby), dk gry (some brn) tailings with vig. fizz. Rare gravel.
8		1.6	6				tailings	
			6	GC94TB2			rare gravel	
9			7	7.5-9.5				
				10-12				
10			2	bags A&B				Lighter med. gry, damp (crumbly), tailings with vig. fizz. Pounds
		1.8	3	2201696			tailings	easily but not very moist.
11			2					
			2					
12								
			2					A bit more moist (breaks), dk gry, very fine-grained, homog., pyritic
13		1.9	2				tailings	tailings. Vig. fizz.
			3	GC94TB2				
14		8/15	4	12.5-14.5				
				15-17				
15			5	bags A&B				Dk gry, damp (breaks), gravelly tailings. Vig. fizz. Gravel is <1" sp
16		1.8	5	2201697			tailings w/	and qz+sr.
			5				gravel	
			5					
17								
			3					Dk gry, moist (beads) tailings. No gravel. Mod. fizz.
18		1.9	4				tailings	
19			6	GC94TB2				
			8	17.5-19.5				
20				20-22				
				bags A&B				Continued on next page.
				2201698				
				2201836				

HOLE: TB2 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20			2					Dk gry, moist (beads), homog. tailings. Mod. fizz. No gravel.
		2.1	3				tailings	
21			5					
			7					
22								
			3					Dk gry, moist (beads), homog. tailings. No gravel.
23		1.9	4				tailings	
			5					
24			5	GC94TB2				
				22.5-24.5				
25			2	25-27				Dk gry-brn (mottled), moist (beads) tailings. Grey portions have
		1.9	2	bags A&B			tailings	vig. fizz and brown portions have mod. fizz. Brn= oxidation
26			3	2201699				
			5					
27								
			2				oxidized	Dk brn-gry, very moist (beads easily), oxidized, homog. tailings.
28		1.9	3				tailings	Mod. fizz. No gravel.
			4					
29			5	GC94TB2				
				27.5-29.5				
30			2	30-32				Dk brn-gry, very moist (beads easily), oxidized, tailings as above.
		1.9	4	bags A&B			oxidized	
31			3	2201700			tailings	
			5	2201837				
32								
			5					Top one foot is same as above then tailings dry out (break-bead)
33		2.0	6					and are greyer. Mod.-vig. fizz.
			13				tailings	
34			14	GC94TB2				
				32.5-34.5				
35			4	35-37				Dk gry-brn, moist (breaks-beads) tailings. Mod. to vig. fizz.
		2.0	5	bags A&B			tailings	
36			4	2201701				
			5					
37								
			2					Dk brn-gry, more moist (beads easily), tailings. Clast of chloritic sr
38		1.8	3				tailings	< 2". Tailings form oxidized brn skin in sample bag.
			7					
39			5	GC94TB2				
				37.5-39.5				
40				40-42				
				bags A&B				
				2201702				Continued on next page.
				2201838				

HOLE: TB2 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
40			3					Top one foot is very moist (beads), dk gry-brn, tailings with mod. fizz.
		1.8	5					Bottom is much dryer (breaks), less brn. Vig. fizz.
41			13					
			12					
42								
			5					Dk gry-brn, dryer (breaks) tailings with mod. fizz. Contains rare <2"
43		2.0	5				tailings	clast of lightning-veined dolomitic massive argillite.
			8				rare dma	
44			5	GC94TB2				
				42.5-44.5				
45			4	45-47				Top 8" are very soft, moist (beads) tailings. Then about 10" of dryer
		2.0	7	bags A&B			tailings	(breaks) tails above 2" of more moist (beads-breaks) tailings.
46			5	2201703				Color is dk gry-brn. Mod. fizz.
			5					
47								
			3					Top 16" are very moist (beads easily, film) tailings above 8" of much
48		2.0	5				tailings	dryer (crumbly), compact tailings. Dk gry-brn. Mod. fizz.
			15					
49			28	GC94TB2				
				47.5-49.5				
50			6	50-52				Moist (beads), dk gry-brn, very fine-grained pyritic tailings. Mod. fizz.
		2.0	7	bags A&B			tailings	Homogeneous. Cuts easily. Makes slurry when water is added.
51			7	2201704				
			7					
52								
			2					Top one foot is moist (beads, film), dk brn-gry tails with mod. fizz.
53		1.9	8				tailings	Bottom foot is dryer (crumbly), dk gry tails with vigorous fizz.
			17					
54			13	GC94TB2				
				52.5-54.5				
55			4	55-57				Soft, moist (beads-breaks), dk gry (minor brn) tailings. Mod fizz.
		2.0	4	57.5-58.5			tailings	
56			5	bags A&B				
			7	2201705				
57				2201839				
			3				tailings	First one foot is moist (beads), dk gry tailings with vig. fizz.
58		2.2	5					
			34	GC94TB2			organics	14" of organics (peat, wood, twigs, <1mm blk spherical seeds).
59			15	58.5-59.5				Organics are red-brn, dense, damp (not soggy).
	EOH			one bag				
60								Well installation: To a depth of 59.5 feet.
								1- 6" plug, 1- 10' screen, 5- 10' risers, 1- 2" cap

WELL LEGEND

- BACKFILL
- WATER LEVEL
- BENTONITE
- .020 SILICA SAND
- SLOTTED PVC
- UNSLOTTED PVC

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TB-3
DATE: 8/10-11/94

DRILLER: Wink International Geotech, Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53368.2909	39963.9035	219.1

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
0								Top 2 feet are mixed tailings and production rock.
							tailings and	
1							cap rock	
							sp, arg, sr	
2		1.1	19				tailings	Six inches of dk brn tailings with gravel. Mod. fizz. Above
			30				oxidized	six inches of gravel. Predom. sericitic phyllite and massive argillite.
3			refusal				sr boulder	Slight fizz with 10% HCl. Refusal on boulder of siliceous rock. Try
				GC94TB3				to drill through it. Two feet of sr. Wreck bit. Move rig five feet to the
4				2-3				west.
				7-9				
5				bags A&B			gravel	
							cap rock	Abund. gravel and rock. Hard drilling. Very pyritic sr, sr, mfb.
6								
7			2				tailings and	Soft, dk brn, tailings mixed with sewage. Smells very poor. Contains
		1.5	2				sewage	abund. hair. Generally unpleasant.
8			2					
			2					
9								
			1				tailings and	10" of tailings and sewage above pyritic sr and much dryer (breaks)
10		1.7	3				sewage	tailings.
			18				pysr, tails	
11			24	GC94TB3				
				9.5-11.5				
12			13	12-14				Dry (crumbly), very fine-grained, pyritic tailings with minor gravel.
		1.9	18	bags A&B			tailings	
13			19	2201706			gravel	
			19					
14								
			9					Homog., very fine-grained, pyritic tailings. No gravel.
15		1.8	13				tailings	
			14					
16			14	GC94TB3				
				14.5-16.5				
17			7	17-19				Dry (breaks, cuts hard), homog. tailings. Vig. fizz.
		1.9	11	bags A&B			tailings	
18			11	2201707				
			11					
19								
			7					
20								Continued on next page.

HOLE: TB3 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20		2.0	9					Moist (dryer 20.5-21.5), very fine-grained, pyritic tailings with vig. fizz
			18				tailings	in 10% HCl.
21			23	GC94TB3				
				19.5-21.5				
22			5	22-24				Very fine-grained, pyritic tailings. Grades from dryer (breaks) to
		1.7	4	bags A&B			tailings	more moist (beads) towards bottom of sample. Encounter several
23			4	2201708			srg	clasts of <1" graphitic siliceous rock. Tails has vig. fizz.
			6					
24								
			4					Top one foot is moist (beads) tailings. Encounter very dry (breaks,
25		1.5	5				tailings	very crumbly) pyritic tailings at 26'. Refusal. Vig. fizz.
			25	GC94TB3				
26			refusal	24.5-26				
				27-29				
27			5	bags A&B				Dry (breaks), very fine-grained, pyritic tailings with vig. fizz for the
		2.0	5	2201709			tailings	upper 1.5 feet. Bottom 6" is moist (beads) tailings with mod. fizz.
28			5					
			5					
29								
			2					Moist (beads, deforms easily), dk gry-brn, very fine-grained, pyritic,
30		2.0	2				tailings	tailings. No gravel.
			4					
31			3	GC94TB3				
				29.5-31.5				
32			2	32-34				
33		2.0	3	bags A&B			tailings	Moist (beads), homog. tailings. Slightly dryer than above. Mod. fizz.
			4	2201710				
			5					
34								
			4					Moist (beads-breaks), dk gry-brn, very fine-grained pyritic tailings.
35		1.9	4				tailings	Vig. fizz. Homogeneous.
			4					
36			4	GC94TB3				
				34.5-36.5				
37			5	37-39				Dryer (breaks), homog. tailings. Some rare gravel. Vig. fizz.
		2.0	10	bags A&B			tailings	
38			10	2201711			rare gravel	
			11					
39								
40			3					
								Continued on next page.

HOLE: TB3 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
40		2.0	3				tailings	Moist (beads-breaks), homog. tailings. Vig. fizz. Rare sp clasts.
			4				rare sp	
41			4	GC94TB3				
				39.5-41.5				
42			2	42-44				Same as above. Some gravel. Red granite? and siliceous slaty arg.
		1.9	3	bags A&B			tailings	
43			6	2201712			granite, ssa	
			5					
44								
			3					Top of sample is moist (beads) tailings. Bottom of sample is much
45		2.0	4				tailings	dryer (breaks, very crumbly), compacted tailings with vig. fizz. One
			5				sr	clast of <1" chloritic siliceous rock.
46			13	GC94TB3				
				44.5-46.5				
47			5	47-49				Moist (breaks-beads), dk gry-brn, homog. tailings with mod. fizz.
		2.1	8	bags A&B			tailings	
48			7	2201713				
			6					
49								
			3					Top 1.25 feet are soft, moist (beads) tailings with mod. fizz. Bottom
50		2.2	5				tailings	portion is hard, crumbly (breaks) tailings. Brown color is located along
			7					wavy planes within the dryer material. Shows channeled flow and
51			16	GC94TB3				oxidation.
				49.5-51.5				
52			7	52-54				Dryer (breaks), homog., dk gry tailings with mod. fizz.
		2.1	9	bags A&B			tailings	
53			9	2201714				
			8					
54								
			2					Much more moist (beads), grey-brn tailings with mod. fizz.
55		2.2	3				tailings	
			3					
56			5	GC94TB3				
				54.5-56.5				
57			2	57-59				Moist (beads), dk gry tailings with vig. fizz.
		2.1	2	bags A&B			tailings	
58			3	2201715				
			4					
59					∇			
60		1.2	11	GC94TB3			organics	Three inches of very saturated tailings with stick included on top of
			30	59.5-60.5			wood	9" of wood (stump?) sticks and organics. Then 2" of tan, coarse,
	EOH		refusal	one bag			clayey sand	clayey sand with black spherical seeds. Not sure what caused
								refusal. Mod. fizz on tailings. No fizz on clayey sand. Muskeg is
								red-brn and smells like methane (cow dung).

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TB-4
DATE: 8/17-18/94

DRILLER: Wink International Geotech. Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53519.4099	39974.0233	224.0

Legend
gravel; tailings; organics; clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
0								
1							cap rock	
2								
			22	GC94TB4			ma, mfb, cr,	Mix of very fine-grained, pyritic tailings and cap rock. Cap rock
		1.7	22	2-4			sp mixed w/	consists of massive argillite, massive fine base metal sulfide, chloritic
3			22	bags A&B			tailings	rock and sericitic phyllite. Tailings shows moderate fizz with 10% HCl.
			12					Bottom of sample contains more tailings.
4								
			11					Relatively dry (crumbly), dk gry, very fine-grained, pyritic tailings.
5		1.8	13				tailings	Oxidation along thin, wavy planes. Vig. fizz. Rare clast of chloritic
			13				rare src	siliceous rock.
6			12	GC94TB4				
				4.5-6.5				
7			3	7-9				Damp (breaks), dk gry, homog., very fine-grained, pyritic tailings.
		2.0	4	bags A&B			tailings	Vig. fizz. No clasts.
8			4	2201716				
			4					
9								
			18					4" of wet (sticky, beads, film), dk gry-brn, tailings with mod. fizz
10		2.2	40				tailings	above 1.8 feet of very hard, dry (crumbly), dk gry tailings. Vig. fizz.
			47				ssa, sa	Inclusions of silic. slaty arg and slaty arg.
11			49	GC94TB4				
				9.5-11.5				
12			16	12-14				Very compacted, crumbly, dk gry tailings. Vig. fizz. Homogeneous.
		1.9	26	bags A&B			tailings	
13			25	2201717				
			30					
14								
			10				wood	Dk gry, dry (crumbly, hard to cut), very compacted tailings. Shread
15		1.9	14				tailings	of wood at top of sample.
			17					
16			20	GC94TB4				
				14.5-16.5				
17			9	17-19				Dk gry (minor brn), crumbly tailings. No inclusions.
		1.8	12	bags A&B			tailings	
18			12	2201718				
			12					
19								
20			10					

Continued on next page.

HOLE: TB4 continued

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20		1.9	11					Same as above except for 6" of moist (beads-breaks) starting at 21'.
			8				tailings	Vig. fizz in 10% HCl. One <1" clast of pyritic sr.
21			14	GC94TB4				
				19.5-21.5				
22			6	22-24	▽		saturated	Very moist (beads, water film), dk gry-brn tailings with vig. fizz.
		2.0	3	bags A&B			tailings	
23			3	2201719				
			6					
24								
			6					Grades from moist (breaks), dk gry-brn tailings to dryer (crumbly),
25		2.0	8				tailings	dk gry tailings down sample. No gravel.
			22	GC94TB4				
26			26	24.5-26				
				27-29				
27			20	bags A&B				Dk gry-brn, very dense, compacted tailings with vig. fizz.
		1.8	24	2201720			tailings with	10% gravel containing sp, ssa, sr.
28			22				10% gravel	
			30				sp, ssa,sr	
29			refusal					
			26					Dk gry, very dense, compacted, dry (crumbly, hard to cut) tailings.
30		1.9	40				tailings, grav	< 2% gravel. sr, sp, one piece of brown, melted glass. Vig. fizz.
			35				sr, sp	.5" beads of tails come up augers as dry tails from below mix with
31			38	GC94TB4				water from above.
				29.5-31.5				
32			11	32-34				Dk gry, crumbly tailings (slightly more moist than above). Dense.
		2.1	14	bags A&B			tailings	Vig. fizz. Some <1" sr gravel.
33			17	2201721			sr	
			20					
34								
			15				gravel and	Dk gry, damp (breaks) tailings with vig. fizz. Some river gravel and
35		2.0	13				tailings	sr (top 6").
			13					
36			13	GC94TB4				
				34.5-36.5				
37			7	37-39				Dk gry-brn, moist (breaks-beads), homog., very fine-grained, pyritic
		1.9	10	bags A&B			tailings	tailings. Vig. fizz. No gravel.
38			8	2201722				
			10					
39								
			4					
40								

Continued on next page.

HOLE: TB4 continued								
depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
40		2.0	6				tailings	Dk gry-brn, moist (breaks-beads, cuts easily), very homog. tailings.
			7					Mod. fizz.
41			8	GC94TB4				
		8/18		39.5-41.5				
42			6	42-44				Dk gry-brn, moist (beads-breaks, cuts easily), very fine-grained,
		2.0	8	bags A&B			tailings	pyritic tailings. Homogeneous. Mod. fizz in 10% HCl.
43			8	2201723				
			8					
44								
			4					1.5 feet of moist (beads), dk brn-gry tailings above 6" of dryer
45		2.1	7				tailings	(crumbly), compacted tailings. Both types have vig. fizz in HCl.
			7					Homogeneous.
46			15	GC94TB4				
				44.5-46.5				
47			11	47-49	∇			Grade from dryer (breaks), dk gry-brn tailings with vig. fizz to more
		2.0	10	bags A&B			tailings	moist (beads), dk brn-gry tailings with mod. fizz. One clast of pyritic
48			11	2201724			pysa	slaty argillite (<1").
			11					
49								
			5	GC94TB4				1.8 feet of moist (beads), dk brn-gry, pyritic, tailings with mod. fizz
50		1.9	5	49.5-51.5				above 1" of smelly ssa (Pit 5) gravel. Gravel smells bad (like french
			7	bags A&B				drain) and contains black coatings. Not satuated.
51			17	2201725				
			refusal				gravel, ssa	
52			14	GC94TB4				Finger drain gravel consisting of crushed (<1") siliceous, slaty, argillite
		0.8	35	52-53				with euhedral, disseminated pyrite (<1mm). Anoxic, sulfur, chemical
53	EOH		refusal	one bag			bedrock	smell. Bottom 6" is silted up with fines. No fizz in HCl. Gravel is soft
								and crumbly at base. Gravel could also be slaty argillite from the mine
54								or bedrock. There is a one inch layer of clayey sp-sa mix between
								the french drain and the crumbly rock below, which also contains
55								rare roots and sticks. Water rises to ~47 feet. Had to drain a
								considerable amount of tailings slurry from the sample.

BOREHOLE SUMMARY

LOCATION: Tailings Impoundment
HOLE: TB-5
DATE: 8/6-7/94

DRILLER: Wink International Geotech, Inc.
DRILL RIG: Nodwell Acker w/ 8" hollow-stem auger
GEOLOGIST: Peter Condon
SAMPLE TYPE: 2.5" split spoon

NORTHING	EASTING	ELEVATION
53665.6072	39973.6131	221.0

Legend
gravel, tailings, organics
clay, silt and/or sand
sericitic phyllite (sp), argillite (arg), quartz (qz)
pyrite (py), massive fine base metal sulfide (mfb),
iron hydroxide (Feox), mottled (mott)
graphitic (gr, g), silicic (si, silic), local (loc)

depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
0								Cap rock is predom. <2" gravel comprised of mostly sericitc phyll*e.
							cap rock	
1							sp	
2								
			3					Dk gry, moist (beads, cuts like butter), very fine-grained, pyritic
3		1.6	3				tailings	tailings. Flakes of sp (<2") and sub-rounded gravel (<1") at base of
			3					sample. Slight fizz.
4			7	GC94TB5			gravel	
				2.5-4.5				
5				5.5-7.0				
			16	bags A&B				Much dryer. Only slightly damp (crumbly), very compacted, pyritic
6		1.5	37	2201726			tailings	tailings. Very hard to hammer. Vig. fizz (more so than above).
			60					Dk gry and very fine-grained.
7			refusal					
8								
			14					Dry (crumbly), dk gry tailings with vig. fizz. One <3cm clast of
		1.8	23				tailings	angular quartz.
9			24				qz	
			23	GC94TB5				
10		8/7		8-10				
			15	10.5-12.5				Dk gry, moist (breaks, cuts easily), very fine-grained, pyritic tailings
11		1.7	13	bags A&B			tailings	with moderate fizz. Rare clasts of imported diorite (<2").
			12	2201727			diorite	
12			13					
13								
			2					Very moist (cuts very easily, still holds together), tailings with
14		1.5	2				tailings	mod. fizz. Contains one <2" clast of imported rock (diorite).
			3				diorite	
			4	GC94TB5				
15				13-15				
			1	15.5-17.5				Same as above. Mod. to slight fizz. Very moist except for the
16		1.5	2	bags A&B			tailings	bottom one inch. Some clasts of imported diorite (<2").
			4	2201728			diorite	
17			7					
18								
			21				tailings	Much dryer (crumbly), very hard (to the point of refusal) tailings
19		1.4	45				diorite	with vig. fizz. Some clasts of <2" imported diorite.
			30	GC94TB5				
			refusal	18-19.5				
20				20.5-22.5				Continued on next page.
				bags A&B				
				2201729				

HOLE: TB5 continued

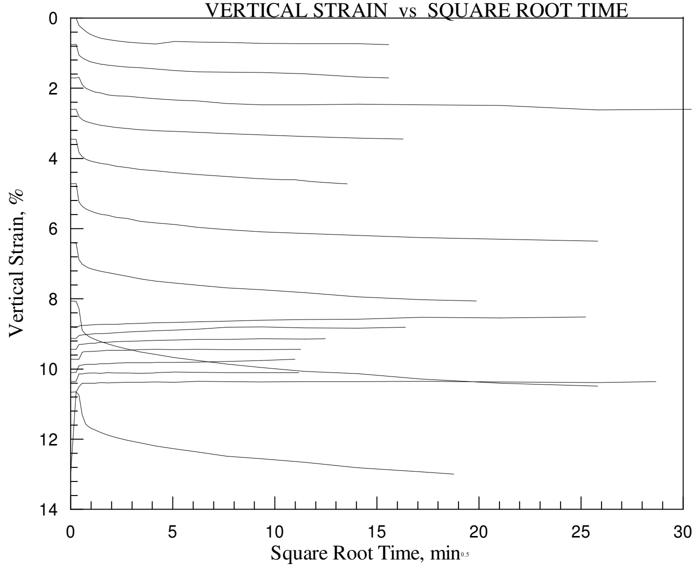
depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
20					∇		saturated	
			3				tails-water	Hit boulder and switch to 1.5" spoon. Abundant water and tails slurry
21		2.0	13				slurry	for the first foot. Then several inches of <1" siliceous rock gravel.
			22				gravel	sr is well-foliated and contains minor <1mm euhed. py. Gravel sits on
22			25				tailings	much dryer, very compacted tailings which fizzes vigorously in 10%
								HCl. No fizz in tailings slurry above.
23			10					
		2.0	14				tailings	Very soggy for the first 8" then firms up a bit but still very moist.
24			12					Very fine-grained, pyritic tailings.
			13	GC94TB5				
25				23-25				
			6	25.5-27.5				As above but dryer (cuts easily).
26		1.8	9	bags A&B			tailings	
			8	2201730				
27			11					
28								
			6					Dk gry, very fine-grained, pyritic tailings. Mod. to vig. fizz. Moist but
		2.0	7				tailings	not soggy (holds together well, cuts easily). Rare <2.5" clasts of
29			8				sp, cr	sericitic phyllite and chloritic rock.
			8	GC94TB5				
30				28-30				
			5	30.5-32.5				6" of slurry above moist (cuts easily) tailings. One <1" clast of
31		1.5	7	bags A&B			tailings	oxidized sp. Very weathered (likely elsewhere).
			9	2201731				
32			11					
33								
			5					2" slurry top above moist, homog. tailings with mod. fizz.
34		1.8	5				tailings	
			5					
			7	GC94TB5				
35				33-35				
			3	35.5-37.5				6" of slurry above moist, very fine-grained, pyritic tailings.
36		2.0	4	bags A&B			tailings	
			5	2201732				Note: Water from the 20' level is mixing with tailings and ponding
37			7					on top of the sample. Material from the first 6" or so is likely not
								representative.
38								
								Note: Driller mistakenly drilled from 35.5' to 48' with five feet of rod
39								extended below the auger bit. This rendered the interval unsample-
								able.
40								
								Continued on next page.

HOLE: TB5 continued								
depth (ft)	sample interval	feet recov.	blows/6" 140 lbs	composite number	oxidation\ water tab.	profile	summary	description and comments
40								Note: Driller mistakenly drilled from 35.5' to 48' with five feet of rod
								extended below the auger bit. This rendered the interval unsample-
41								able.
42								
43								
44								
45								
46								
							inferred	Missed contact but the over-extended rod did not have muskeg on
47							contact	it when it came up. Driller noted a change in torque at 47'.
48								
			4	GC94TB5				Top of sample has slight methane smell, which indicates it was close
49		2.2	4	48.5-50.5			organics	to the paleosurface. Material is moist, soft, decomposed grass, roots,
			8	one bag				sticks and bark. Nothing over 1cm in diameter. Dk red-brn. Bottom
50			9					of core has no methane smell.
51								
							inferred	
52							contact	
53								
							sandy clay	
			10	GC94TB5				The entire interval is med gry, sandy clay. Becomes very compact
54		1.5	20	53.5-55.5				at 55'. One clast of yellow-green (epidote-altered?) rock and several
			30	one bag				pieces of brn-blk grassy-looking organics. Material looks very
55	EOH		refusal					impermeable. Mod. to vig. fizz.
								Note: The sample is coated with tailings, which suggests that
								there may be a fair amount of cross-contamination in the wetter holes.
								This sample also has a ponded slurry on top of it. Water runs down
								the inside of the auger and gains access to the sample when the
								plug is removed.

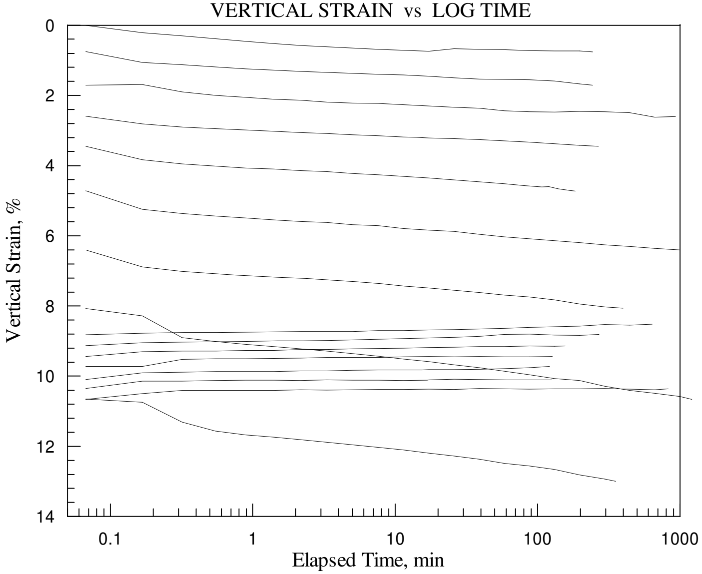
APPENDIX VI

2005 Laboratory Test Data
(Tests Conducted after August 16, 2005)

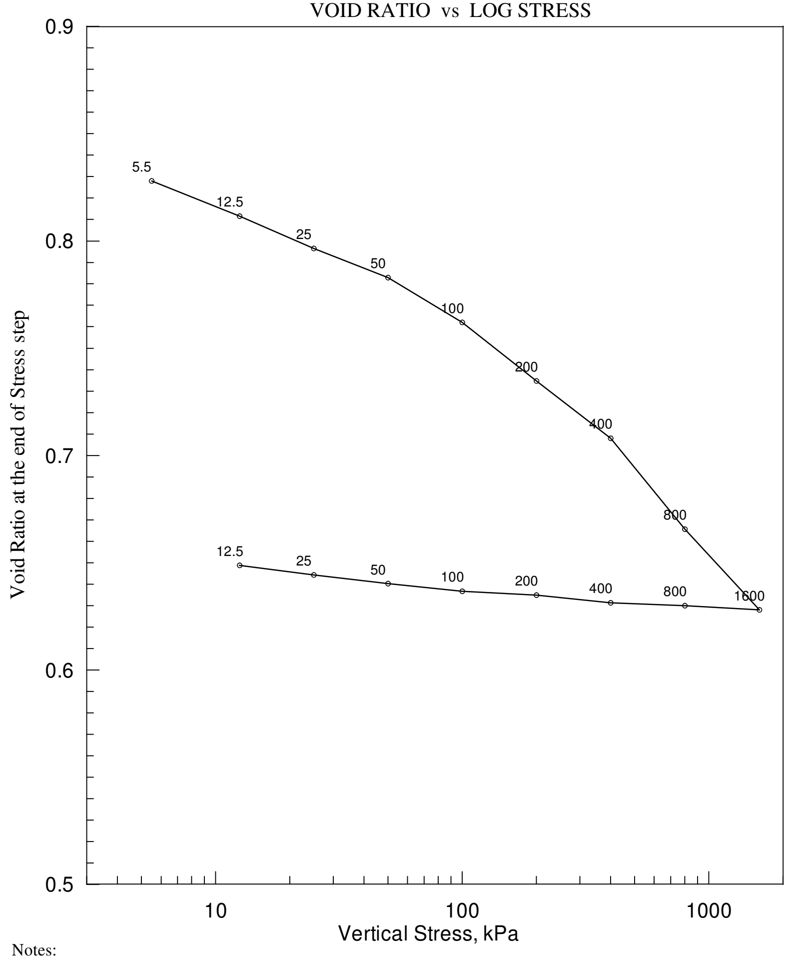
VERTICAL STRAIN vs SQUARE ROOT TIME



VERTICAL STRAIN vs LOG TIME



VOID RATIO vs LOG STRESS



Notes:
1. Based on the t_{90} obtained from Taylor's Method.
2. Based on C_v, M_v and t_{90}.
3. System compliance calibrated according to ASTM D 2435.

SUMMARY RESULTS

Compression Index, C_c	
Stress range 5 to 50 kPa	0.047
Stress range 50 to 400 kPa	0.083
Stress range 400 to 1600 kPa	0.133
Swell Index, C_s	
Stress range 1600 to 13 kPa	0.010
Recompression Index, C_r	N/A

Average Stress, kPa	Coefficient of Consolidation, C_{v_1}, cm²/sec	Permeability k_2 cm/sec	Void Ratio
9.0	2.4E-02	3.8E-06	0.820
18.8	2.3E-02	2.7E-06	0.804
37.5	2.3E-02	1.4E-06	0.790
75.0	1.1E-02	5.1E-07	0.772
150.0	1.1E-02	3.3E-07	0.748
300.0	1.0E-02	1.6E-07	0.721
600.0	6.7E-03	8.1E-08	0.687
1200.0	6.3E-03	3.5E-08	0.647

Specimen Information	Values	Units
Sample ID	West Tails	-
Depth	-	m
Initial Water Content	26.3	%
Initial Dry Unit Weight	1867	kg/m³
Specific Gravity	3.44	-
Specimen Initial Height	18.57	mm
Specimen Area	3848	mm²
Initial Void Ratio	0.842	-
Final Water Content	18.7	%
Final Dry Unit Weight	2095	kg/m³

Klohn Crippen		Date
Tested	GG	Aug '05
Drawn	GG	Aug '05
Recommended		
Checked		

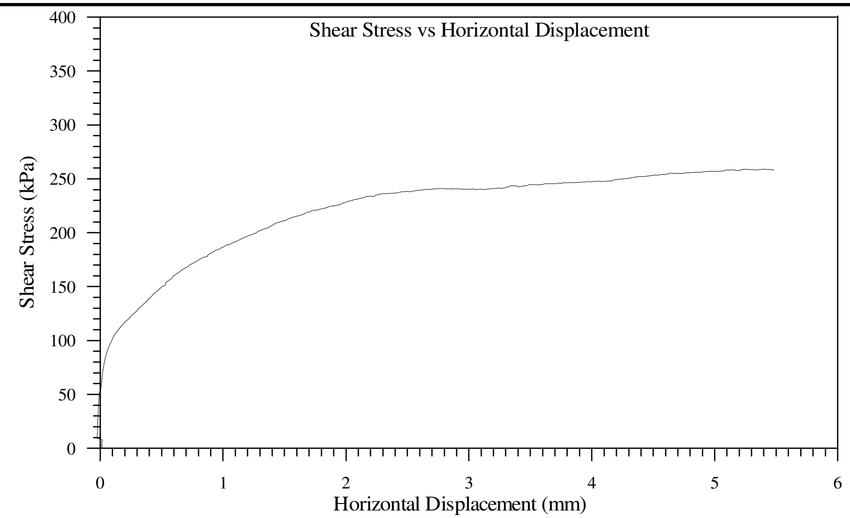
CLIENT
KENNECOTT MINERALS

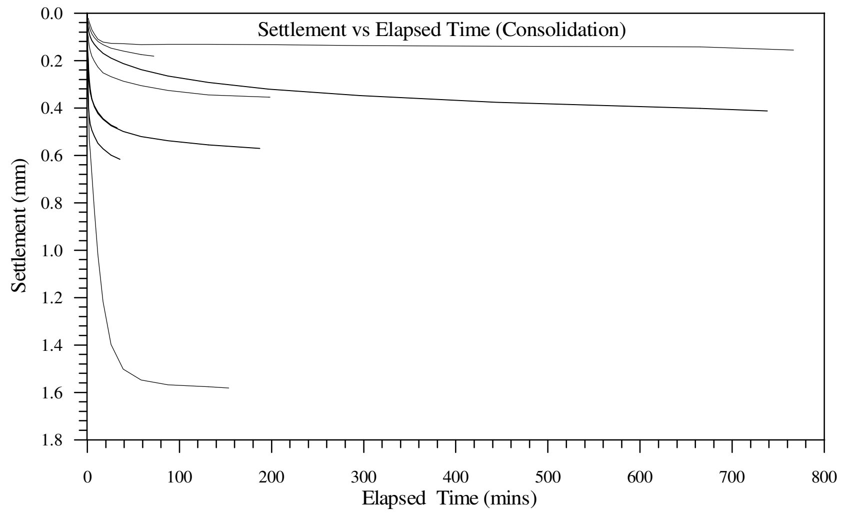
KLOHN CRIPPEN

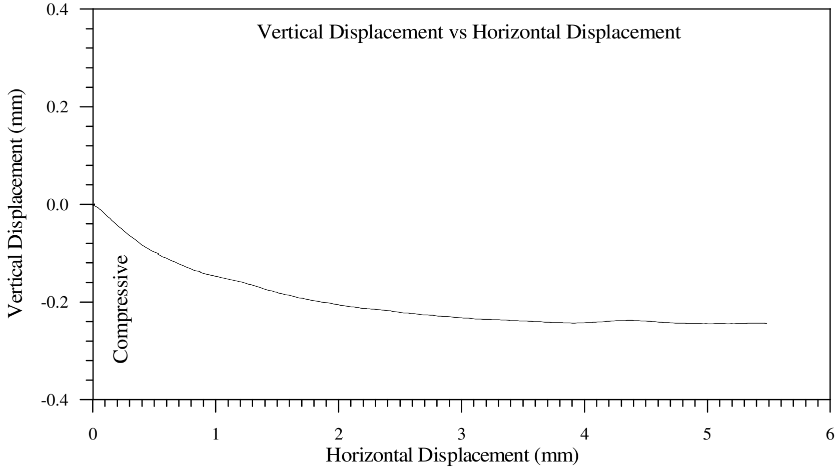
PROJECT
KENNECOTT GREENS CREEK MINE

TITLE
ONE DIMENSIONAL CONSOLIDATION on NEW TAILINGS (WEST)

DATE OF TEST	PROJECT No.	DWG No.	REV.
August, 2005	M07802 A40	FIG . VI -1	







SPECIMEN INFORMATION	DATA	UNITS
SAMPLE	New Tailings	-
	(late Aug 2004)	-
INITIAL SPECIMEN HEIGHT	33.0	mm
SPECIMEN DIAMETER	60	mm
STRAIN RATE	0.0	mm/min
INITIAL WATER CONTENT	25.0	%
INITIAL DRY DENSITY	1724	kg/m^3
CONSOLIDATION STRESS	250	kPa
- AFTER CONSOLIDATION	-	-
DRY DENSITY	1987	kg/m^3
FINAL WATER CONTENT	17.1	%

TO BE READ WITH KLOHN-CRIPPEN REPORT DATED ______________

KLOHN CRIPPEN		DATE
DESIGNED		
DRAWN	Ganan	Sept '05
TESTED	Ganan	Sept '05
CHECKED		
RECOMMENDED		
APPROVED		

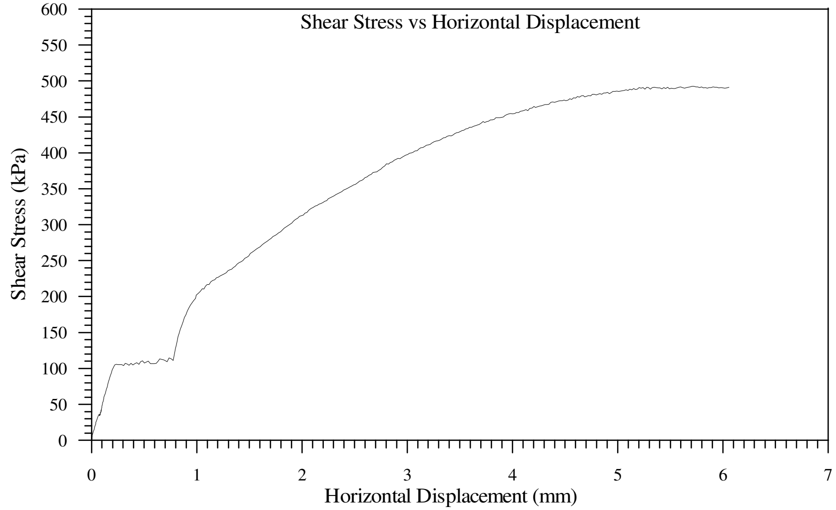
CLIENT: **KENNECOTT MINERALS**

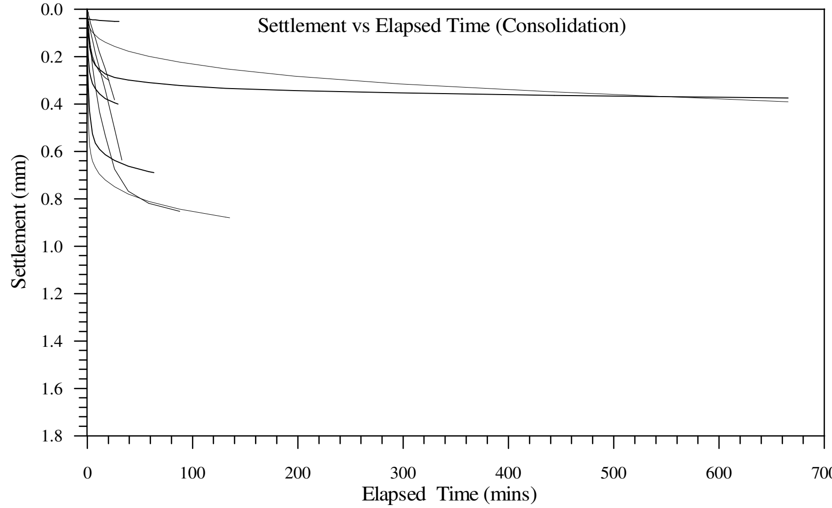
KLOHN CRIPPEN

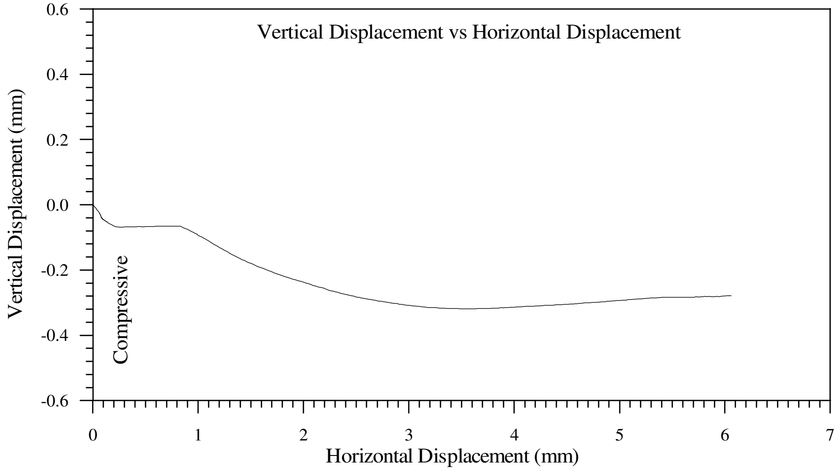
PROJECT: **KENNECOTT GREENS CREEK MINE**

TITLE: Direct Shear Test on Tailings

DATE OF TEST	PROJECT No.	FIG No.	REV.
August, 2005	M07802 A40	FIG. VI - 2	







SPECIMEN INFORMATION	DATA	UNITS
SAMPLE	New Tailings	-
	(late Aug 2004)	-
INITIAL SPECIMEN HEIGHT	33.0	mm
SPECIMEN DIAMETER	60	mm
STRAIN RATE	0.0	mm/min
INITIAL WATER CONTENT	27.1	%
INITIAL DRY DENSITY	1773	kg/m^3
CONSOLIDATION STRESS	500	kPa
- AFTER CONSOLIDATION	-	-
DRY DENSITY	2086	kg/m^3
FINAL WATER CONTENT	16.7	%

TO BE READ WITH KLOHN-CRIPPEN REPORT DATED ____________

KLOHN CRIPPEN		DATE
DESIGNED		
DRAWN	Ganan	Sept '05
TESTED	Ganan	Sept '05
CHECKED		
RECOMMENDED		
APPROVED		

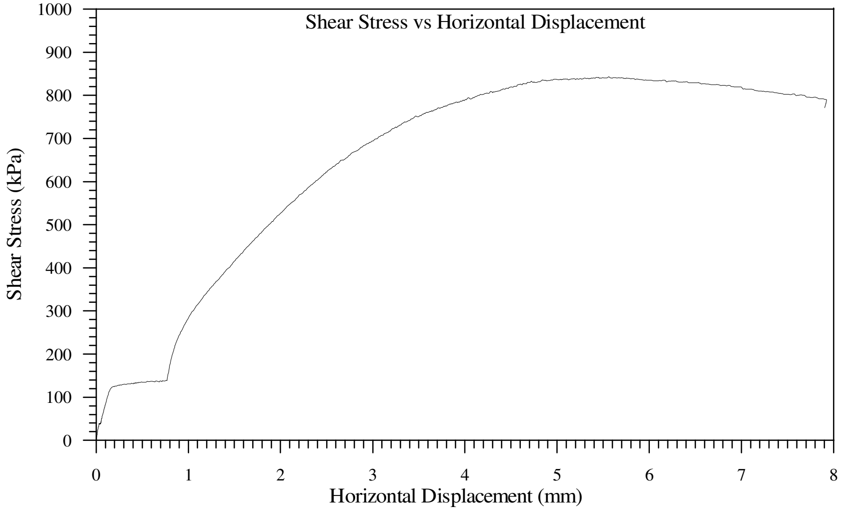
CLIENT
KENNECOTT MINERALS

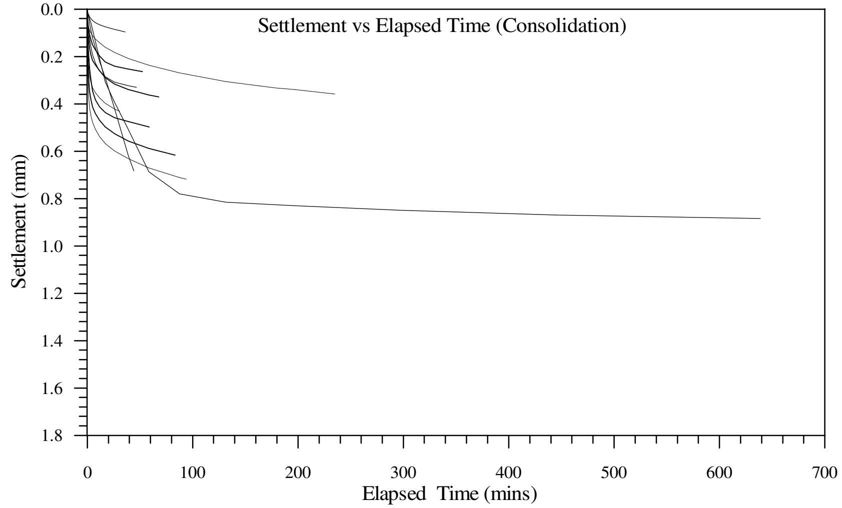
KLOHN CRIPPEN

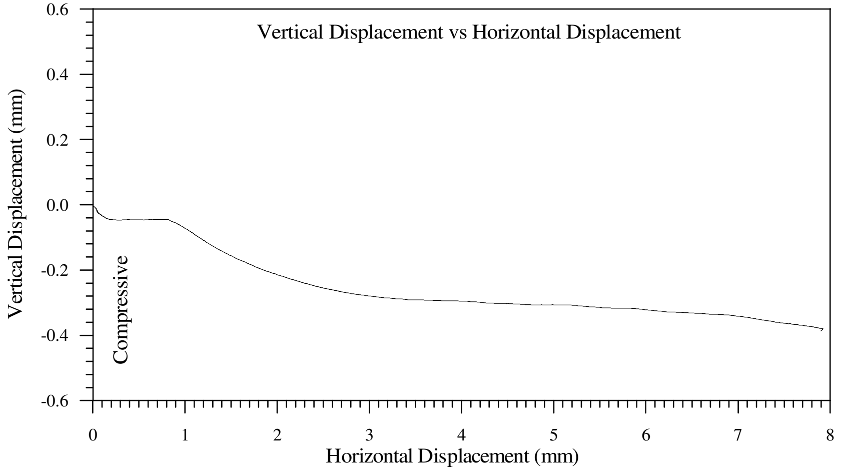
PROJECT
KENNECOTT GREENS CREEK MINE

TITLE
Direct Shear Test on Tailings

DATE OF TEST	PROJECT No.	FIG No.	REV.
August, 2005	M07802 A40	FIG VI - 3	







SPECIMEN INFORMATION	DATA	UNITS
SAMPLE	New Tailings	-
	(late Aug 2004)	-
INITIAL SPECIMEN HEIGHT	33.0	mm
SPECIMEN DIAMETER	60	mm
STRAIN RATE	0.0	mm/min
INITIAL WATER CONTENT	26.5	%
INITIAL DRY DENSITY	1753	kg/m^3
CONSOLIDATION STRESS	1000	kPa
- AFTER CONSOLIDATION	-	-
DRY DENSITY	2084	kg/m^3
FINAL WATER CONTENT	15.33	%

TO BE READ WITH KLOHN-CRIPPEN REPORT DATED ____________

KLOHN CRIPPEN		DATE
DESIGNED		
DRAWN	Ganan	Sept '05
TESTED	Ganan	Sept '05
CHECKED		
RECOMMENDED		
APPROVED		

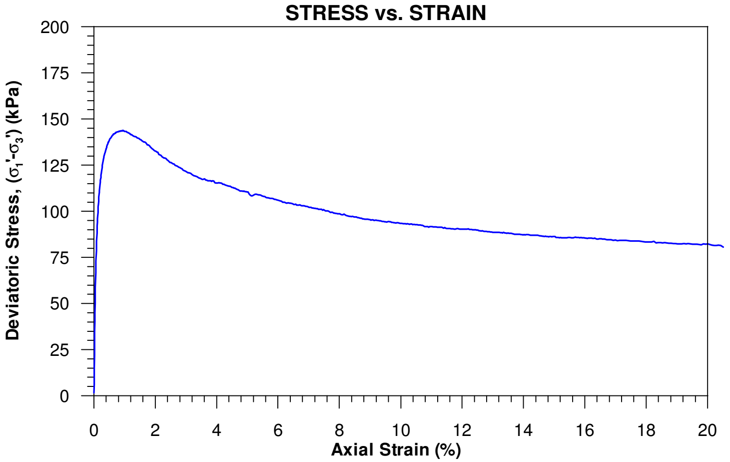
CLIENT
KENNECOTT MINERALS

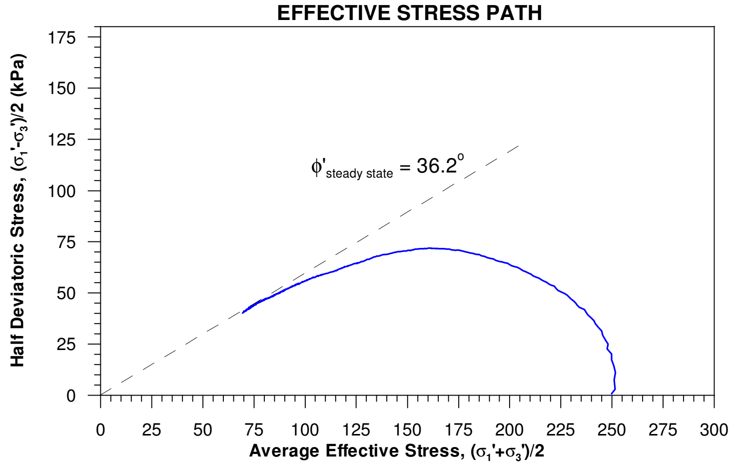
KLOHN CRIPPEN

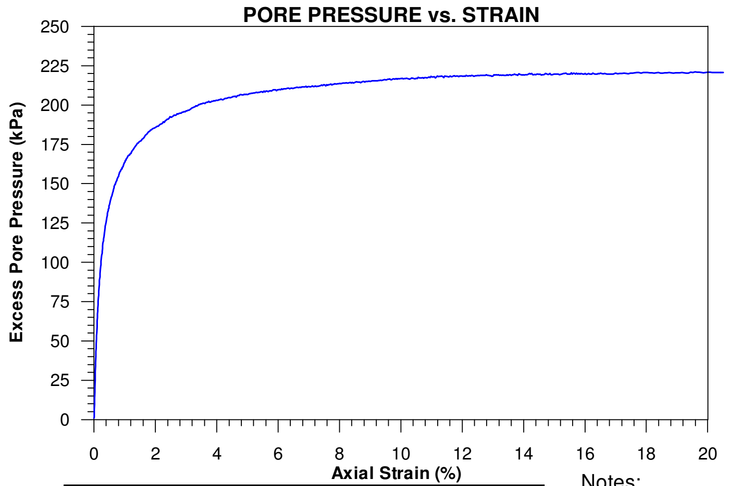
PROJECT
KENNECOTT GREENS CREEK MINE

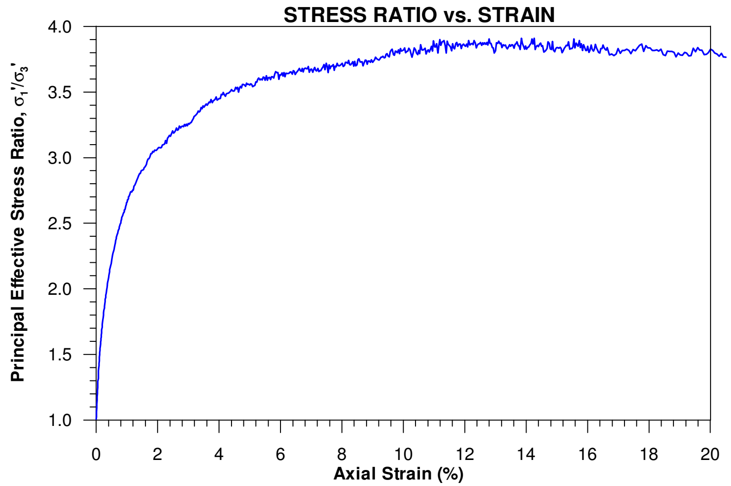
TITLE
Direct Shear Test on Tailings

DATE OF TEST	PROJECT No.	FIG No.	REV.
August, 2005	M07802 A40	FIG VI - 4	









SPECIMEN INFORMATION	UNITS	DATA
Tailings Sample (Dec 2005)		
Initial Water Content	%	11.2
Initial Dry Density	kg/m^3	1880
Skempton's B Parameter		0.98
Back Pressure	kPa	200
Consolidation Stress (σ_3')	kPa	250
(at start of shear)		
Dry Density	kg/m^3	1976
Specimen Height	mm	126.6
Specimen Area	mm^2	3038.7
Final Water Content	%	21.4

Notes:

1. Test specimen was prepared by moist tamping at 11.2% moisture content.

TO BE READ WITH KLOHN-CRIPPEN REPORT DATED ____________

KLOHN-CRIPPEN		DATE
DESIGNED		
DRAWN		
CHECKED		
RECOMMENDED		
APPROVED		

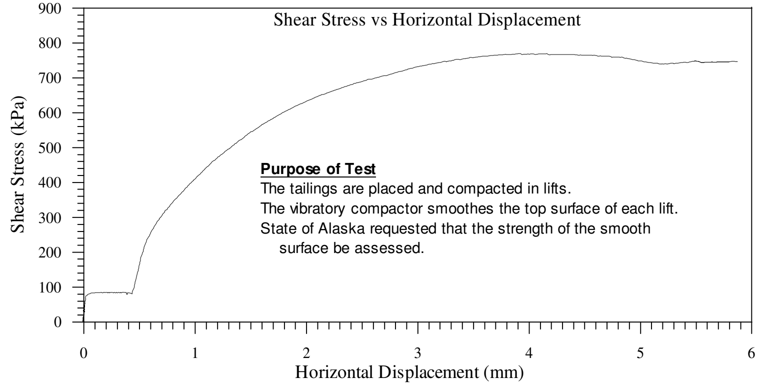
KLOHN CRIPPEN

CLIENT

PROJECT
Kennecott Greens Creek Mine

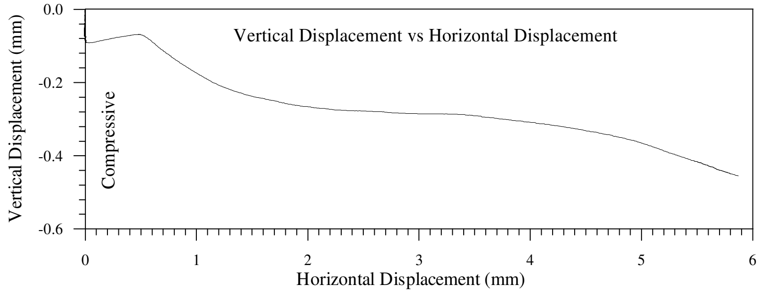
TITLE
ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST TAILINGS SAMPLE (Dec 2005)

DATE OF ISSUE	PROJECT No.	DWG. No.	REV.
January 2006	M07802A40	FIG. VI - 5	



Purpose of Test

The tailings are placed and compacted in lifts.
The vibratory compactor smoothes the top surface of each lift.
State of Alaska requested that the strength of the smooth surface be assessed.



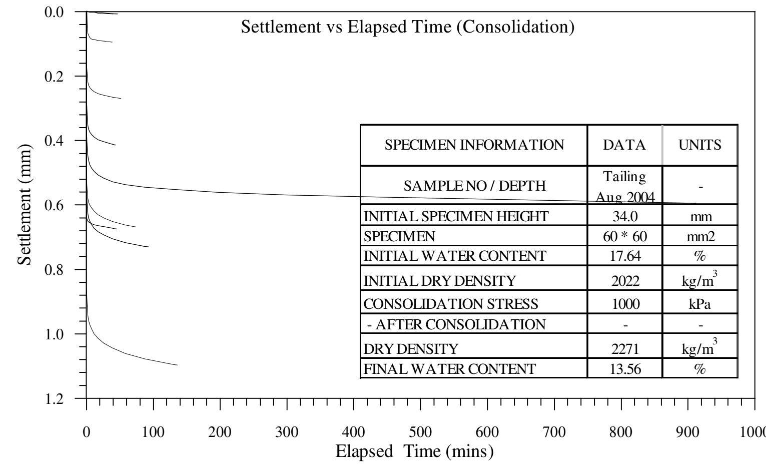
Test Procedure

1. Prepared remoulded sample at 90% standard proctor. Used "New" tailings from August 2004.
2. Consolidated the sample at 1000 kPa.
3. Pre-sheared the sample along the shear box plane using a fine wire.
4. Polished both sides of sample using a smooth piece of plastic until the surfaces appeared shiny and smooth (pulled plastic only in the direction of shear on each surface).
5. Re-positioned sample in shear box such that upon reloading the smoothed surface aligned with the shear plane.
6. Re-loaded to 1000 kPa.
7. Sheared sample for several cycles until residual strength was indicated. Sheared at slow rate of 0.016 mm/min.

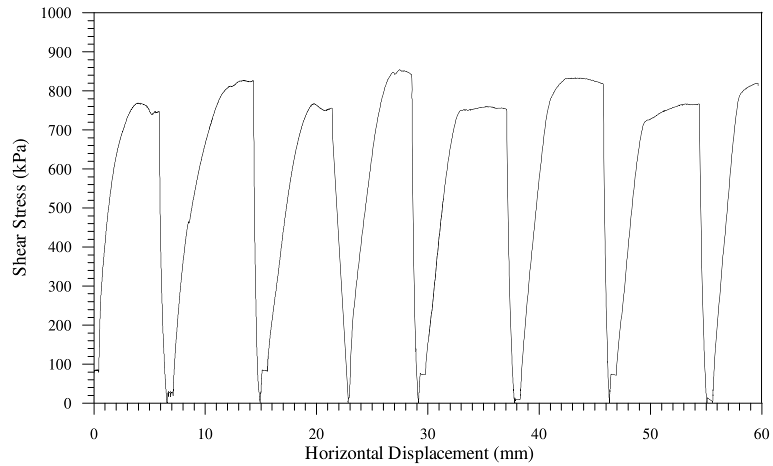
Test Result

37.6 degrees <u>peak</u> direct shear strength of smoothed sample (This is comparable to the average residual direct shear strength of "new" tailings tested in the normal manner.)

This shear test was carried out by pre-smoothing both the lower and upper surfaces of the failure plane.
In the field only the bottom surface of the lift joints is smoothed by the compactor.



SPECIMEN INFORMATION	DATA	UNITS
SAMPLE NO / DEPTH	Tailing Aug 2004	-
INITIAL SPECIMEN HEIGHT	34.0	mm
SPECIMEN	60 * 60	mm2
INITIAL WATER CONTENT	17.64	%
INITIAL DRY DENSITY	2022	kg/m^3
CONSOLIDATION STRESS	1000	kPa
- AFTER CONSOLIDATION	-	-
DRY DENSITY	2271	kg/m^3
FINAL WATER CONTENT	13.56	%



TO BE READ WITH KLOHN-CRIPPEN REPORT DATED ____________

KLOHN CRIPPEN		DATE
DESIGNED		
DRAWN	Juan	Oct '05
TESTED	Juan	Oct '05
CHECKED		
RECOMMENDED		
APPROVED		

CLIENT
KENNECOTT MINERALS

KLOHN CRIPPEN

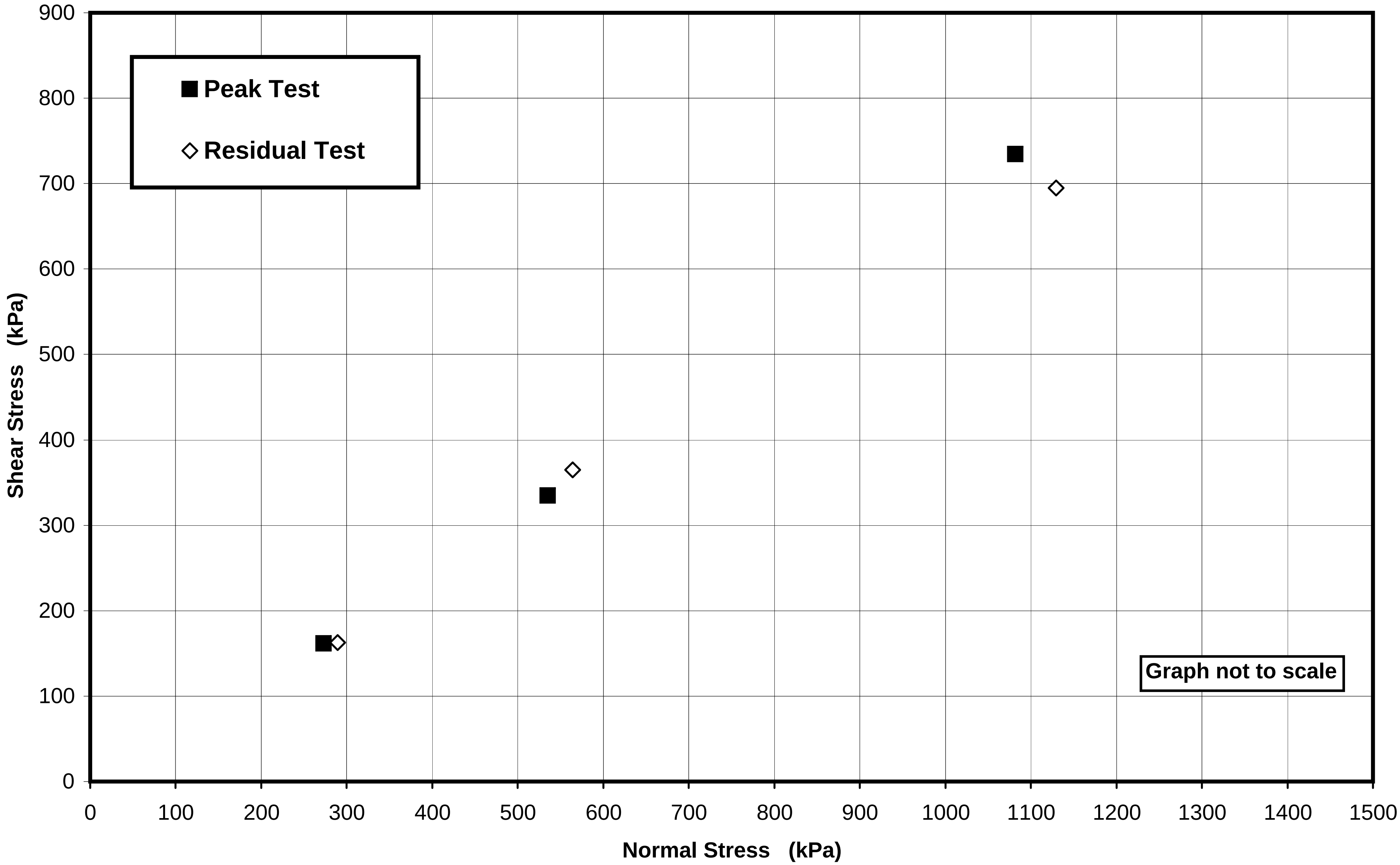
PROJECT
KENNECOTT GREENS CREEK MINE

TITLE
Direct Shear Test on Tailings

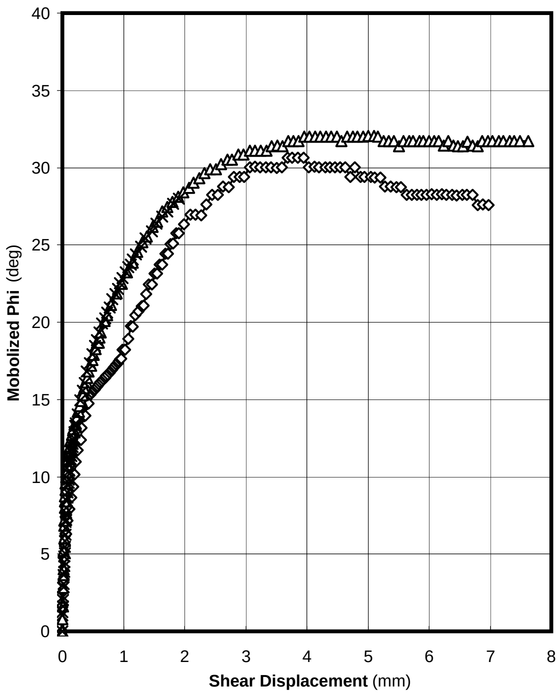
DATE OF TEST	PROJECT No.	FIG No.	REV.
October, 2005	M07802 A40	Figure	

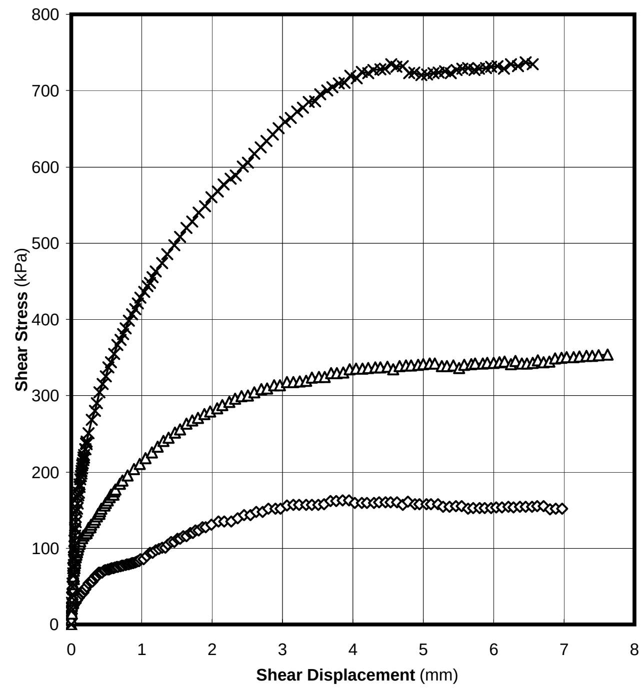
Direct Shear Test of Soils Under Consolidated Drained Conditions ASTM D3080-90

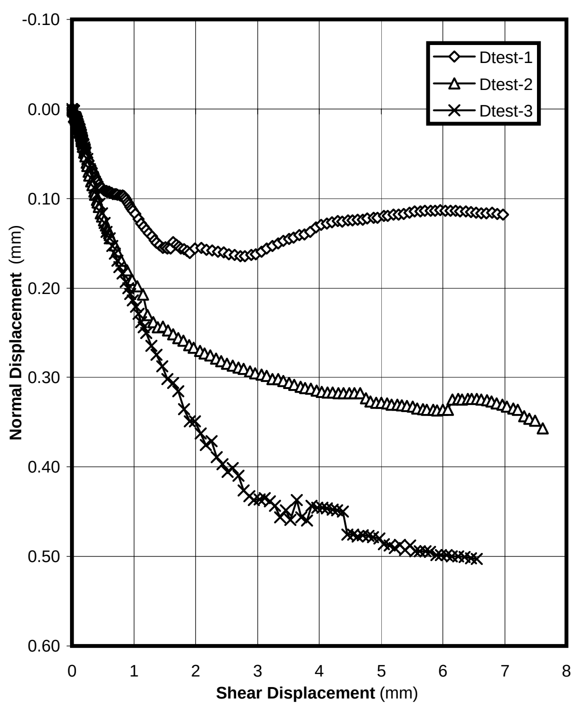
05-1416-158 Klohn Crippen/Proj# M07802A40/Burnaby Greens Creek Old Tailings Sample



Direct Shear Test of Soils Under Consolidated Drained Condtions
ASTM D3080-90





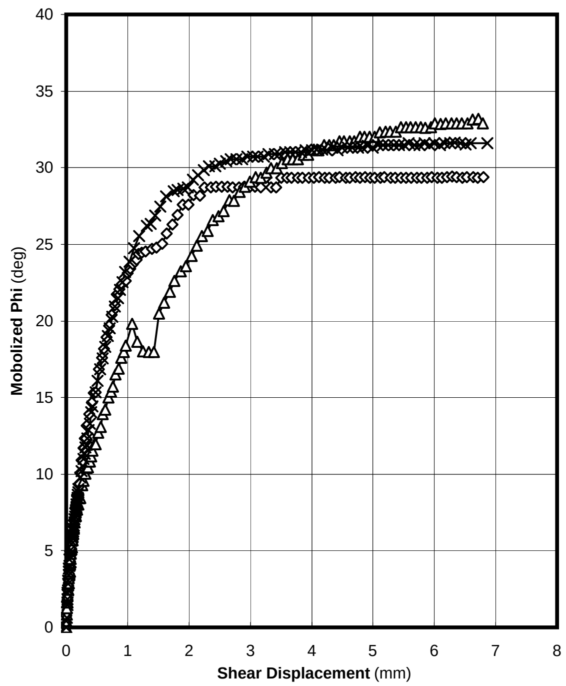


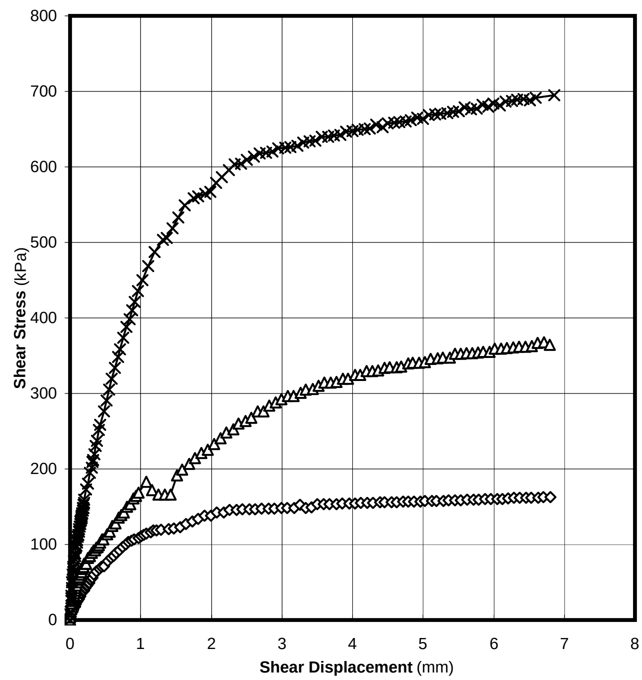
Direct Shear Test Summary

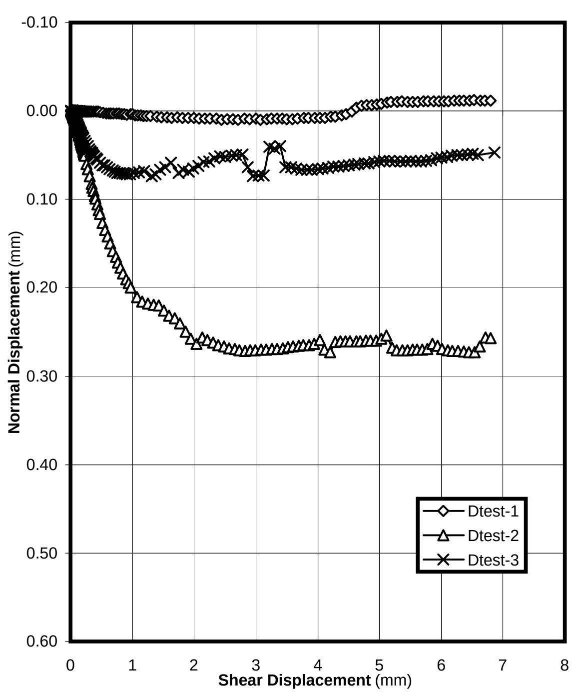
	Dtest-1	Dtest-2	Dtest-3	
$w_{initial}$	18.1	18.1	18.1	%
w_{final}	16.9	17.5	14.8	%
γ_{dry} initial	2015	2015	2015	Kg/M^3
γ_{dry} @ σ_n	2105	2247	2226	Kg/M^3
t_{50}	0.04	0.04	0.12	min
Feed Rate	0.016	0.016	0.016	mm/min
Peak Values @ Failure				
σ_n	273	535	1082	kPa
τ_{max}	162	334	734	kPa
Residual Values @ Failure				
σ_n	289	564	1129	kPa
$\tau_{residual}$	163	365	695	kPa

Proj #	04-1416-158	ID	Greens Creek Old Tailings Sample	Peak Test
Project	Project# M07801A40	Test Date	Oct-05-05	Remoulded Sample
Client	Klohn Crippen	Tech	HA/LL	Drained Test
Location	Burnaby			

Direct Shear Test of Soils Under Consolidated Drained Condtions
ASTM D3080-90







Direct Shear Test Summary				
	Dtest-1	Dtest-2	Dtest-3	
$w_{initial}$	18.1	18.1	18.1	%
w_{final}	16.9	17.5	14.8	%
γ_{dry} initial	2015	2015	2015	Kg/M^3
γ_{dry} @ σ_n	2105	2247	2226	Kg/M^3
t_{50}	0.04	0.04	0.12	min
Feed Rate	0.0160	0.0160	0.0160	mm/min
Peak Values @ Failure				
σ_n	273	535	1082	kPa
τ_{max}	162	334	734	kPa
Residual Values @ Failure				
σ_n	289	564	1129	kPa
$\tau_{residual}$	163	365	695	kPa

Proj #	**04-1416-158**	**ID**	**Greens Creek Old Tailings Sample**	**Residual Test**
Project	**Project# M07801A40**	**Test Date**	**Oct-05-05**	**Remoulded Sample**
Client	**Klohn Crippen**	**Tech**	**HA/LL**	**Drained Test**
Location	**Burnaby**			**Residual test after 5 manual pre-shears**

Direct Shear Test of Soils Under Consolidated Drained Conditions
ASTM D3080-90

									Dtest-1
Project#	04-1416-158		**Dimensions**			**Summary**	**Peak**	**Residual**	
Sch#	218		Length =	59.50	mm	σ_n =	273.1	289.3	kPa
Sample ID:			Width =	60.00	mm	τ_{max} =	161.7	162.8	kPa
Greens Creek Old Tailings Sample			Area =	35.70	cm²	**Phi =**	30.6	29.4	deg
						Feed Rate	0.016	0.016	mm/min
			Applied Normal Stress			**Remarks**			
$W_{(initial)}$ =	18.1	%	Hanger Load =	0.2322	kN	**- Remoulded sample**			
$W_{(final)}$ =	16.9	%	Plate Wt =	0.6824	kN	**- Drained Test**			
$\gamma_{dry\ (initial)}$ =	2015	Kg/M^3	σ_{Load} =	0.9146	kN	**- Residual Test after 5 manual pre-shears**			
$\gamma_{dry\ (ac)}$ =	2105	Kg/M^3	σ_n =	256.2	kPa				

Peak Test					Residual Test				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	**Normal Displacement** (mm)	**Mobolized Phi** (deg)	**Shear Displacement** (mm)	σ_n (kPa)	τ (kPa)	**Normal Displacement** (mm)	**Mobolized Phi** (deg)
0.00	256.2	0.0	0.000	0.0	0.000	256.2	0.0	0.000	0.0
0.00	256.2	0.0	0.000	0.0	0.004	256.2	3.6	0.000	0.8
0.00	256.2	7.4	0.000	1.7	0.012	256.3	7.4	0.000	1.7
0.01	256.2	14.0	0.002	3.1	0.021	256.3	11.0	0.000	2.5
0.02	256.3	21.6	0.004	4.8	0.026	256.3	10.8	-0.001	2.4
0.03	256.3	21.4	0.005	4.8	0.034	256.4	10.9	-0.001	2.4
0.04	256.4	25.2	0.008	5.6	0.041	256.4	13.9	0.000	3.1
0.05	256.4	28.3	0.010	6.3	0.048	256.4	13.8	0.000	3.1
0.07	256.5	28.3	0.015	6.3	0.054	256.4	17.6	-0.001	3.9
0.08	256.6	32.2	0.018	7.1	0.058	256.5	17.5	0.000	3.9
0.10	256.6	35.7	0.022	7.9	0.066	256.5	17.5	0.000	3.9
0.11	256.7	35.6	0.025	7.9	0.068	256.5	17.6	-0.001	3.9
0.14	256.8	39.1	0.032	8.7	0.074	256.5	21.3	0.000	4.7
0.16	256.9	42.5	0.035	9.4	0.076	256.5	21.5	0.000	4.8
0.17	257.0	42.3	0.040	9.3	0.081	256.6	21.4	0.000	4.8
0.18	257.0	46.1	0.042	10.2	0.085	256.6	21.3	0.000	4.8
0.20	257.1	46.0	0.046	10.1	0.091	256.6	25.1	0.000	5.6
0.22	257.1	49.9	0.049	11.0	0.095	256.6	24.9	0.000	5.5
0.25	257.3	53.4	0.054	11.7	0.102	256.6	25.0	0.000	5.6
0.27	257.4	56.5	0.058	12.4	0.106	256.7	25.1	0.000	5.6
0.30	257.5	56.6	0.064	12.4	0.113	256.7	27.9	-0.001	6.2
0.32	257.6	60.2	0.067	13.2	0.118	256.7	27.9	0.000	6.2
0.34	257.7	64.0	0.071	13.9	0.126	256.7	27.8	0.000	6.2
0.35	257.7	63.9	0.073	13.9	0.131	256.8	31.5	0.000	7.0
0.38	257.8	64.0	0.076	13.9	0.140	256.8	31.9	0.000	7.1
0.39	257.9	67.5	0.079	14.7	0.146	256.8	31.8	0.000	7.0
0.41	258.0	67.6	0.081	14.7	0.156	256.9	35.3	0.000	7.8
0.43	258.1	67.9	0.084	14.7	0.162	256.9	35.5	0.000	7.9
0.46	258.2	70.8	0.087	15.3	0.173	257.0	39.2	0.000	8.7
0.48	258.3	71.4	0.089	15.4	0.179	257.0	39.0	0.000	8.6
0.52	258.5	71.9	0.091	15.6	0.189	257.0	39.1	0.000	8.7
0.54	258.5	72.5	0.092	15.7	0.196	257.1	39.0	0.000	8.6
0.56	258.6	73.0	0.092	15.8	0.205	257.1	42.2	0.000	9.3
0.57	258.7	73.6	0.093	15.9	0.211	257.1	42.3	0.000	9.3
0.60	258.8	74.1	0.093	16.0	0.219	257.2	42.3	0.000	9.3
0.62	258.9	74.7	0.094	16.1	0.224	257.2	45.7	0.000	10.1
0.65	259.0	75.2	0.094	16.2	0.237	257.2	45.8	0.000	10.1
0.67	259.1	75.8	0.095	16.3	0.247	257.3	45.7	0.000	10.1
0.70	259.2	76.3	0.095	16.4	0.253	257.3	49.5	0.000	10.9
0.72	259.3	76.9	0.095	16.5	0.262	257.3	49.5	0.000	10.9
0.74	259.4	77.5	0.096	16.6	0.270	257.4	49.5	0.000	10.9

Direct Shear Test of Soils Under Consolidated Drained Conditions
ASTM D3080-90

Peak Test					Residual Test				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)	Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)
0.78	259.6	78.0	0.097	16.7	0.280	257.4	53.3	0.000	11.7
0.79	259.7	78.6	0.097	16.8	0.291	257.5	53.3	0.000	11.7
0.81	259.8	79.1	0.096	16.9	0.304	257.5	56.3	0.000	12.3
0.84	259.9	79.7	0.097	17.0	0.321	257.6	56.3	0.000	12.3
0.86	260.0	80.2	0.100	17.2	0.339	257.7	60.2	0.001	13.2
0.88	260.0	80.8	0.102	17.3	0.371	257.8	64.0	0.001	13.9
0.91	260.2	81.4	0.105	17.4	0.413	258.0	67.6	0.001	14.7
0.92	260.2	81.9	0.107	17.5	0.444	258.1	70.6	0.001	15.3
0.95	260.3	82.6	0.109	17.6	0.480	258.3	70.8	0.001	15.3
0.97	260.4	82.8	0.112	17.6	0.532	258.5	78.2	0.002	16.8
0.98	260.5	85.8	0.114	18.2	0.579	258.7	82.0	0.002	17.6
1.03	260.7	85.9	0.118	18.2	0.620	258.9	85.1	0.002	18.2
1.08	260.9	89.6	0.123	18.9	0.656	259.1	88.8	0.002	18.9
1.12	261.1	93.7	0.128	19.7	0.697	259.2	92.7	0.003	19.7
1.16	261.3	93.6	0.132	19.7	0.744	259.5	96.5	0.003	20.4
1.20	261.5	97.5	0.135	20.4	0.789	259.6	99.8	0.003	21.0
1.25	261.7	99.0	0.139	20.7	0.826	259.8	103.5	0.004	21.7
1.29	261.9	100.8	0.142	21.0	0.866	260.0	105.4	0.004	22.1
1.33	262.1	100.9	0.147	21.1	0.911	260.2	107.2	0.004	22.4
1.37	262.2	104.9	0.150	21.8	0.968	260.4	108.5	0.004	22.6
1.42	262.5	108.4	0.152	22.4	1.006	260.6	111.2	0.004	23.1
1.47	262.7	108.5	0.156	22.4	1.041	260.8	112.9	0.005	23.4
1.51	262.9	112.4	0.155	23.2	1.083	261.0	114.6	0.005	23.7
1.55	263.1	112.5	0.156	23.2	1.143	261.2	116.2	0.005	24.0
1.59	263.2	115.8	0.156	23.7	1.181	261.4	118.4	0.006	24.4
1.64	263.5	115.9	0.149	23.7	1.227	261.6	118.9	0.006	24.5
1.69	263.7	119.7	0.151	24.4	1.292	261.9	119.5	0.006	24.5
1.73	263.9	119.9	0.154	24.4	1.396	262.4	120.7	0.006	24.7
1.77	264.0	123.5	0.156	25.1	1.468	262.7	121.3	0.007	24.8
1.81	264.2	123.6	0.157	25.1	1.561	263.1	122.9	0.007	25.0
1.86	264.5	127.6	0.159	25.8	1.639	263.5	126.8	0.008	25.7
1.91	264.7	127.8	0.161	25.8	1.731	263.9	130.3	0.007	26.3
1.99	265.1	131.2	0.156	26.3	1.814	264.3	134.2	0.008	26.9
2.09	265.5	135.1	0.156	27.0	1.902	264.7	138.2	0.008	27.6
2.17	265.9	135.3	0.157	27.0	1.993	265.1	138.4	0.008	27.6
2.27	266.4	135.4	0.158	26.9	2.077	265.5	142.5	0.008	28.2
2.36	266.8	139.6	0.159	27.6	2.176	265.9	142.5	0.008	28.2
2.45	267.2	143.5	0.160	28.2	2.256	266.3	145.9	0.009	28.7
2.54	267.6	143.7	0.162	28.2	2.358	266.8	146.1	0.008	28.7
2.63	268.0	147.3	0.163	28.8	2.440	267.2	146.6	0.010	28.7
2.72	268.5	147.3	0.164	28.7	2.537	267.6	146.8	0.009	28.7
2.80	268.9	151.4	0.165	29.4	2.625	268.0	147.0	0.009	28.7
2.90	269.3	151.7	0.163	29.4	2.715	268.5	147.2	0.010	28.7
2.97	269.7	151.9	0.162	29.4	2.812	268.9	147.4	0.009	28.7
3.07	270.1	156.1	0.160	30.0	2.898	269.3	147.7	0.009	28.7
3.15	270.5	156.5	0.156	30.1	2.996	269.8	148.1	0.009	28.8
3.24	271.0	156.7	0.153	30.0	3.077	270.2	148.4	0.010	28.8
3.33	271.4	156.7	0.150	30.0	3.173	270.6	148.2	0.009	28.7
3.41	271.8	157.0	0.148	30.0	3.252	271.0	152.3	0.009	29.3
3.51	272.3	157.2	0.145	30.0	3.342	271.5	148.7	0.009	28.7
3.59	272.6	157.5	0.144	30.0	3.423	271.8	149.0	0.009	28.7
3.68	273.1	161.7	0.141	30.6	3.503	272.2	153.1	0.009	29.4
3.76	273.5	162.0	0.140	30.6	3.595	272.7	153.4	0.009	29.4
3.86	274.0	162.3	0.137	30.6	3.675	273.1	153.6	0.008	29.4
3.95	274.4	162.6	0.132	30.6	3.773	273.6	153.8	0.008	29.3

Direct Shear Test of Soils Under Consolidated Drained Conditions
ASTM D3080-90

Peak Test					Residual Test				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)	Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)
4.03	274.8	158.9	0.130	30.0	3.853	273.9	154.0	0.008	29.3
4.13	275.3	159.2	0.128	30.0	3.950	274.4	154.2	0.008	29.3
4.20	275.7	159.3	0.127	30.0	4.032	274.8	154.6	0.008	29.4
4.30	276.2	159.6	0.125	30.0	4.122	275.3	154.9	0.008	29.4
4.37	276.5	159.9	0.126	30.0	4.212	275.7	155.1	0.007	29.4
4.47	277.0	160.1	0.124	30.0	4.289	276.1	155.3	0.006	29.3
4.54	277.4	160.3	0.125	30.0	4.387	276.6	155.6	0.005	29.4
4.62	277.8	160.5	0.124	30.0	4.456	276.9	155.9	0.004	29.4
4.71	278.2	156.8	0.124	29.4	4.553	277.4	156.0	0.001	29.3
4.78	278.6	161.0	0.122	30.0	4.622	277.8	156.2	-0.003	29.3
4.88	279.1	157.3	0.121	29.4	4.712	278.2	156.6	-0.006	29.4
4.94	279.4	157.3	0.122	29.4	4.790	278.6	156.8	-0.006	29.4
5.04	279.9	157.7	0.120	29.4	4.870	279.0	157.0	-0.006	29.4
5.11	280.3	157.7	0.120	29.4	4.959	279.5	157.2	-0.008	29.4
5.21	280.8	158.1	0.118	29.4	5.028	279.9	157.4	-0.008	29.4
5.28	281.2	154.4	0.118	28.8	5.125	280.4	157.7	-0.009	29.4
5.37	281.6	154.6	0.117	28.8	5.188	280.7	157.9	-0.010	29.4
5.46	282.1	154.8	0.116	28.8	5.289	281.2	158.0	-0.010	29.3
5.54	282.5	154.8	0.115	28.7	5.360	281.6	158.3	-0.011	29.3
5.64	283.0	151.9	0.114	28.2	5.453	282.1	158.7	-0.010	29.4
5.71	283.4	152.3	0.114	28.3	5.536	282.5	158.9	-0.010	29.4
5.80	283.9	152.5	0.114	28.2	5.623	282.9	159.1	-0.010	29.3
5.87	284.3	152.6	0.114	28.2	5.716	283.4	159.3	-0.011	29.3
5.96	284.7	152.9	0.113	28.2	5.790	283.8	159.5	-0.011	29.3
6.04	285.2	153.3	0.114	28.3	5.882	284.3	159.8	-0.011	29.3
6.12	285.6	153.4	0.114	28.2	5.957	284.7	160.3	-0.011	29.4
6.21	286.0	153.9	0.114	28.3	6.046	285.2	160.4	-0.011	29.4
6.28	286.4	153.8	0.114	28.2	6.119	285.6	160.5	-0.011	29.3
6.38	287.0	154.2	0.115	28.3	6.207	286.0	161.0	-0.012	29.4
6.45	287.4	154.2	0.116	28.2	6.289	286.5	161.5	-0.012	29.4
6.54	287.9	154.5	0.116	28.2	6.366	286.9	161.6	-0.012	29.4
6.62	288.3	154.7	0.117	28.2	6.457	287.4	161.6	-0.012	29.4
6.71	288.8	155.0	0.117	28.2	6.534	287.8	162.0	-0.012	29.4
6.80	289.3	151.2	0.116	27.6	6.629	288.3	162.2	-0.012	29.4
6.88	289.7	151.5	0.117	27.6	6.704	288.7	162.4	-0.012	29.4
6.97	290.2	151.6	0.118	27.6	6.800	289.3	162.8	-0.012	29.4

Direct Shear Test of Soils Under Consolidated Drained Conditions
ASTM D3080-90

				Summary			**Dtest-2**
Project#	04-1416-158	**Dimensions**			Peak	Residual	
Sch#	218	Length =	59.50 mm	σ_n =	535.2	564.0	kPa
Sample ID:		Width =	60.00 mm	τ_{max} =	334.4	364.8	kPa
Greens Creek Old Tailings Sample		Area =	35.70 cm²	Phi =	32.0	32.9	deg
				Feed Rate	0.016	0.016	mm/min

		Applied Normal Stress			**Remarks**
$W_{(initial)}$ =	18.1 %	Hanger Load =	0.2322	kN	- Remoulded sample
$W_{(final)}$ =	17.5 %	Plate Wt =	1.5513	kN	- Drained Test
$\gamma_{dry\ (initial)}$ =	2015 Kg/M^3	σ_{Load} =	1.7834	kN	- Residual Test after 5 manual pre-shears
$\gamma_{dry\ (ac)}$ =	2247 Kg/M^3	σ_n =	499.6	kPa	

Peak Test					**Residual Test**				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)	Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)
0.00	499.6	0.0	0.000	0.0	0.000	499.6	0.0	0.000	0.0
0.00	499.6	6.8	0.000	0.8	0.002	499.6	3.7	0.000	0.4
0.01	499.7	14.0	0.001	1.6	0.007	499.7	7.3	0.000	0.8
0.01	499.7	24.4	0.001	2.8	0.010	499.7	10.9	0.000	1.3
0.02	499.7	31.7	0.001	3.6	0.013	499.7	14.1	0.001	1.6
0.02	499.8	35.0	0.003	4.0	0.016	499.7	14.3	0.001	1.6
0.02	499.8	45.8	0.001	5.2	0.021	499.8	17.8	0.002	2.0
0.03	499.8	52.6	0.001	6.0	0.023	499.8	18.0	0.003	2.1
0.03	499.9	59.8	0.002	6.8	0.028	499.8	21.6	0.003	2.5
0.04	499.9	63.0	0.002	7.2	0.032	499.9	21.6	0.005	2.5
0.04	499.9	70.3	0.003	8.0	0.037	499.9	25.3	0.006	2.9
0.04	500.0	74.0	0.003	8.4	0.040	499.9	25.2	0.006	2.9
0.05	500.0	76.9	0.004	8.7	0.047	500.0	28.3	0.007	3.2
0.05	500.0	80.6	0.005	9.2	0.050	500.0	30.8	0.009	3.5
0.06	500.1	84.2	0.006	9.6	0.058	500.1	31.9	0.010	3.6
0.06	500.1	88.1	0.008	10.0	0.062	500.1	35.7	0.010	4.1
0.07	500.2	91.0	0.008	10.3	0.071	500.2	39.2	0.012	4.5
0.07	500.2	91.1	0.009	10.3	0.077	500.3	42.2	0.013	4.8
0.07	500.2	94.9	0.011	10.7	0.087	500.3	46.1	0.016	5.3
0.08	500.3	94.9	0.012	10.7	0.094	500.4	45.9	0.017	5.2
0.09	500.3	98.6	0.015	11.1	0.103	500.5	49.7	0.019	5.7
0.09	500.4	98.5	0.017	11.1	0.110	500.5	53.4	0.021	6.1
0.10	500.4	102.3	0.018	11.6	0.119	500.6	56.3	0.023	6.4
0.10	500.5	102.3	0.020	11.5	0.125	500.7	56.5	0.024	6.4
0.11	500.6	105.2	0.023	11.9	0.134	500.7	60.0	0.026	6.8
0.13	500.7	109.1	0.026	12.3	0.139	500.8	60.4	0.027	6.9
0.16	500.9	112.8	0.035	12.7	0.147	500.8	63.9	0.029	7.3
0.18	501.1	116.4	0.042	13.1	0.152	500.9	63.9	0.030	7.3
0.20	501.3	119.4	0.049	13.4	0.159	500.9	63.9	0.033	7.3
0.21	501.4	119.6	0.053	13.4	0.164	501.0	67.6	0.033	7.7
0.23	501.6	123.5	0.060	13.8	0.170	501.0	67.6	0.035	7.7
0.25	501.7	126.9	0.064	14.2	0.174	501.1	67.6	0.037	7.7
0.27	501.9	127.1	0.070	14.2	0.180	501.1	70.7	0.038	8.0
0.28	502.0	130.7	0.075	14.6	0.183	501.2	70.8	0.039	8.0
0.31	502.3	133.8	0.081	14.9	0.189	501.2	70.6	0.040	8.0
0.33	502.4	137.4	0.085	15.3	0.194	501.2	70.5	0.042	8.0
0.36	502.7	141.5	0.092	15.7	0.203	501.3	74.3	0.044	8.4
0.38	502.8	145.1	0.096	16.1	0.208	501.4	74.3	0.046	8.4
0.40	503.0	145.2	0.102	16.1	0.216	501.4	74.3	0.048	8.4
0.41	503.1	148.2	0.105	16.4	0.220	501.5	74.3	0.049	8.4
0.43	503.3	151.9	0.109	16.8	0.225	501.5	74.5	0.050	8.5

Direct Shear Test of Soils Under Consolidated Drained Conditions
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Peak Test					Residual Test				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)	Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)
0.47	503.6	155.8	0.117	17.2	0.258	501.8	81.9	0.060	9.3
0.49	503.7	159.5	0.120	17.6	0.279	502.0	84.8	0.065	9.6
0.52	504.0	162.7	0.125	17.9	0.308	502.2	88.8	0.074	10.0
0.55	504.3	166.5	0.131	18.3	0.338	502.5	92.4	0.082	10.4
0.57	504.4	170.3	0.133	18.7	0.354	502.6	92.5	0.087	10.4
0.58	504.5	170.4	0.136	18.7	0.369	502.7	96.1	0.090	10.8
0.60	504.7	170.6	0.139	18.7	0.390	502.9	96.3	0.096	10.8
0.61	504.8	174.0	0.141	19.0	0.406	503.0	99.1	0.099	11.1
0.63	504.9	177.1	0.144	19.3	0.430	503.2	102.9	0.106	11.6
0.69	505.5	184.6	0.152	20.1	0.458	503.5	106.7	0.112	12.0
0.73	505.8	188.6	0.159	20.4	0.476	503.6	106.8	0.116	12.0
0.80	506.4	195.7	0.169	21.1	0.522	504.0	113.5	0.126	12.7
0.89	507.2	203.6	0.180	21.9	0.561	504.4	117.3	0.135	13.1
0.97	507.9	210.3	0.191	22.5	0.598	504.7	125.0	0.142	13.9
1.06	508.6	218.2	0.198	23.2	0.640	505.0	127.9	0.150	14.2
1.15	509.4	225.4	0.208	23.9	0.687	505.4	135.6	0.158	15.0
1.23	510.1	233.2	0.230	24.6	0.730	505.8	139.3	0.165	15.4
1.31	510.9	240.3	0.238	25.2	0.765	506.1	142.5	0.171	15.7
1.39	511.5	244.4	0.244	25.5	0.805	506.5	150.1	0.177	16.5
1.47	512.3	251.6	0.243	26.2	0.852	506.9	153.8	0.184	16.9
1.55	513.0	255.8	0.248	26.5	0.898	507.3	160.7	0.190	17.6
1.64	513.7	263.5	0.252	27.2	0.938	507.6	164.6	0.195	18.0
1.72	514.5	267.3	0.256	27.5	0.974	507.9	168.7	0.200	18.4
1.80	515.2	271.3	0.259	27.8	1.078	508.8	183.4	0.210	19.8
1.89	516.0	275.5	0.264	28.1	1.160	509.5	172.1	0.216	18.7
1.97	516.8	279.5	0.267	28.4	1.256	510.4	165.8	0.218	18.0
2.07	517.6	283.2	0.270	28.7	1.342	511.1	165.7	0.219	18.0
2.15	518.3	287.4	0.274	29.0	1.428	511.9	166.2	0.220	18.0
2.24	519.2	291.8	0.276	29.3	1.514	512.7	191.5	0.226	20.5
2.33	519.9	295.8	0.279	29.6	1.591	513.3	199.1	0.232	21.2
2.42	520.8	299.5	0.282	29.9	1.688	514.2	206.6	0.234	21.9
2.51	521.6	300.1	0.285	29.9	1.765	514.9	214.3	0.240	22.6
2.60	522.4	304.2	0.287	30.2	1.864	515.8	221.3	0.250	23.2
2.70	523.3	308.5	0.289	30.5	1.946	516.5	225.5	0.258	23.6
2.78	524.1	309.2	0.291	30.5	2.039	517.3	233.0	0.263	24.2
2.88	525.0	313.6	0.293	30.8	2.131	518.2	240.8	0.256	24.9
2.97	525.8	314.0	0.295	30.8	2.214	518.9	248.1	0.259	25.6
3.06	526.7	317.6	0.297	31.1	2.308	519.8	252.2	0.262	25.9
3.15	527.6	318.1	0.299	31.1	2.388	520.5	260.2	0.264	26.6
3.24	528.4	318.7	0.302	31.1	2.484	521.4	264.0	0.266	26.9
3.34	529.3	319.4	0.302	31.1	2.563	522.1	268.0	0.268	27.2
3.42	530.1	323.6	0.304	31.4	2.658	523.0	276.5	0.269	27.9
3.52	531.0	324.3	0.306	31.4	2.737	523.7	276.5	0.270	27.8
3.60	531.8	324.6	0.308	31.4	2.825	524.5	284.1	0.271	28.4
3.69	532.7	329.2	0.310	31.7	2.912	525.3	288.2	0.270	28.8
3.78	533.5	329.6	0.312	31.7	2.994	526.1	292.6	0.270	29.1
3.87	534.3	330.1	0.313	31.7	3.088	527.0	296.5	0.270	29.4
3.96	535.2	334.4	0.315	32.0	3.168	527.7	296.9	0.270	29.4
4.04	536.0	335.1	0.316	32.0	3.263	528.6	301.1	0.269	29.7
4.14	536.9	335.7	0.317	32.0	3.341	529.3	305.2	0.269	30.0
4.22	537.7	336.0	0.317	32.0	3.434	530.2	305.7	0.268	30.0
4.31	538.7	336.8	0.317	32.0	3.518	531.0	310.3	0.267	30.3
4.39	539.4	337.3	0.318	32.0	3.604	531.8	314.0	0.266	30.6
4.49	540.4	337.7	0.317	32.0	3.693	532.7	314.4	0.266	30.5
4.57	541.2	334.4	0.318	31.7	3.771	533.4	314.9	0.265	30.6

Direct Shear Test of Soils Under Consolidated Drained Conditions
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Peak Test					Residual Test				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)	Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)
4.66	542.1	339.0	0.318	32.0	3.865	534.3	319.3	0.264	30.9
4.75	543.0	339.5	0.323	32.0	3.939	535.0	319.8	0.263	30.9
4.83	543.8	340.0	0.327	32.0	4.035	536.0	324.2	0.259	31.2
4.93	544.7	340.6	0.328	32.0	4.112	536.7	324.5	0.270	31.2
5.00	545.5	341.3	0.329	32.0	4.201	537.6	329.3	0.273	31.5
5.10	546.4	341.8	0.329	32.0	4.284	538.4	329.6	0.261	31.5
5.17	547.1	342.1	0.330	32.0	4.363	539.1	330.1	0.261	31.5
5.26	548.1	338.7	0.330	31.7	4.455	540.0	333.8	0.261	31.7
5.34	548.9	339.1	0.331	31.7	4.526	540.7	334.5	0.261	31.7
5.42	549.7	339.9	0.332	31.7	4.624	541.7	334.9	0.260	31.7
5.51	550.6	336.1	0.332	31.4	4.689	542.3	335.3	0.260	31.7
5.58	551.3	340.7	0.335	31.7	4.786	543.3	339.9	0.260	32.0
5.68	552.3	341.3	0.335	31.7	4.857	544.0	340.3	0.260	32.0
5.74	552.9	341.7	0.336	31.7	4.942	544.9	340.7	0.260	32.0
5.84	554.0	342.3	0.336	31.7	5.027	545.7	341.3	0.258	32.0
5.91	554.7	342.8	0.337	31.7	5.109	546.5	345.8	0.254	32.3
6.00	555.7	343.4	0.337	31.7	5.207	547.5	346.6	0.268	32.3
6.08	556.5	343.7	0.337	31.7	5.283	548.3	347.1	0.270	32.3
6.16	557.3	344.3	0.325	31.7	5.374	549.2	347.7	0.270	32.3
6.24	558.2	341.0	0.324	31.4	5.450	550.0	352.2	0.270	32.6
6.31	558.9	345.5	0.325	31.7	5.537	550.9	352.9	0.270	32.6
6.40	559.8	341.9	0.324	31.4	5.612	551.6	353.2	0.270	32.6
6.47	560.5	342.2	0.324	31.4	5.694	552.5	353.8	0.270	32.6
6.55	561.4	342.8	0.324	31.4	5.776	553.3	354.4	0.269	32.6
6.62	562.2	346.7	0.325	31.7	5.853	554.1	354.7	0.263	32.6
6.71	563.1	343.9	0.326	31.4	5.940	555.0	355.3	0.266	32.6
6.79	564.0	344.3	0.327	31.4	6.013	555.8	359.3	0.269	32.9
6.87	564.8	348.9	0.329	31.7	6.105	556.7	359.7	0.270	32.9
6.96	565.8	349.5	0.331	31.7	6.182	557.5	360.3	0.271	32.9
7.04	566.6	350.3	0.332	31.7	6.275	558.5	361.0	0.271	32.9
7.14	567.7	350.6	0.334	31.7	6.360	559.4	361.6	0.272	32.9
7.21	568.5	351.3	0.337	31.7	6.447	560.3	362.2	0.272	32.9
7.32	569.6	352.0	0.343	31.7	6.539	561.3	362.8	0.272	32.9
7.40	570.5	352.6	0.346	31.7	6.620	562.2	367.4	0.266	33.2
7.49	571.6	353.3	0.348	31.7	6.716	563.2	368.1	0.256	33.2
7.62	573.0	354.2	0.357	31.7	6.795	564.0	364.8	0.257	32.9

Direct Shear Test of Soils Under Consolidated Drained Conditions
ASTM D3080-90

									Dtest-3
Project#	04-1416-158		Dimensions			Summary	Peak	Residual	
Sch#	218		Length =	59.50	mm	σ_n =	1082.0	1129.5	kPa
Sample ID:			Width =	60.00	mm	τ_{max} =	734.2	695.2	kPa
Greens Creek Old Tailings Sample			Area =	35.70	cm²	Phi =	34.2	31.6	deg
						Feed Rate	0.016	0.016	mm/min
			Applied Normal Stress			Remarks			
$W_{(initial)}$ =	18.1	%	Hanger Load =	0.2322	kN	- Remoulded sample			
$W_{(final)}$ =	14.8	%	Plate Wt =	3.3349	kN	- Drained Test			
$\gamma_{dry\ (initial)}$ =	2015	Kg/M^3	σ_{Load} =	3.5671	kN	- Residual Test after 5 manual pre-shears			
$\gamma_{dry\ (ac)}$ =	2226	Kg/M^3	σ_n =	999.3	kPa				

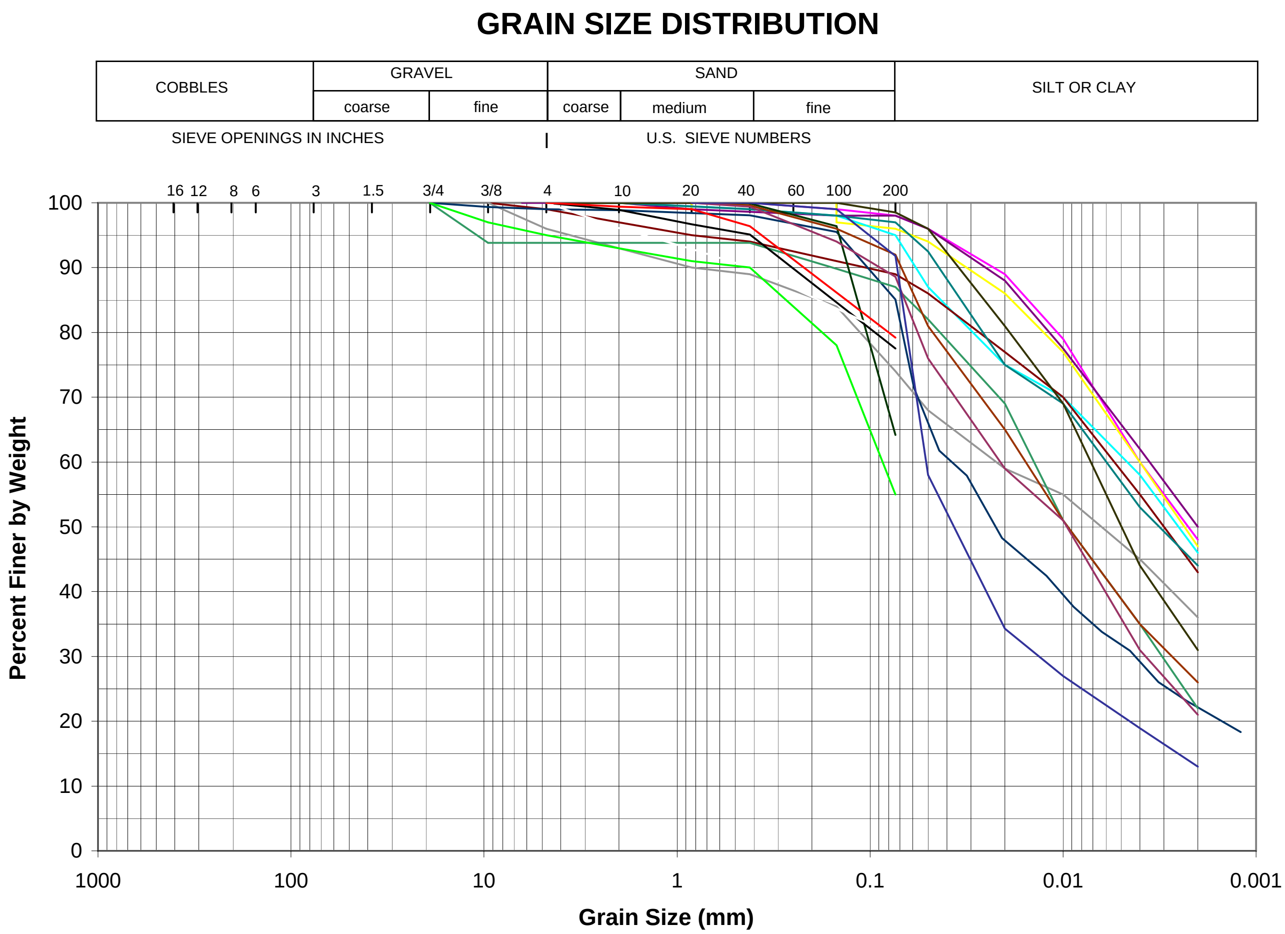
Peak Test					Residual Test				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)	Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)
0.00	999.3	0.0	0.000	0.0	0.000	999.3	0.0	0.000	0.0
0.00	999.3	17.9	0.000	1.0	0.002	999.3	7.5	0.000	0.4
0.00	999.3	21.8	0.001	1.2	0.007	999.4	11.1	0.000	0.6
0.01	999.4	28.3	0.001	1.6	0.017	999.5	25.4	0.000	1.5
0.01	999.5	35.6	0.001	2.0	0.019	999.6	28.5	0.001	1.6
0.01	999.5	39.3	0.001	2.2	0.021	999.6	32.1	0.001	1.8
0.02	999.6	49.8	0.002	2.9	0.025	999.7	39.4	0.001	2.3
0.02	999.6	53.5	0.002	3.1	0.027	999.7	42.2	0.001	2.4
0.03	999.7	63.7	0.002	3.6	0.031	999.8	49.6	0.001	2.8
0.03	999.7	67.5	0.002	3.9	0.034	999.8	53.4	0.002	3.1
0.03	999.8	74.1	0.002	4.2	0.038	999.9	60.2	0.002	3.4
0.03	999.8	78.1	0.002	4.5	0.041	1000.0	63.7	0.003	3.6
0.04	999.9	84.6	0.008	4.8	0.045	1000.0	67.5	0.003	3.9
0.04	999.9	88.3	0.009	5.0	0.049	1000.1	70.4	0.004	4.0
0.04	1000.0	95.6	0.009	5.5	0.056	1000.2	77.8	0.005	4.4
0.04	1000.0	102.5	0.009	5.9	0.061	1000.3	81.4	0.006	4.7
0.05	1000.1	109.7	0.010	6.3	0.068	1000.4	84.5	0.007	4.8
0.05	1000.1	116.6	0.010	6.7	0.074	1000.5	88.4	0.008	5.0
0.06	1000.2	123.9	0.010	7.1	0.082	1000.7	95.5	0.011	5.5
0.06	1000.2	126.8	0.011	7.2	0.088	1000.7	98.7	0.011	5.6
0.06	1000.3	134.3	0.011	7.6	0.096	1000.9	102.5	0.013	5.8
0.07	1000.4	140.7	0.012	8.0	0.103	1001.0	106.1	0.014	6.1
0.07	1000.5	148.2	0.013	8.4	0.112	1001.1	109.9	0.016	6.3
0.08	1000.6	152.0	0.015	8.6	0.117	1001.2	113.0	0.017	6.4
0.09	1000.7	158.6	0.016	9.0	0.125	1001.4	116.6	0.018	6.6
0.09	1000.8	166.1	0.017	9.4	0.131	1001.5	120.1	0.019	6.8
0.10	1001.0	169.1	0.019	9.6	0.138	1001.6	124.1	0.021	7.1
0.11	1001.0	173.0	0.021	9.8	0.143	1001.7	126.8	0.022	7.2
0.11	1001.2	180.2	0.022	10.2	0.150	1001.8	130.7	0.022	7.4
0.12	1001.3	183.4	0.023	10.4	0.154	1001.9	132.0	0.023	7.5
0.13	1001.4	190.6	0.025	10.8	0.159	1001.9	134.6	0.024	7.7
0.13	1001.5	194.5	0.027	11.0	0.162	1002.0	136.6	0.024	7.8
0.14	1001.6	197.5	0.029	11.2	0.167	1002.1	138.3	0.026	7.9
0.15	1001.7	201.2	0.030	11.4	0.170	1002.1	141.2	0.026	8.0
0.15	1001.9	204.8	0.032	11.6	0.175	1002.2	141.1	0.027	8.0
0.16	1001.9	208.8	0.034	11.8	0.178	1002.3	145.1	0.027	8.2
0.17	1002.1	211.5	0.035	11.9	0.183	1002.4	148.4	0.028	8.4
0.17	1002.1	215.2	0.037	12.1	0.187	1002.4	148.5	0.028	8.4
0.18	1002.2	218.9	0.039	12.3	0.193	1002.5	152.3	0.029	8.6
0.18	1002.3	218.9	0.039	12.3	0.197	1002.6	155.2	0.030	8.8
0.19	1002.4	222.6	0.040	12.5	0.203	1002.7	159.0	0.031	9.0

Direct Shear Test of Soils Under Consolidated Drained Conditions
ASTM D3080-90

Peak Test					Residual Test				
Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)	Shear Displacement (mm)	σ_n (kPa)	τ (kPa)	Normal Displacement (mm)	Mobolized Phi (deg)
0.20	1002.6	229.6	0.043	12.9	0.231	1003.2	173.2	0.034	9.8
0.20	1002.7	229.4	0.045	12.9	0.251	1003.5	180.6	0.037	10.2
0.21	1002.8	236.9	0.047	13.3	0.279	1004.0	194.8	0.040	11.0
0.22	1002.9	236.7	0.049	13.3	0.304	1004.4	201.5	0.044	11.3
0.22	1003.0	239.8	0.050	13.4	0.318	1004.6	209.2	0.045	11.8
0.25	1003.4	250.8	0.056	14.0	0.330	1004.8	212.1	0.047	11.9
0.29	1004.2	268.3	0.068	15.0	0.347	1005.1	219.5	0.048	12.3
0.33	1004.9	279.7	0.079	15.6	0.362	1005.4	230.1	0.050	12.9
0.37	1005.4	289.9	0.088	16.1	0.382	1005.7	237.4	0.051	13.3
0.40	1006.1	304.3	0.096	16.8	0.412	1006.2	251.6	0.053	14.0
0.45	1006.8	314.9	0.106	17.4	0.430	1006.5	258.5	0.055	14.4
0.49	1007.6	325.6	0.117	17.9	0.478	1007.3	276.5	0.059	15.4
0.53	1008.3	336.9	0.127	18.5	0.515	1008.0	290.7	0.061	16.1
0.56	1008.8	343.9	0.137	18.8	0.551	1008.6	305.0	0.062	16.8
0.61	1009.6	354.6	0.145	19.4	0.591	1009.3	319.5	0.064	17.6
0.65	1010.4	366.1	0.153	19.9	0.636	1010.1	333.8	0.066	18.3
0.70	1011.1	372.9	0.162	20.2	0.679	1010.8	348.2	0.067	19.0
0.73	1011.7	380.7	0.169	20.6	0.712	1011.4	358.9	0.069	19.5
0.77	1012.4	388.1	0.177	21.0	0.751	1012.0	373.5	0.070	20.3
0.82	1013.2	398.6	0.184	21.5	0.795	1012.8	387.9	0.070	21.0
0.87	1014.0	406.4	0.193	21.8	0.841	1013.6	398.7	0.071	21.5
0.91	1014.8	413.5	0.200	22.2	0.880	1014.3	410.3	0.071	22.0
0.94	1015.3	421.3	0.207	22.5	0.915	1014.9	420.9	0.071	22.5
0.99	1016.1	428.7	0.214	22.9	0.957	1015.6	435.3	0.071	23.2
1.03	1017.0	436.3	0.221	23.2	1.029	1016.9	450.3	0.070	23.9
1.08	1017.8	443.7	0.229	23.6	1.104	1018.2	469.0	0.069	24.7
1.12	1018.4	447.7	0.238	23.7	1.194	1019.7	487.5	0.068	25.6
1.16	1019.1	455.4	0.244	24.1	1.313	1021.8	503.3	0.074	26.2
1.20	1019.9	462.6	0.251	24.4	1.374	1022.9	506.2	0.072	26.3
1.29	1021.4	473.9	0.264	24.9	1.454	1024.3	518.7	0.067	26.9
1.36	1022.7	485.8	0.275	25.4	1.533	1025.7	533.2	0.063	27.5
1.46	1024.4	497.4	0.287	25.9	1.627	1027.4	549.1	0.059	28.1
1.54	1025.8	508.4	0.302	26.4	1.749	1029.5	558.4	0.070	28.5
1.64	1027.6	520.1	0.306	26.8	1.817	1030.7	561.1	0.067	28.6
1.72	1029.0	528.5	0.315	27.2	1.912	1032.4	564.0	0.069	28.6
1.81	1030.7	540.2	0.335	27.7	1.983	1033.7	566.7	0.065	28.7
1.90	1032.3	548.3	0.349	28.0	2.066	1035.2	578.8	0.062	29.2

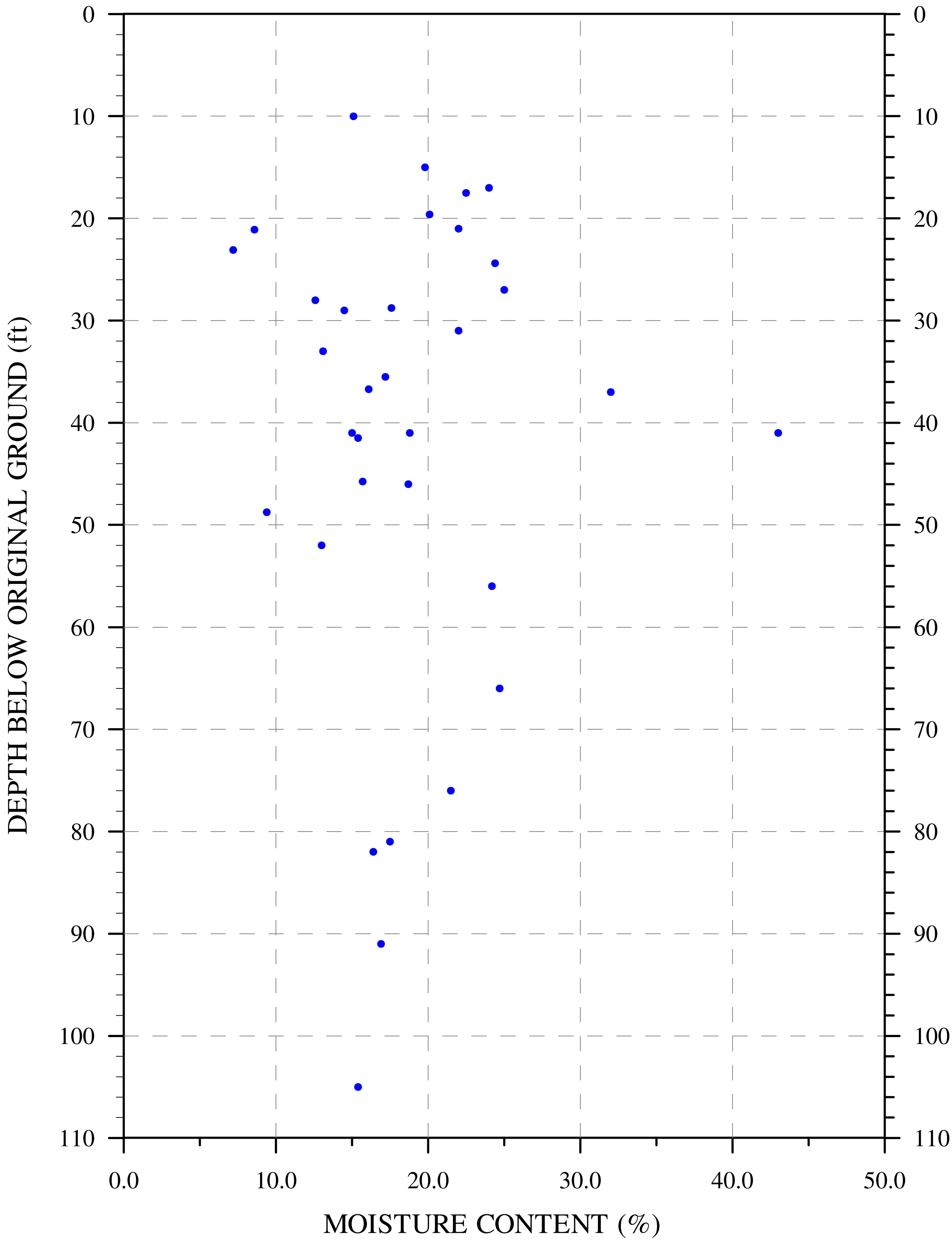
APPENDIX VII

Laboratory Test Data Summary



KLOHN CRIPPEN

PROJECT No.:	M07802A41
PROJECT:	Stage 2 Tailings Expansion Overall Stability Update
SUBJECT:	Figure VII-1 - Silt/Clay Gradations
DATE:	24-Jan-06
DRAWN BY:	Rick Friedel
CHECKED BY:	



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CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

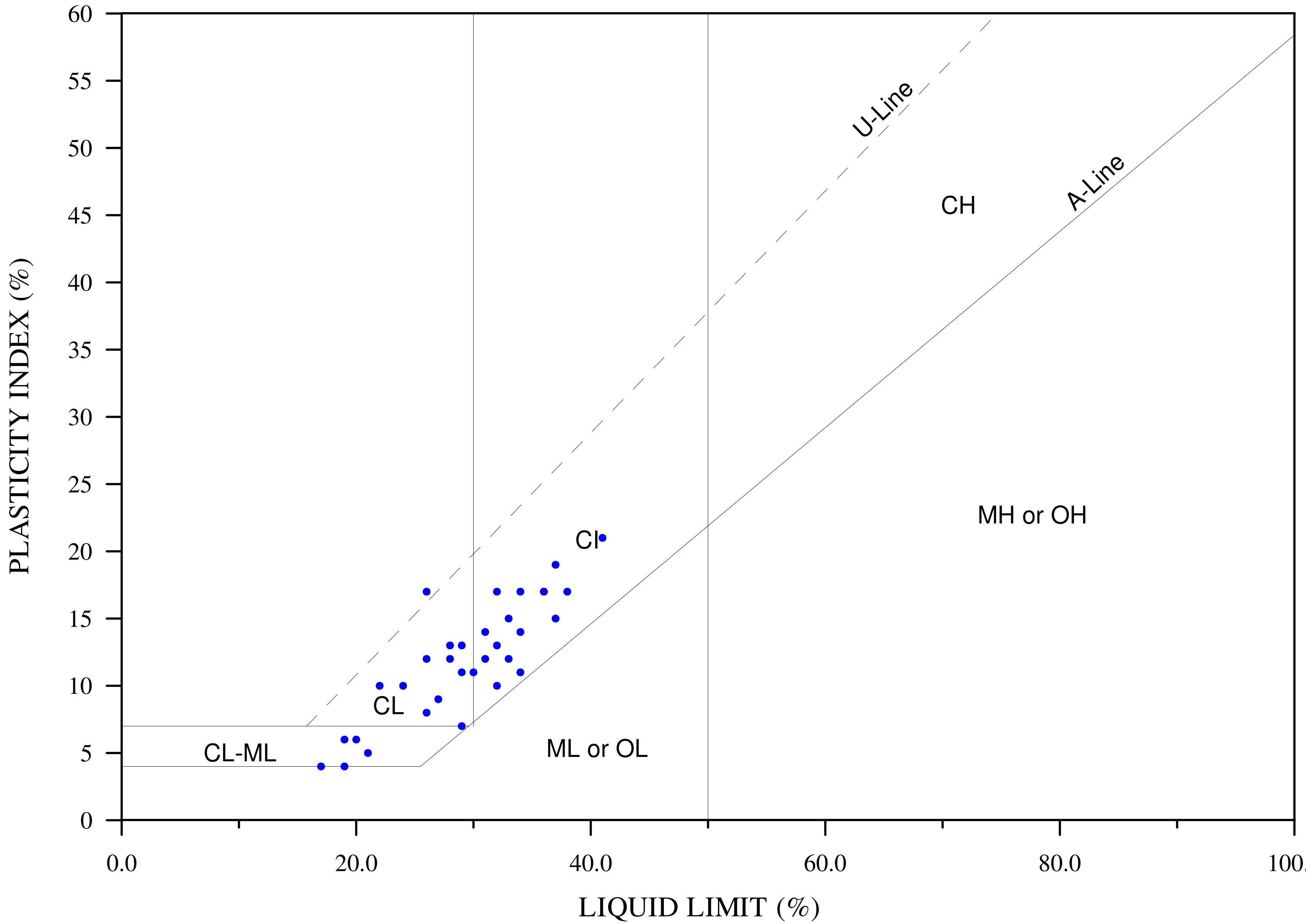
KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION
OVERALL STABILITY UPDATE

TITLE
SILT/CLAY -
MOISTURE CONTENT VS. DEPTH

PROJECT NO. M07802A41

FIGURE NO. VII-2



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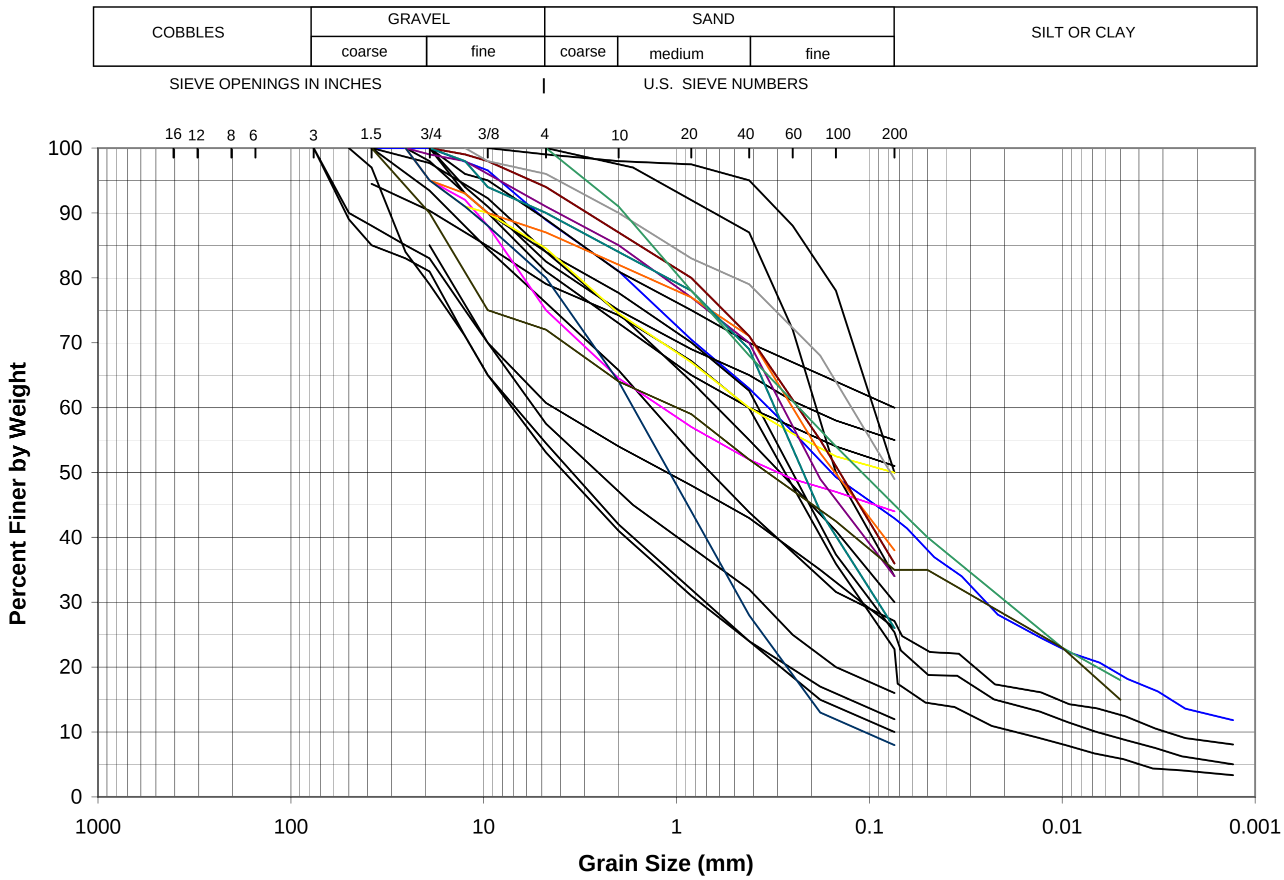
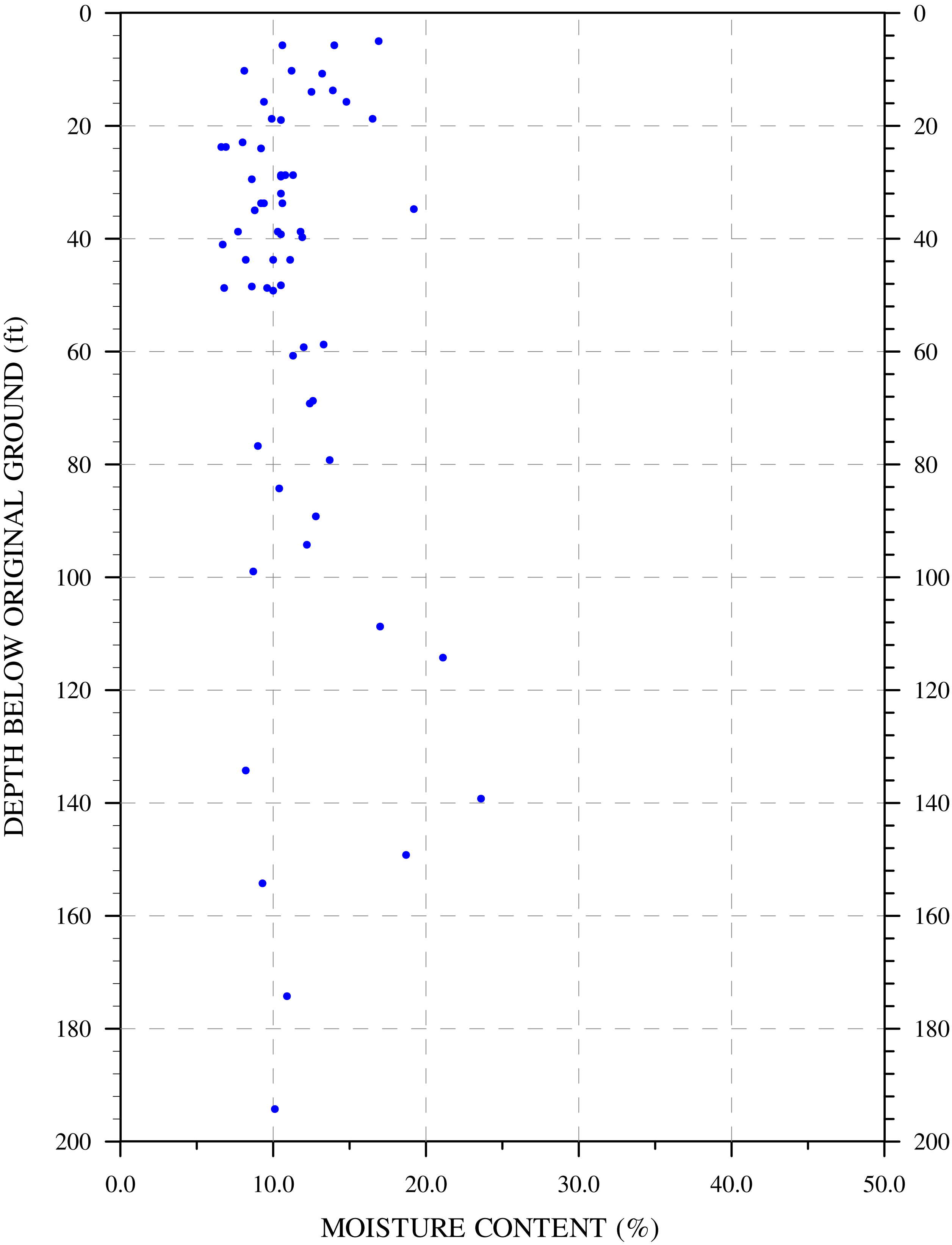
CLIENT KENNECOTT GREENS CREEK MINING COMPANY	PROJECT STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE	
KLOHN CRIPPEN	TITLE SILT/CLAY - PLASTICITY CHART	
	PROJECT NO. M07802A41	FIGURE NO. VII-3

GRAIN SIZE DISTRIBUTION



KLOHN CRIPPEN

PROJECT No.:	**M07802A41**
PROJECT:	**Stage 2 Tailings Expansion Overall Stability Update**
SUBJECT:	**Figure VII-4 - Silty Sand (Till) Gradations**
DATE:	24-Jan-06
DRAWN BY:	Rick Friedel **CHECKED BY:**



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KENNECOTT GREENS CREEK MINING COMPANY

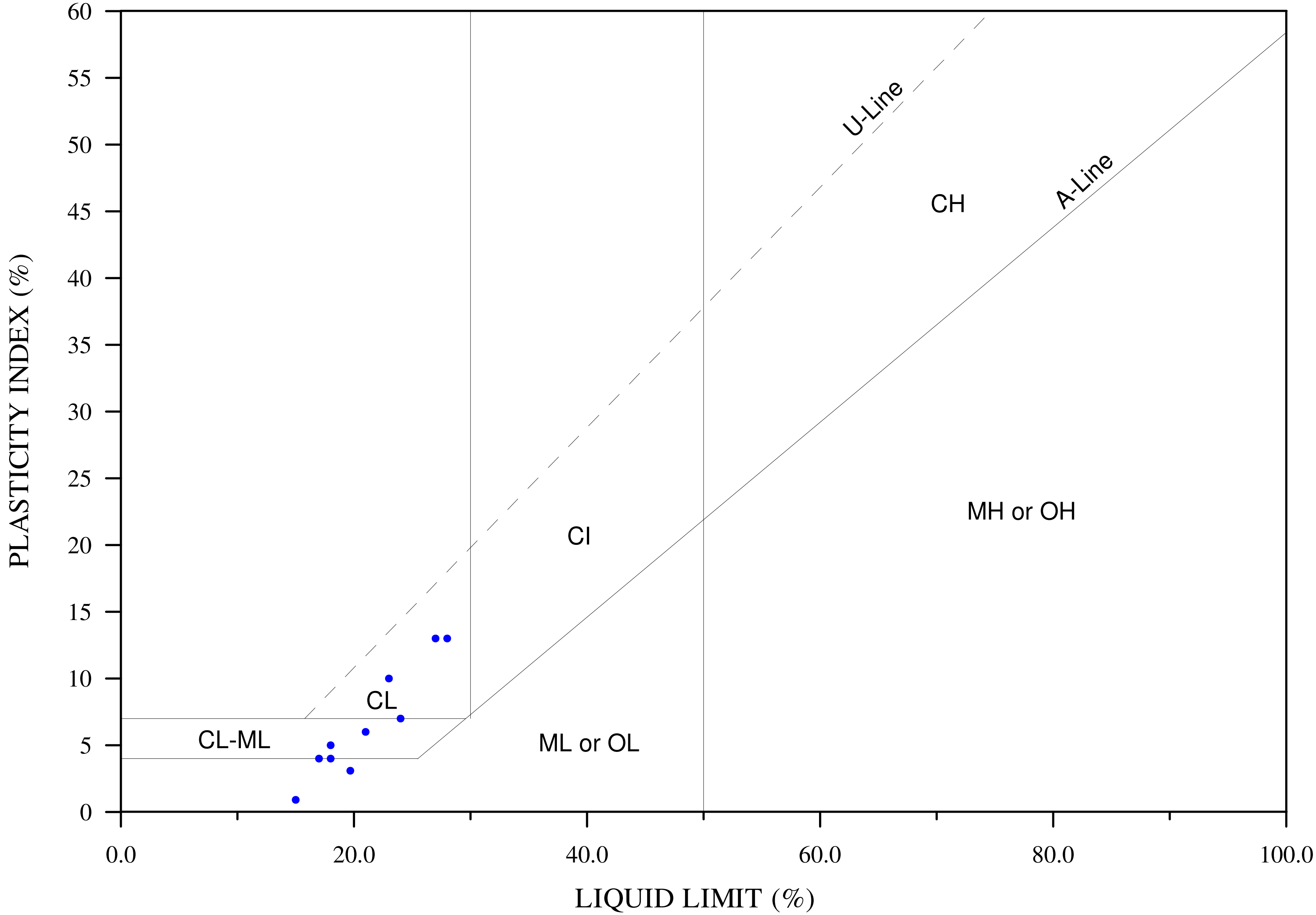
KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE
SILTY SAND - MOISTURE CONTENT VS. DEPTH

PROJECT NO. M07802A41

FIGURE NO. VII-5



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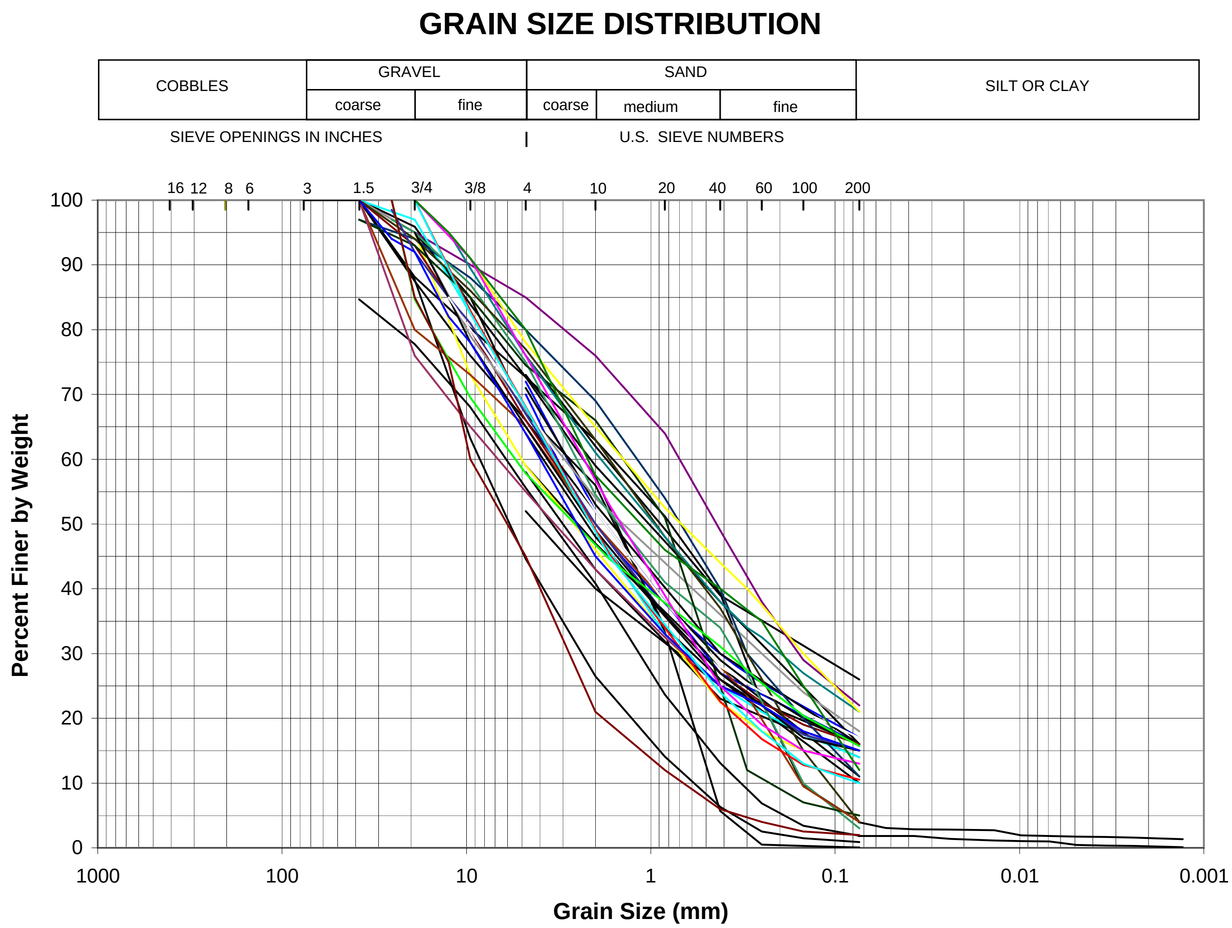
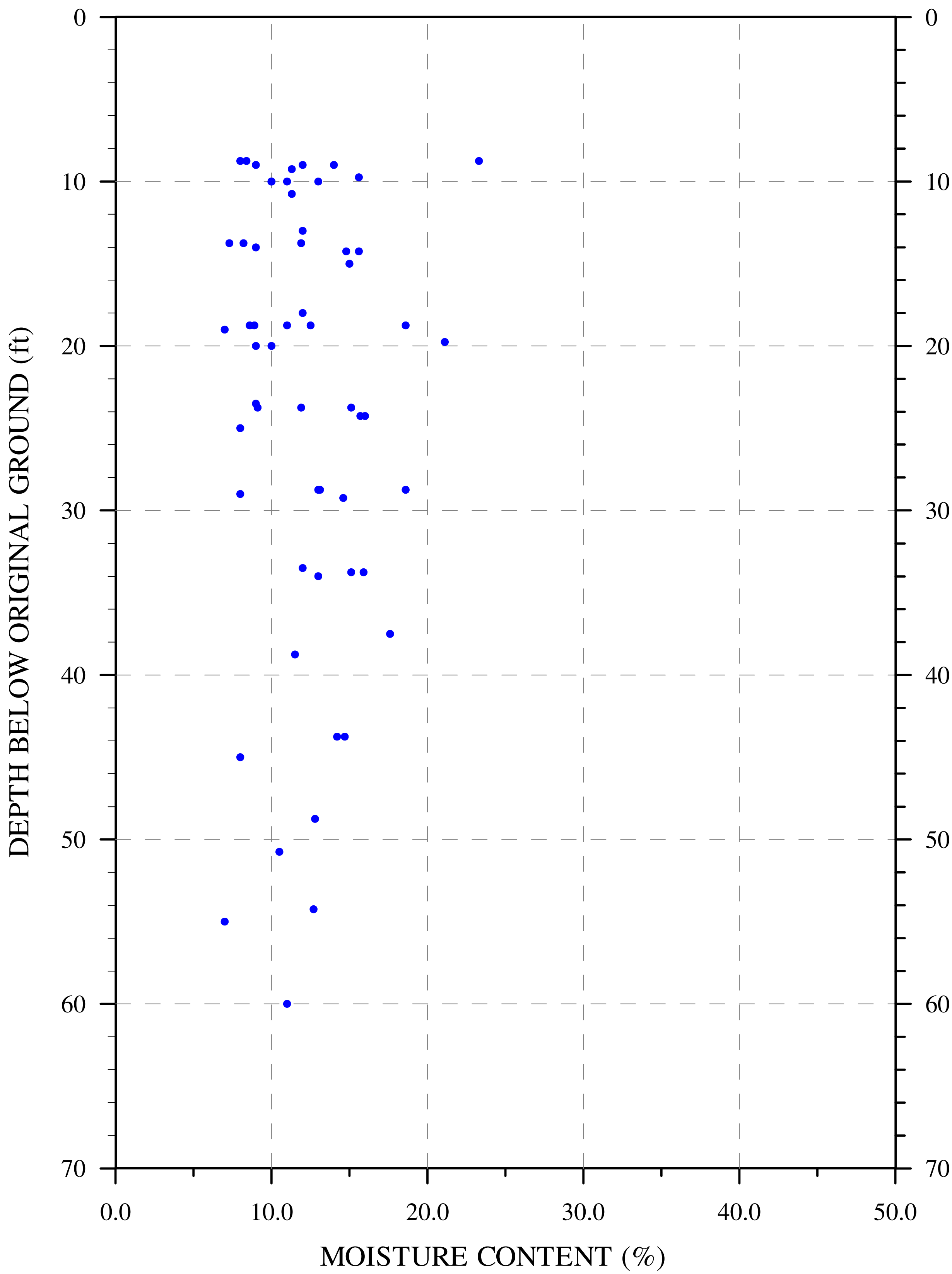
CLIENT	PROJECT
KENNECOTT GREENS CREEK MINING COMPANY	STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE
KLOHN CRIPPEN	TITLE: SILTY SAND - PLASTICITY CHART
	PROJECT NO. M07802A41 — FIGURE NO. VII-6



KLOHN CRIPPEN	**PROJECT No.:** M07802A41 0101
	PROJECT: **Stage 2 Tailings Expansion Overall Stability Update**
	SUBJECT: **Figure VII-7 - Sand and Gravel Gradations**
	DATE: 24-Jan-06
	DRAWN BY: Rick Friedel **CHECKED BY:**



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CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

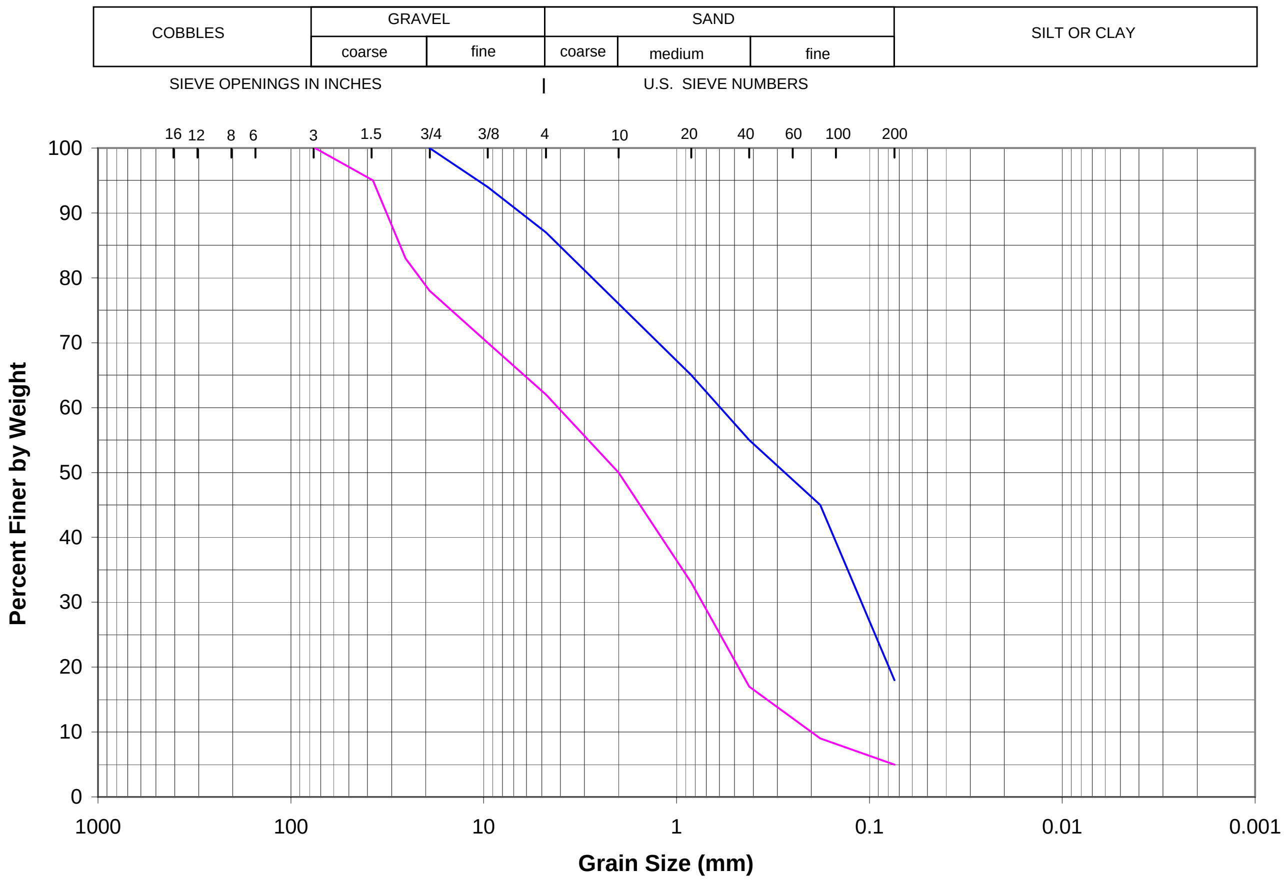
PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: SAND AND GRAVEL - MOISTURE CONTENT VS. DEPTH

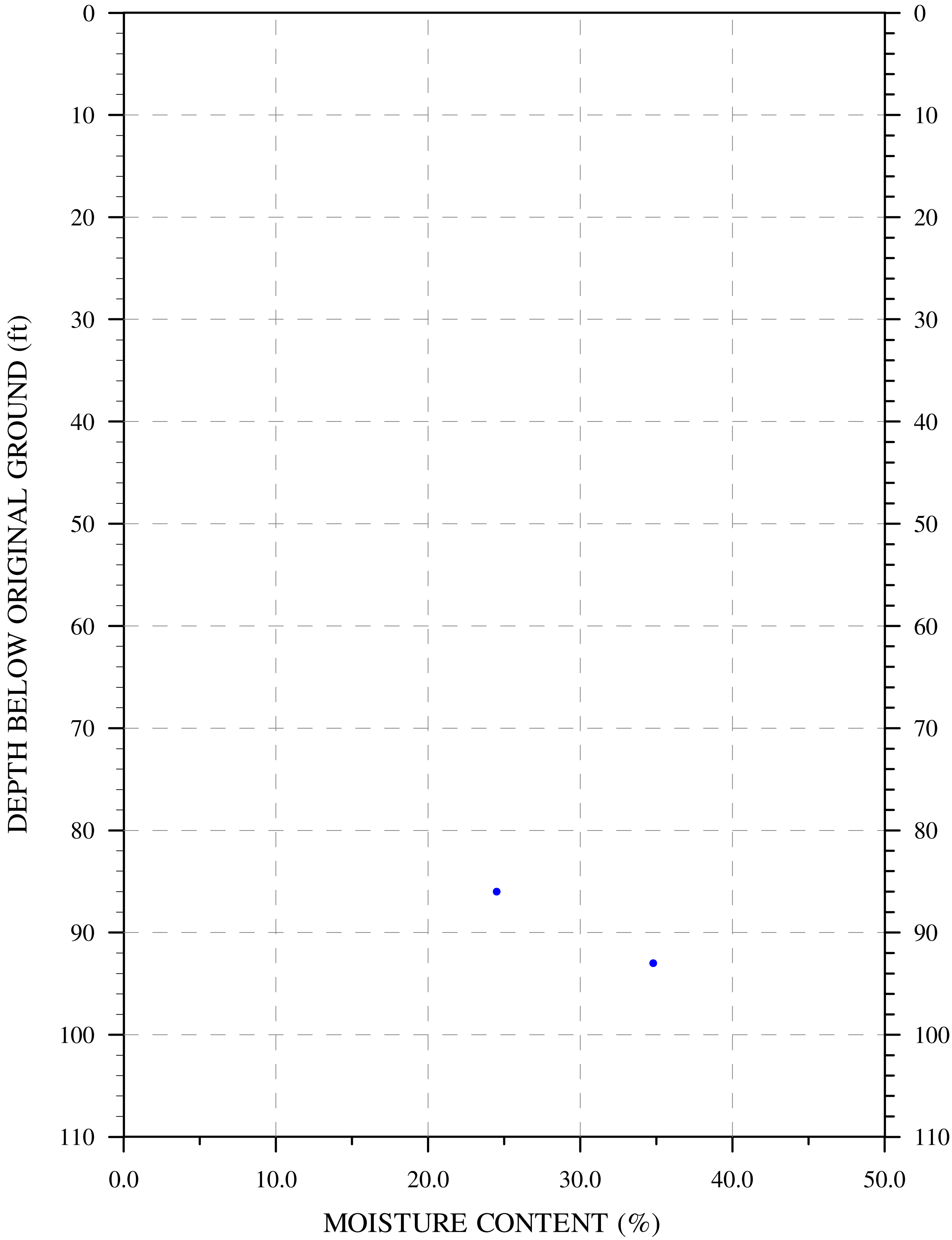
PROJECT NO. M07802A41

FIGURE NO. VII-8

GRAIN SIZE DISTRIBUTION



KLOHN CRIPPEN	**PROJECT No.:** **M07802A41**
	PROJECT: **Stage 2 Tailings Expansion Overall Stability Update**
	SUBJECT: **Figure VII-9 - Peat Gradations**
	DATE: 24-Jan-06
	DRAWN BY: Rick Friedel **CHECKED BY:**



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KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

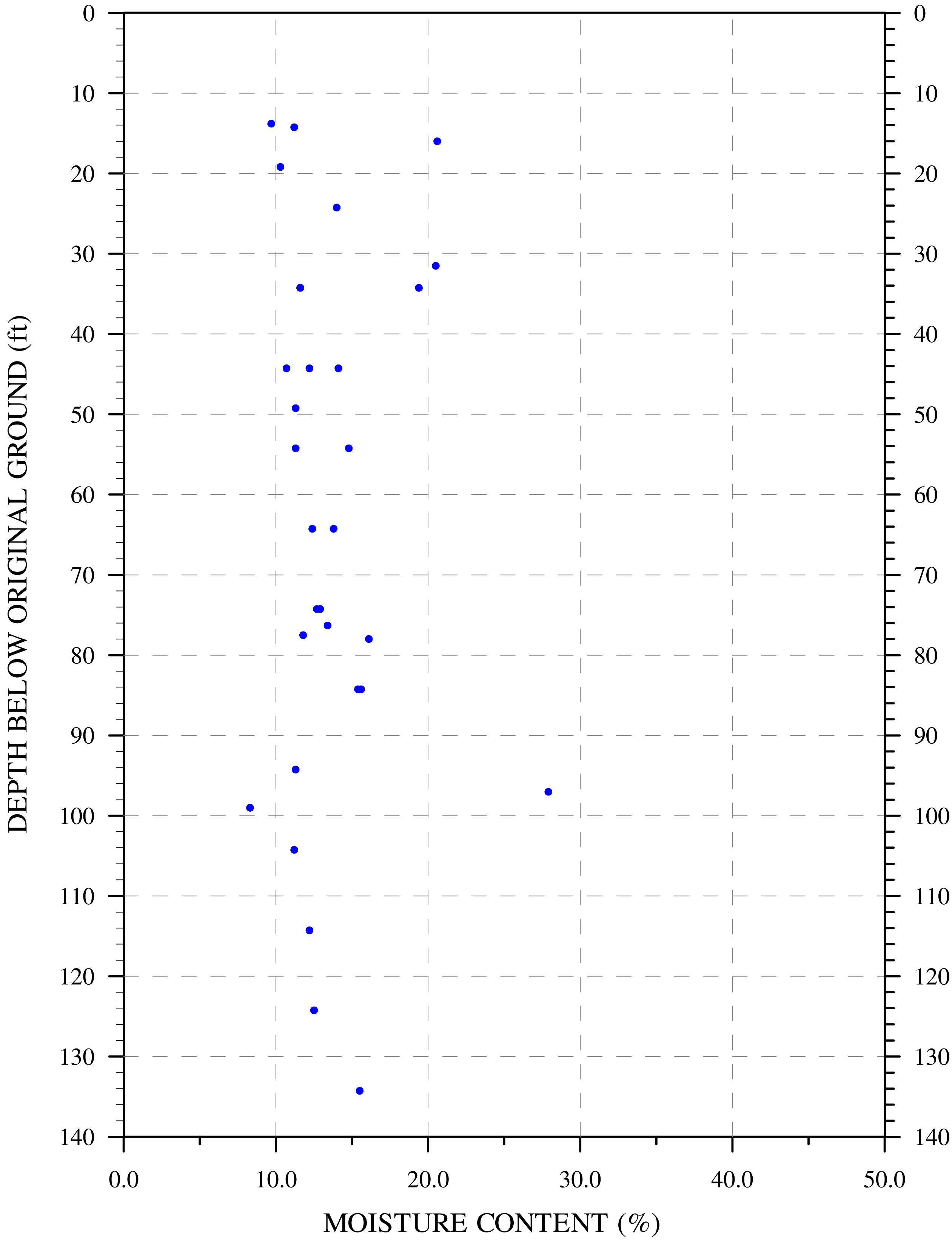
PROJECT
STAGE 2 TAILINGS EXPANSION
OVERALL STABILITY UPDATE

TITLE
PEAT -
MOISTURE CONTENT VS. DEPTH

PROJECT NO. M07802A41

FIGURE NO. VII-10

KLOHN CRIPPEN	**PROJECT No.:** M07802A41 0101
	PROJECT: Stage 2 Tailings Expansion Overall Stability Update
	SUBJECT: Figure VII-11 - Sand(SW) Gradations
	DATE: 24-Jan-06
	DRAWN BY: Rick Friedel **CHECKED BY:**



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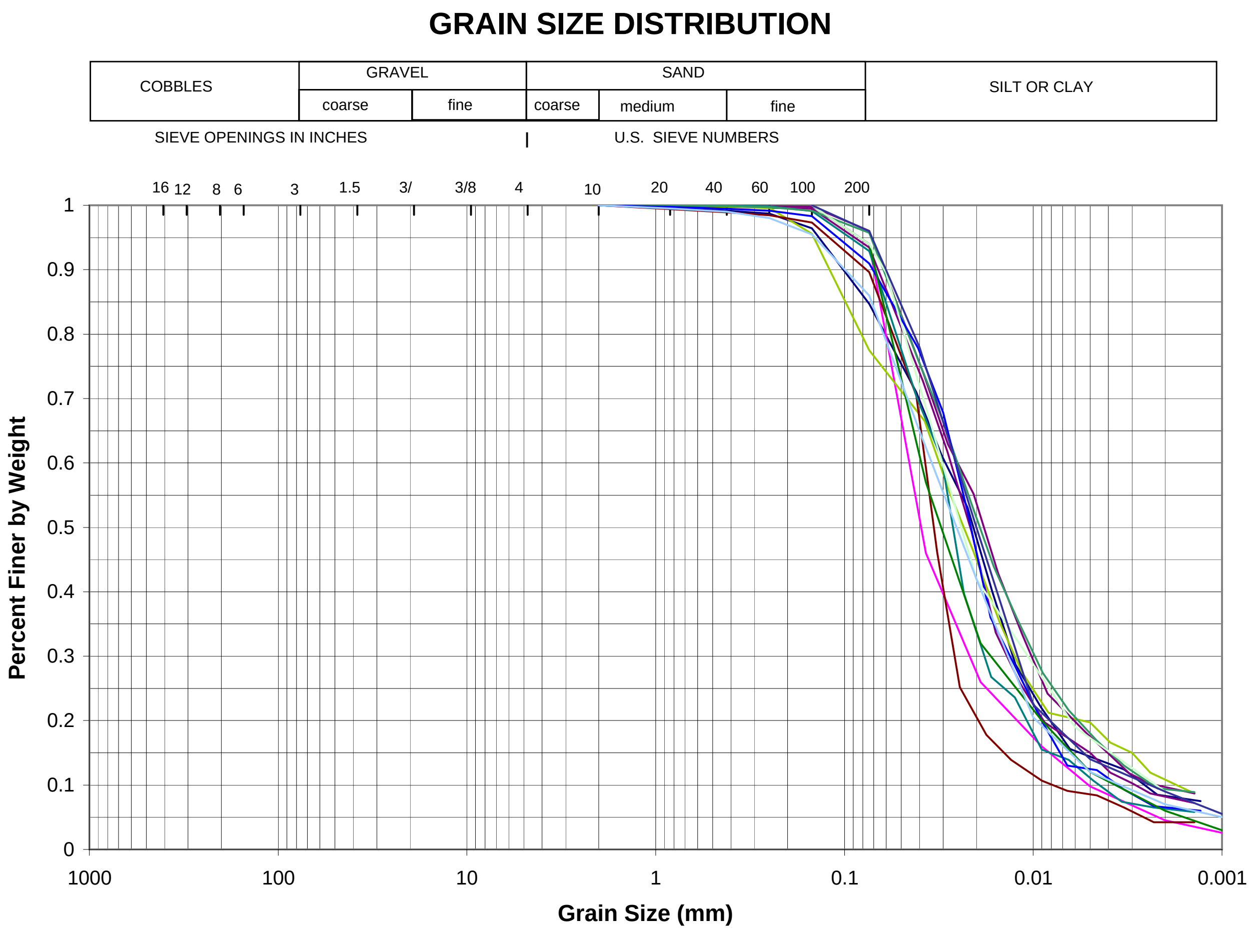
KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION
OVERALL STABILITY UPDATE

TITLE
SAND (SW) -
MOISTURE CONTENT VS. DEPTH

PROJECT NO. M07802A41

FIGURE NO. VII-12



KLOHN CRIPPEN

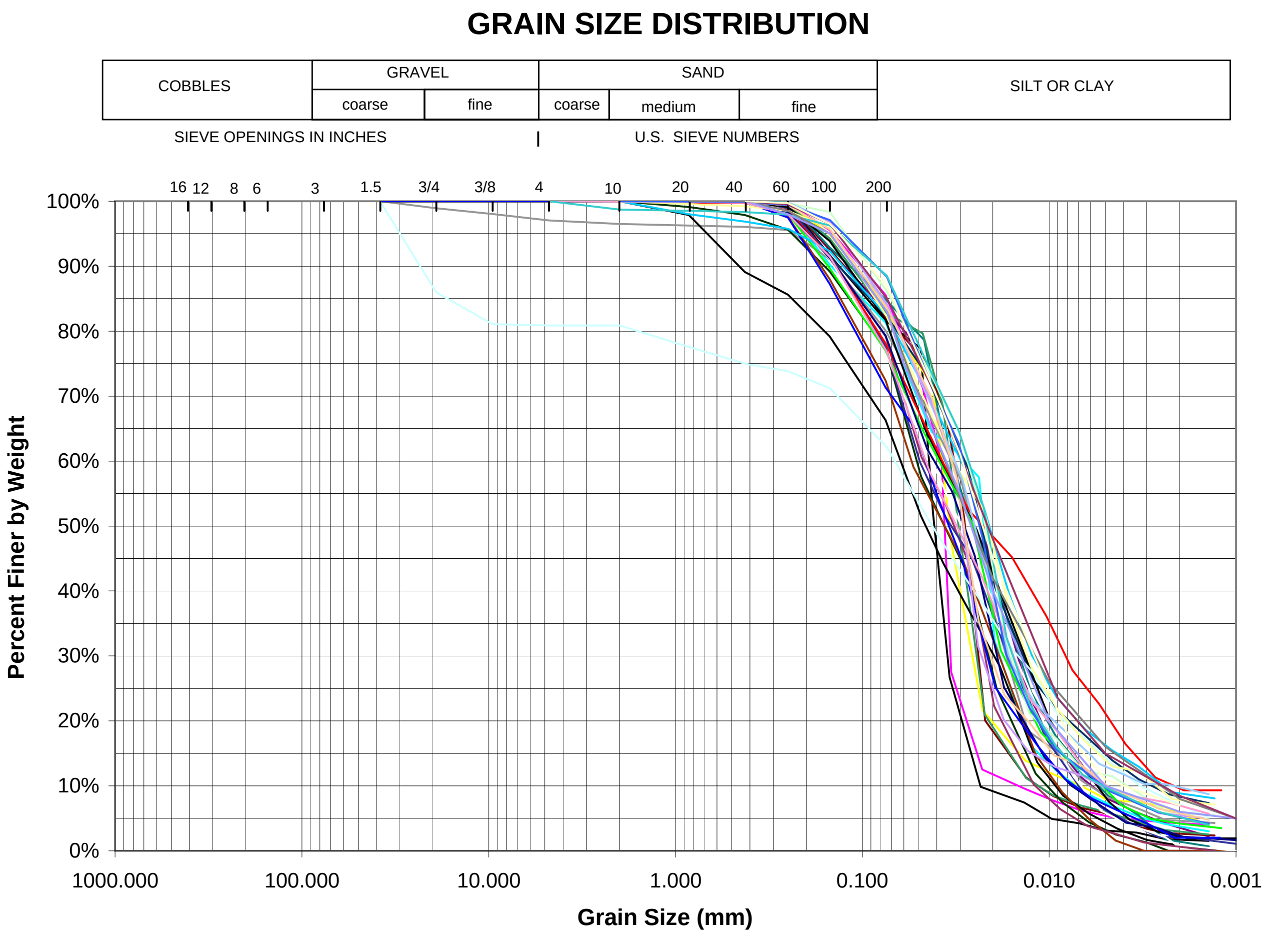
PROJECT No.: M07802A41

PROJECT: Stage 2 Tailings Expansion Overall Stability Update

SUBJECT: Figure VII-13 - Old Tailings Gradations

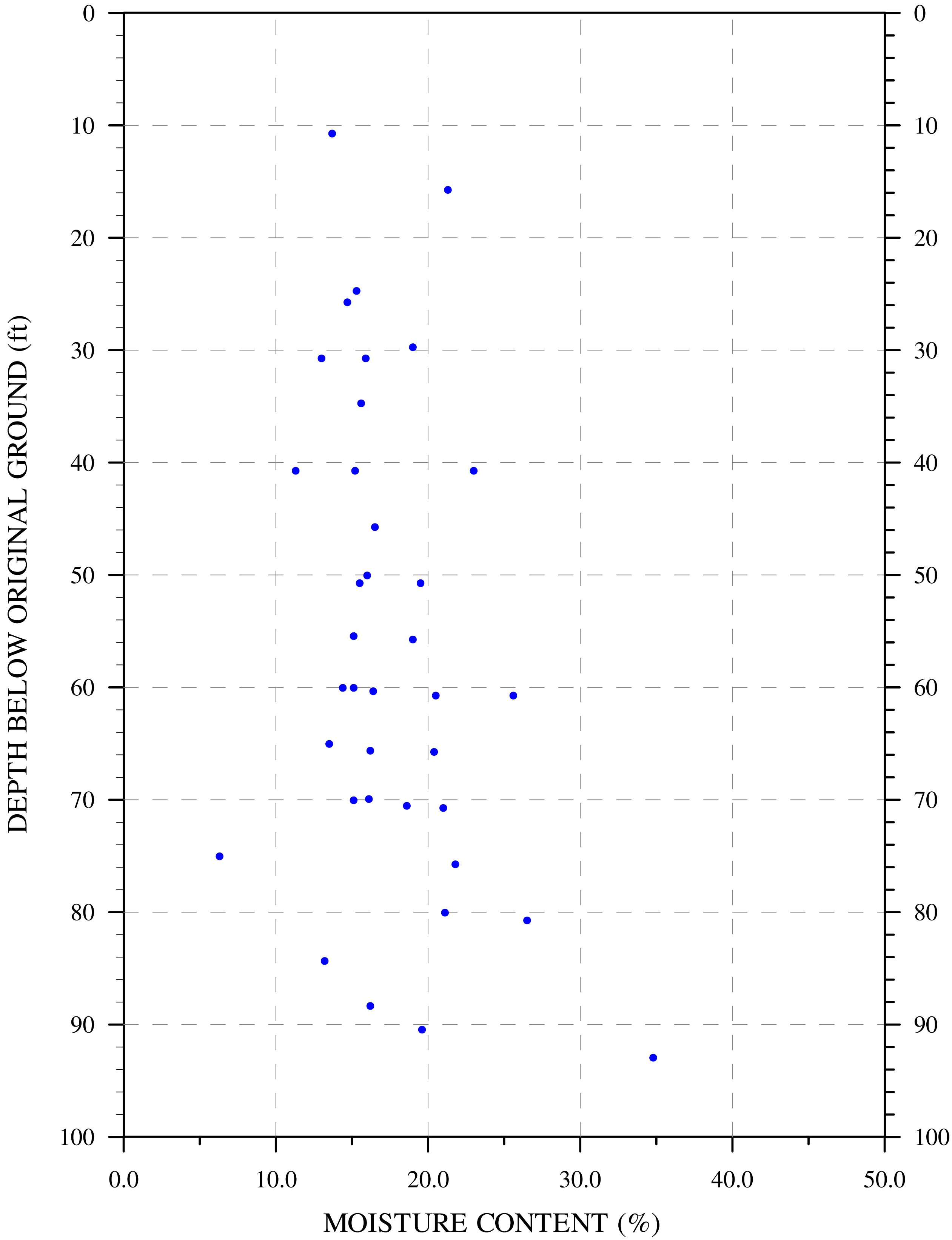
DATE: 24-Jan-06

DRAWN BY: Rick Friedel **CHECKED BY:**



KLOHN CRIPPEN

PROJECT No.:	M07802A41
PROJECT:	Stage 2 Tailings Expansion Overall Stability Update
SUBJECT:	Figure VII-14 - New Tailings Gradations
DATE:	24-Jan-06
DRAWN BY:	Rick Friedel CHECKED BY:



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CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

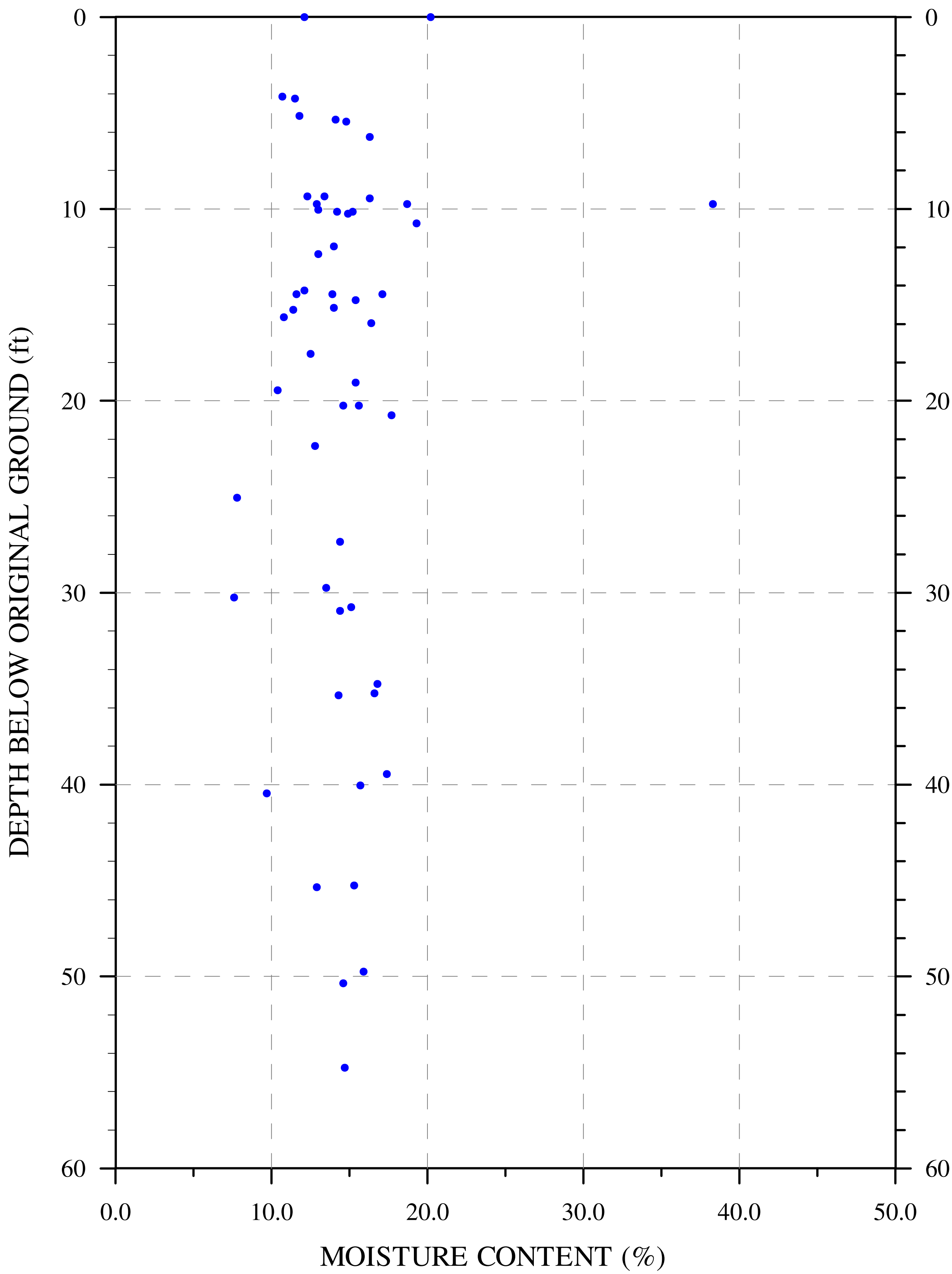
KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION
OVERALL STABILITY UPDATE

TITLE
OLD TAILINGS -
MOISTURE CONTENT VS. DEPTH

PROJECT NO. M07802A41

FIGURE NO. VII-15



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PROJECT
STAGE 2 TAILINGS EXPANSION
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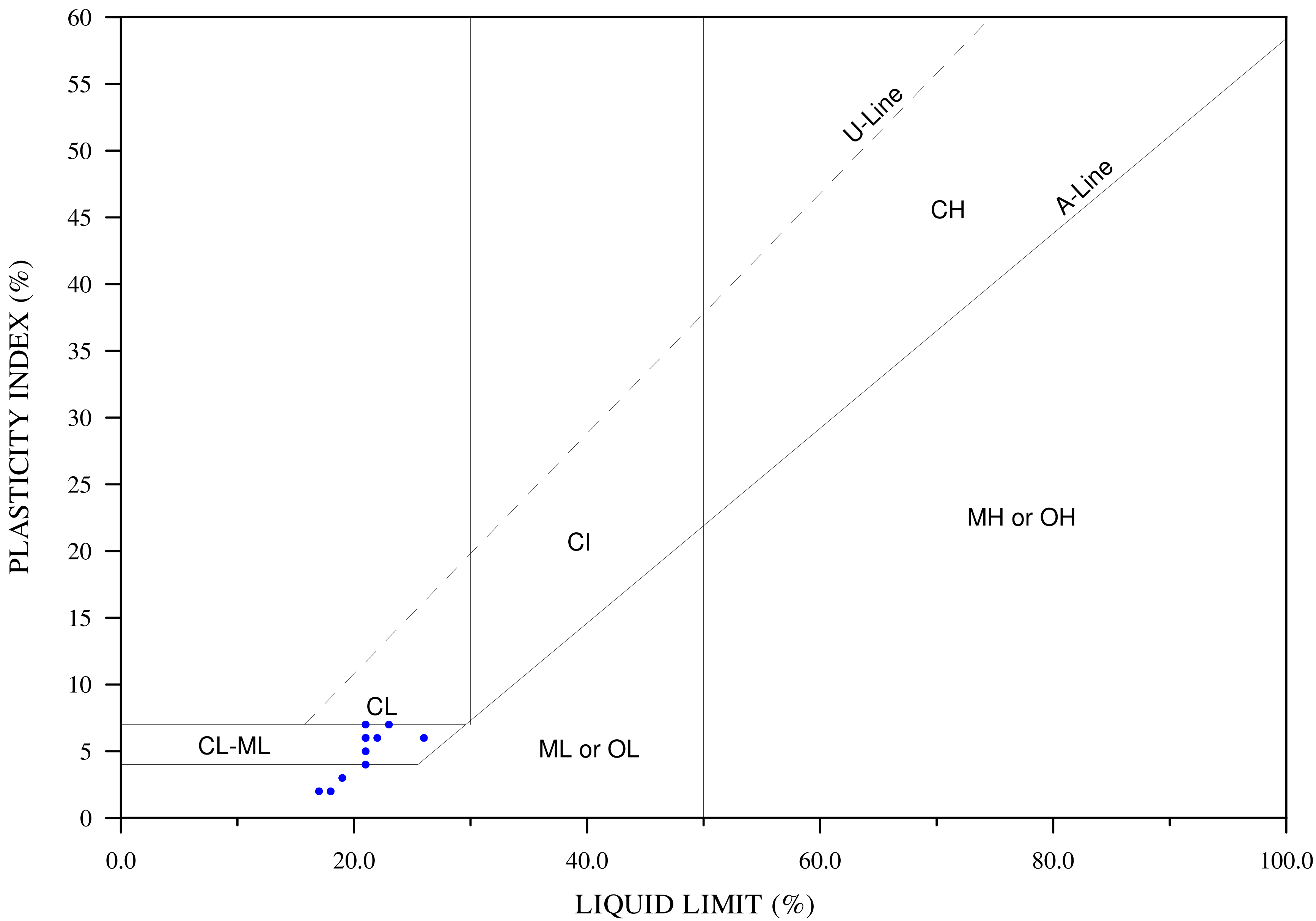
TITLE
NEW TAILINGS -
MOISTURE CONTENT VS. DEPTH

PROJECT NO. M07802A41

FIGURE NO. VII-16



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CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

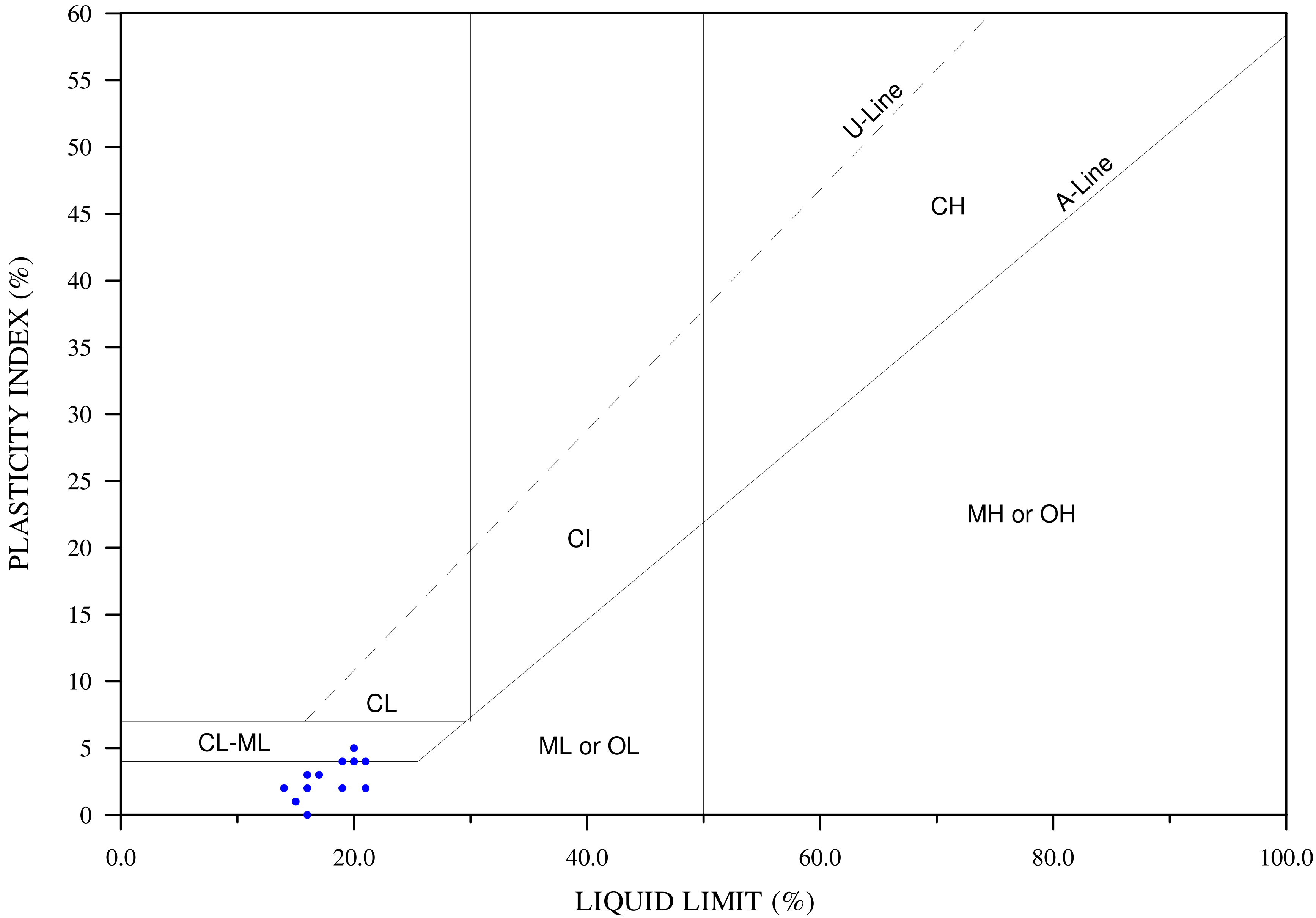
KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE
OLD TAILINGS - PLASTICITY CHART

PROJECT NO. M07802A41

FIGURE NO. VII-17



TO BE READ WITH KLOHN-CRIPPEN REPORT DATED January 24, 2006

CLIENT	PROJECT	
KENNECOTT GREENS CREEK MINING COMPANY	STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE	
KLOHN CRIPPEN	TITLE: NEW TAILINGS - PLASTICITY CHART	
	PROJECT NO. M07802A41	FIGURE NO. VII-18

APPENDIX VIII

Slope Stability Sections and Sensitivity Analysis

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APPENDIX VIII
Slope Stability Sections and Sensitivity Analysis

VIII-1. STATIC SENSITIVITY ANALYSIS

A sensitivity analysis was carried out on material properties and piezometric levels of the "Old" and "New" tailings in West Buttress Section 4 for static (peak) design case, to determine their relative influence on overall slope stability. Section 4 was selected because it has the greatest amount of old tailings and a high phreatic surface, relative to the other stability sections. The analysis was carried out by varying one of the following parameters while holding the others at their design values:

- Friction angle (new tailings);
- Friction angle (old tailings);
- Unit weight (new tailings);
- Unit weight (old tailings); and
- Tailings phreatic surface level.

For each variation in parameter, the minimum factor of safety (FOS) against slope failure was determined using static material properties in all soil units.

VIII-1.1 Friction Angle

Lower bound friction angles have been selected for design. Based on available test data the lower bound peak strengths are 39° for the new tailings and 33° for the old tailings. For the sensitivity analyses, the friction angle was varied from 32° to 42° in new tailings and from 28° to 33° in old tailings. Results of the analyses are plotted in Figure 1.

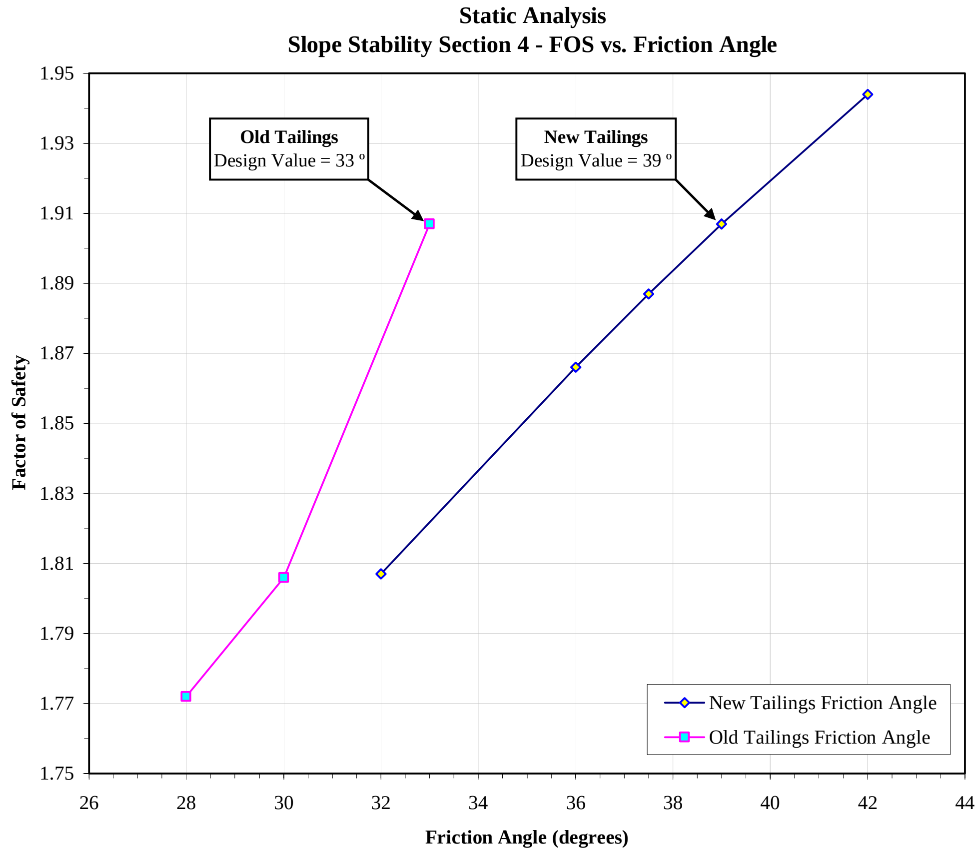


Figure 1 Factor of Safety vs. Tailings Friction Angle

The results show that the slope stability FOS does not fall below the design criteria of 1.5, for a reasonable range of strengths. For this section the friction angle of the old tailings had a greater impact on FOS than the friction angle in the new tailings. This may vary between sections depending on the section geometry and the location of the critical slip surface.

VIII-1.2 Unit Weight

Design total unit weights are 128 pcf for new tailings and 120 pcf for old tailings. The unit weight was varied independently from 118 pcf to 138 pcf in new tailings and from 110 pcf to 130 pcf in old tailings. Results of the analyses are plotted in Figure 2.

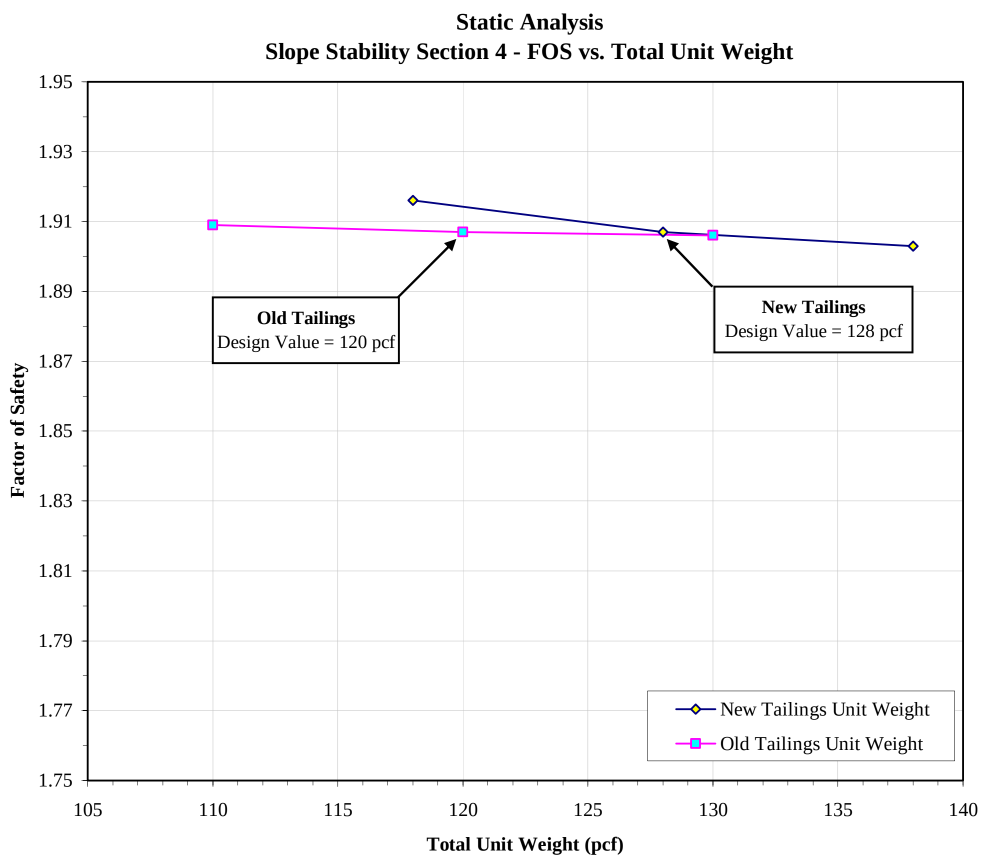


Figure 2 Factor of Safety vs. Tailings Total Unit Weight

The results indicate that the slope stability is not sensitive to variations in new or old tailings total unit weight.

VIII-1.3 Phreatic Surface Level

Design long-term water levels for Section 4 in the tailings range from 30ft to 35ft above the foundation. These levels are assumed to apply where the tailings surface is relatively horizontal over the central area of the pile. This is also where depth of tailings is greatest. Beneath the outer 3H:1V slopes, where the tailing pile slopes down to the toe, the water level is assumed to reduce linearly (as shown on the stability sections) to 3 ft above the foundation at the toe.

For the sensitivity analysis, the water level in the central area of the pile is raised or lowered by the specified increment and the water level depth beneath the outer 3H:1V slopes reduces linearly to 3 ft above the foundation at the toe (as indicated above). The elevation of the tailings piezometric surface near the centre of the tailings pile was varied from 10ft below design level (El. 180ft) to 30ft above design level (El. 220ft). Results of the analyses are plotted in Figure 3.

Similar to the results of the friction angle analysis the slope stability is sensitive to the phreatic surface level but, for the sensitivity range examined, did not lower the FOS below design criteria of 1.5.

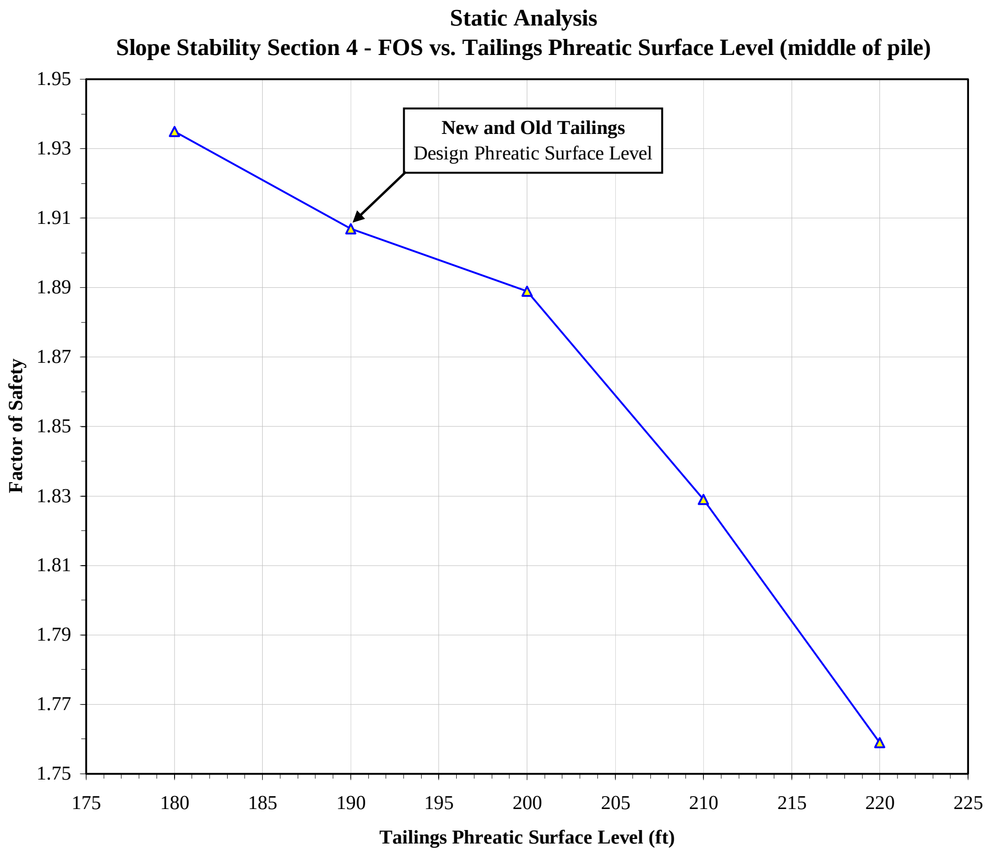


Figure 3 Factor of Safety vs. Tailings Phreatic Surface Level

VIII-2. POST-LIQUEFACTION SENSITIVITY ANALYSIS

The following analysis considers a reasonable worst-case scenario for the pile upon closure. The best data from our current liquefaction assessment indicates that the new tailings will not liquefy under design earthquake loading under DBE and MDE. However, due to ongoing research and development in the assessment of liquefaction of fine-grained materials such as tailings, this conclusion may change in the future. The purpose of the following analysis is to demonstrate that in the unlikely event that liquefaction of the new tailings does occur, it would not be a fatal flaw to the pile design, and that reasonable action can be taken to accommodate liquefaction, if required.

In our judgment, the following approach to pile stability assessment due to liquefaction deals with uncertainty in the data and uncertainty in the available analytical techniques, and identifies a means of stabilizing the pile, should future data or developments in the understanding of liquefaction lead to a requirement for remedial work:

- Assume all tailings below the predicted water level liquefy. Based on current data this appears to be a worst case scenario.
- Assume that the tailings post-liquefaction undrained strength at depth is equivalent to the high stress laboratory strengths measured in post-liquefaction monotonic extension from the cyclic triaxial tests.
- Assume that the tailings post-liquefaction undrained strength reduces with stress to median values, as proposed by Seed and Harder, 1990, based on average tailings SPT $(N_1)_{60\text{-cs}}$.
- Recommend continued installation and monitoring of instruments to measure long term water and saturation levels to confirm the possible extent of liquefaction or softening.
- Prepare conceptual designs for remedial measures (e.g., rock fill toe berms) to cover a reasonable range of assumed tailings pile water levels.
- Liaise with designers of the tailings closure cover to stress the importance of a low post closure water level in the pile.

VIII-2.1 Results of Post-Liquefaction Stability Analyses

The results of the post-liquefaction stability analyses are presented in Table 2.1. The sections with stratigraphy, critical slip surfaces, material properties, and SPT liquefaction results are shown on Figures VIII-1 through VIII-10.

Table 2.1 Post-Liquefaction Stability Results

STABILITY SECTION	LOCATION	MINIMUM FOS	SAND AND GRAVEL PROPERTIES	COMMENTS
1	Northeast	1.3	Liquefied	Assumes partial excavation of Sand and Gravel at toe (Klohn Crippen 2004) and replaced with rockfill.
1b	Northeast	1.1	Liquefied	Based on Section 1; Tailings slope is extended to ultimate elevation 330ft at 3H:1V slope.
2	North	1.4	Liquefied	
3	South Slope	1.1	Static	
4	West Buttress	0.9	Static	No remedial berm
4	West Buttress	1.1	Static	Includes 50ft-high remedial rockfill berm on toe
4	West Buttress	1.2	Static	Includes 65ft-high remedial rockfill berm on toe
4b	West Buttress	1.0	Static	
5a	Northwest	1.2	N/A	
5b	Northwest	0.9	N/A	
5c	Northwest	1.1	N/A	Based on Section 5a; Tailings slope was steepened to 3H:1V, from toe to elevation 280ft.
6	Southeast	1.1	Static	Residual liner strength was used.

The sections analyzed meet the minimum required FOS of 1.0 for post-liquefaction conditions, except for Section 4 and 5b. To determine possible remedial measures, West Buttress Section 4 was analyzed to calculate the size of toe-berm required to achieve a FOS equal to 1.0. Figure VIII-13 shows a plan view of the final tailings surface with a 50 ft high West Buttress toe berm, which is sufficient to achieve a post-liquefied FOS of 1.1. This toe berm could be designed as the perimeter road. Design modifications can be made to the Northwest expansion area (Section 5b) prior to construction to raise this FOS above 1.0 if required for post-closure.

VIII-2.2 Seismic Deformation

Although the tailings pile generally meets acceptable safety factors against limit equilibrium slope stability failure during the MDE, some deformation is expected.

Seismic deformations of the tailings pile were assessed using pseudo-static methods (Hynes-Griffin, et. al. 1984) assuming liquefaction in all saturated tailings (post-liquefaction condition). The analysis was applied to stability Section 3 and Section 6. Section 6 is representative of the tailings pile areas with a geosynthetic liner system, and Section 3 represents those areas of the tailings pile without the liner system. Newmark's sliding block model, which provides the basis for the Hynes-Griffin deformation prediction, is a good representation of tailings sitting on a lined foundation. A summary of yield accelerations and estimated deformations is in Table 2.2.

Table 2.2 Predicted Deformations

SECTION	STATE	STATIC FOS	YIELD ACCEL. FOR FOS = 1.0	PREDICTED DEFORMATION (inches) MDE (PGA=0.30g) Mean	PREDICTED DEFORMATION (inches) MDE (PGA=0.30g) Upper Bound
Section 3	Post-liquefaction	1.1	0.025	24	268
Section 6	Post-liquefaction	1.1	0.01	71	394

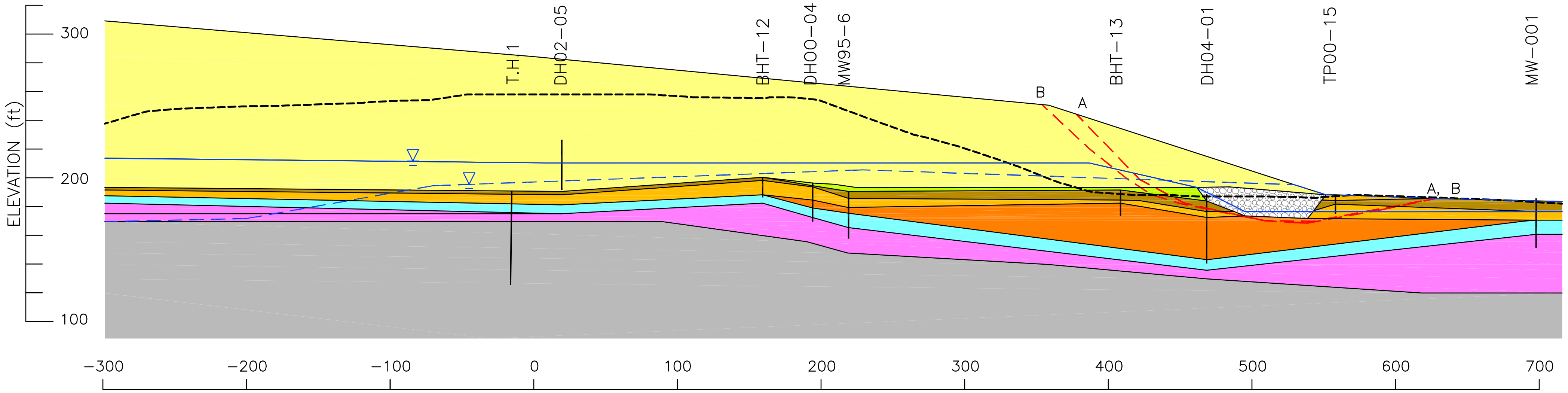
The Hynes-Griffin method assumes a relatively large base amplification factor based on case histories from many sites. The South and Southeast areas of the pile are largely founded on rock or dense till, and hence, the base amplification and calculated deformation in these areas is expected to be at the low end of the ranges presented in Table 2.2, that is, expected values are in the "mean" column.

The expected deformation will not significantly affect the pile stability. However, there could be some disruption to the liner system and to underdrain pipes, as discussed in the main text Section 10.2. The larger deformations noted in Table 2.2 occur in the post-liquefaction condition, and should be concentrated within the liquefied tailings and not in the relatively drained liner system. If the liner should tear, a loss of underdrain water could occur through the localized openings. Regular sampling from the under-liner lysimeters will confirm the integrity of the liner system.

VIII-3. RECOMMENDATIONS

- Contingency remedial measures have been identified in concept using the approach to liquefaction and pile stability assessment, as discussed above. The remedial measures (e.g., rockfill toe berms) would be constructed upon mine closure, if needed. The conceptual designs should cover a reasonable range in assumed tailings pile water levels.

- If a perimeter road is planned around the west side of the TSF, consideration should be given to having the road fill, if any, placed so that it can be incorporated into a future berm if needed.



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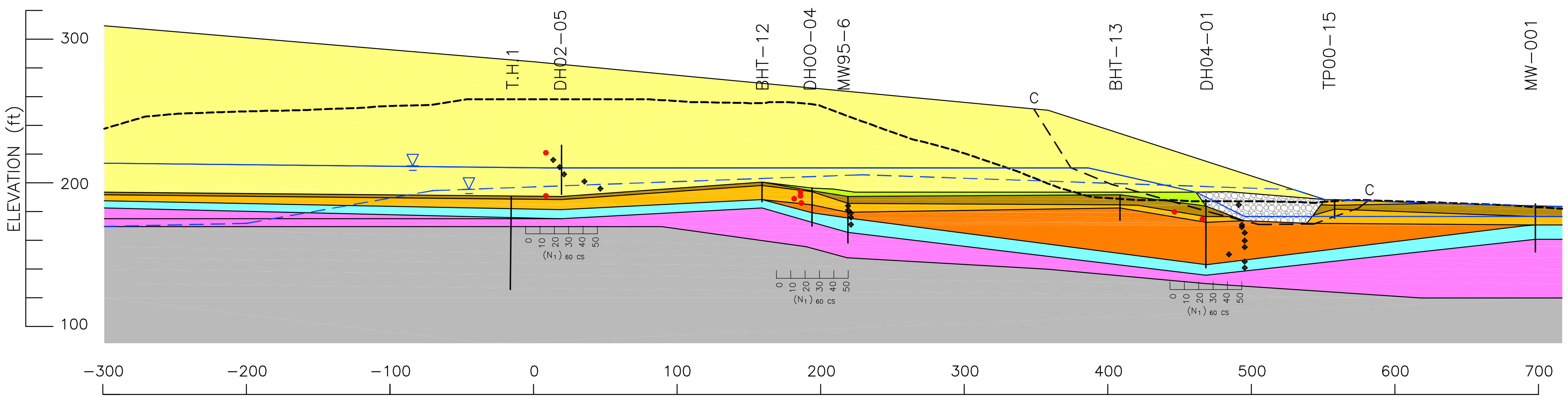

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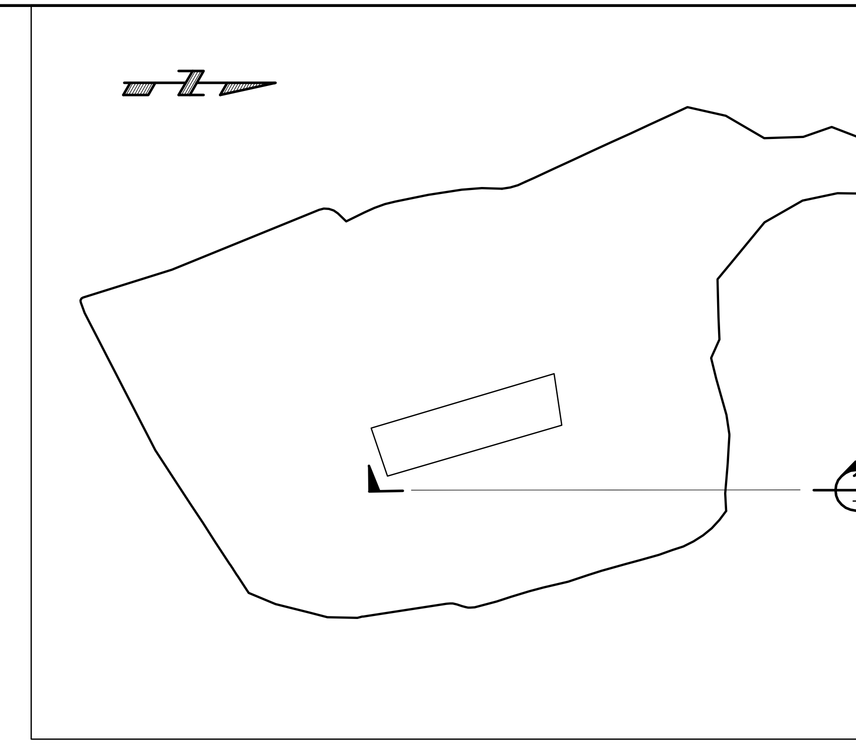

KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- FAILURE SURFACE – POST-LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1
- TAILINGS SURFACE DEC. 2005

NOTES:

1. POST-LIQUEFACTION TAILINGS STRENGTH FUNCTION VARIES LINEARLY BETWEEN:
   $Sr = 324$ psf @ $\sigma_N = 0$ psf; AND
   $Sr = 2297$ psf @ $\sigma_N \geq 18880$ psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST-LIQUEFACTION ANALYSIS.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	OLD TAILINGS	120	33	0	32	0	NOTES (1,2)		1
	ROCK FILL	120	40	0	40	0	N/A	N/A	1
	PEAT (UNIT 6)	67	27	0	27	0	N/A	N/A	2
	SAND (UNIT 5)	120	33	0	33	0	$\phi' = 33°$; Max 1640		2
	DENSE SAND AND GRAVEL (UNIT 4)	120	33	0	33	0	N/A	N/A	2
	SILT/CLAY (UNIT 3)	120	30	0	30	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	2
	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	2

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC PEAK	FACTOR OF SAFETY STATIC RESIDUAL	FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A–A	1.7		
B–B		1.6	
C–C			1.3

NOT FOR CONSTRUCTION

100 0 100 FT

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SCALE

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PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: STABILITY SECTION 1a – NORTHEAST (TRUE SLOPE)

PROJECT No. M07802A41

FIG. No. VIII-1

KCR-B

Date: 3/1/2006
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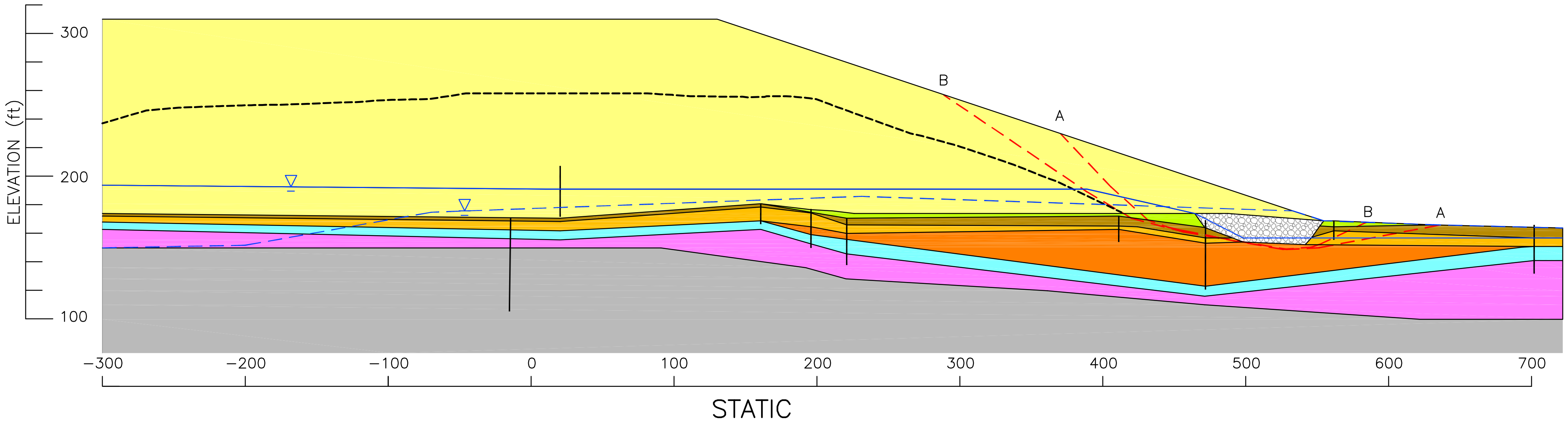

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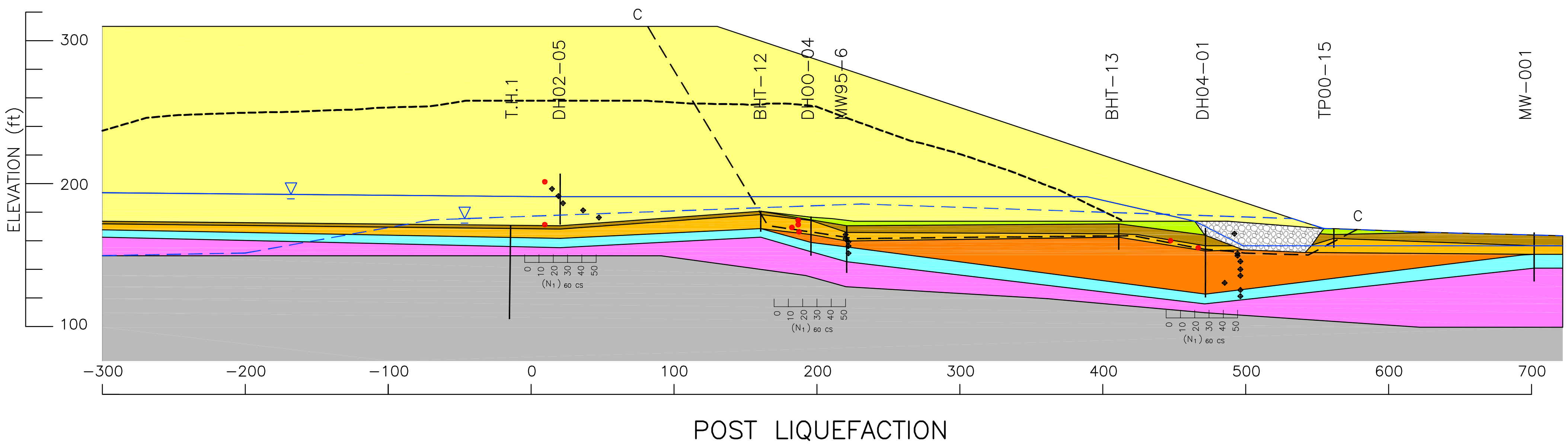

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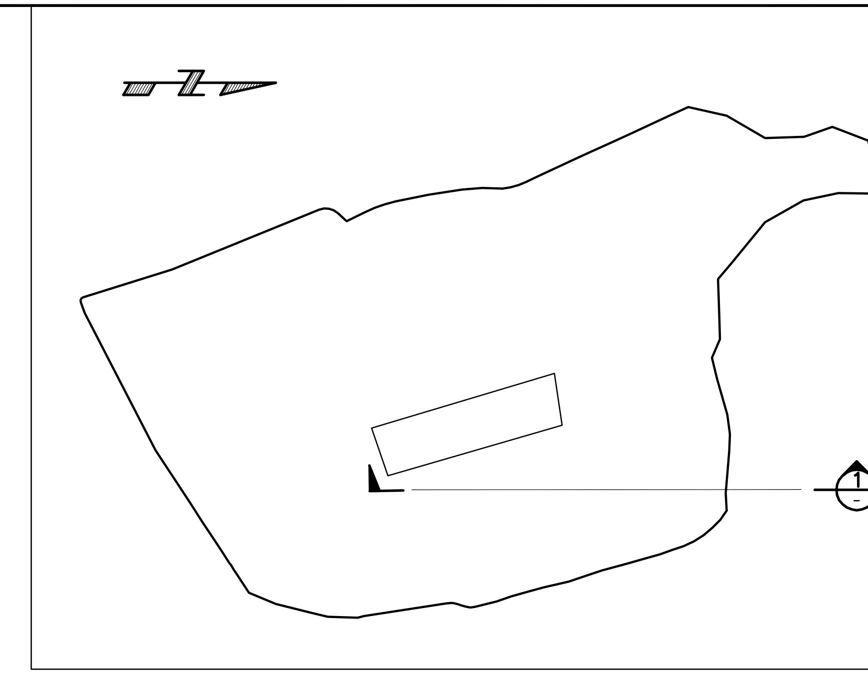

KEY PLAN

LEGEND

- FAILURE SURFACE − STATIC
- FAILURE SURFACE − POST−LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1
- TAILINGS SURFACE DEC. 2005

NOTES:

1. POST−LIQUEFACTION TAILINGS STRENGTH FUNCTION VARIES LINEARLY BETWEEN:
   $Sr = 324$ psf @ $\sigma_N = 0$ psf; AND
   $Sr = 2297$ psf @ $\sigma_N \geq 18880$ psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. SECTION 1b IS BASED ON SECTION 1 STRATIGRAPHY, BUT WITH THE TAILINGS SLOPE EXTENDED TO THE ULTIMATE PILE ELEVATION 330 ft.
4. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST−LIQUEFACTION ANALYSIS.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC−PEAK FRICTION ANGLE (deg)	STATIC−PEAK COHESION (psf)	STATIC−RESIDUAL FRICTION ANGLE (deg)	STATIC−RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	OLD TAILINGS	120	33	0	32	0	NOTES (1,2)		1
	ROCK FILL	120	40	0	40	0	N/A	N/A	1
	PEAT (UNIT 6)	67	27	0	27	0	N/A	N/A	2
	SAND (UNIT 5)	120	33	0	33	0	ϕ'=33°; Max 1640		2
	DENSE SAND AND GRAVEL (UNIT 4)	120	33	0	33	0	N/A	N/A	2
	SILT/CLAY (UNIT 3)	120	30	0	30	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	2
	BEDROCK (UNIT 1)	−	−	−	−	−	N/A	N/A	2

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC PEAK	FACTOR OF SAFETY STATIC RESIDUAL	FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A−A	1.7		
B−B		1.6	
C−C			1.1

NOT FOR CONSTRUCTION

100 0 100 FT

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PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: STABILITY SECTION 1b − NORTHEAST (330 ft CREST)

PROJECT No. M07802A41

FIG. No. VIII−2

KCR−B

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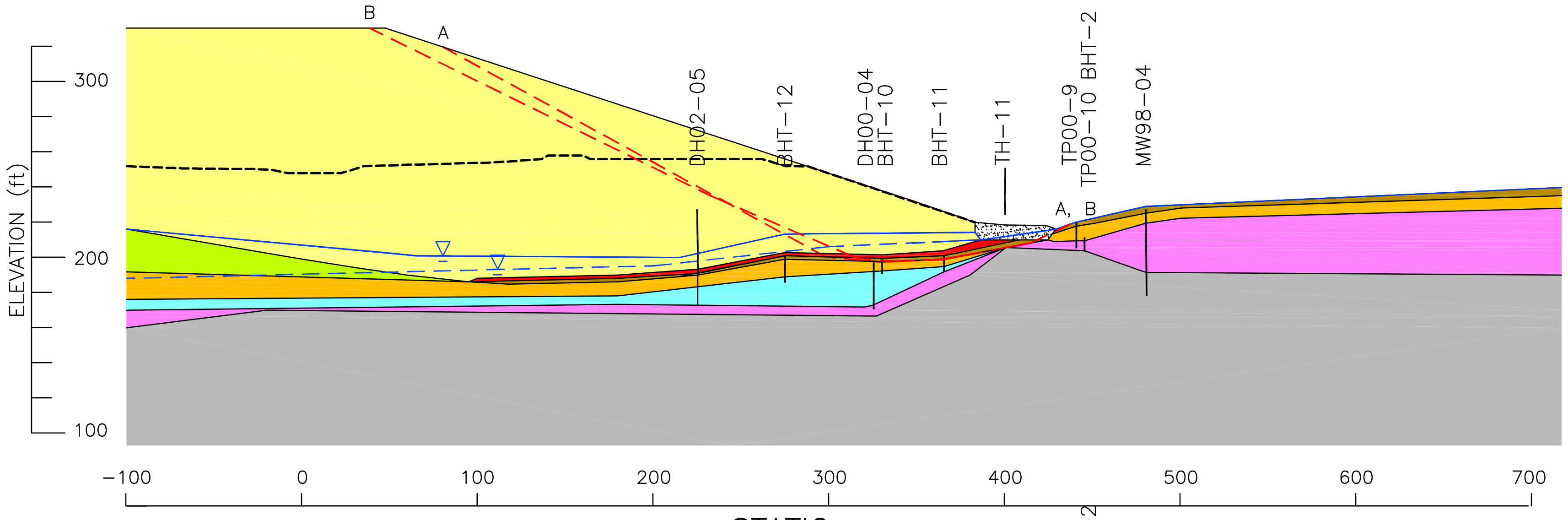

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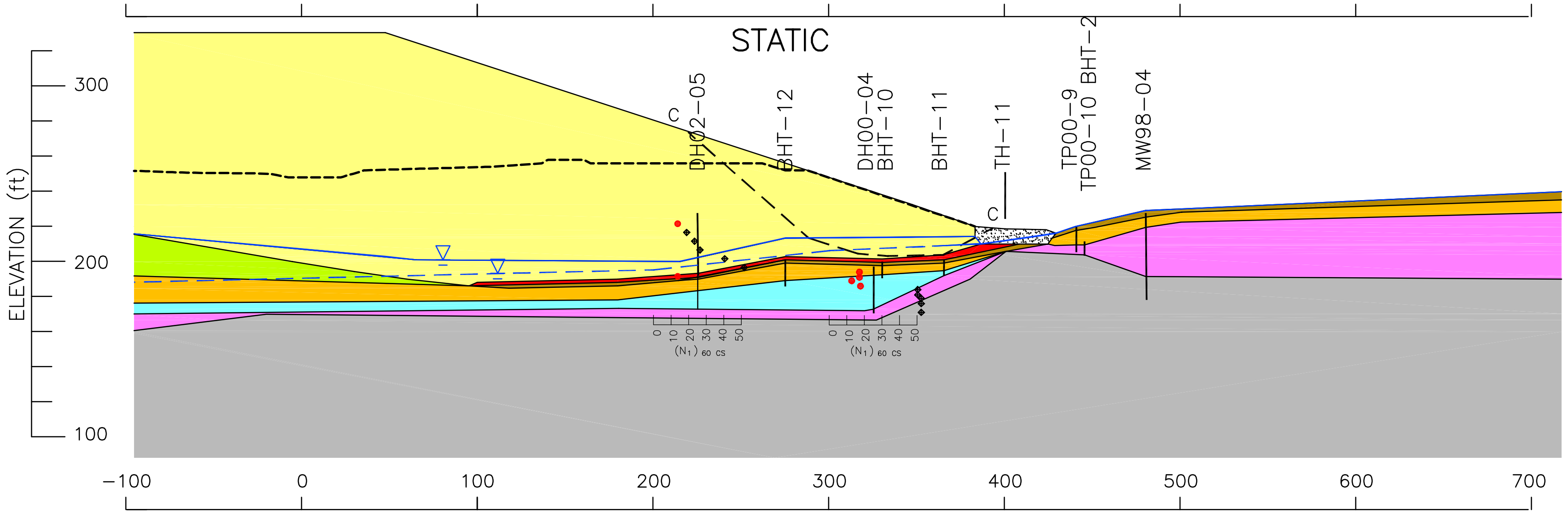
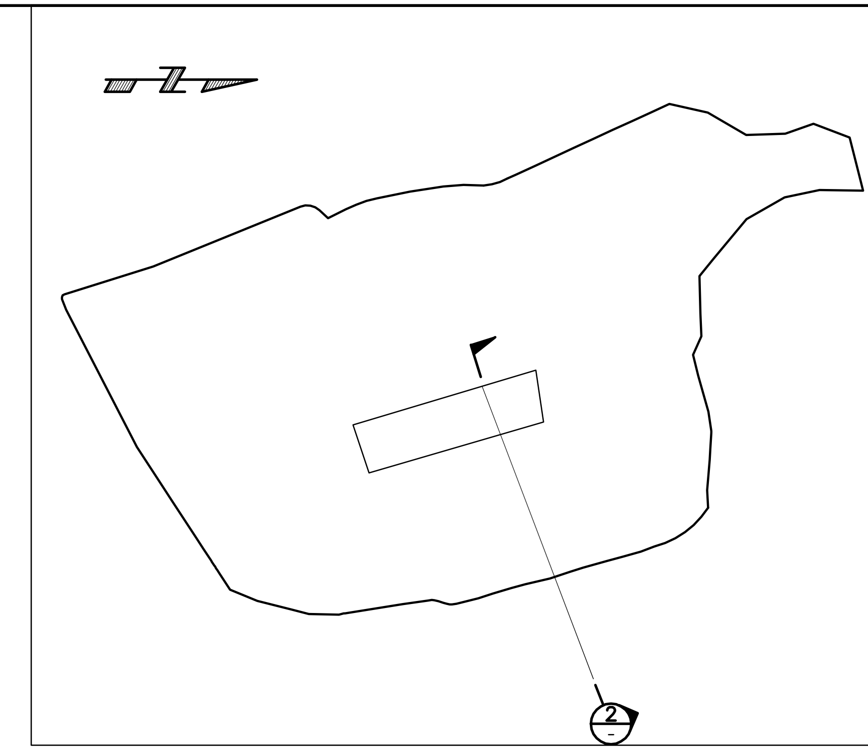


POST LIQUEFACTION



KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- FAILURE SURFACE – POST–LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1
- TAILINGS SURFACE DEC. 2005

NOTES:

1. POST–LIQUEFACTION TAILINGS STRENGTH FUNCTION VARIES LINEARLY BETWEEN:
   $Sr = 324$ psf @ $\sigma_N = 0$ psf; AND
   $Sr = 2297$ psf @ $\sigma_N \geq 18880$ psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	OLD TAILINGS	120	33	0	32	0	NOTES (1,2)		1
	SAND DRAINAGE BLANKET	120	40	0	40	0	N/A	N/A	1
	PEAT (UNIT 6)	67	27	0	27	0	N/A	N/A	2
	SAND (UNIT 5)	120	33	0	33	0	ϕ'=33°; Max 1640		2
	SILT/CLAY (UNIT 3)	120	30	0	30	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	2
	ROADFILL	120	40	0	40	0	N/A	N/A	2
	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	2

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC PEAK	FACTOR OF SAFETY STATIC RESIDUAL	FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A–A	2.3		
B–B		1.9	
C–C			1.4

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SCALE: 100 0 100 FT

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

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PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: STABILITY SECTION 2 – NORTHEAST

PROJECT No. M07802A41

FIG. No. VIII–3

KCR–B

Date: 3/1/2006
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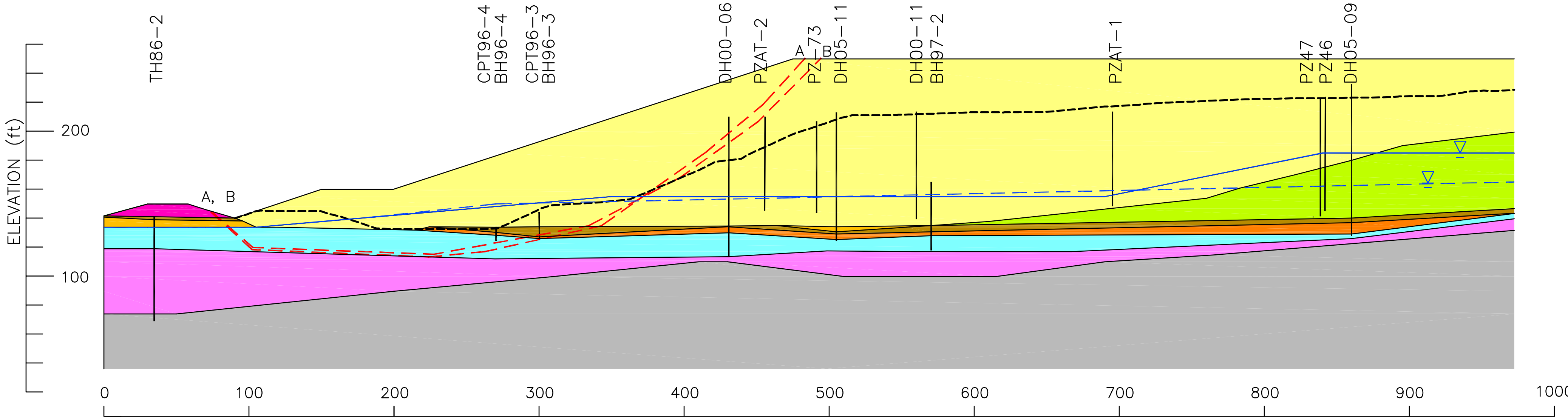

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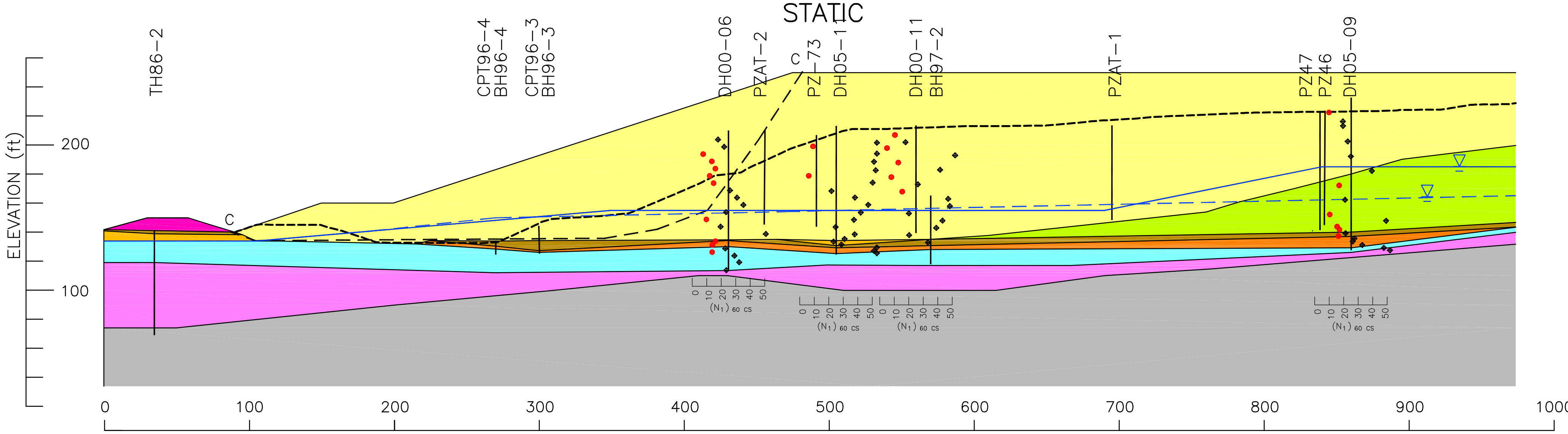
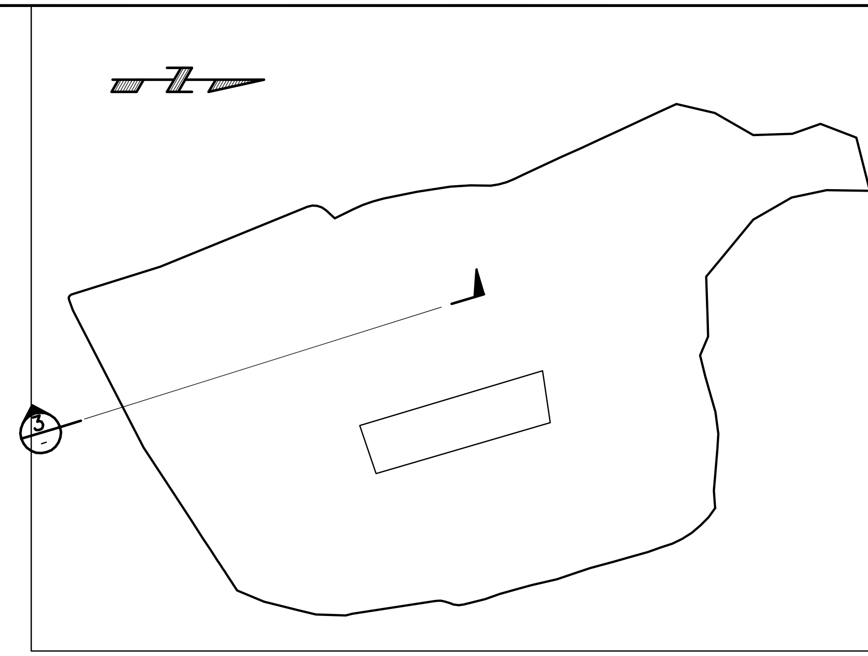


POST LIQUEFACTION



KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- FAILURE SURFACE – POST–LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1
- TAILINGS SURFACE DEC. 2005

NOTES:

1. POST–LIQUEFACTION TAILINGS STRENGTH FUNCTION VARIES LINEARLY BETWEEN:
   S_r = 324 psf @ σ_N = 0 psf; AND
   S_r = 2297 psf @ $\sigma_N \geq$ 18880 psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	OLD TAILINGS	120	33	0	32	0	NOTES (1,2)		1
	MAIN EMBANKMENT TILL	120	33	0	33	0	N/A	N/A	2
	PEAT (UNIT 6)	67	27	0	27	0	N/A	N/A	2
	DENSE SAND AND GRAVEL (UNIT 4)	120	33	0	33	0	N/A	N/A	2
	SILT/CLAY (UNIT 3)	120	30	0	30	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	2
	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	2

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC PEAK	FACTOR OF SAFETY STATIC RESIDUAL	FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A–A	2.3		
B–B		2.0	
C–C			1.1

NOT FOR CONSTRUCTION

100 0 100 FT

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

SCALE

CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

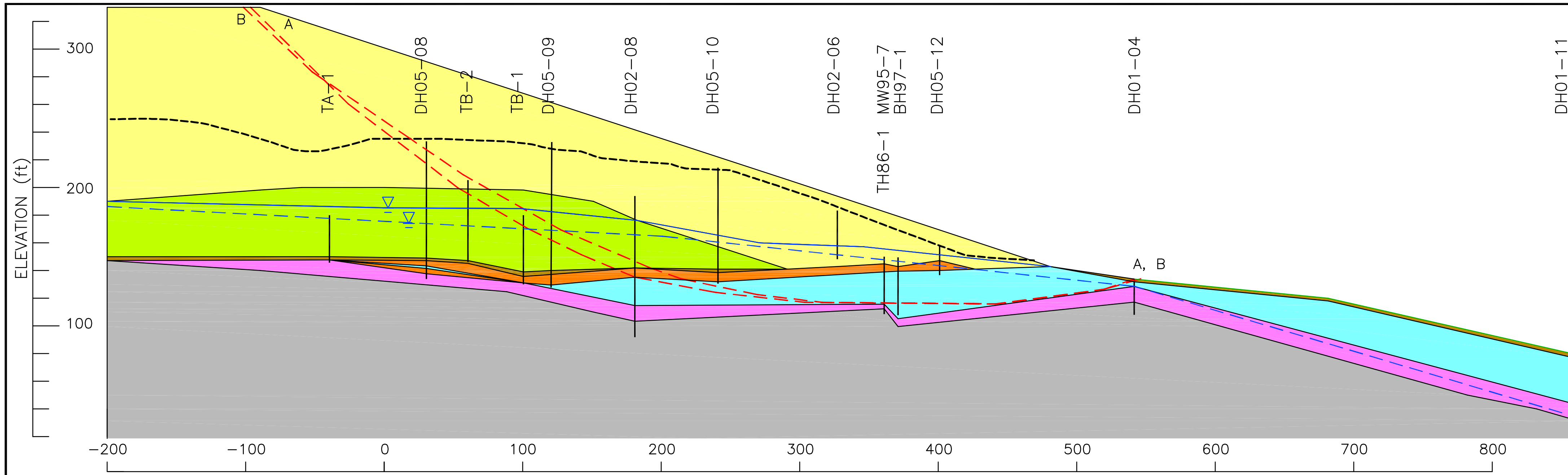
TITLE
STABILITY SECTION 3 – SOUTH SLOPE

PROJECT No. M07802A41

FIG. No. VIII–4

KCR-B

Date: 3/1/2006
Scale: 1=0(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\B-FIG_StabilitySectionsRA2006r1.dwg (cwong)



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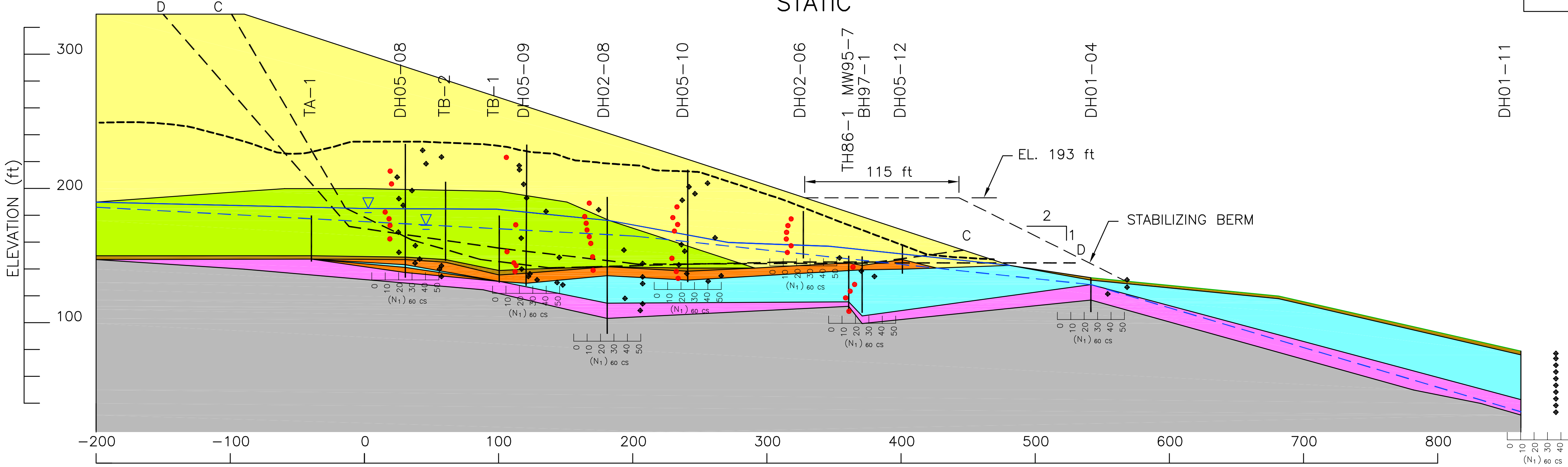
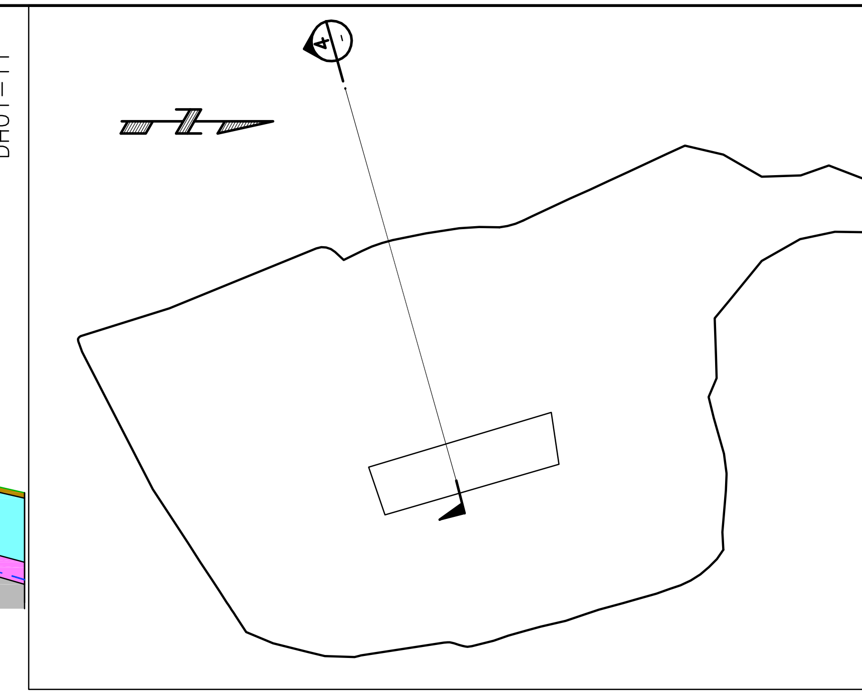


POST LIQUEFACTION



KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- FAILURE SURFACE – POST–LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1
- TAILINGS SURFACE DEC. 2005

NOTES:

1. POST–LIQUEFACTION TAILINGS STRENGTH FUNCTION VARIES LINEARLY BETWEEN:
   S_r = 324 psf @ σ_N = 0 psf; AND
   S_r = 2297 psf @ σ_N ≥ 18880 psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	OLD TAILINGS	120	33	0	32	0	NOTES (1,2)		1
	PEAT (UNIT 6)	67	27	0	27	0	N/A	N/A	2
	DENSE SAND AND GRAVEL (UNIT 4)	120	33	0	33	0	N/A	N/A	2
	SILT/CLAY (UNIT 3)	120	30	0	30	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	2
	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	2

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY – STATIC – PEAK	FACTOR OF SAFETY – STATIC – RESIDUAL	POST LIQUEFACTION (MDE)	POST LIQUEFACTION (MDE) WITH BERM
A–A	1.9			
B–B		1.8		
C–C			0.9	
D–D				1.1

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

SCALE: 100 0 100 FT

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

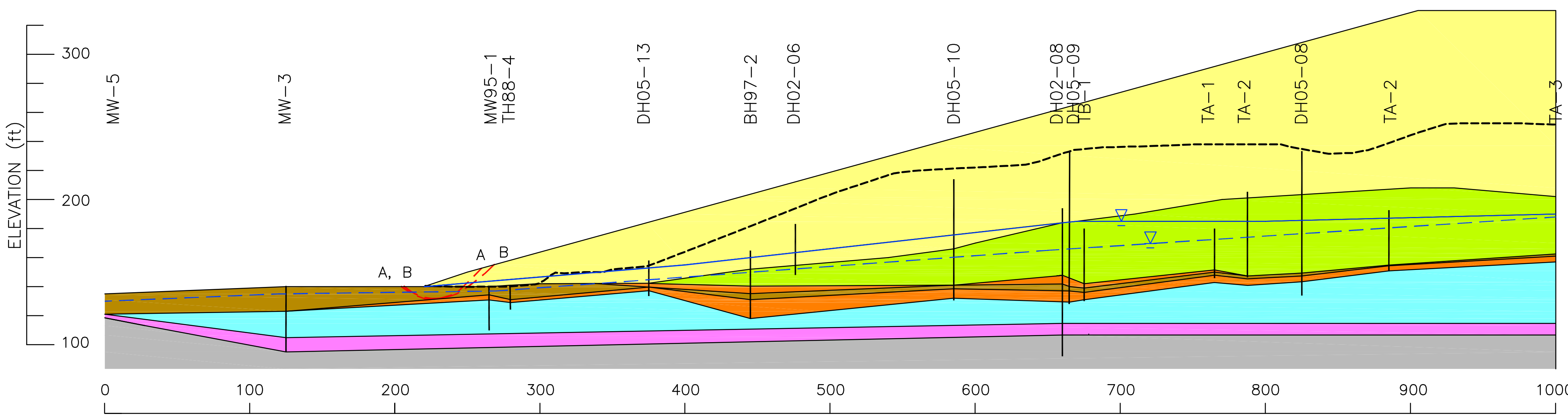
KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: STABILITY SECTION 4 – WEST BUTTRESS

PROJECT No. M07802A41

FIG. No. VIII–5



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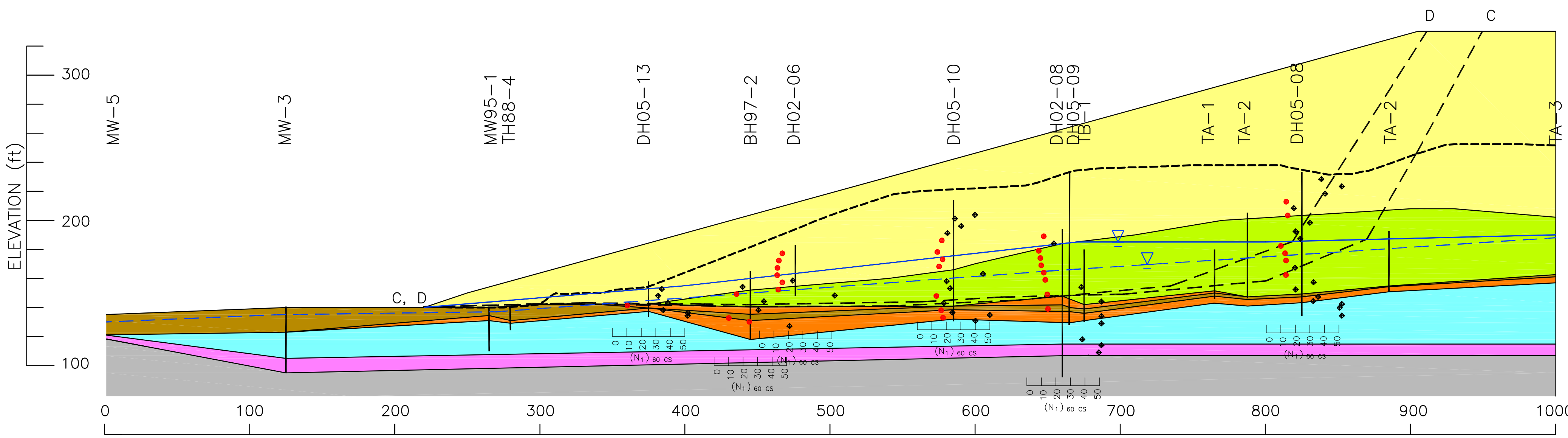
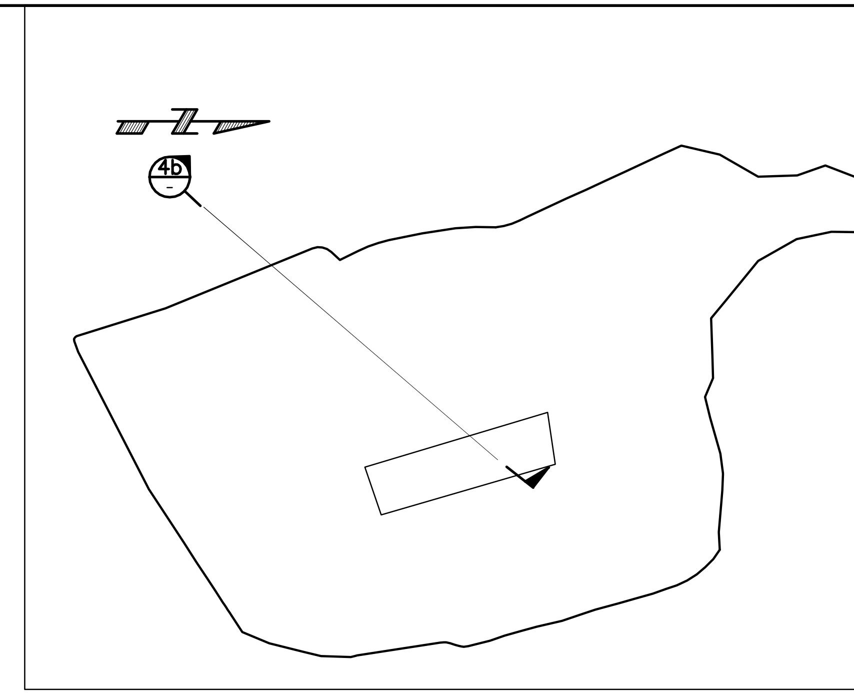


POST LIQUEFACTION



KEY PLAN

LEGEND

FAILURE SURFACE – STATIC

FAILURE SURFACE – POST–LIQUEFACTION

PIEZOMETRIC SURFACE 1

PIEZOMETRIC SURFACE 2

$(N_1)_{60\ cs}$ WHERE FS MDE < 1.1

$(N_1)_{60\ cs}$ WHERE FS MDE > 1.1

TAILINGS SURFACE DEC. 2005

NOTES:

1. POST–LIQUEFACTION STRENGTH FUNCTION FOR NEW AND OLD TAILINGS AND POST–EARTHQUAKE STRENGTH FUNCTION FOR OLD TAILINGS VARIES LINEARLY BETWEEN:
   Sr = 324 psf @ σ_N = 0 psf; AND
   Sr = 2297 psf @ σ_N > 18880 psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.
4. POST–EARTHQUAKE ANALYSIS USES THE FOLLOWING PROPERTIES FOR TAILINGS BELOW THE WATER TABLE:
   - POST–LIQUEFACTION STRENGTH FOR OLD TAILINGS
   - PEAK STATIC STRENGTH REDUCED BY 33% FOR NEW TAILINGS

   TAILINGS ABOVE THE WATER TABLE AND ALL OTHER MATERIALS ARE ASSIGNED RESIDUAL STATIC STRENGTHS.

100 0 100 FT

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	OLD TAILINGS	120	33	0	32	0	NOTES (1,2)		1
	PEAT (UNIT 6)	67	27	0	27	0	N/A	N/A	1
	DENSE SAND AND GRAVEL (UNIT 4)	120	33	0	33	0	N/A	N/A	2
	SILT/CLAY (UNIT 3)	120	30	0	30	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	2
	BEDROCK (UNIT 1)	–	–	–	–	0	N/A	N/A	2

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY – STATIC PEAK	STATIC RESIDUAL	POST EARTHQUAKE (MDE) 4	POST LIQUEFACTION (MDE)
A–A	1.7			
B–B		1.6		
C–C			1.1	
D–D				1.0

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

SCALE

CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

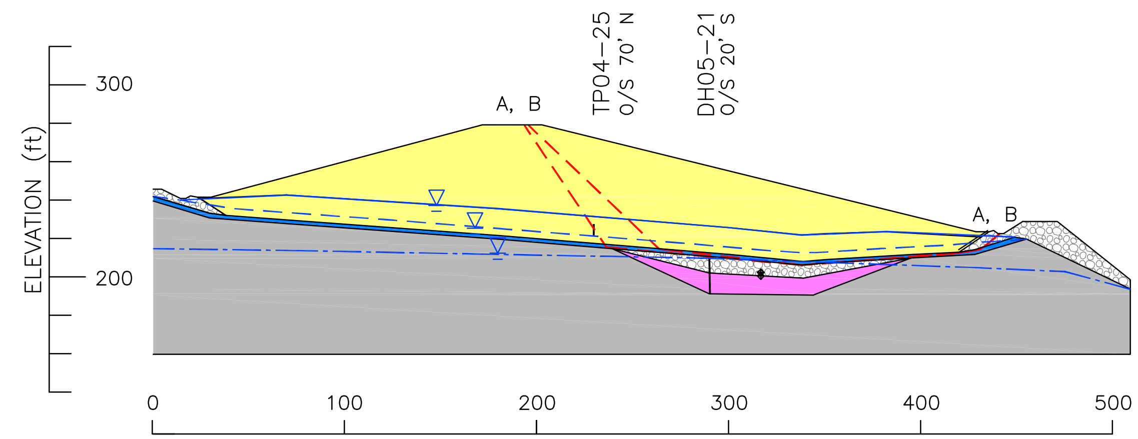
TITLE
STABILITY SECTION 4b – WEST BUTTRESS

PROJECT No. M07802A41

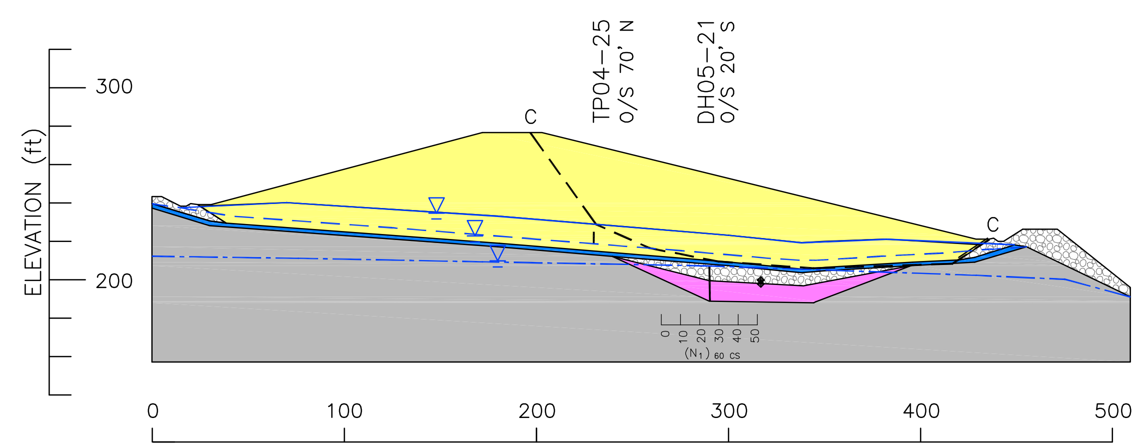
FIG. No. VIII–6

KCR–B

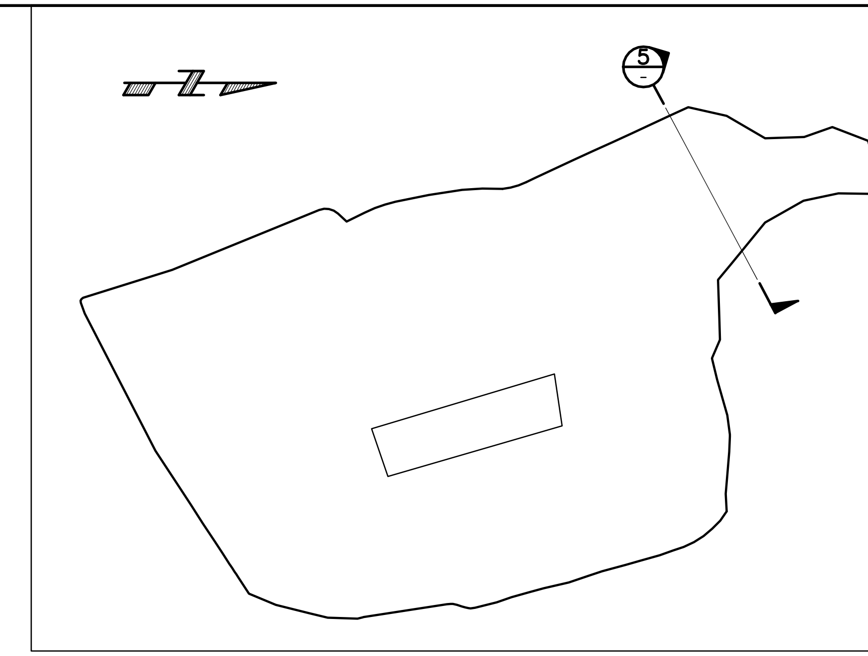
Date: 3/1/2006
Scale: 1=0(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\B–FIG_StabilitySections\RA2006r1.dwg (cwong)



STATIC



POST LIQUEFACTION



KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- FAILURE SURFACE – POST–LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- PIEZOMETRIC SURFACE 3
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1

NOTES:

1. POST–LIQUEFACTION STRENGTH FUNCTION FOR NEW AND OLD TAILINGS AND POST–EARTHQUAKE STRENGTH FUNCTION FOR OLD TAILINGS VARIES LINEARLY BETWEEN:
   Sr = 324 psf @ σ_N = 0 psf; AND
   Sr = 2297 psf @ σ_N > 18880 psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.
4. NORTHWEST LINER SYSTEM WILL BE DESIGNED TO HAVE 16° MINIMUM RESIDUAL STRENGTH.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIUAL FRICTION ANGLE (deg)	STATIC–RESIUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	GEOSYNTHETIC LINER SYSTEM	125	24.2	0	16 [4]	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	3
	ROCK FILL	120	40	0	40	0	N/A	N/A	3
	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	3

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC		FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A–A	2.7		
B–B		1.8	
C–C			1.3

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

SCALE 100 0 100 FT

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

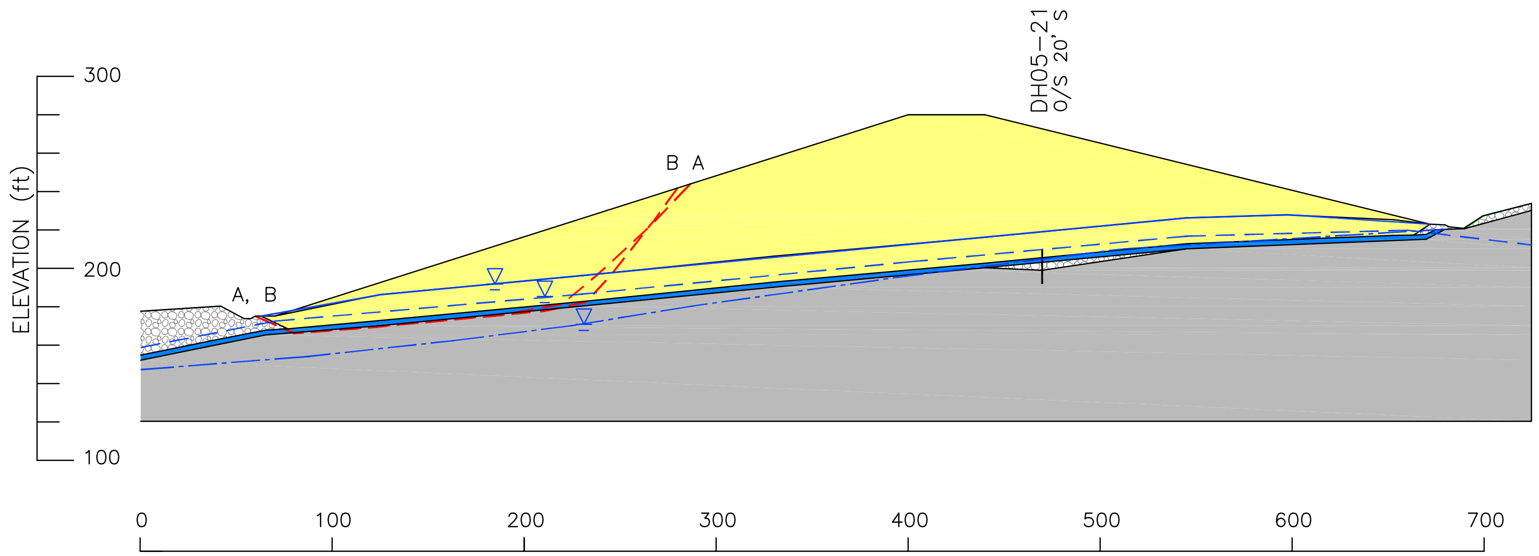
TITLE: STABILITY SECTION 5a – NORTHWEST

PROJECT No. M07802A41

FIG. No. VIII–7

KCR-B

Date: 3/1/2006
Scale: 1=0(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\B-FIG_StabilitySectionsRA2006r1.dwg (cwong)



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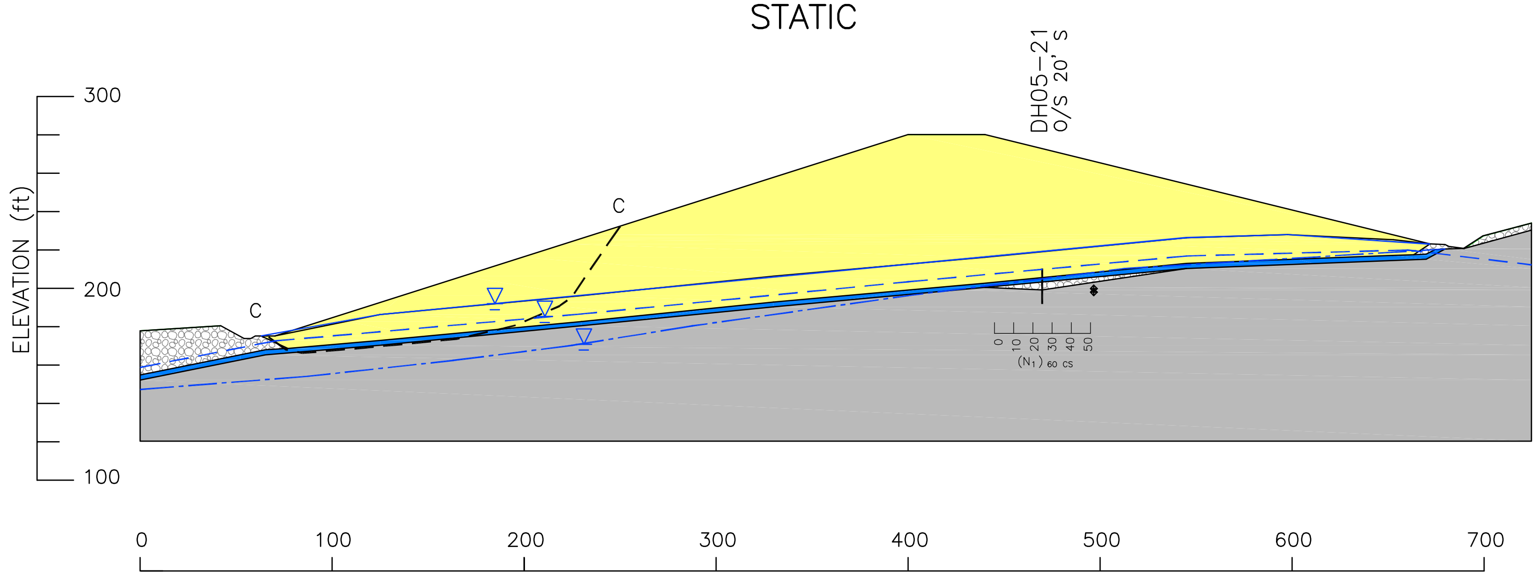
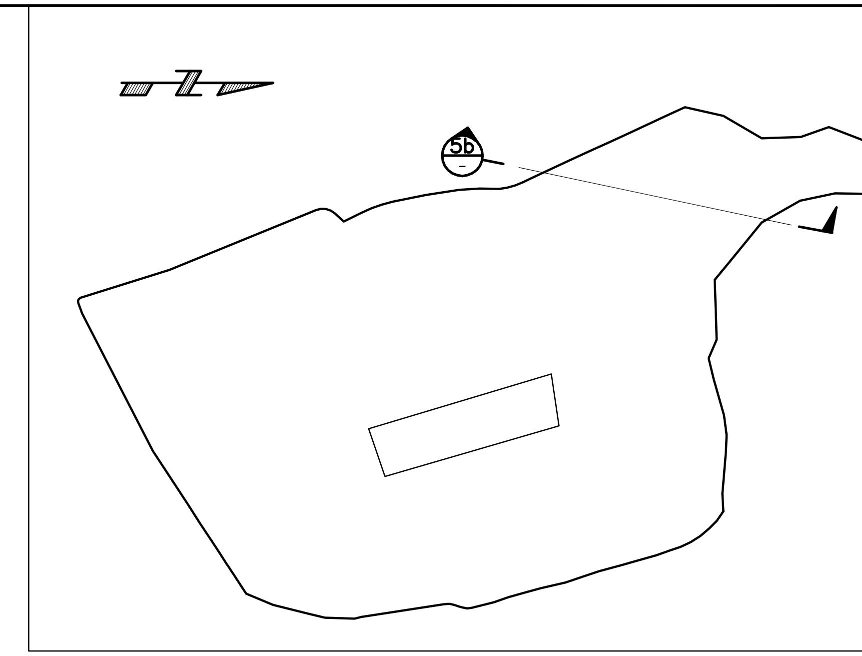


POST LIQUEFACTION



KEY PLAN

LEGEND

- - - - - - FAILURE SURFACE – STATIC

- - - - - - FAILURE SURFACE – POST–LIQUEFACTION

PIEZOMETRIC SURFACE 1

PIEZOMETRIC SURFACE 2

PIEZOMETRIC SURFACE 3

• $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1

• $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1

NOTES:

1. POST–LIQUEFACTION STRENGTH FUNCTION FOR NEW AND OLD TAILINGS AND POST–EARTHQUAKE STRENGTH FUNCTION FOR OLD TAILINGS VARIES LINEARLY BETWEEN:
   S_r = 324 psf @ σ_N = 0 psf; AND
   S_r = 2297 psf @ σ_N > 18880 psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.
4. NORTHWEST LINER SYSTEM WILL BE DESIGNED TO HAVE 16° MINIMUM RESIDUAL STRENGTH.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
(yellow)	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
(blue)	GEOSYNTHETIC LINER SYSTEM	125	24.2	0	16 [4]	0	N/A	N/A	2
(hatched)	ROCK FILL	120	40	0	40	0	N/A	N/A	3
(grey)	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	3

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC		FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A–A	1.8		
B–B		1.3	
C–C			1.0

NOT FOR CONSTRUCTION

100 0 100 FT

SCALE

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE
STABILITY SECTION 5b – NORTHWEST

PROJECT No. M07802A41

FIG. No. VIII–8

KCR-B

Date: 3/1/2006
Scale: 1=0(PS)
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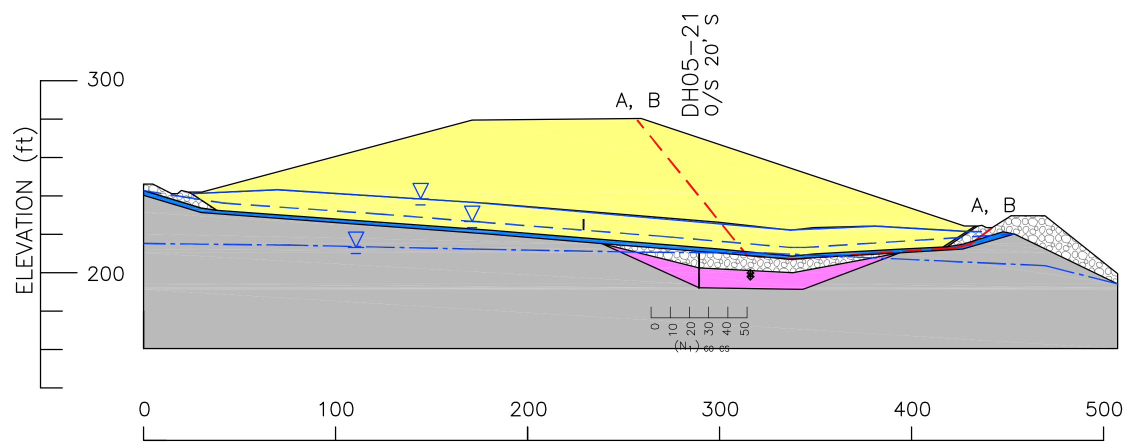

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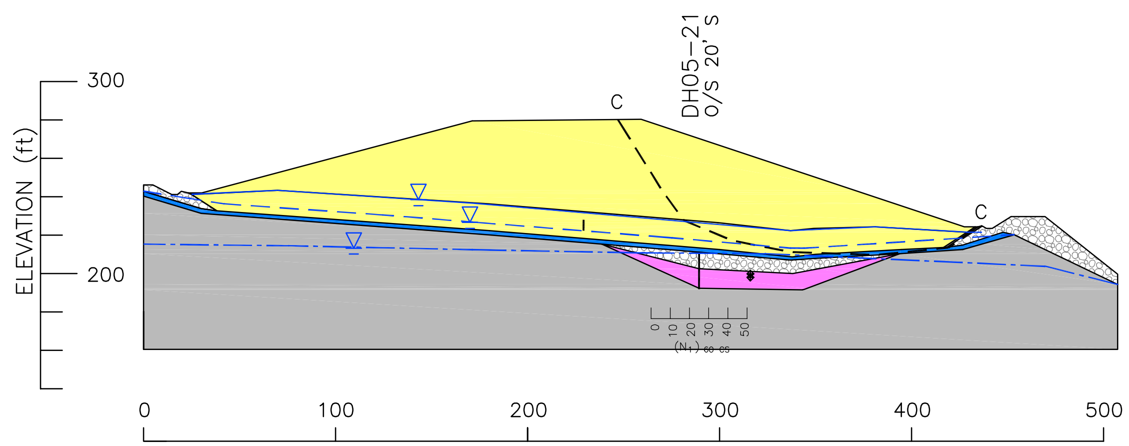
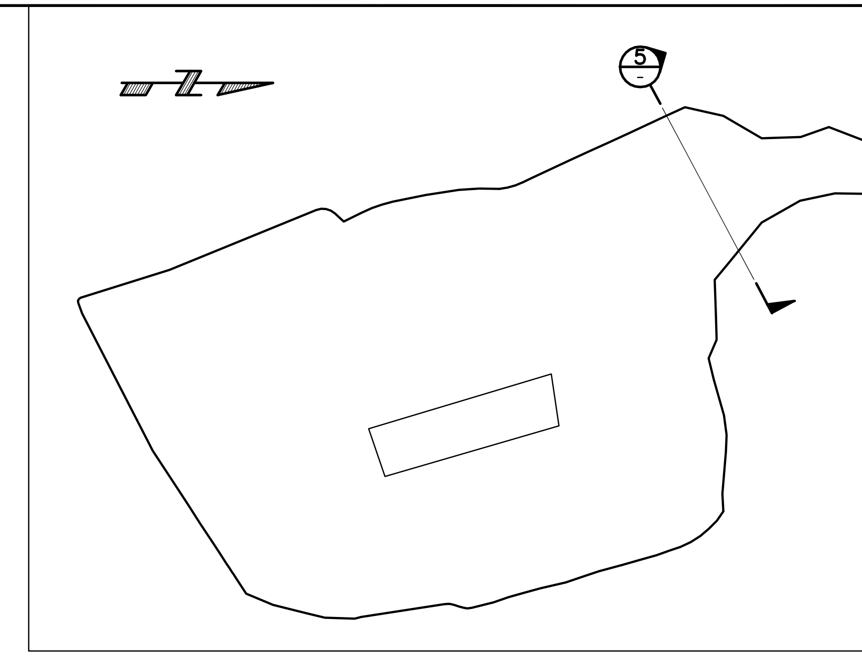


POST LIQUEFACTION



KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- FAILURE SURFACE – POST–LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- PIEZOMETRIC SURFACE 3
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	NOTES (1,2)		1
	GEOSYNTHETIC LINER SYSTEM	125	24.2	0	16 [4]	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	3
	ROCK FILL	120	40	0	40	0	N/A	N/A	3
	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	3

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC		FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A–A	2.5		
B–B		1.7	
C–C			1.2

NOTES:

1. POST–LIQUEFACTION STRENGTH FUNCTION FOR NEW AND OLD TAILINGS AND POST–EARTHQUAKE STRENGTH FUNCTION FOR OLD TAILINGS VARIES LINEARLY BETWEEN:
   $Sr = 324$ psf @ $\sigma_N = 0$ psf; AND
   $Sr = 2297$ psf @ $\sigma_N > 18880$ psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.
4. NORTHWEST LINER SYSTEM WILL BE DESIGNED TO HAVE 16° MINIMUM RESIDUAL STRENGTH.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

SCALE 100 0 100 FT

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

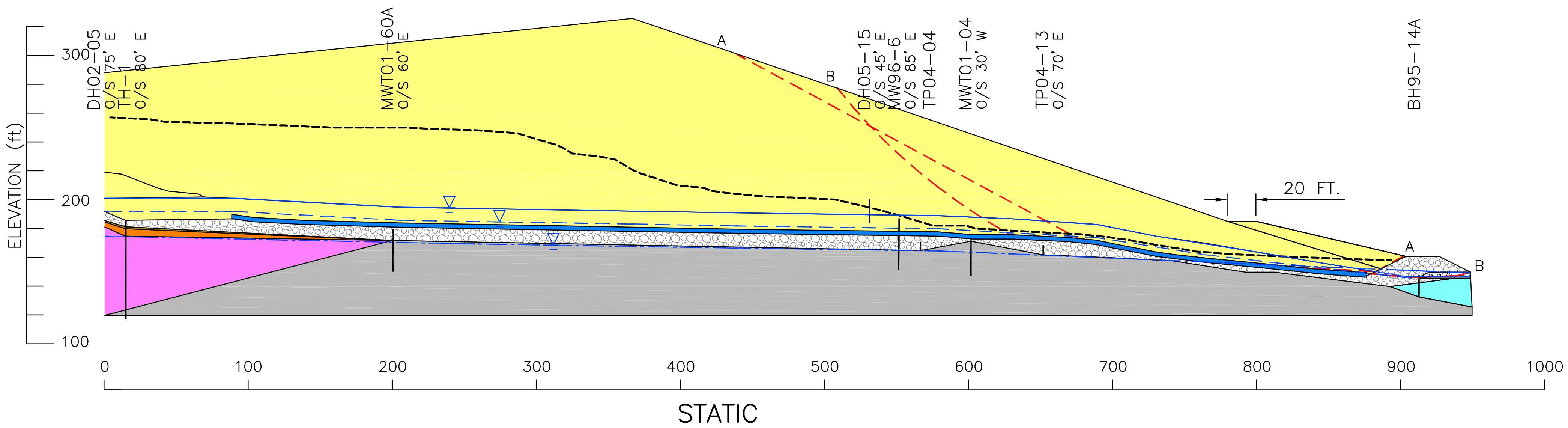
TITLE: STABILITY SECTION 5c – NORTHWEST

PROJECT No. M07802A41

FIG. No. VIII–9

KCR-B

Date: 3/1/2006
Scale: 1=0(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\B-FIG_StabilitySectionsRA2006r1.dwg (cwong)



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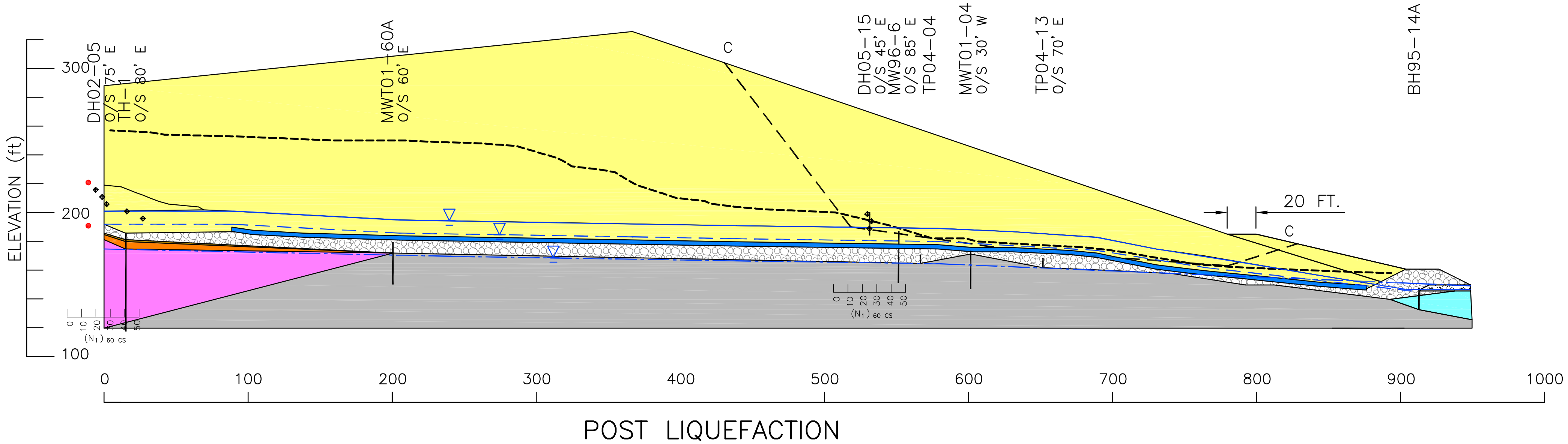
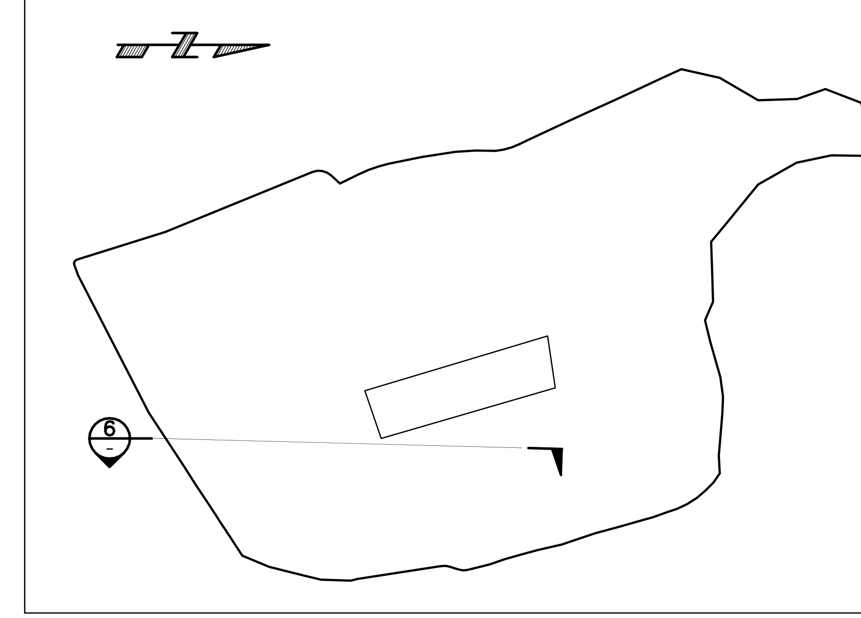


POST LIQUEFACTION



KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- FAILURE SURFACE – POST–LIQUEFACTION
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- PIEZOMETRIC SURFACE 3
- $(N_1)_{60\ cs}$ WHERE FS MDE < 1.1
- $(N_1)_{60\ cs}$ WHERE FS MDE > 1.1
- TAILINGS SURFACE DEC. 2005

NOTES:

1. POST–LIQUEFACTION STRENGTH FUNCTION FOR NEW AND OLD TAILINGS AND POST–EARTHQUAKE STRENGTH FUNCTION FOR OLD TAILINGS VARIES LINEARLY BETWEEN:
   Sr = 324 psf @ σ_N = 0 psf; AND
   Sr = 2297 psf @ σ_N > 18880 psf
2. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
3. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	POST LIQUEFACTION FRICTION ANGLE (deg)	POST LIQUEFACTION COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	42	0	32	0	NOTES (1,2)		1
	GEOSYNTHETIC LINER SYSTEM	120	24.2	0	12.5	0	NOTES (1,2)		2
	COMPACTED ROCK FILL	120	40	0	40	0	N/A	N/A	3
	PEAT (UNIT 6)	67	27	0	27	0	N/A	N/A	2
	DENSE SAND AND GRAVEL (UNIT 4)	120	33	0	33	0	N/A	N/A	2
	SILT/CLAY (UNIT 3)	120	30	0	30	0	N/A	N/A	2
	SILTY SANDY TILL (UNIT 2)	120	33	0	33	0	N/A	N/A	2
	BEDROCK (UNIT 1)	–	–	–	–	–	N/A	N/A	2
	ROADFILL/NATIVE	130	36	0	36	0	N/A	N/A	2

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC		FACTOR OF SAFETY POST LIQUEFACTION (MDE)
A–A	2.1		
B–B		1.3	
C–C			1.1

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

SCALE 100 0 100 FT

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

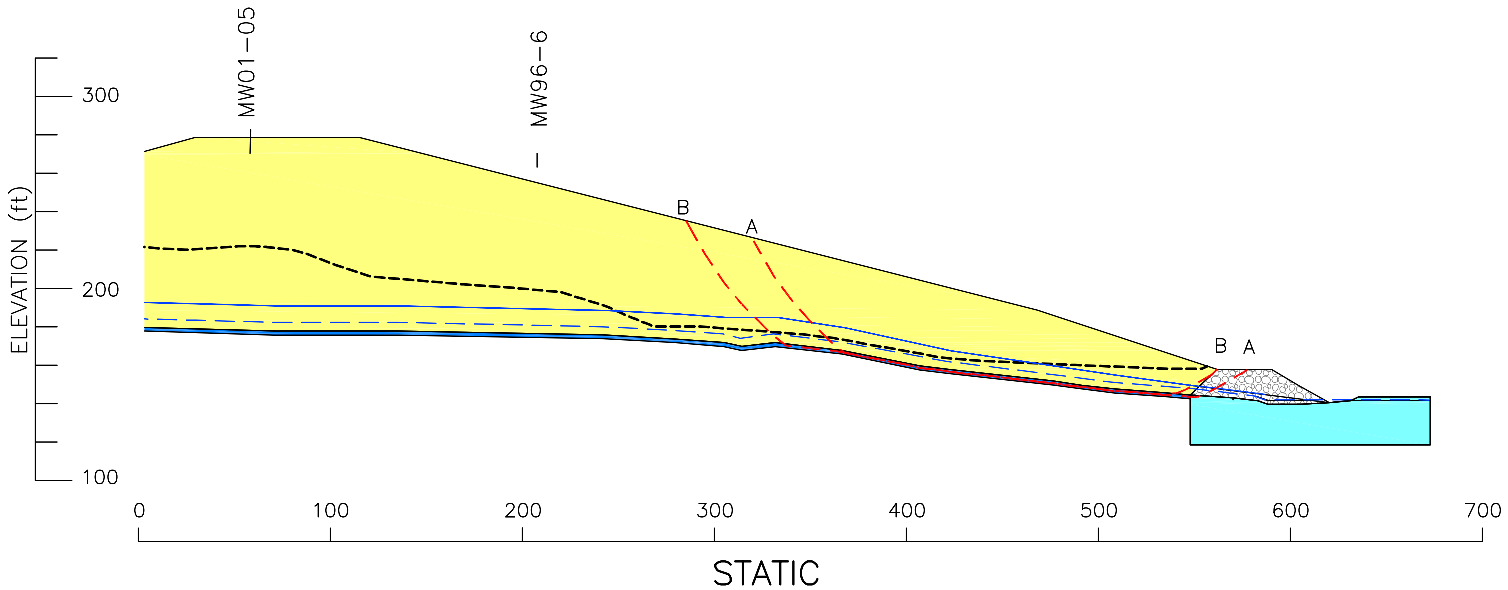
PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

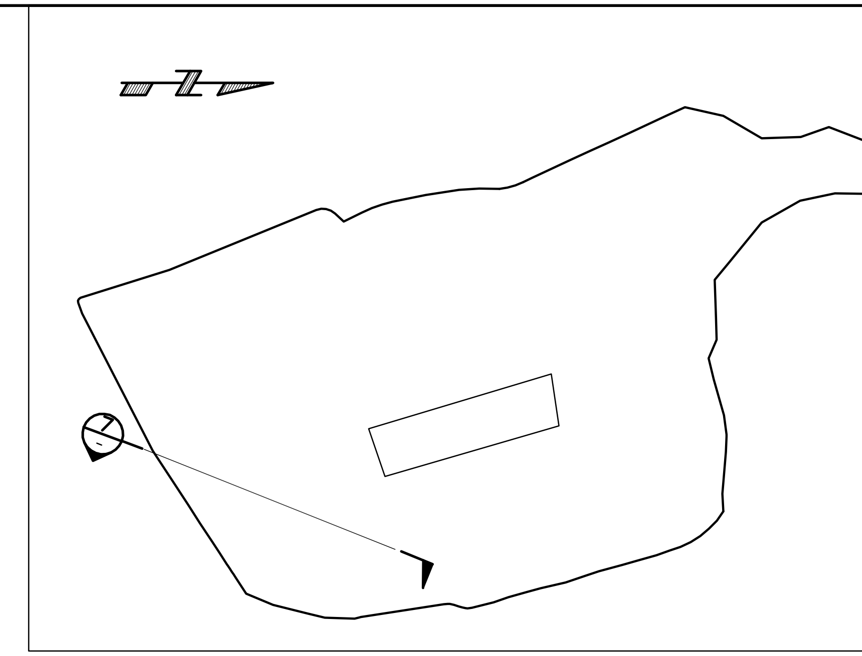
TITLE: STABILITY SECTION 6 – SOUTHEAST

PROJECT No. M07802A41

FIG. No. VIII–10

Date: 3/1/2006
Scale: 1=0(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\B-FIG_StabilitySectionsRA2006r1.dwg (cwong)
KCR-B





KEY PLAN

LEGEND

- FAILURE SURFACE – STATIC
- PIEZOMETRIC SURFACE 1
- PIEZOMETRIC SURFACE 2
- TAILINGS SURFACE DEC. 2005

NOTES:

1. STATIC STRENGTHS APPLY ABOVE PIEZOMETRIC SURFACE 1.
2. N/A INDICATES THAT THE SOIL DOES NOT LIQUEFY DURING THE MDE, THEREFORE THE STATIC PROPERTIES ARE USED IN THE POST–LIQUEFACTION ANALYSIS.
3. FOUNDATION MATERIALS ARE NOT SHOWN BECAUSE FAILURE SURFACE WAS PURPOSEFULLY FORCED ALONG THE LINER.

SOIL TYPE AND DESCRIPTION

ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	STATIC–PEAK FRICTION ANGLE (deg)	STATIC–PEAK COHESION (psf)	STATIC–RESIDUAL FRICTION ANGLE (deg)	STATIC–RESIDUAL COHESION (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	39	0	32	0	1
	GEOSYNTHETIC LINER SYSTEM	125	24.2	0	12.5	0	2
	COMPACTED ROCK FILL	120	40	0	40	0	1
	SILT/CLAY (UNIT 3)	120	30	0	30	0	1

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY STATIC	
A–A	2.1	
B–B		1.3

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

SCALE: 100 0 100 FT

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

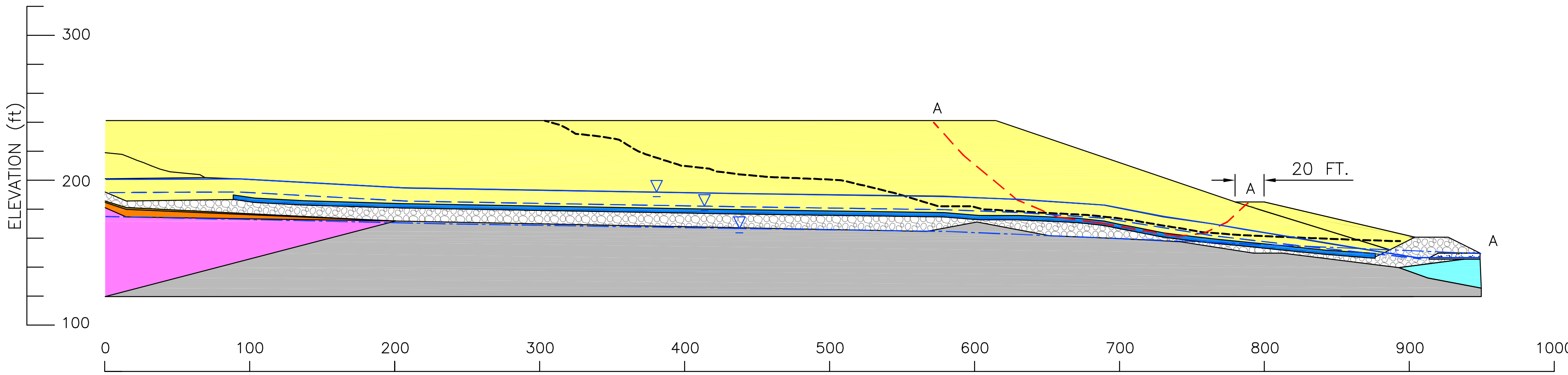
TITLE: SOUTHEAST 2 – SECTION 7 INTERIM CONSTRUCTION STABILITY ANALYSIS – STATIC

PROJECT No. M07802A41

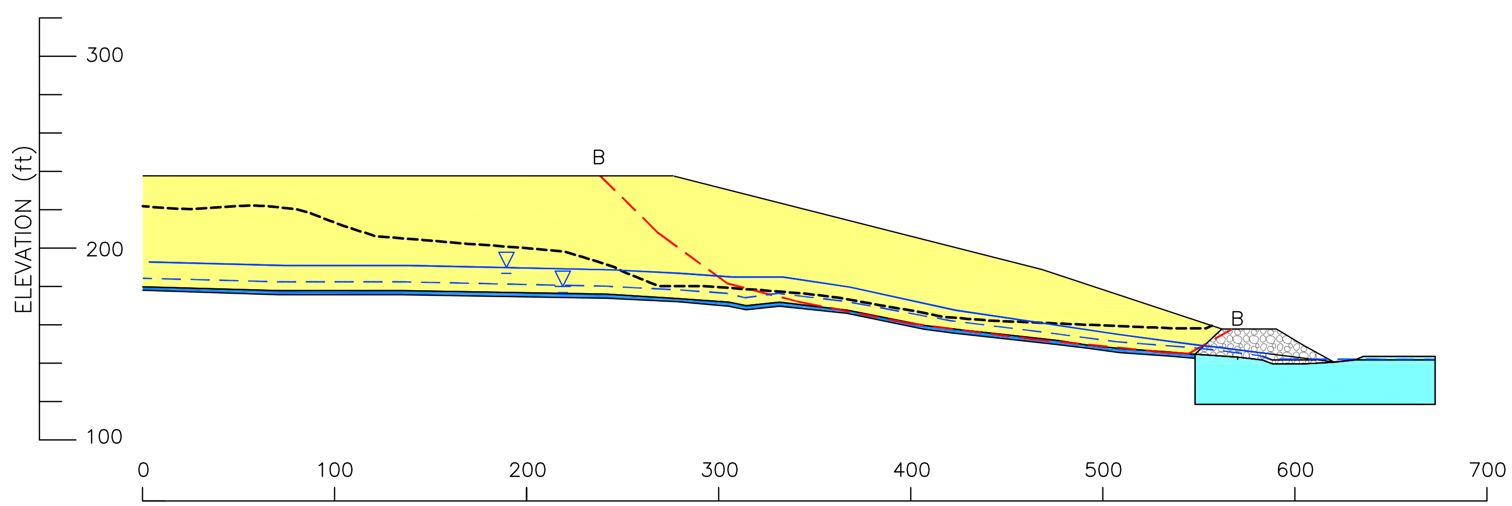
FIG. No. VIII–11

KCR-B

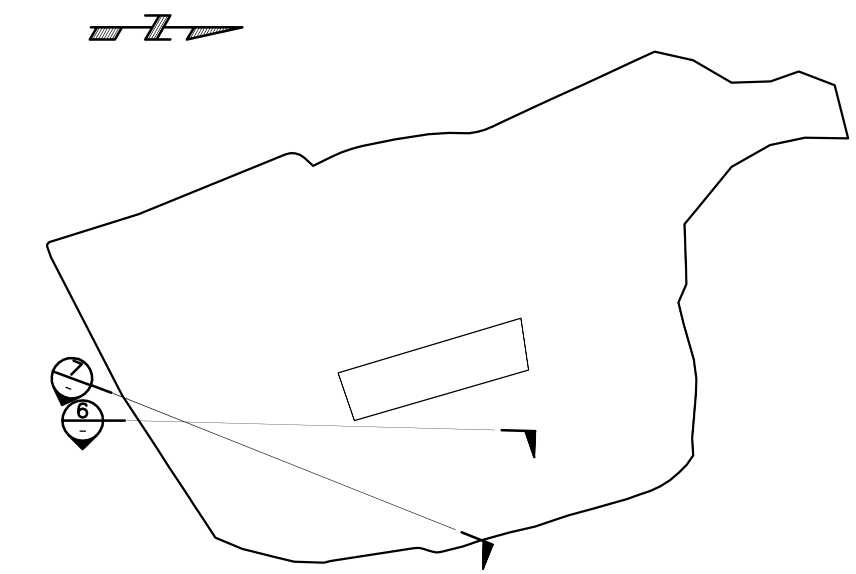
Date: 3/1/2006
Scale: 1=0(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\B-FIG_StabilitySectionsRA2006r1.dwg (cwong)



SECTION 6 – UNDRAINED



SECTION 7 – UNDRAINED



KEY PLAN

LEGEND

FAILURE SURFACE – STATIC

PIEZOMETRIC SURFACE 1

PIEZOMETRIC SURFACE 2

PIEZOMETRIC SURFACE 3

TAILINGS SURFACE DEC. 2005

NOTES:

1. IN THIS ANALYSIS, UNDRAINED STRENGTH WAS USED FOR THE TAILINGS AND PEAK STRENGTHS WERE USED FOR ALL OTHER MATERIALS.
2. SECTION 7 FOUNDATION MATERIALS ARE NOT SHOWN BECAUSE FAILURE SURFACE WAS PURPOSEFULLY FORCED ALONG THE LINER.

SOIL TYPE AND DESCRIPTION

			STATIC–PEAK[1]		UNDRAINED STRENGTH[1]	
ZONE COLOUR	SOIL TYPE	UNIT WEIGHT (pcf)	FRICTION ANGLE (deg)	COHESION (psf)	Su (psf)	PIEZOMETRIC SURFACE
	NEW TAILINGS	128	–	–	1500	–
	GEOSYNTHETIC LINER SYSTEM	125	24.2	0	–	2
	COMPACTED ROCK FILL	120	40	0	–	3
	PEAT (UNIT 6)	67	27	0	–	3
	DENSE SAND AND GRAVEL (UNIT 4)	120	33	0	–	3
	SILT/CLAY (UNIT 3)	120	30	0	–	3
	SILTY SANDY TILL (UNIT 2)	120	33	0	–	3
	BEDROCK (UNIT 1)	–	–	–	–	3
	ROADFILL/NATIVE	130	36	0	–	3

CALCULATED FACTOR OF SAFETY

SLIP SURFACE	FACTOR OF SAFETY	
	SECTION 6	SECTION 7
A–A	1.6	
B–B		1.5

NOT FOR CONSTRUCTION

SCALE: 100 0 100 FT

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

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CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

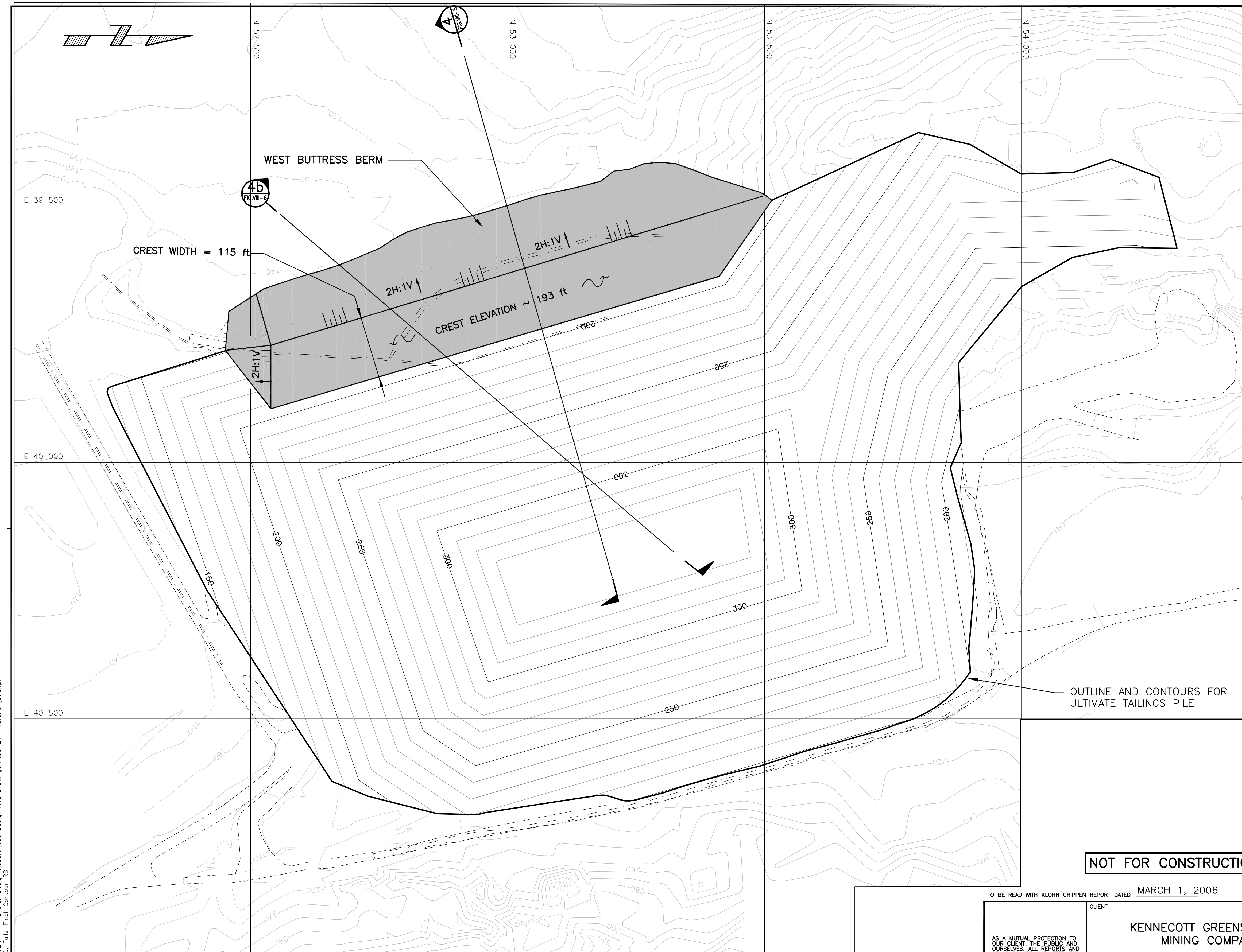
TITLE: SOUTHEAST 2 INTERIM CONSTRUCTION STABILITY ANALYSIS – UNDRAINED STRENGTH

PROJECT No. M07802A41

FIG. No. VIII–12

KCR–B

Date: 3/1/2006
Scale: 1=0(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\B-FIG_StabilitySectionsRA2006r1.dwg (cwong)



LEGEND

WEST BUTTRESS BERM

SLURRY WALL

ROAD

NOTES:

1. BASE TOPOGRAPHY PROVIDED BY KGCMC OCTOBER 2003.
2. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.

NOT FOR CONSTRUCTION

0 500 FT

SCALE

TO BE READ WITH KLOHN CRIPPEN REPORT DATED MARCH 1, 2006

CLIENT

KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT

STAGE 2 TAILINGS EXPANSION
OVERAL STABILITY UPDATE

TITLE

FINAL TAILINGS PILE PLAN
SHOWING WEST BUTTRESS BERM

PROJECT No. M07802 A41

FIG. No. VIII–13

APPENDIX IX

Stage 2 Expansion Design Drawings

D-35003	**Stage 2 Final Configuration Plan**
D-35010	**Northwest Corner Excavation and Grading Plan**
D-35012	**Northwest Corner Ditch and Road Sections**
D-35013	**Northeast Corner Excavation and Grading Plan**
D-35015	**Southeast Corner, Pond 6, and Southwest Corner Grading Plan**
D-39011	**Southeast 2 Final Grading – Plan**
D-39025	**Pond 7 – Grading Surface and Berm Layout**
	Tails As-Built 11-30-05 (KGCMC Drawing)

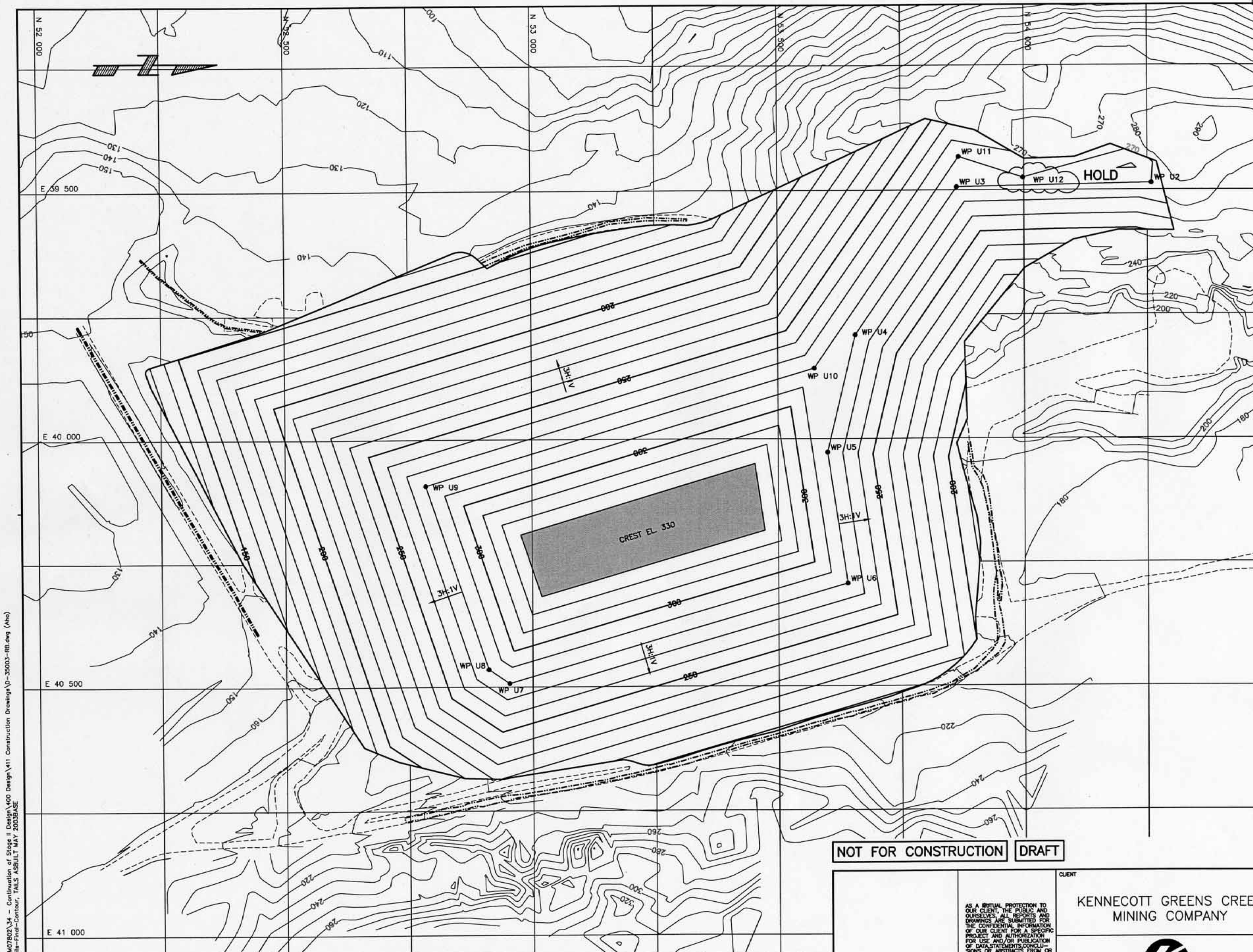


TABLE OF WORK POINTS

WORK POINT No.	EASTING (FT)	NORTHING (FT)	ELEVATION (FT)
U2	39 486	54 258	280
U3	39 494	53 863	280
U4	39 790	53 657	280
U5	40 027	53 601	280
U6	40 292	53 640	280
U7	40 493	52 952	280
U8	40 465	52 910	280
U9	40 093	52 785	280
U10	39 857	53 574	280
U11	39 433	53 867	280
U12	39 475	53 997	280

HOLD

NOTES:

1. BASE TOPOGRAPHY PROVIDED BY KGCMC OCTOBER 2003.
2. ALL DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.

LEGEND:

- ULTIMATE NOMINAL TAILINGS TOELINE
- —250— ULTIMATE TAILINGS PILE CONTOUR
- DITCH CENTRELINE
- EXISTING ROAD
- EXISTING SLURRY WALL

SCALE: 100 0 100 FT

NOT FOR CONSTRUCTION | DRAFT

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

PROJECT: STAGE 2 EXPANSION OF TAILINGS FACILITY

TITLE: STAGE 2 FINAL CONFIGURATION PLAN

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802A35	D-35003	B

CANCEL PRINTS BEARING PREVIOUS REVISION

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
B	APR 2, 2004	75% COMPLETE – ISSUED FOR DESIGN OVERVIEW REPORT				
A	JAN 30, 2004	75% COMPLETE – ISSUED FOR CLIENT REVIEW				

DRAWING NO.	REFERENCE DRAWING

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Xrefs: Ponds, Tails-Final-Contour, TAILS ASBUILT MAY 2003BASE

KCC-D

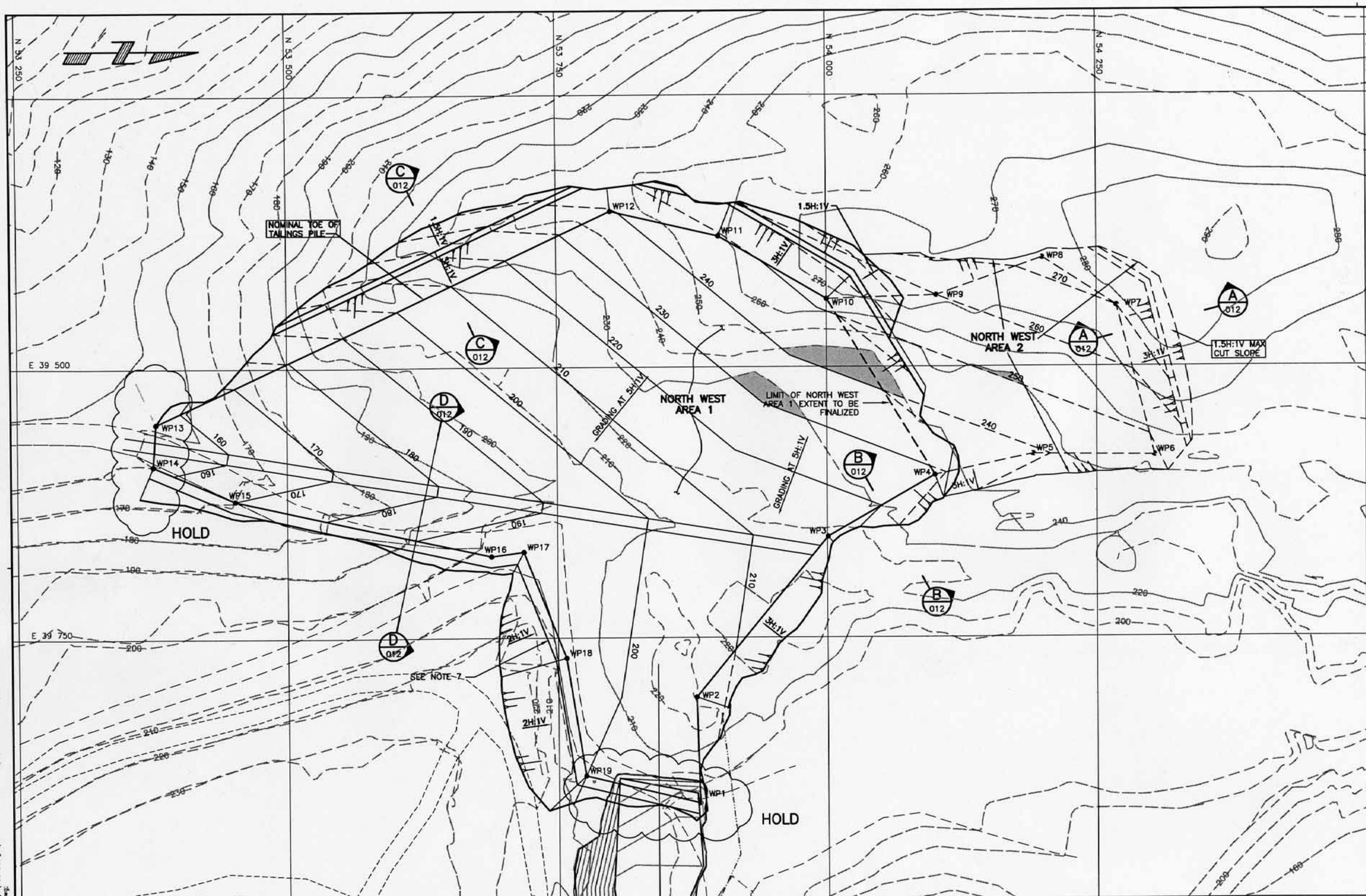


TABLE OF WORK POINTS

WORK POINT No.	EASTING (FT)	NORTHING (FT)	ELEVATION (FT)	
1	39 903	53 881		HOLD
2	39 805	53 878	207	
3	39 657	54 001	214	
4	39 600	54 100	229	
5	39 581	54 191	239	
6	39 582	54 304	247	
7	39 444	54 269	270	
8	39 400	54 200	273	
9	39 434	54 102	260	
10	39 438	54 000	252	
11	39 380	53 900	249	
12	39 357	53 800	239	
13	39 551	53 379	156	
14	39 590	53 376	157	
15	39 622	53 452	170	
16	39 675	53 688	191	
17	39 671	53 719	191	
18	39 769	53 758	196	
19	39 878	53 775		HOLD

NOTES:

1. GRADING SURFACE IS PROPOSED BASE OF EXCAVATION. IF ROCK OR COMPETENT SOIL IS NOT EXPOSED, SUB-EXCAVATE BELOW GRADING SURFACE AND THEN PLACE COMPACTED ROCKFILL TO BUILD BACK TO GRADING SURFACE.
2. IN AREAS WHERE GRADING SURFACE IS ABOVE GROUND LEVEL, EXCAVATE TO SOUND MATERIAL AND BACKFILL WITH COMPACTED ROCKFILL TO ACHIEVE GRADING SURFACE ELEVATIONS.
3. LAYOUT EXCAVATION SLOPE AT 3H:1V PERPENDICULAR TO NOMINAL PILE TOE. TRUE SLOPE WILL BE STEEPER DUE TO SLOPE OF GRADING SURFACE.
4. LAYOUT BASED ON TOPOGRAPHIC SURVEY DATED OCTOBER 2003.
5. UTILITY LOCATION AND RE-ROUTING NOT SHOWN. KGCM TO PREPARE PLAN FOR REMOVAL/RELOCATION.
6. DEVELOPMENT SCHEDULE OF NORTH WEST AREA 1 AND NORTH WEST AREA 2 TO BE REVIEWED. THIS DRAWING TO BE REVISED ONCE SCHEDULE IS DEFINED.
7. EXTENT OF LINER ON THE SOUTH SIDE MAY BE EXTENDED IF RE-EXCAVATION AND REPLACEMENT OF EXISTING TAILINGS IS REQUIRED. THIS IS CURRENTLY UNDER REVIEW BY KGCM.
8. BASE TOPOGRAPHY PROVIDED BY KGCMC OCTOBER 2003.
9. ALL DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.

LEGEND:

- 220 — GRADING CONTOUR
- EXISTING ROAD
- EXISTING SLURRY WALL
- GRADING SURFACE IS ABOVE EXISTING GROUND

ALL DIMENSIONS AND ELEVATIONS ARE APPROXIMATE ONLY AND NEED TO BE CHECKED AND FINALIZED IN FINAL DESIGN

NOT FOR CONSTRUCTION **DRAFT**

SCALE: 50 0 50 FT

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
B	APR 2, 2004	75% COMPLETE – ISSUED FOR DESIGN OVERVIEW REPORT				
A	JAN 30, 2004	75% COMPLETE – ISSUED FOR CLIENT REVIEW				

DRAWING NO. | REFERENCE DRAWING

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

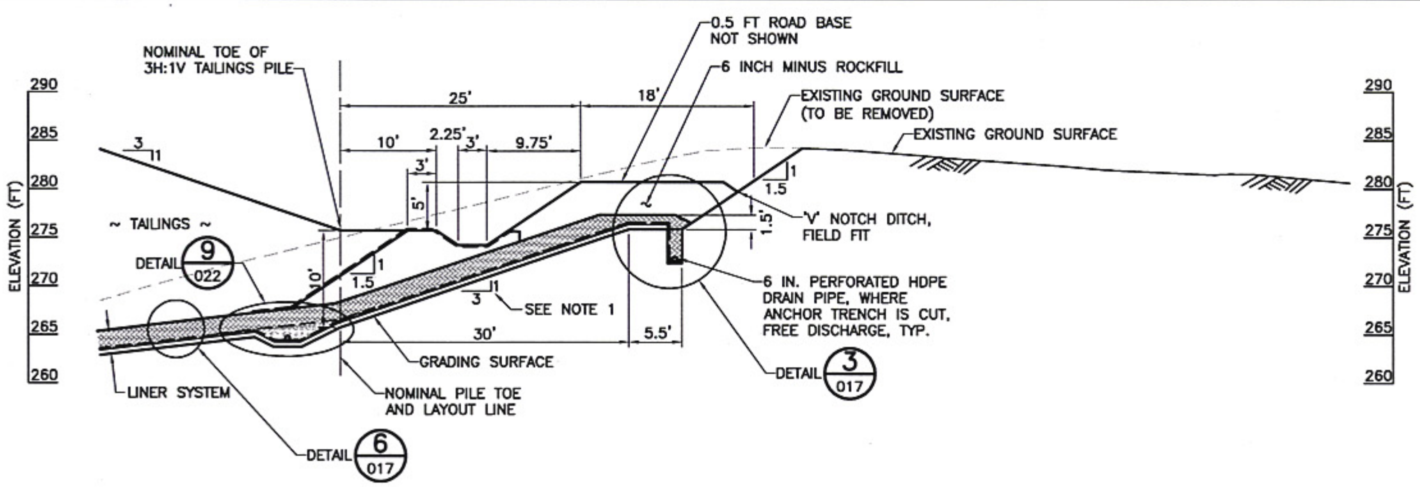
KLOHN CRIPPEN

PROJECT: STAGE 2 EXPANSION OF TAILINGS FACILITY

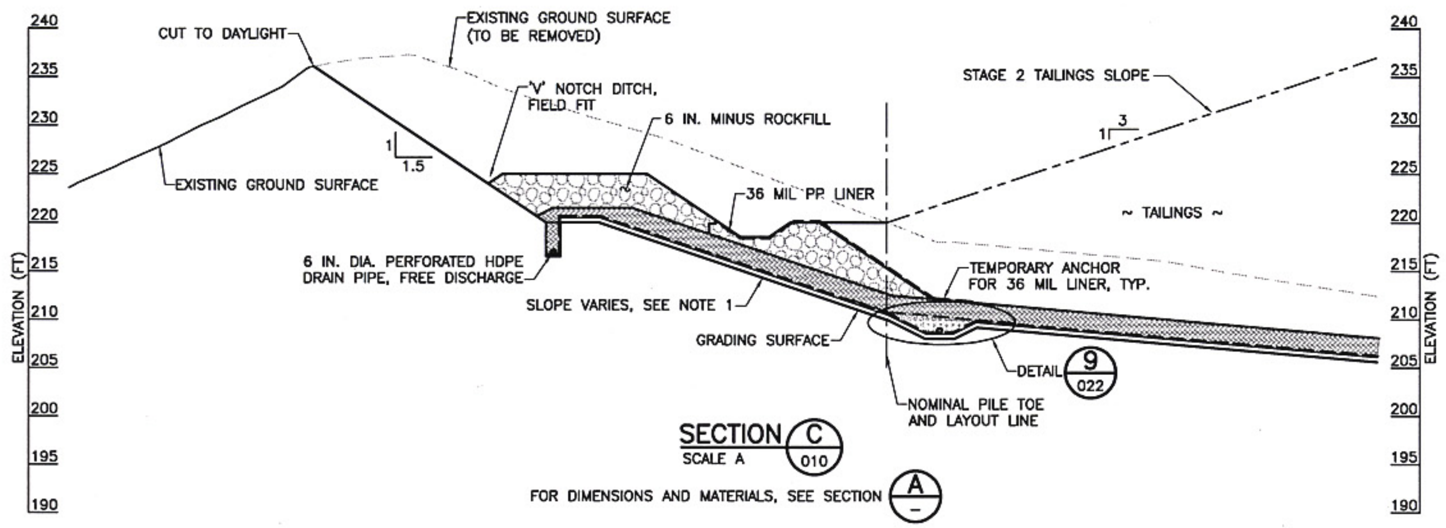
TITLE: NORTHWEST CORNER EXCAVATION AND GRADING PLAN

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802A35	D-35010	B

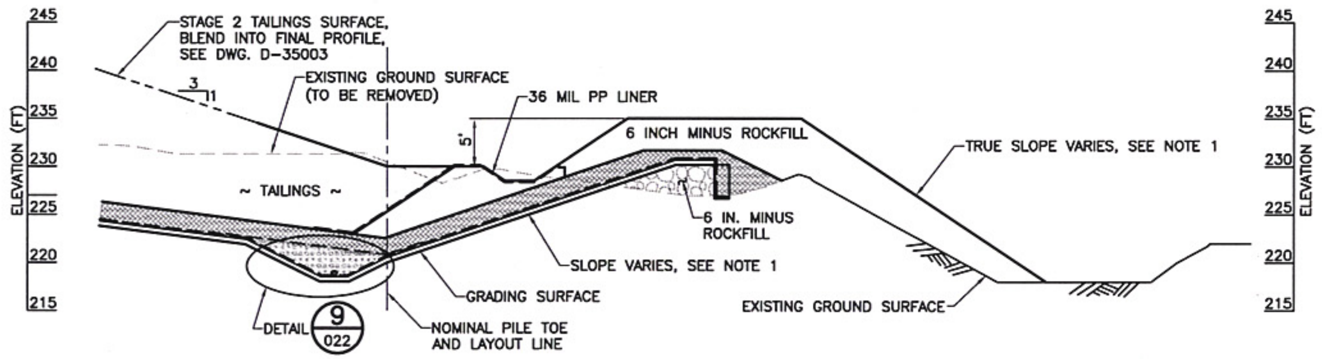
CANCEL PRINTS BEARING PREVIOUS REVISION



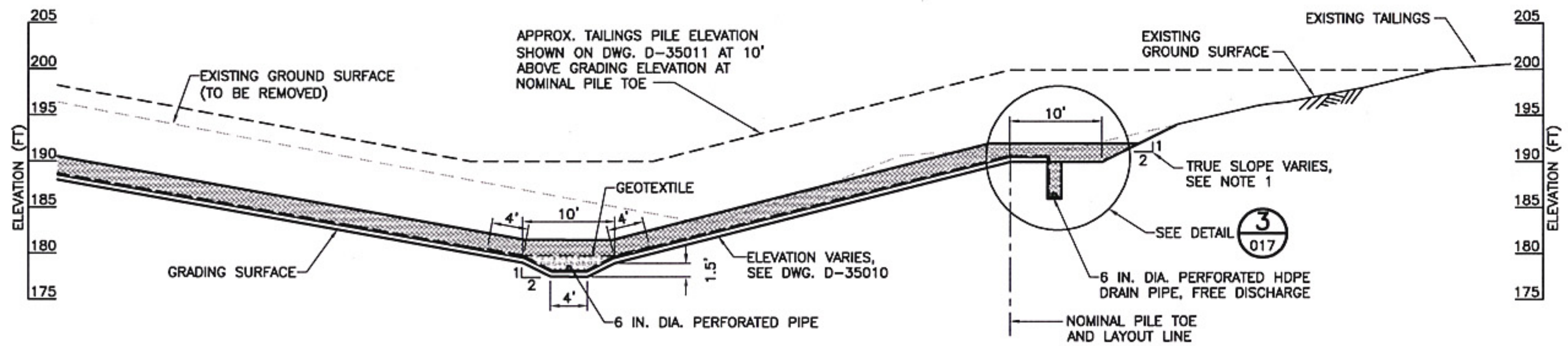
SECTION A 010
SCALE A



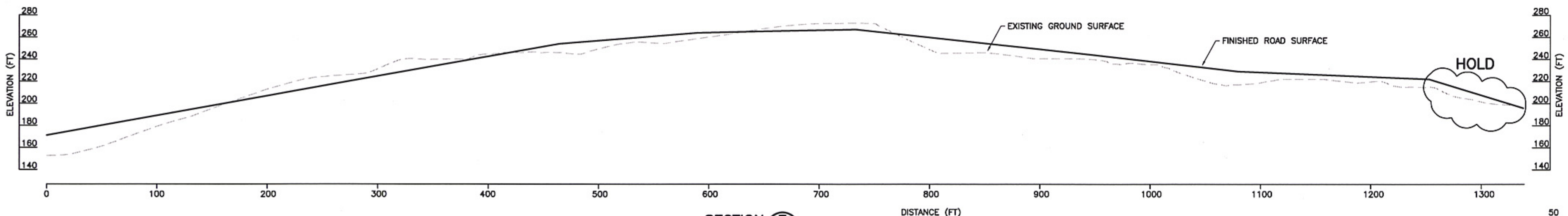
SECTION C 010
SCALE A
FOR DIMENSIONS AND MATERIALS, SEE SECTION A



SECTION B 010
SCALE A
FOR DIMENSIONS AND MATERIALS, SEE SECTION A



SECTION D 010
SCALE A



SECTION E 010
SCALE B

SCALE:
SCALE B 50 0 50 FT
SCALE A 10 0 10 FT

NOTES:

1. THE LAYOUT IS BASED ON SLOPES DEFINED ON THE DRAWING FOR CUT AND FILL. FOR EXAMPLE, A CUT SLOPE 3H:1V SET PERPENDICULAR TO THE NOMINAL PILE TOE, FOR A SLOPE LENGTH OF 30 FT AS NOTED ON SECTION A. THE SLOPE ABOVE THE ROAD CAN BE 1.5H:1V IF SOIL/ROCK CONDITIONS PERMIT. THE TRUE SLOPE VARIES DEPENDING ON THE SLOPE OF THE GRADING SURFACE, SEE DRAWING D-35010.
2. ALL DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.

ALL DIMENSIONS AND ELEVATIONS ARE APPROXIMATE ONLY AND NEED TO BE CHECKED AND FINALIZED IN FINAL DESIGN

NOT FOR CONSTRUCTION **DRAFT**

DRAWING NO.	REFERENCE DRAWING
D-35010	NORTHWEST CORNER – EXCAVATION AND GRADING PLAN

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
C	MAY 6, 2004	80% COMPLETE – ISSUED WITH QUANTITIES LETTER				
B	APR 2, 2004	75% COMPLETE – ISSUED FOR DESIGN OVERVIEW REPORT				
A	JAN 30, 2004	75% COMPLETE – ISSUED FOR CLIENT REVIEW				

CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT
STAGE 2 EXPANSION OF TAILINGS FACILITY

TITLE
NORTHWEST CORNER
DITCH AND ROAD SECTIONS

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802A35	D-35012	C

CANCEL PRINTS BEARING PREVIOUS REVISION

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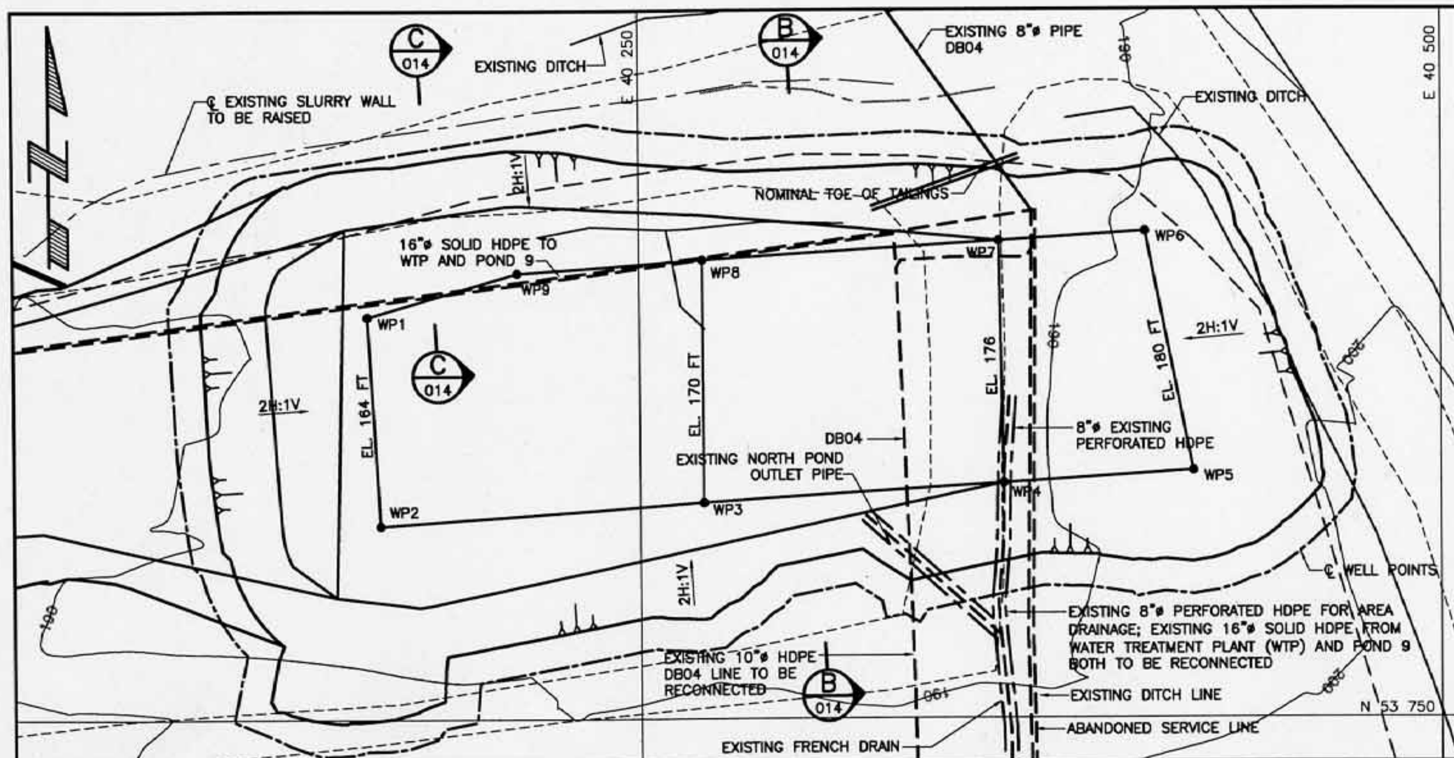
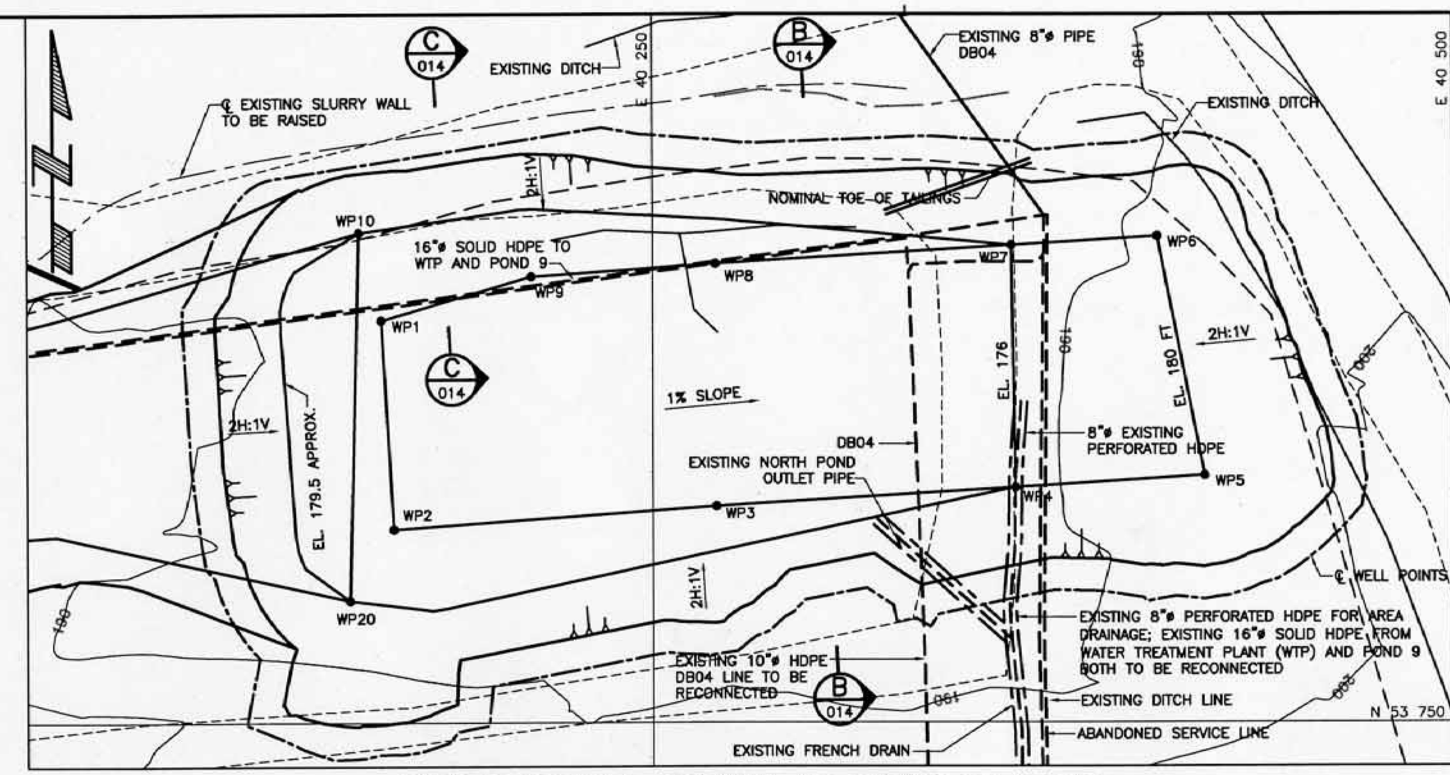
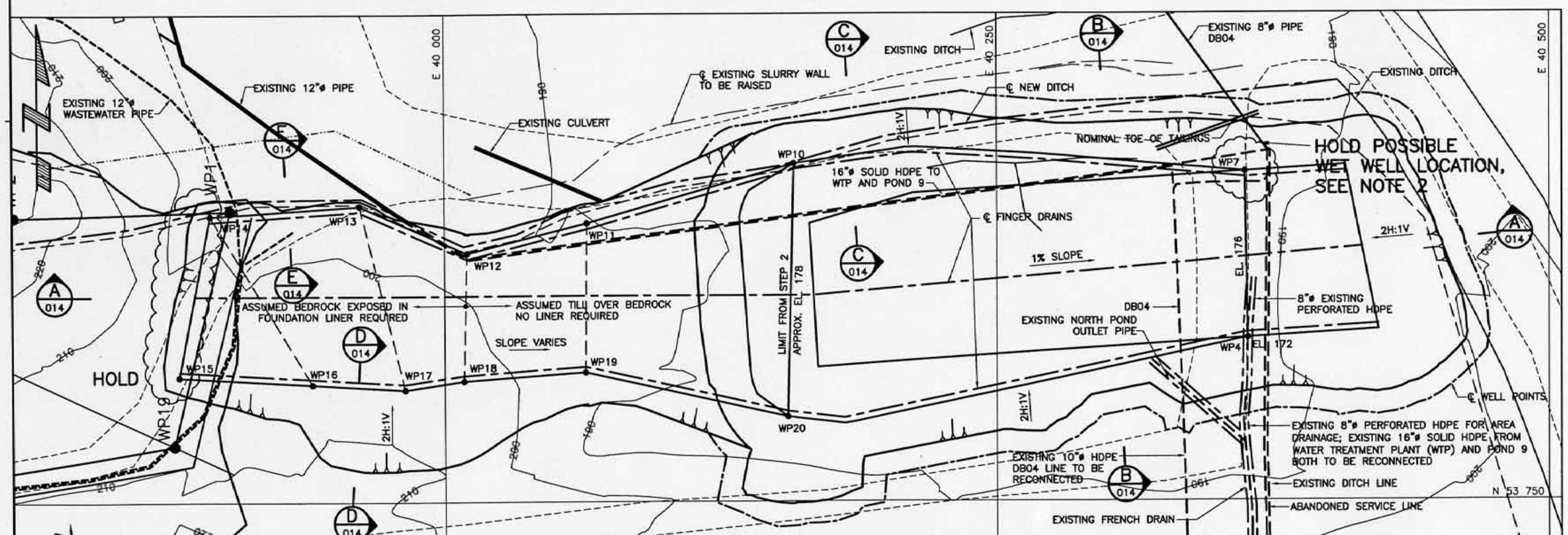


STEP 1- EXCAVATION OF LOOSE FOUNDATION



STEP 2-ROCKFILL, BACKFILL TO CREATE 1% SLOPE



STEP 3 – FINAL GRADING PLAN

TABLE OF WORK POINTS

WORK POINT No.	NORTHING (FT)	EASTING (FT)	ELEVATION AT NOTED STAGE (FT)	STEP No.
WP1	53 875	40 165	164	1
WP2	53 810	40 169	164	1
WP3	53 817	40 270	170	1
WP4	53 823	40 364	176	1, 2, 3
WP5	53 827	40 423	180	1
WP6	53 901	40 408	180	1
WP7	53 898	40 362	176	1, 2, 3
WP8	53 892	40 270	170	1
WP9	53 888	40 212	167	1
WP10	53 902	40 158	178	2, 3
WP11	53 875	40 064	185	3
WP12	53 862	40 010	190	3
WP13	53 883	39 962	192	3
WP14	53 879	39 894	195	3
WP15	53 806	39 880	200	3
WP16	53 803	39 940	195	3
WP17	53 800	39 982	192	3
WP18	53 804	40 009	190	3
WP19	53 808	40 064	185	3
WP20	53 788	40 155	178	2, 3

NOTES:

1. UTILITY RELOCATIONS NOT SHOWN PAVING DEVELOPMENT OF A PLAN BY KGCMC.
2. WET WELL AND CLEAN OUT MANHOLE FOR UTILITIES TO BE DETERMINED BY KGCMC.
3. BASE TOPOGRAPHY PROVIDED BY KGCMC OCTOBER 2003.
4. ALL DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.

LEGEND:

- EXISTING SLURRY WALL
- EXISTING ROAD
- ℄ DITCH LINE
- ℄ WELL POINT

ALL DIMENSIONS AND ELEVATIONS ARE APPROXIMATE ONLY AND NEED TO BE CHECKED AND FINALIZED IN FINAL DESIGN

NOT FOR CONSTRUCTION

DRAFT

SCALE: 30 0 30 FT

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
B	APR 2, 2004	75% COMPLETE – ISSUED FOR DESIGN OVERVIEW REPORT				
A	JAN 30, 2004	75% COMPLETE – ISSUED FOR CLIENT REVIEW				

DRAWING NO. | REFERENCE DRAWING

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

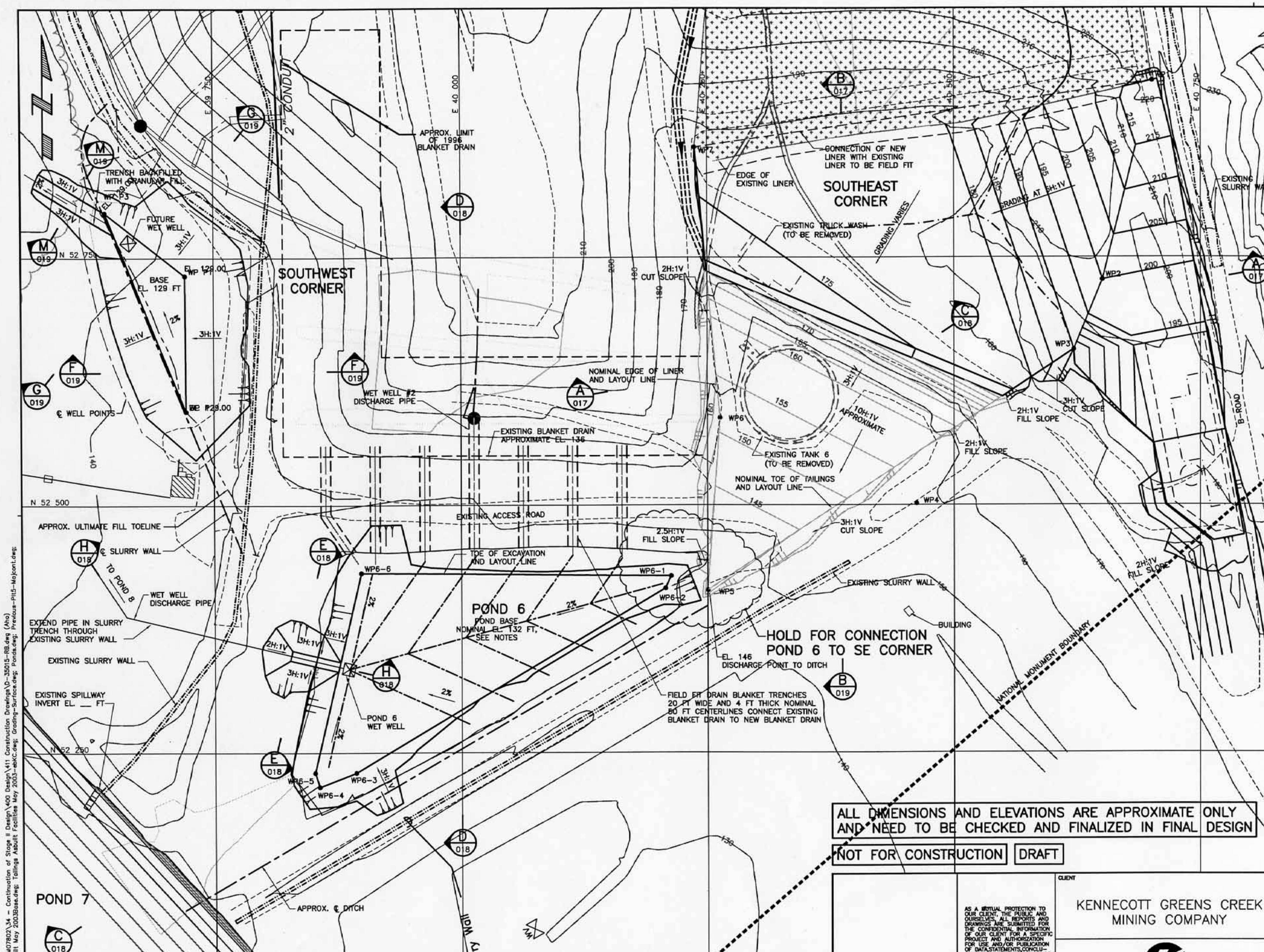
KLOHN CRIPPEN

PROJECT: STAGE 2 EXPANSION OF TAILINGS FACILITY

TITLE: NORTHEAST CORNER EXCAVATION AND GRADING PLAN

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802A35	D-35013	B

CANCEL PRINTS BEARING PREVIOUS REVISION



ALL DIMENSIONS AND ELEVATIONS ARE APPROXIMATE ONLY AND NEED TO BE CHECKED AND FINALIZED IN FINAL DESIGN

NOT FOR CONSTRUCTION | DRAFT

TABLE OF WORK POINTS SOUTHEAST CORNER

WORK POINT No.	EASTING (FT)	NORTHING (FT)	ELEVATION (FT)
1	40 701	52 927	223
2	40 651	52 727	200
3	40 622	52 657	190
4	40 462	52 502	157
5	40 250	52 414	150
6	40 263	52 588	156
7	40 237	52 861	162

TABLE OF WORK POINTS POND 6

WORK POINT No.	EASTING (FT)	NORTHING (FT)	ELEVATION (FT)
6-1	40 213	52 428	132
6-2	40 207	52 416	132
6-3	39 893	52 230	132
6-4	39 856	52 216	132
6-5	39 851	52 230	132
6-6	39 900	52 432	132

TABLE OF WORK POINTS SOUTHWEST CORNER

WORK POINT No.	EASTING (FT)	NORTHING (FT)	ELEVATION (FT)
P1	39 721	52 732	129
P2	39 722	52 595	129
P3	39 641	52 795	129

NOTES:

1. GRADING SURFACE IS MINIMUM BASE OF EXCAVATION. IF ROCK OR COMPETENT SOIL IS NOT EXPOSED, SUB-EXCAVATE BELOW GRADING SURFACE THEN PLACE COMPACTED ROCKFILL TO BUILD BACK TO GRADING SURFACE.
2. IN AREAS WHERE GRADING SURFACE IS ABOVE GROUND LEVEL EXCAVATE TO SOUND MATERIAL AND BACKFILL WITH COMPACTED ROCKFILL TO ACHIEVE GRADING SURFACE ELEVATIONS.
3. SLOPE EXCAVATION BASE TO ACHIEVE MINIMUM 1% GRADE TO SUMP.
4. BASE TOPOGRAPHY PROVIDED BY KGCMC OCTOBER 2003.
5. ALL DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.

LEGEND:

- 200 — GRADING CONTOUR
- EXISTING ROAD
- EXISTING SLURRY WALL
- EXISTING LINER
- GRADING SURFACE IS ABOVE EXISTING GROUND

SCALE: 50 0 50 FT

DRAWING NO.	REFERENCE DRAWING	NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
		B	APR 2, 2004	75% COMPLETE – ISSUED FOR DESIGN OVERVIEW REPORT				
		A	JAN 30, 2004	75% COMPLETE – ISSUED FOR CLIENT REVIEW				

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 EXPANSION OF TAILINGS FACILITY

TITLE: SOUTHWEST CORNER, SOUTHEAST CORNER AND POND 6 GRADING PLAN

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802A35	D-35015	B

CANCEL PRINTS BEARING PREVIOUS REVISION

Time: 1/41:41
Date: 3/22/2004
Scale: 1=0(PS)
Drawing File: M:\M07802\34 – Continuation of Stage II Design\400 Design\411 Construction Drawings\D-35015-RB.dwg (Ano)
xref: Tails Asbuilt May 2003Base.dwg; Tailings Asbuilt Facilities May 2003-ebKC.dwg; Grading-Surface.dwg; Ponds.dwg; Previous-Pit5-Mojcont.dwg

TABLE 1 – WORK POINTS ON LAYOUT REFERENCE LINE

WORK POINT NO.	EAST	NORTH	FINAL GRADE ELEVATION	TYPICAL LOCATION
WP1	40301.3	52447.6	145.0	SECTION D & E, SHT 12
WP2	40516.1	52588.1	167.0	SECTION D & E, SHT 12
WP3	40240.0	52831.1	173.5	SECTION F, SHT 13
WP4	40270.1	52658.4	158.0	SECTION F, SHT 13
WP5	40275.4	52577.4	151.0	SECTION F, SHT 13
WP6	40269.8	52523.7	146.0	SECTION F, SHT 13
WP7	40257.1	52470.5	146.0	SECTION F, SHT 13

LEGEND:

- EXISTING ROAD
- FOUNDATION FRENCH DRAIN (TYP.) TO BE FIELD FIT
- EXISTING FRENCH DRAIN
- PERFORATED (SOLID BEYOND LINER) DRAIN PIPE
- DITCH LINE
- WELL LOCATION

NOTES:

1. FIELD FIT CONNECTION OF SOUTHEAST 2 LINER TO SOUND EXISTING LINER, TRANSITION TO DESIGN SLOPES AS INDICATED.
2. CUT/FILL SLOPES ARE SET PERPENDICULAR TO THE TOE OR CREST LINE RESPECTIVELY. TRUE SLOPES MAY BE STEEPER.
3. TAILINGS PILE SURVEY SEPTEMBER 2004. OUTLYING AREA TOPOGRAPHY MEASURED JUNE 2004. TOPOGRAPHY PROVIDED BY KGCMC.
4. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.
5. THERE ARE BURIED SERVICES NEAR AND ALONG THE WEST EDGE OF SE AREA 2, SOME OF WHICH MIGHT BE FOUNDED ON COMPRESSIBLE PEAT. THESE SERVICES WILL BE PRESERVED AND RE-ROUTED AS REQUIRED ON TO COMPETENT FOUNDATION, AND EXTENDED TO POND 6. ALIGNMENT OF THE WEST SIDE OF SE AREA 2 WILL BE ADJUSTED IN THE FIELD, AS DIRECTED, TO MINIMIZE TAILINGS AND PEAT EXCAVATION AND TO ESTABLISH A FIRM BASE FOR THE SERVICE LINES.
6. ASSIST WITH INSTALLATION OF A MINIMUM OF 2 VIBRATING WIRE PIEZOMETERS AND 2 LYSIMETERS BELOW THE LINER AS DIRECTED BY OWNER.
7. ASSIST WITH CONNECTION OF INSTRUMENTATION LEADS TO NEW INSTRUMENTS AS DIRECTED BY OWNER.
8. ASSIST WITH EXTENSION OF INSTRUMENTATION LEADS OUTSIDE LINER BOUNDARY AS DIRECTED BY OWNER.

SCALE: 30 0 30 FT

APPROVED FOR CONSTRUCTION

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
0	APR 13, 2005	APPROVED FOR CONSTRUCTION				

DRAWING NO.	REFERENCE DRAWING

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 EXPANSION OF TAILINGS FACILITY

TITLE: SOUTHEAST 2 FINAL GRADING–PLAN

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802A39	D–39011	0

CANCEL PRINTS BEARING PREVIOUS REVISION

Date: 4/13/2005
Scale: 1=1(PS)
Drawing File: W:\M07802A39 – 2005 Tailings Facility Design & Specifications\400 Design\411 Construction Drawings\D–39011–R0.dwg (cwong)
Xrefs: DRAWING 2 – GENERAL ARRANGEMENT050215

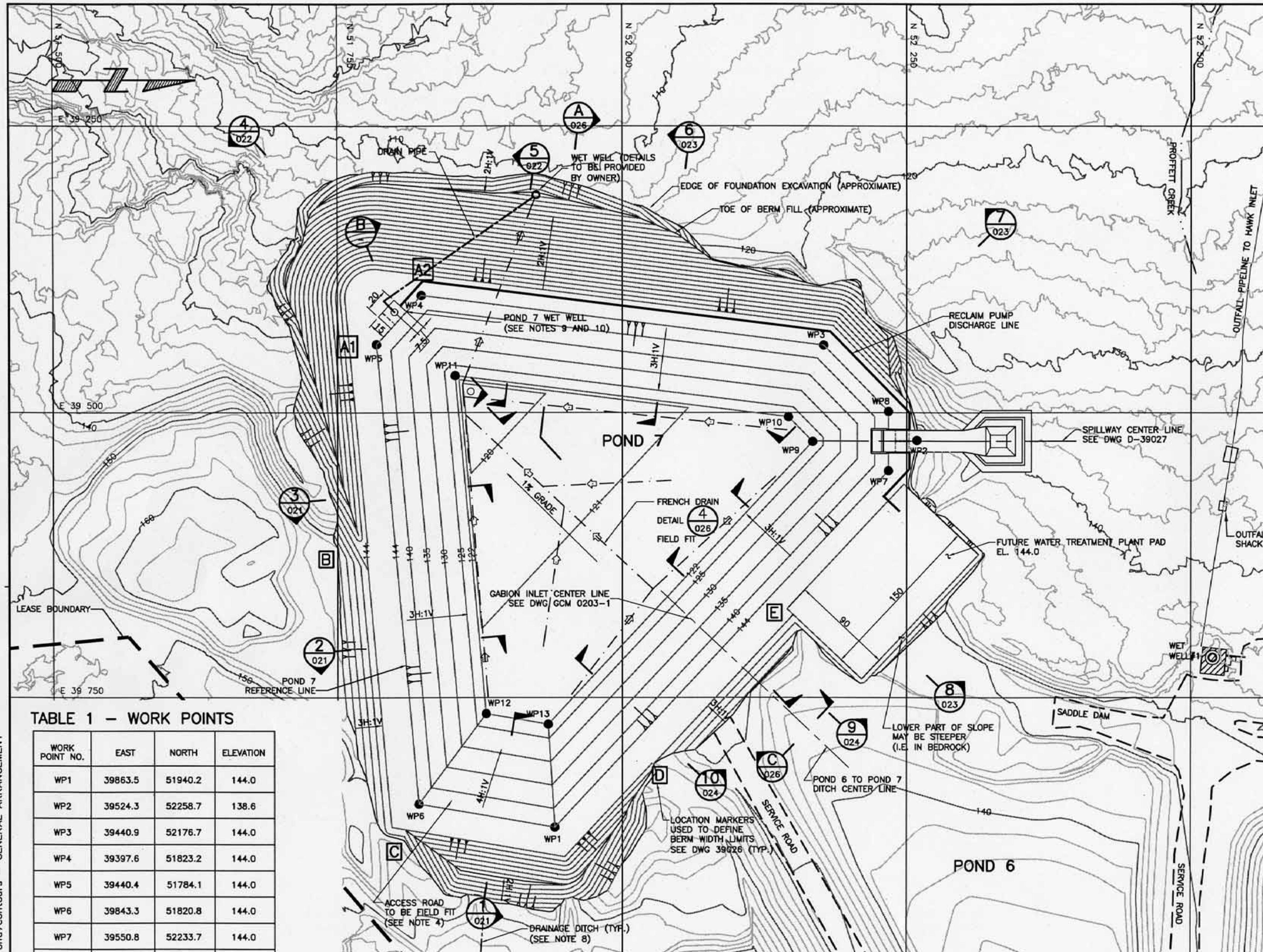
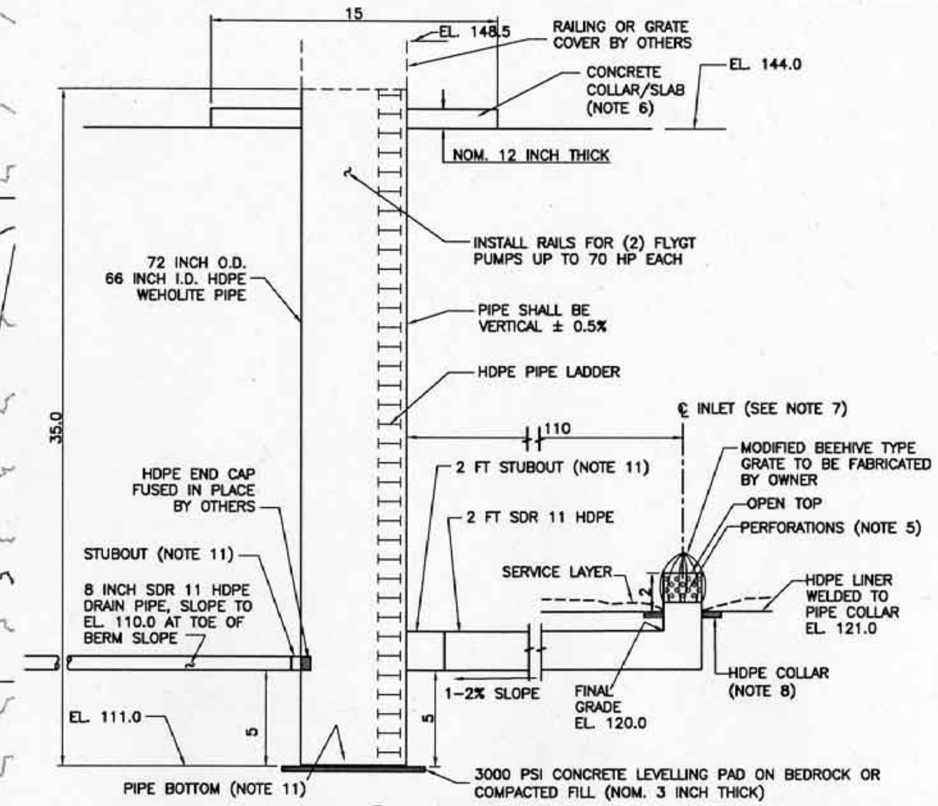


TABLE 1 – WORK POINTS

WORK POINT NO.	EAST	NORTH	ELEVATION
WP1	39863.5	51940.2	144.0
WP2	39524.3	52258.7	138.6
WP3	39440.9	52176.7	144.0
WP4	39397.6	51823.2	144.0
WP5	39440.4	51784.1	144.0
WP6	39843.3	51820.8	144.0
WP7	39550.8	52233.7	144.0
WP8	39498.9	52233.7	144.0
WP9	39525.3	52167.2	122.0
WP10	39503.7	52145.8	122.0
WP11	39467.7	51852.8	122.0
WP12	39764.0	51879.8	122.0
WP13	39773.3	51934.4	122.0



SECTION B/025 NTS POND 7 WET WELL PROVIDED BY KGCMC

NOTES:

1. POND 7 CONTOURS ARE THE FINAL GRADE SURFACE.
2. POND 7 AREA SURVEY WAS TAKEN SEPTEMBER 2004. OUTLYING AREA TOPOGRAPHY WAS CARRIED OUT JUNE 2004. TOPOGRAPHY WAS PROVIDED BY KGCMC.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE. COORDINATES AND ELEVATIONS ARE REFERENCED TO MINE DATUM.
4. A 4H:1V SLOPE ACCESS ROAD IS TO BE CONSTRUCTED OF 6 INCH MINUS ROCKFILL WITH MIN 1.5 FT BEDDING SAND USED TO PROTECT LINER SURFACE. MIN. 3 FT DEPTH OF ROAD FILL IS REQUIRED. FILL PLACEMENT FROM THE TOP DOWN WILL NOT BE ALLOWED.
5. PERFORATIONS SHALL BE 1 INCH HOLES AT 4 INCH CENTERS ON EACH ROW. ROWS ARE AT 4 INCH CENTERS. STAGGER HOLE PATTERN ON ADJACENT ROWS.
6. CONCRETE STRENGTH = 4500 PSI, COLLAR SHALL BE REINFORCED WITH A GRID OF #5 REBAR ON 12 INCH CENTERS PLACED 3 INCHES BELOW SLAB SURFACE AND WITH A GRID OF #5 REBAR ON 12 INCH CENTERS PLACED 8 INCHES BELOW SLAB SURFACE.
7. SET RECLAIM PUMP INLET APPROXIMATELY 5 FT FROM TOE OF SLOPE.
8. HAND PLACE NON-SHRINK SAND/CEMENT GROUT AS DIRECTED TO PROVIDE CONTINUOUS SUPPORT FOR THE HDPE COLLAR. SEAL DETAIL TO FOLLOW.
9. PUMP WELL AND OUTLET PIPE SHALL BE EMBEDDED IN COMPACTED SAND MIN. 1 FT THICK. INLET PIPE SHALL BE EMBEDDED IN DRAIN GRAVEL FOR MIN. 45 FT DOWNSTREAM FROM PIPE INLET, REMAINDER SHALL BE EMBEDDED IN SAND.
10. PLACEMENT AND COMPACTION OF ALL FILL MATERIALS WITHIN A 10 FT RADIUS OF THE PUMP WELL SHALL BE CARRIED OUT AS DIRECTED.
11. WELDED HDPE PIPE BOTTOM AND STUBOUTS FABRICATED BY MANUFACTURER.
12. FRENCH DRAINS UNDER THE BERM AND IN THE POND WILL BE FIELD FIT AS DIRECTED.
13. DRAINS AND INSTRUMENTATION LEADS WILL BE EXTENDED TO THE TOE OF THE BERM AS DIRECTED.
14. INSTALL 12 SUCTION LYSIMETERS BELOW THE WATER TABLE IN THE BASE OF THE POND (~75–100 FOOT SPACING). ROUTE ¼ INCH PLASTIC TUBING FROM THE LYSIMETERS THROUGH 1" PVC CONDUIT TO OUTER EDGE OF THE POND BERM. MINIMIZE ROUTING DISTANCE.

    INSTALL THREE VIBRATING WIRE PIEZOMETERS EQUALLY SPACED ALONG THE THREE SIDES OF THE BOTTOM OF THE POND. ROUTE THE CABLE LEADS THROUGH 1" PVC CONDUIT TO THE SPILLWAY END OF THE POND TO ENABLE CONNECTION WITH THE OUTFALL SHACK CR10 SYSTEM.
15. EXACT LOCATION OF INSTRUMENTS WILL BE DETERMINED BY FINAL FOUNDATION CONDITIONS.
16. SWITCH FRENCH DRAIN PIPING FROM PERFORATED TO SOLID FROM THE JUNCTION OF THE TWO DOWNSTREAM LATERALS TO THE TERMINUS AT THE TOE OF THE POND BERM.

APPROVED FOR CONSTRUCTION

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
0	APR 13, 2005	APPROVED FOR CONSTRUCTION				

DRAWING NO. | REFERENCE DRAWING

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

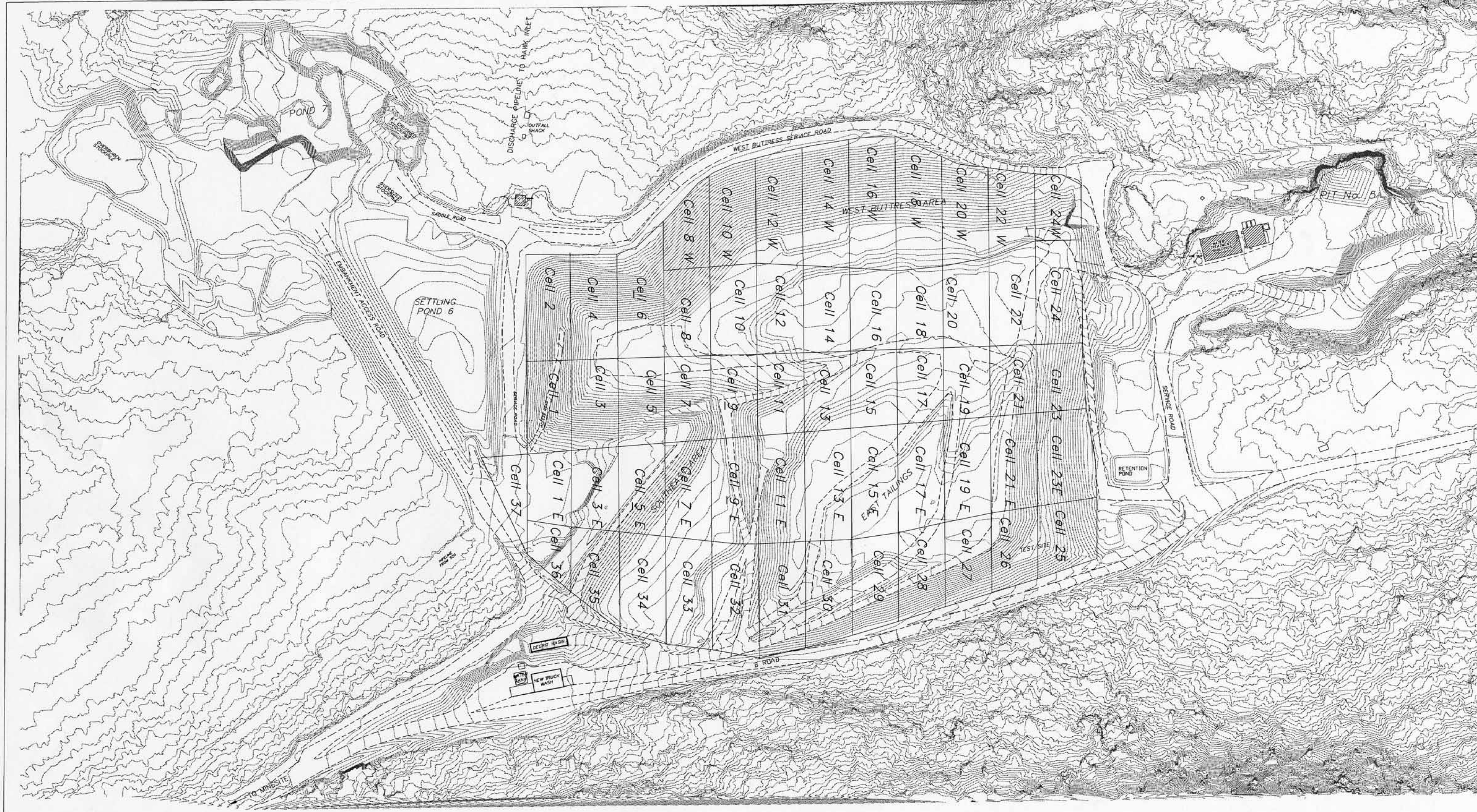
PROJECT: STAGE 2 EXPANSION OF TAILINGS FACILITY

TITLE: POND 7 GRADING SURFACE–PLAN AND BERM LAYOUT

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802A39	D-39025	0

CANCEL PRINTS BEARING PREVIOUS REVISION

Scale: 1=1(PS)
Drawing File: M:\M07802\A39 – 2005 Tailings Facility Design & Specifications\400 Design\411 Construction Drawings\D-39025-R0.dwg (awong)
Xrefs: POND7_SPILLWAY_MAR8_05, Pond7contours – GENERAL ARRANGEMENT



LEGEND:	
ROADS/DITCHES	
BUILDINGS	
WATER UTILS	
ELECT UTILS	
FRENCH DRAIN	

SYMBOLS:

FIRE HYDRANT — MONITORING POINT
BOLLARDS — POWER POLES
Wells — CATCH BASIN

KENNECOTT MINERALS

DATE: 8-31-05
DRAWING BY: Shelby Edwards
DESIGN BY: ----
REVIEWED BY: ----
PROJ OR REF. ----

KENNECOTT GREENS CREEK MINING CO.
P.O. BOX 32199 JUNEAU, ALASKA 99803
PHONE: (907)790-8441 FAX: (907)790-8448

Tails Asbuilt

GRAPHIC SCALE: 1" = 100'
40 100 200

SHEET: 1 OF 1

APPENDIX X

Liner Interface Shear Testing

- **Precision Geosynthetic Laboratories, 2002**
- **TRI/Environmental Inc., 2005**

Precision Geosynthetic Laboratories, 2002

March 11, 2002

Mr. Bob Chambers

KLOHN CRIPPEN CONSULTANTS

10200 Shellbridge Way
Richmond, BC V6X2W7
Canada

Dear Mr. Chambers: **RE: Wide Corner Quarry Liner Design**

Thank you for consulting Precision Geosynthetic Laboratories for your material testing needs.

It should be noted that the test specimen and test sample used for this report was believed to be representative of the material produced under the designation herein stated. However, these results are indicative of only the specimens that were actually tested. The testing herein is based upon accepted industry practice as well as the test method listed. Precision Geosynthetic Laboratories neither accepts responsibility for nor makes claims to the final use and purpose of the material.

By accepting the data and results represented on this report, Client agrees to limit the liability of Precision Geosynthetic Laboratories from Client and all other parties for claims arising out of the use of this data to the cost for the respective test(s) represented in this report, and Client agrees to indemnify and hold harmless Precision Geosynthetic Laboratories from and against all liability in excess of the aforementioned limit.

The test data and all associated project information provided shall be held in confidence and disclosed to other parties only with the authorization of Client or Precision Geosynthetic Laboratories.

It is a company policy to keep the physical records of each job for 2 years since the receipt of the samples and keep the electronic file for 7 years. We will dispose the samples two weeks after the final report is faxed to you. Should you want us to keep them for some period of time, please advise us immediately.

If you have any questions or if we may be of further service, please do not hesitate to call at 800-522-4599.

Sincerely,

PRECISION GEOSYNTHETIC LABORATORIES

Edith Pintor
Quality Assurance

Cora B. Queja
Vice President

Enclosure: (Job No.020090)

Precision Geosynthetic Laboratories

CLIENT: Klohn CRIPPEN CONSULTANTS

PROJECT: Wide Corner Quarry Liner Design

INTERFACE SHEAR TEST RESULTS

(PGL Job No. 020090)

SAMPLE IDENTIFICATIONS:

SAMPLE ID	PRECISION CONTROL NUMBER	DATE RECEIVED	ORIGIN OF MATERIAL
Fines	74302	2/27/02	Klohn Crippen
HDPE R#GR-8-01-0097-37	74305	2/28/02	Northwest Linings
HDPE R#GR-8-01-0214-37	74306	2/28/02	Northwest Linings
D/S Geocomposite R#C2-1016-2-01-2-200	74307	2/28/02	Northwest Linings
S/S Geocomposite R#C1-10-2-00-72-175	74308	2/28/02	Northwest Linings
Geotextile R100	74309	2/28/02	Northwest Linings

TESTS REQUIRED:

TEST METHOD	DESCRIPTION
ASTM D5321	Interface Shear

TEST CONDITIONS: The samples were conditioned for a minimum one hour in the laboratory at 22 $\pm$ 2°C (71.6 $\pm$ 3.6°F) and at 60 $\pm$ 10% relative humidity prior to test.

TEST RESULTS:

The test results are summarized in Tables 1 through 5. The units in which the data are reported are included
on the tables.

PRECISION GEOSYNTHETIC LABORATORIES

Edith Pintor
Quality Assurance

Cora B. Queja
Vice President

1160 North Gilbert Street, Anaheim, CA. 92801, Tel# 714-520-9631, Fax# 714-520-9637

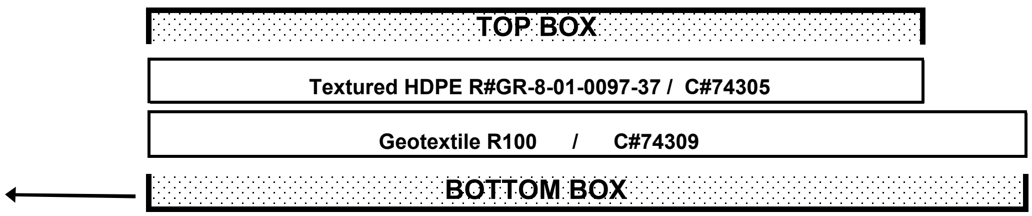
TABLE 1

CLIENT: Klohn Crippen Consultants
PROJECT: Wide Corner Quarry Liner Design

INTERFACE SHEAR TEST RESULT (ASTM D5321)
PGL Job No. G020090

QC'd by: ___________
12-Mar-02

TEST CONFIGURATION # 1



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimen were cut along machine direction to 14" x 18".

CONSOLIDATION:

1. Each set of specimen was sprayed with water, then consolidated under drained condition for 15min @ normal load before shearing.

SHEAR TEST:

1. Shear test was conducted @ 0.20 in/ min.
2. Sheared @ minimum 3.0 inch horizontal displacement.
3. The test specimens were sheared in drained condition.
4. Test were performed in general accordance with ASTM D5321-92
   using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

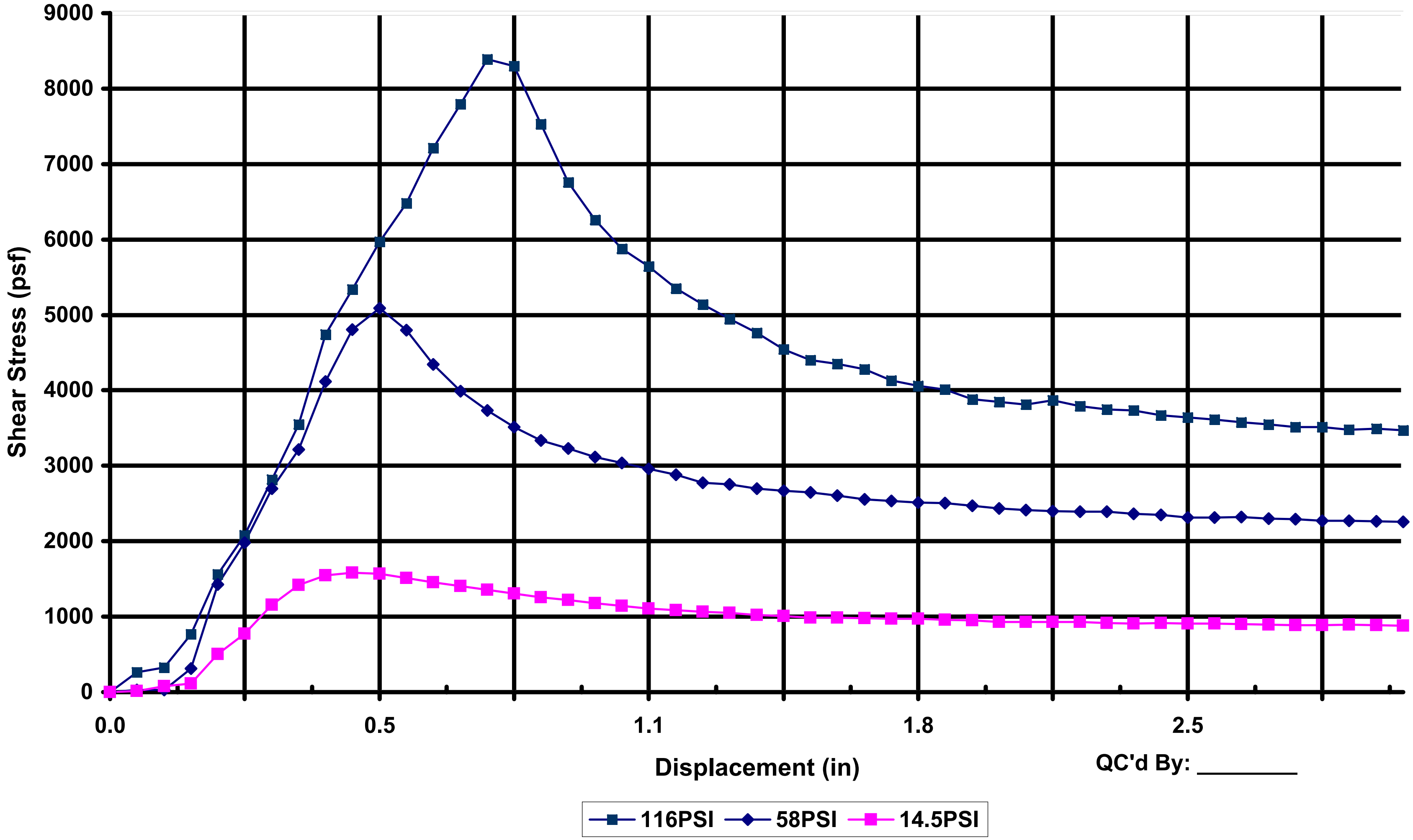
Normal Stress (psi)	Normal Stress (psf)	PEAK STRENGTH: SHEAR STRESS (psf)	PEAK STRENGTH: PEAK SECANT ANGLE (degrees)	3.0 " DISPLACEMENT STRENGTH: SHEAR STRESS (psf)	3.0 " DISPLACEMENT STRENGTH: 3.0 " DISPLACEMENT SECANT ANGLE (degrees)
14.50	2088	1582	37	881	23
58.00	8352	5091	31	2260	15
116.00	16704	8390	27	3470	12
COHESION (psf):		**841**		**617**	
COEFFICIENT OF FRICTION:		**0.46**		**0.18**	
FRICTION ANGLE(degrees):		**24.8**		**9.9**	

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

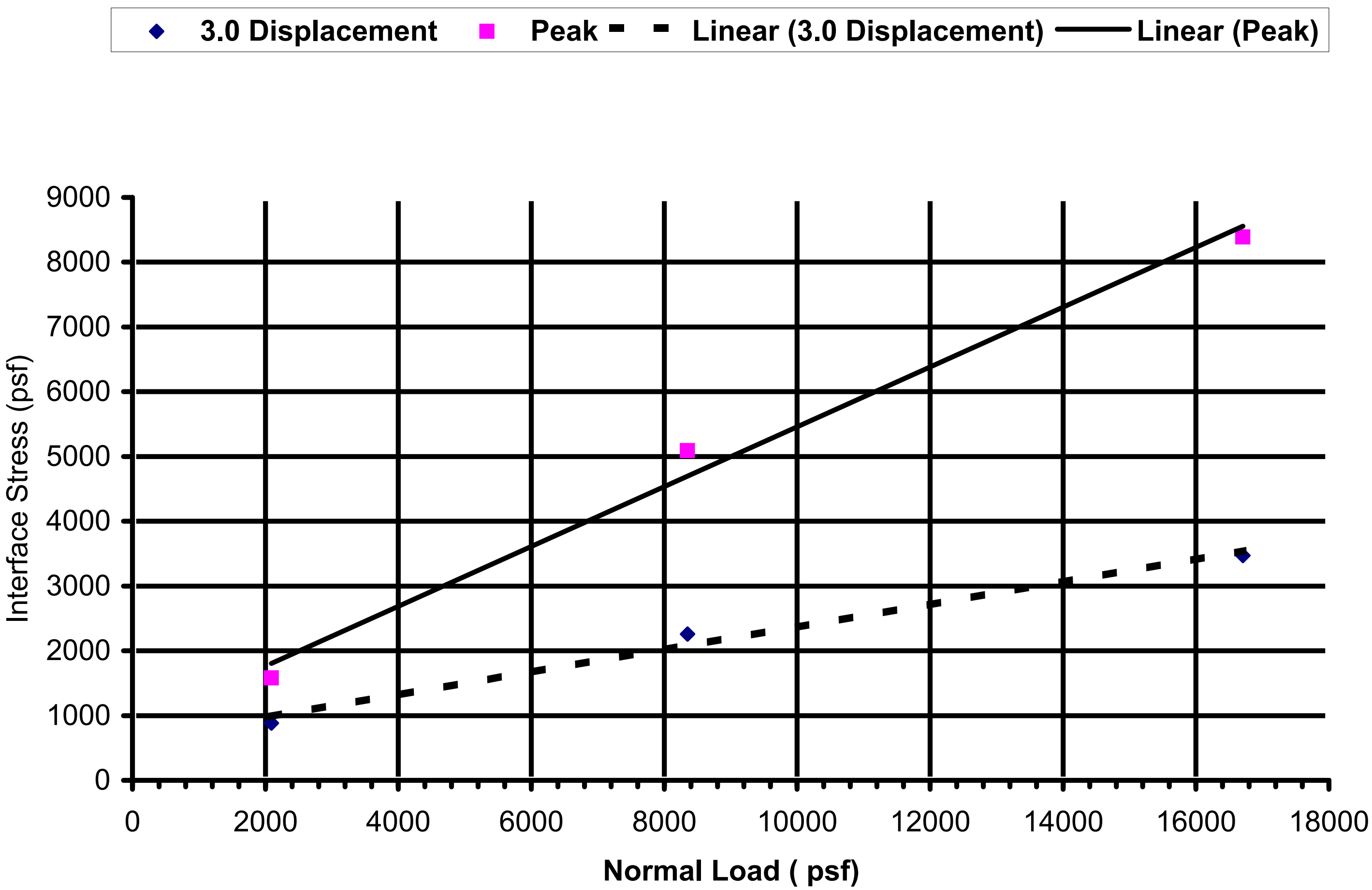
See Figure #1 and #2
Slight pulling of threads of the geotextile was observed at all normal loads. The textured HDPE was slightly abraded at 58psi, and 116psi . No change was observed with the HDPE at 14.5 psi.

Figure #1
Shear Stress/ Displacement Curve
PGL Job No. 020090_1
Textured HDPE R#GR-8-01-0097-37, C#74305 / Geotextile R100, C#74309



QC'd By: ________

Figure #2
Normal Stress/ Interface Stress
PGL Job No. 020090_1
Textured HDPE R#GR-8-01-0097-37, C#74305 / Geotextile R100, C#74309



QC'd By: ________

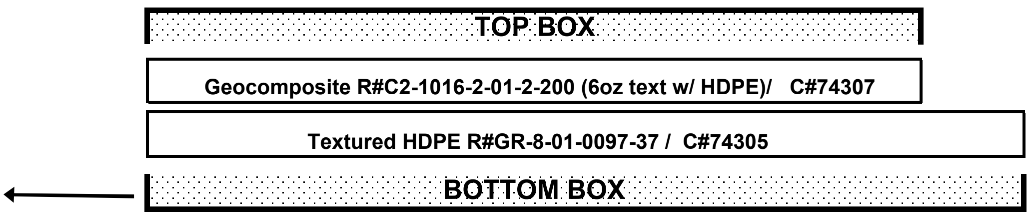
TABLE 2

CLIENT: Klohn Crippen Consultants
PROJECT: Wide Corner Quarry Liner Design

INTERFACE SHEAR TEST RESULT (ASTM D5321)
PGL Job No. G020090

QC'd by: ___________
6-Mar-02

TEST CONFIGURATION # 2



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimen were cut along machine direction to 14" x 18".

CONSOLIDATION:

1. Each set of specimen was sprayed with water, then consolidated under drained condition for 15min @ normal load before shearing.

SHEAR TEST:

1. Shear test was conducted @ 0.20 in/ min.
2. Sheared @ minimum 3.0 inch horizontal displacement.
3. The test specimens were sheared in drained condition.
4. Test were performed in general accordance with ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

Normal Stress (psi)	Normal Stress (psf)	PEAK STRENGTH: SHEAR STRESS (psf)	PEAK STRENGTH: PEAK SECANT ANGLE (degrees)	3.0 " DISPLACEMENT STRENGTH: SHEAR STRESS (psf)	3.0 " DISPLACEMENT STRENGTH: 3.0 " DISPLACEMENT SECANT ANGLE (degrees)
14.50	2088	1441	35	780	20
58.00	8352	4464	28	1846	12
116.00	16704	7440	24	2870	10

	Peak	3.0 " Displacement
COHESION (psf):	**761**	**548**
COEFFICIENT OF FRICTION:	**0.41**	**0.14**
FRICTION ANGLE(degrees):	**22.2**	**8.1**

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2

Partial pulling of threads of the 6oz geotextile was observed at all normal loads. The textured HDPE was slightly abraded at 58psi, and 116psi . No change was observed with the HDPE at 14.5 psi.

Figure #1
Shear Stress/ Displacement Curve
PGL Job No. 020090_2
Geocomposite R#C2-1016-2-01-2-200 (6oz textile w/ HDPE), C#74307 / Textured HDPE
R#GR-8-01-0097-37, C#74305

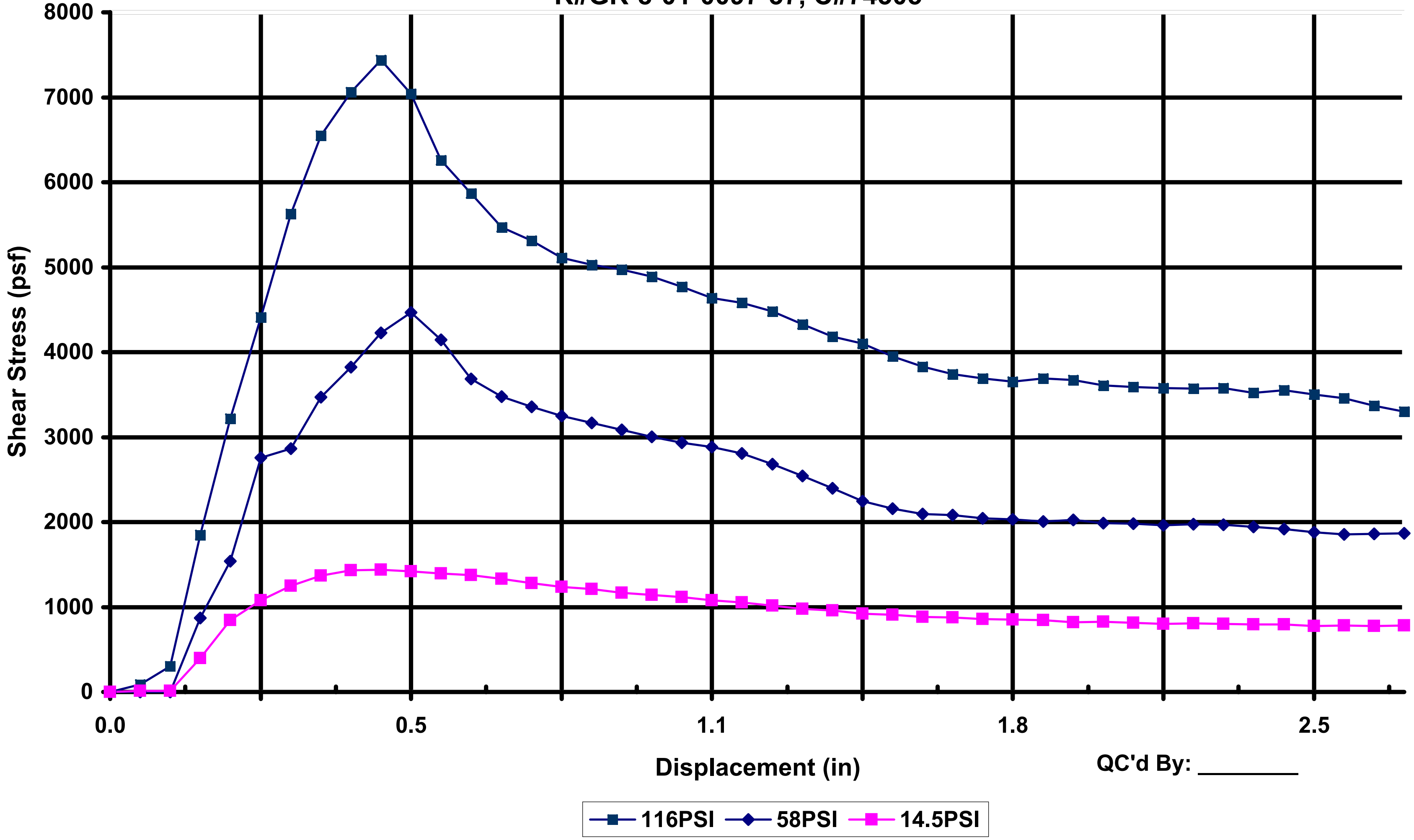
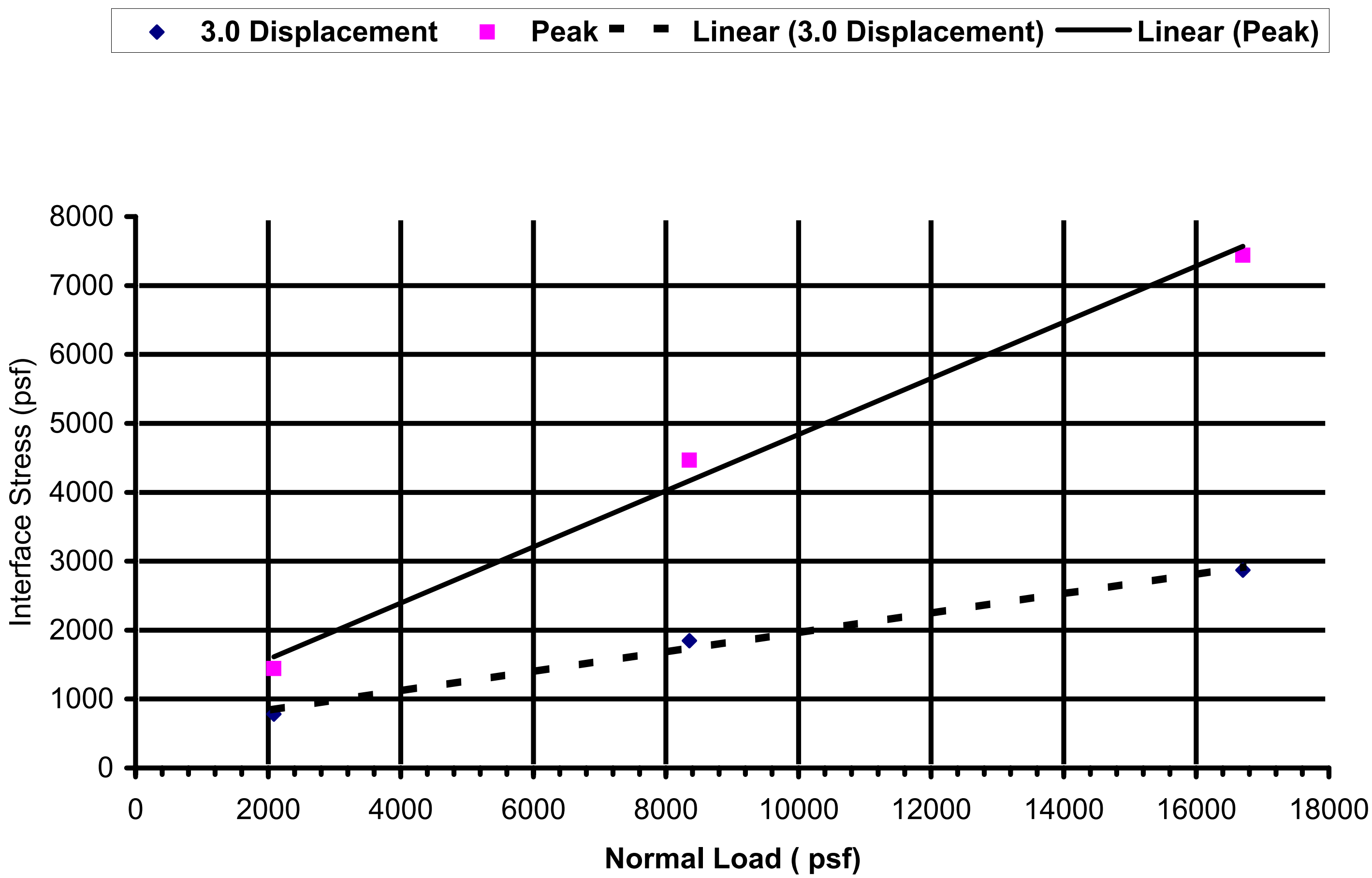


Figure #2
Normal Stress/ Interface Stress
PGL Job No. 020090_2
Geocomposite R#C2-1016-2-01-2-200 (6oz textile w/ HDPE),C#74307 / Textured HDPE R#GR-8-01-0097, C#74305



QC'd By: ________

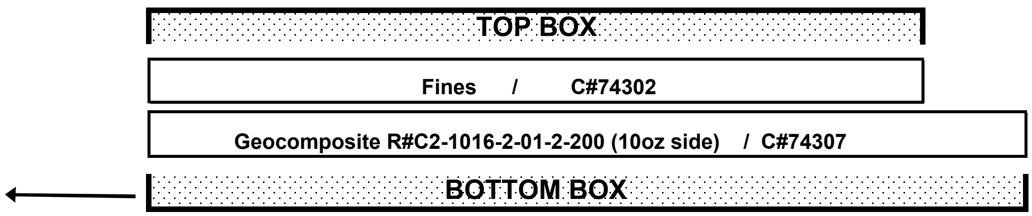
TABLE 3

CLIENT: Klohn Crippen Consultants
PROJECT: Wide Corner Quarry Liner Design

INTERFACE SHEAR TEST RESULT (ASTM D5321)
PGL Job No. G020090

QC'd by: __________
8-Mar-02

TEST CONFIGURATION # 3



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimen were cut along machine direction to 14" x 18".
2. Fines was lightly tamped into the top box at 104.5 pcf at 10.08% moisture content.

CONSOLIDATION:

1. Each set of specimen was consolidated under drained condition for 1 hour @ normal load before shearing.

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 3.0 inch horizontal displacement.
3. The test specimens were sheared in drained condition.
4. Test were performed in general accordance with ASTM D5321-92
   using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

Normal Stress (psi)	Normal Stress (psf)	% MC Fines	PEAK STRENGTH: SHEAR STRESS (psf)	PEAK STRENGTH: PEAK SECANT ANGLE (degrees)	3.0 " DISPLACEMENT STRENGTH: SHEAR STRESS (psf)	3.0 " DISPLACEMENT STRENGTH: 3.0 " DISPLACEMENT SECANT ANGLE (degrees)
14.50	2088	10.3	1451	35	1427	
58.00	8352	10.42	5173	32	5780	
116.00	16704	10.18	9073	29	3860	

COHESION (psf): 540.6
COEFFICIENT OF FRICTION: 0.52
FRICTION ANGLE(degrees): 27.4

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2
At 116psi, geotexile portion stretched and was completely separated from the geonet portion of the geocomposite. At 14.5psi and 58psi, no stretching or tearing was observed .
Residual secant and friction angles were not calculated because failure occurred between geotextile and the geonet portion of the geocomposite, and not on the interface between sand and the geotextile.

Figure #1
Shear Stress/ Displacement Curve
PGL Job No. 020090_3
Fines, C#74302 / Geocomposite R#C2-1016-2-01-2-200 (10oz Side), C#74307

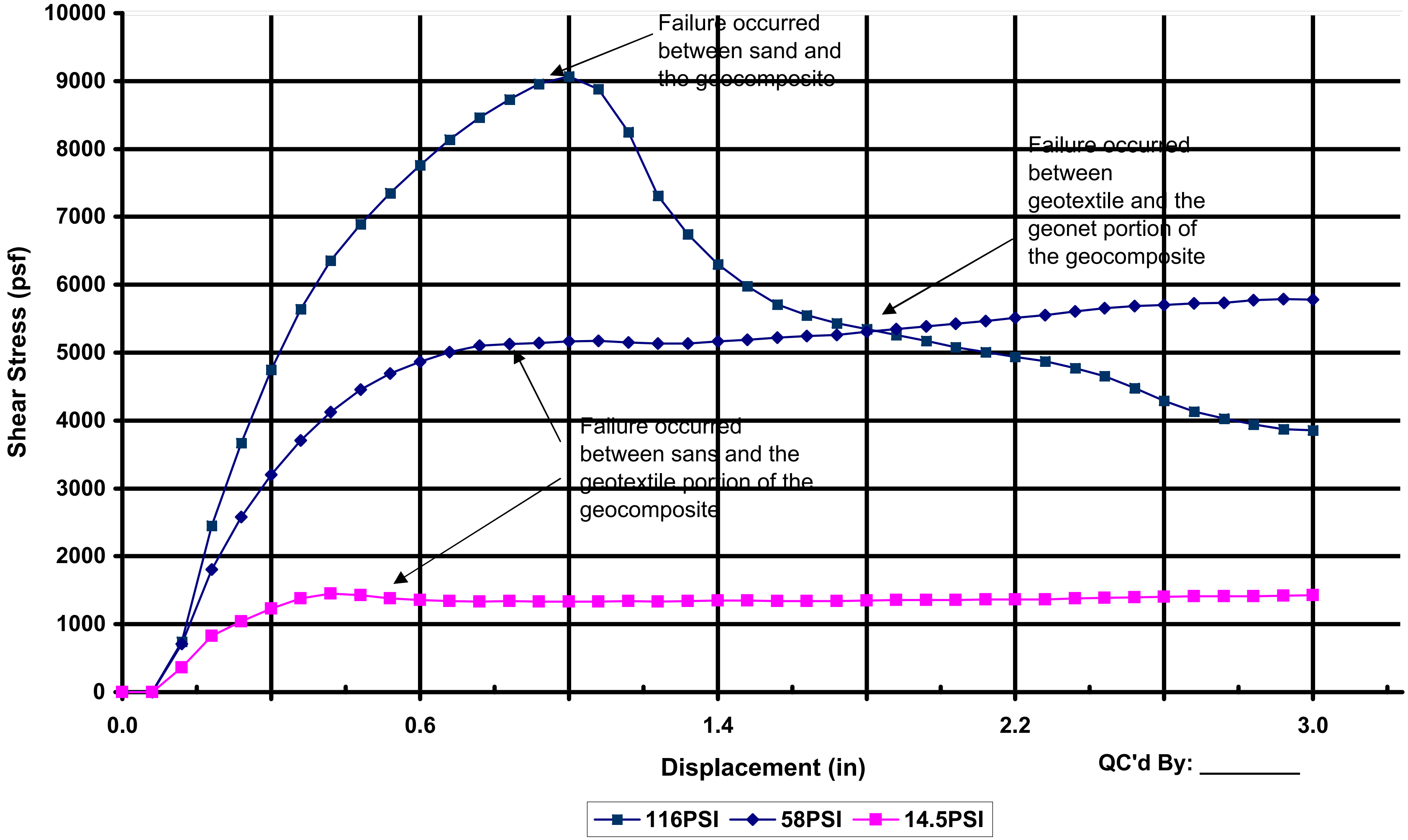
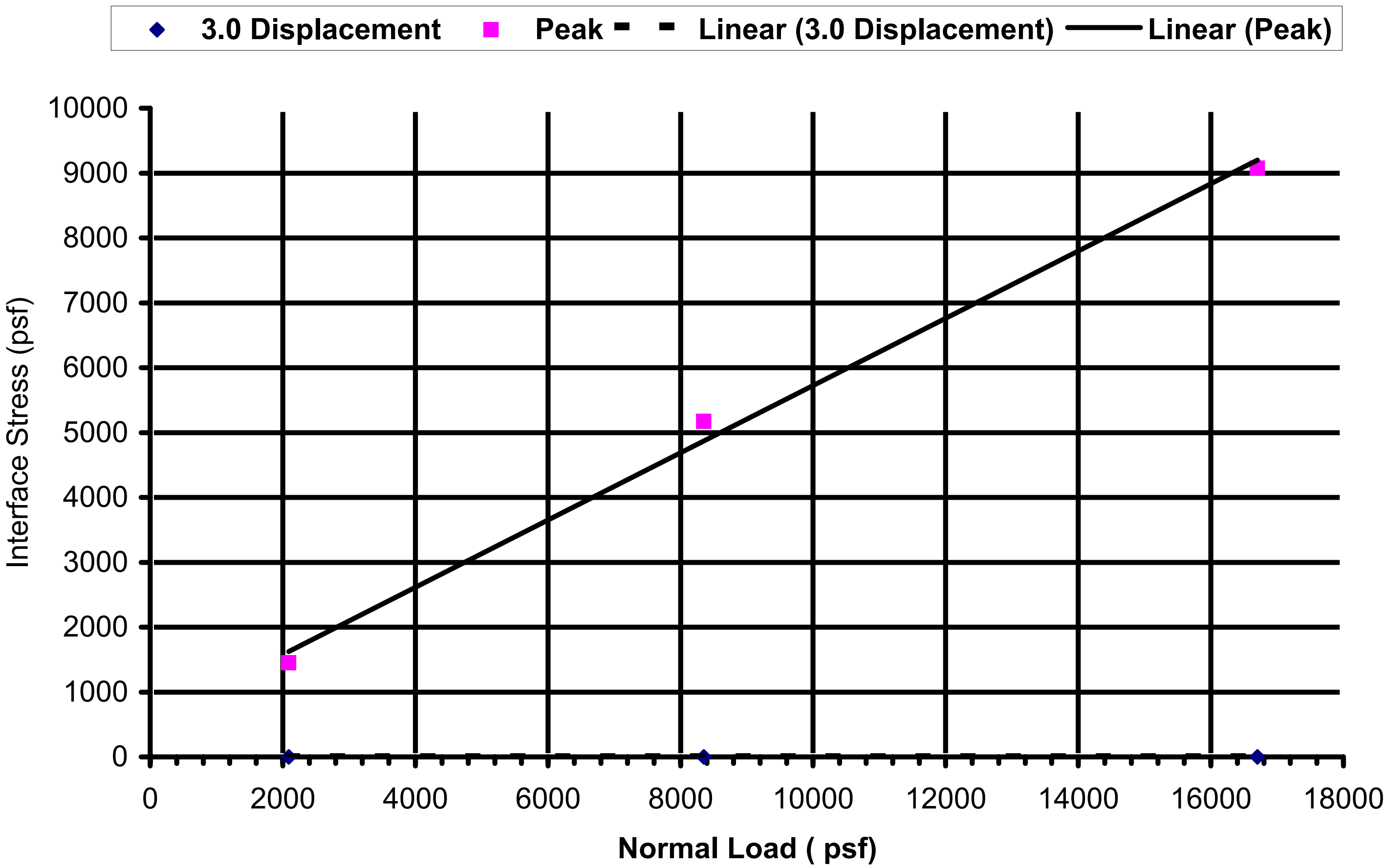


Figure #2
Normal Stress/ Interface Stress
PGL Job No. 020090_3
Fines, C#74302 / Geocomposite R#C2-1016-2-01-2-200
(10 oz Geotextile side),C#74307



QC'd By: ________

TABLE 4

CLIENT: KLOHN CRIPPEN CONSULTANTS

PROJECT: Wide Corner Quarry Liner Design

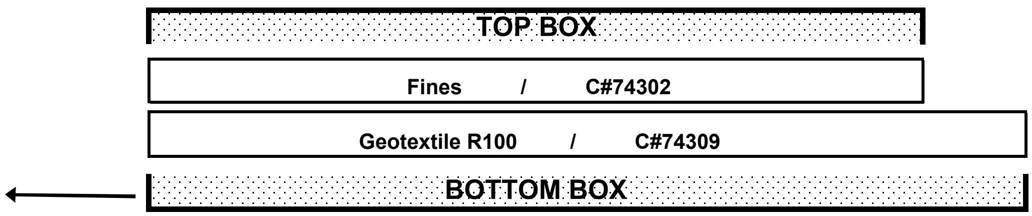
INTERFACE SHEAR TEST RESULT (ASTM D 5321)

PGL Job No. G020090

QC'd by: ___________

13-Mar-02

TEST CONFIGURATION # 4



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimen were cut along machine direction to 14" x 18".
2. Sand was lightly tamped into the top box at 104.5pcf and 10.11% moisture content.

CONSOLIDATION:

1. Geotextile was sprayed with water, then consolidated under drained condition for 1 hour at normal load before shearing.

SHEAR TEST:

1. Shear test was conducted @ 0.04 in/ min.
2. Sheared @ minimum 3.0 inch horizontal displacement.
3. The test specimens were sheared in drained condition.
4. Test were performed in general accordance with ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

Normal Stress (psi)	Normal Stress (psf)	FINAL MC Sand (%)	PEAK STRENGTH: SHEAR STRESS (psf)	PEAK STRENGTH: PEAK SECANT ANGLE (degrees)	3.0 " DISPLACEMENT STRENGTH: SHEAR STRESS (psf)	3.0 " DISPLACEMENT STRENGTH: 3.0 " DISPLACEMENT SECANT ANGLE (degrees)
14.50	2088	10.25	1250	31	1181	29
58.00	8352	10.23	5101	31	4354	28
116.00	16704	10.11	9370	29	6090	20

	PEAK STRENGTH	3.0 " DISPLACEMENT STRENGTH
COHESION (psf):	**235.4**	**898.7**
COEFFICIENT OF FRICTION:	**0.55**	**0.33**
FRICTION ANGLE(degrees):	**28.9**	**18.2**

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2

At 14.5psi and 58 psi, there was no stretching of the geotextile. However, at 116psi, stretching was evident.

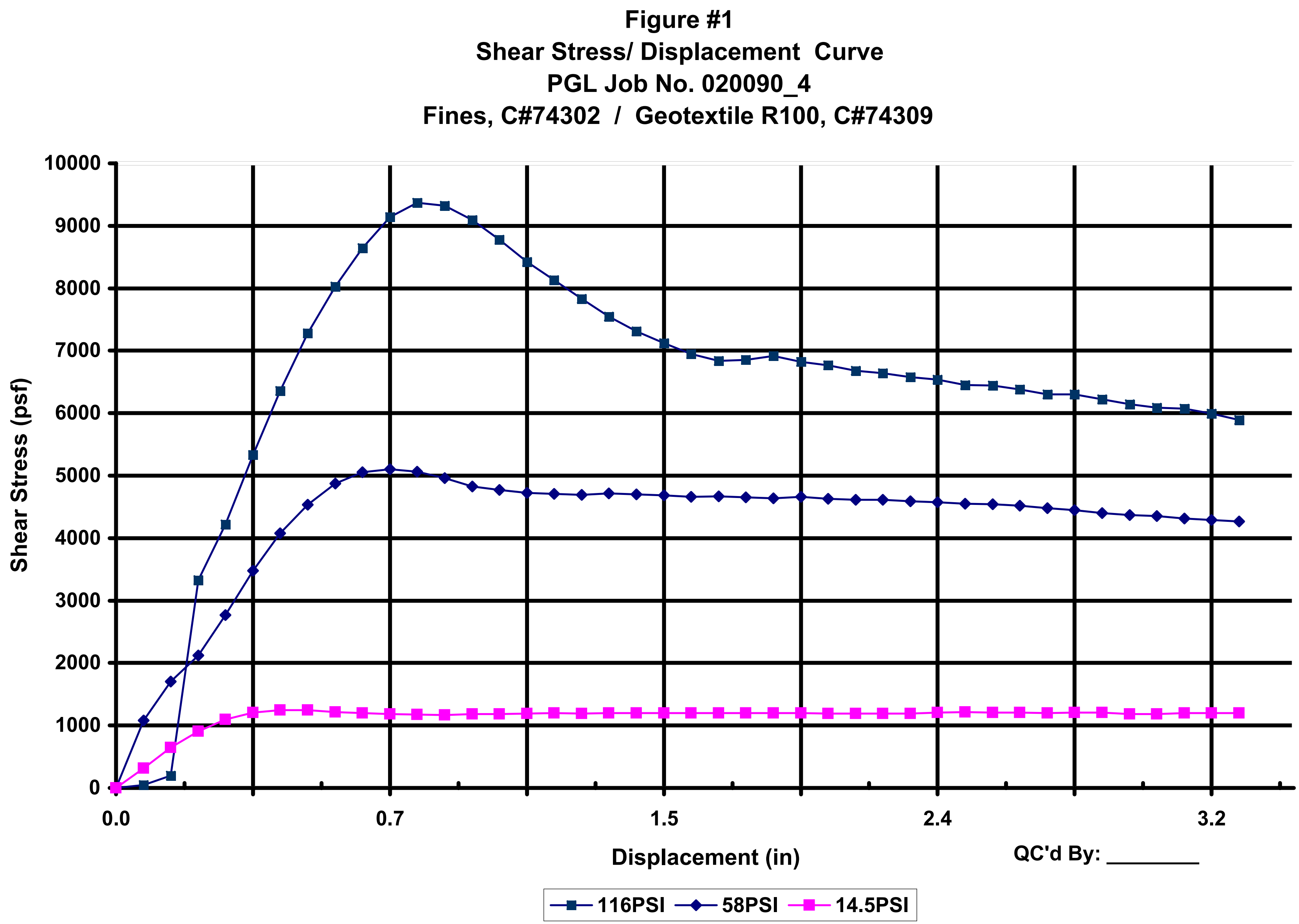
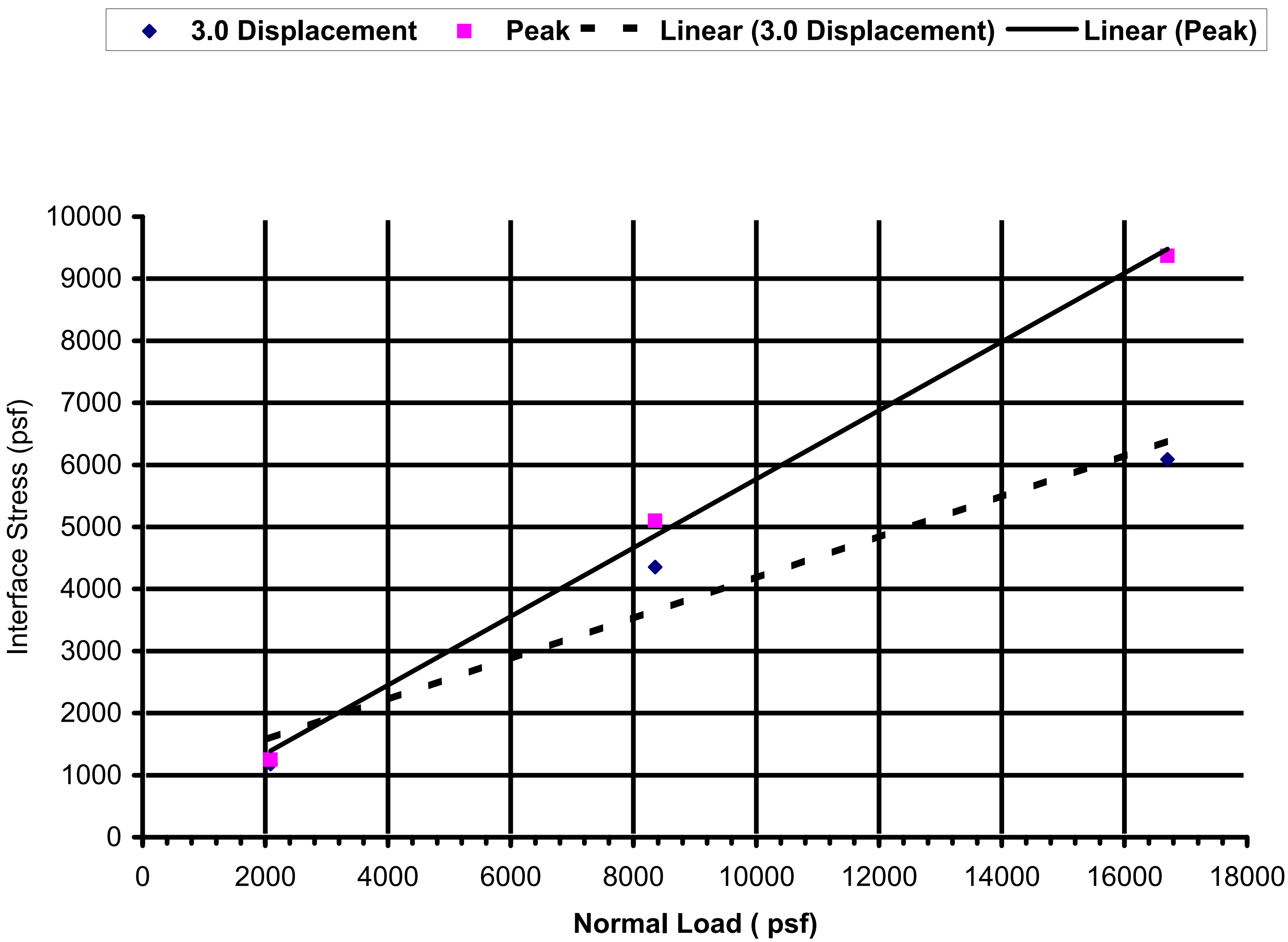


Figure #1
Shear Stress/ Displacement Curve
PGL Job No. 020090_4
Fines, C#74302 / Geotextile R100, C#74309
Shear Stress (psf)
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
0.0
0.7
1.5
2.4
3.2
Displacement (in)
QC'd By: ________
116PSI
58PSI
14.5PSI

Figure #2
Normal Stress/ Interface Stress
PGL Job No. 020090_4
Fines, C#74302 / Geotextile R100, C#74309



QC'd By: ________

TABLE 5

CLIENT: Klohn Crippen Consultants

PROJECT: Wide Corner Quarry Liner Design

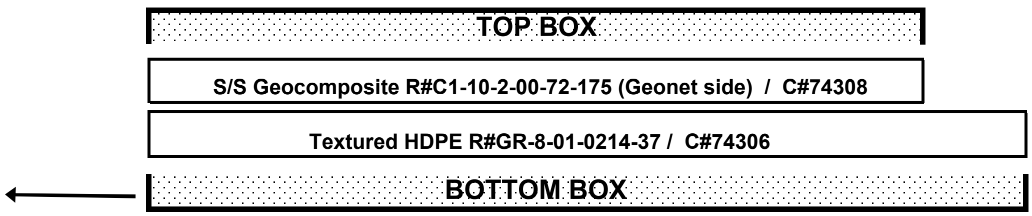
INTERFACE SHEAR TEST RESULT (ASTM D5321)

PGL Job No. G020090

QC'd by: ___________

6-Mar-02

TEST CONFIGURATION # 5



TEST CONDITIONS:

SAMPLE PREPARATION:

1. Specimen were cut along machine direction to 14" x 18".

CONSOLIDATION:

1. Each set of specimen was sprayed with water, then consolidated under drained condition for 15min @ normal load before shearing.

SHEAR TEST:

1. Shear test was conducted @ 0.20 in/ min.
2. Sheared @ minimum 3.0 inch horizontal displacement.
3. The test specimens were sheared in drained condition.
4. Test were performed in general accordance with ASTM D5321-92 using Brainard-Kilman LG-112 Direct Shear machine with effective area of 12 in X 12 in.

TEST RESULTS:

Normal Stress (psi)	Normal Stress (psf)	PEAK STRENGTH: SHEAR STRESS (psf)	PEAK STRENGTH: PEAK SECANT ANGLE (degrees)	3.0 " DISPLACEMENT STRENGTH: SHEAR STRESS (psf)	3.0 " DISPLACEMENT STRENGTH: 3.0 " DISPLACEMENT SECANT ANGLE (degrees)
14.50	2088	594	16	502	14
58.00	8352	2307	15	1789	12
116.00	16704	4990	17	3990	13

	PEAK STRENGTH	3.0 " DISPLACEMENT STRENGTH
COHESION (psf):	**-101**	**-78**
COEFFICIENT OF FRICTION:	**0.30**	**0.24**
FRICTION ANGLE(degrees):	**16.8**	**13.5**

NOTE: The friction angles and cohesion results given here are based on mathematically determined best fit line.

OBSERVATIONS:

See Figure #1 and #2

No stretching, abrasion or tearing was observed with either the Geocomposite or the HDPE specimens.

Figure #1
Shear Stress/ Displacement Curve
PGL Job No. 020090_5
S/S Geocomposite R#C2-10-2-00-72-175 (Geonet Side), C#74308 / Textured HDPE
R#GR-8-01-0214-37, C#74306

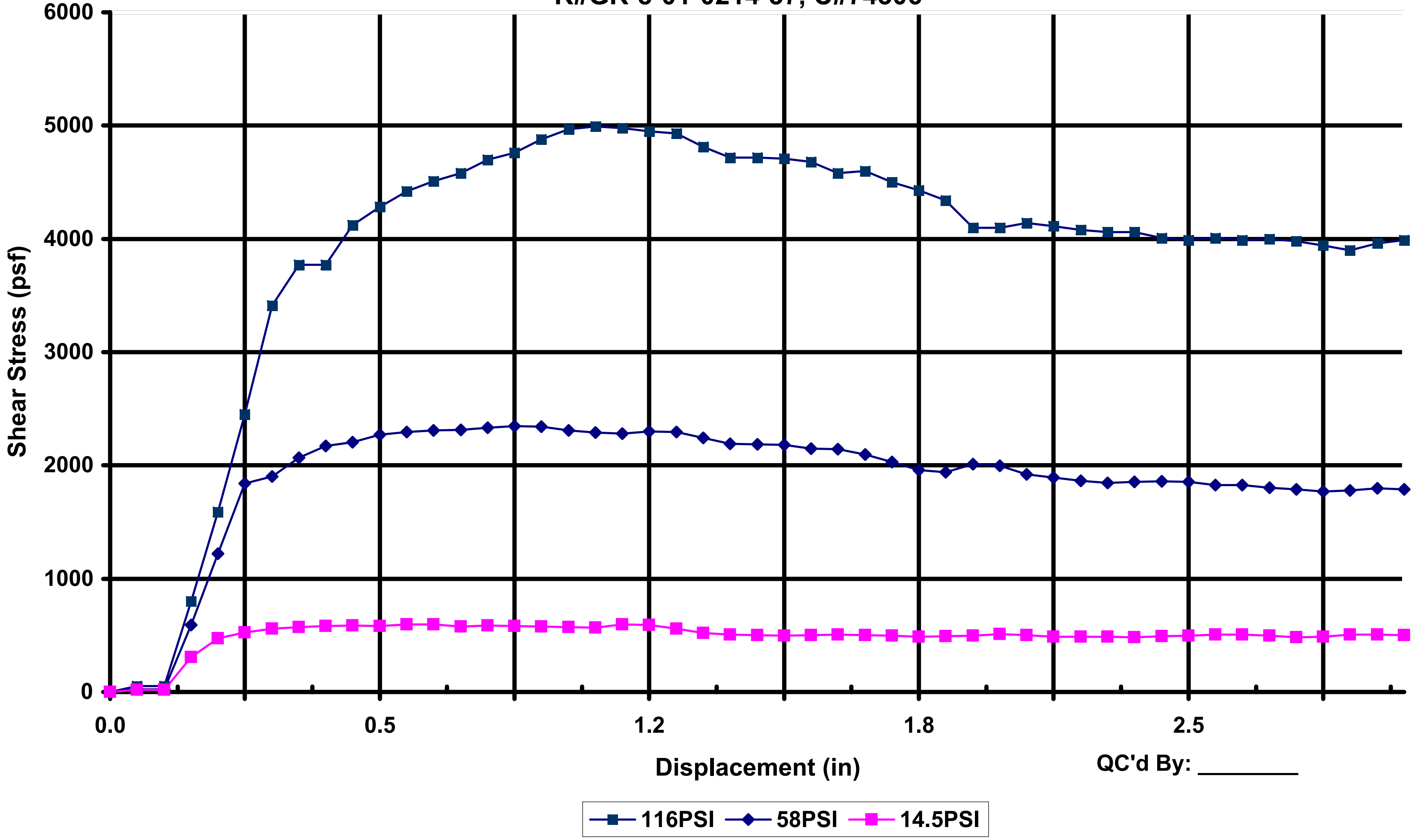
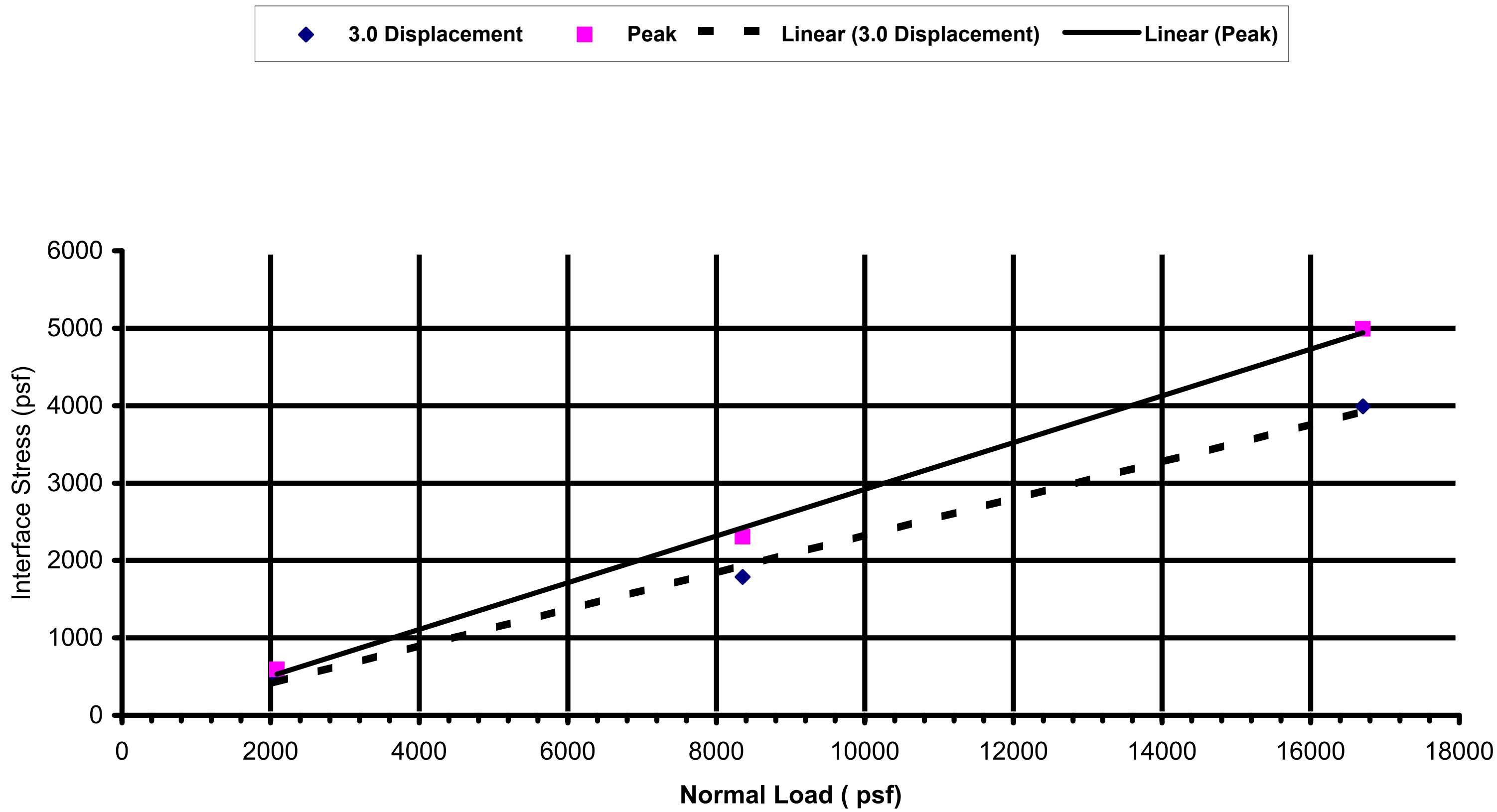


Figure #2
Normal Stress/ Interface Stress
PGL Job No. 020090_5
S/S Geocomposite R#C2-10-2-00-72-175 (Geonet side),C#74308 / Textured HDPE
R#GR-8-01-0214, C#74306



QC'd By: _________

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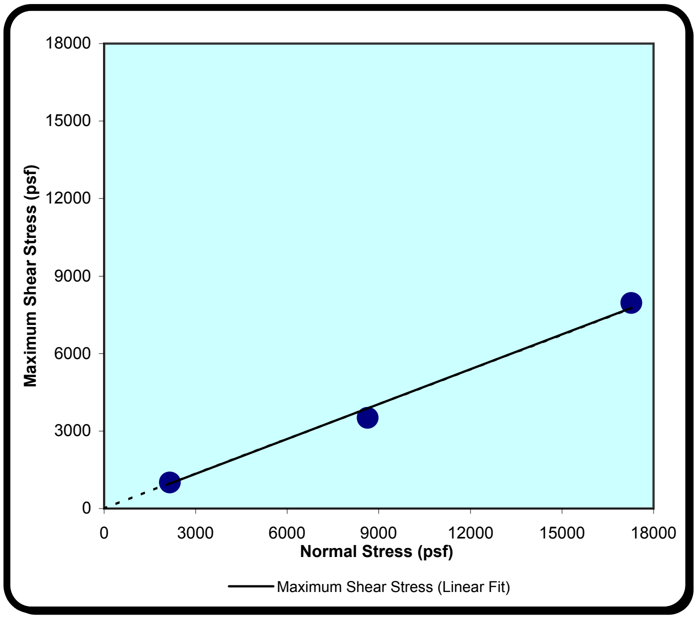
INTERFACE FRICTION TEST REPORT

Client:	**Klohn Crippen**	TRI Log#:	E2201-83-06
Project:	**Kennecott Greens Creek Mining Co.**	Test Method:	ASTM D 5321
Test Date:	8/05-8/05/05		

Tested Interface: Ploy Flex Doublesided Geocomposite (1002-602) vs. Huitex 80 mil Textured HDPE Geomembrane



Upper Box:	Ploy Flex double-sided geocomposite
Lower Box:	Huitex 80 mil textured HDPE geomembrane
Interface Conditioning:	Interface soaked and loading applied for a minimum of 1 hours prior to shear
Box Dimension:	12"x12"x4"
Test Condition:	Wet
Shearing Rate:	0.2 inches/minute

Trial Number	1	2	3
Bearing Slide Resistance (lbs)	29	90	172
Normal Stress (psf)	2160	8640	17280
Maximum Shear Stress (psf)	1032	3590	8126
Corrected Shear Stress (psf)	1003	3500	7954
Secant Angle (degrees)	24.9	22.1	24.7

RESULTS: Maximum Friction Angle and Y-intercept

Regression Friction Angle (degrees):		**24.2**		
Y-intercept or Regression Adhesion (psf):		**0**		
Regression Line:	Y=	0.449	* X +	0
Regression Coefficient (r squared):		0.994		

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Note: The regression line includes the origin.

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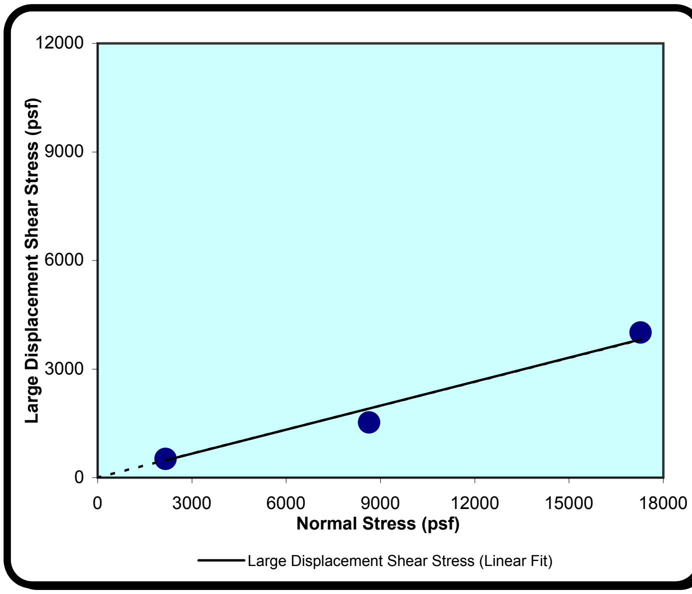
INTERFACE FRICTION TEST REPORT

Client: **Klohn Crippen**
Project: **Kennecott Greens Creek Mining Co.**
Test Date: 8/05-8/05/05

TRI Log#: E2201-83-06
Test Method: ASTM D 5321

Tested Interface: Ploy Flex Doublesided Geocomposite (1002-602) vs. Huitex 80 mil Textured HDPE Geomembrane



Upper Box: Ploy Flex double-sided geocomposite

Lower Box: Huitex 80 mil textured HDPE geomembrane

Interface Conditioning: Interface soaked and loading applied for a minimum of 1 hours prior to shear

Box Dimension: 12"x12"x4"

Test Condition: Wet

Shearing Rate: 0.2 inches/minute

Trial Number	1	2	3
Bearing Slide Resistance (lbs)	29	90	172
Normal Stress (psf)	2160	8640	17280
Large Displacment Shear Stress (psf)	544	1614	4182
Corrected Shear Stress (psf)	515	1524	4010
Secant Angle (degrees)	13.4	10.0	13.1

RESULTS: Large Displacement Friction Angle and Y-intercept at 3.3-in. of Displacement

Regression Friction Angle (degrees):		**12.5**		
Y-intercept or Regression Adhesion (psf):		**0**		
Regression Line:	Y=	0.221	* X +	0
Regression Coefficient (r squared):		0.977		

Note: The regression line includes the origin.

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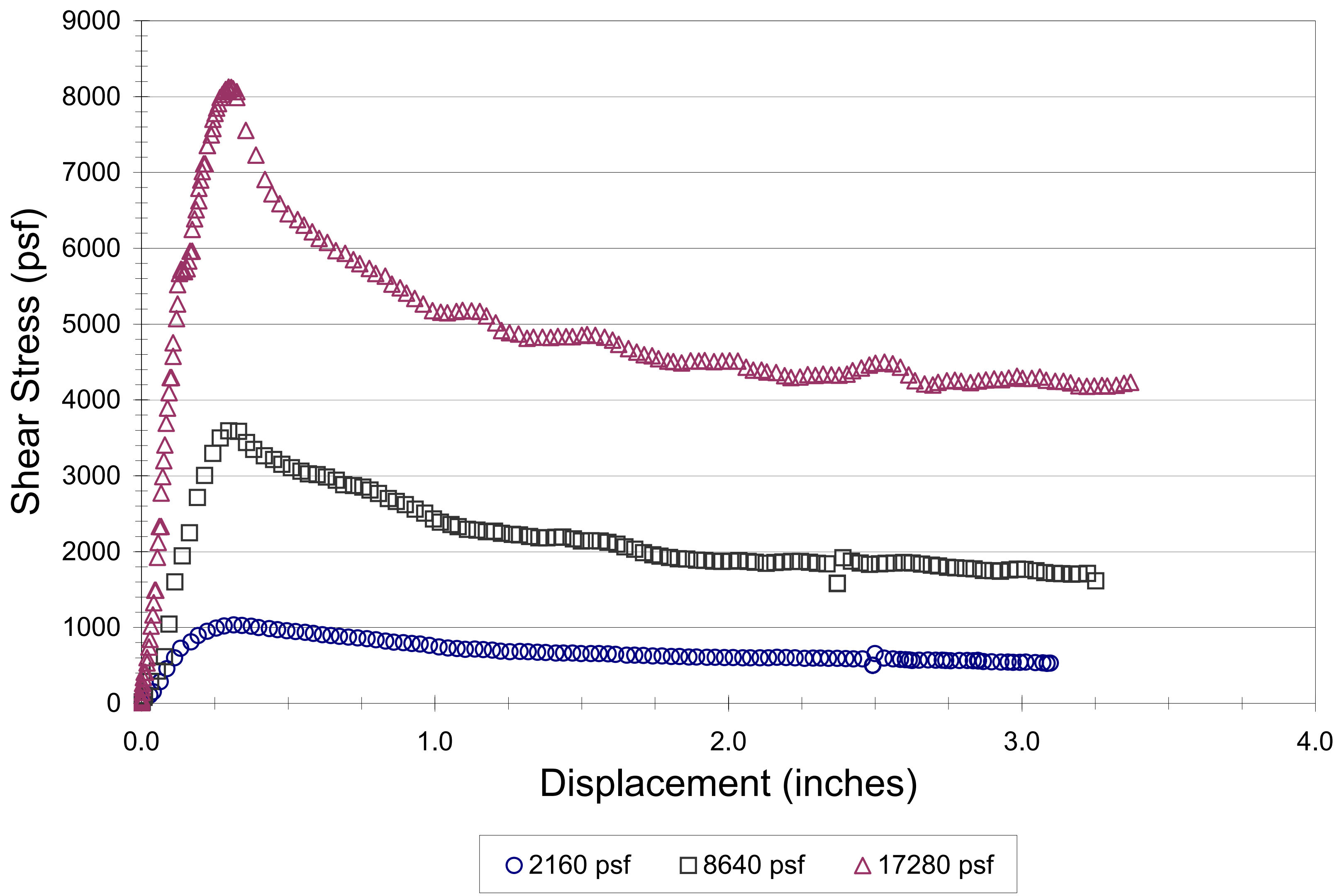
Klohn Crippen INTERFACE FRICTION TEST
Ploy Flex Doublesided Geocomposite (1002-602) vs. Huitex 80 mil Textured HDPE Geomembrane



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TRI Log No. E2201-83-06

TRI/ENVIRONMENTAL, INC.
A Texas Research International Company

INTERFACE FRICTION TEST REPORT

Client: **Klohn Crippen**
Project: **Kennecott Greens Creek Mining Co.**
Test Date: 8/05-8/05/05

TRI Log#: E2201-83-06
Test Method: ASTM D 5321

Tested Interface: Ploy Flex Doublesided Geocomposite (1002-602) vs. Huitex 80 mil Textured HDPE Geomembrane

Figure 1 Interface as tested and removed from the direct shear box

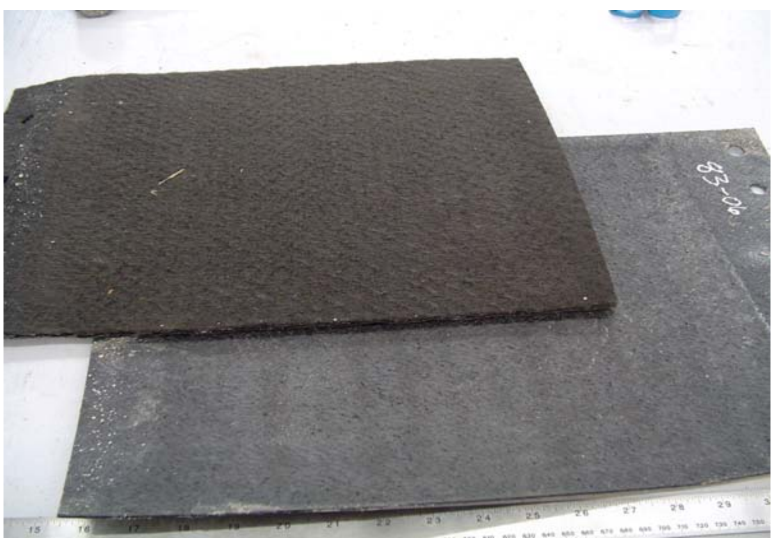

Figure 2 Double-side geocomposite turned over. There was no delaminating of the geotextile from the geonet.

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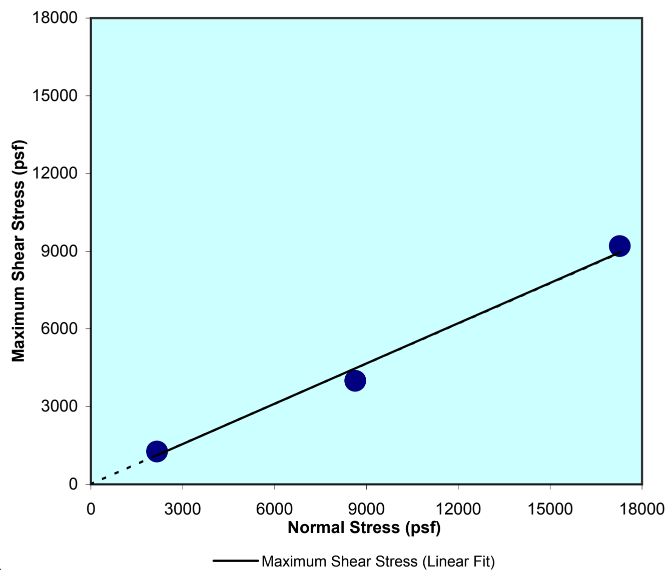
INTERFACE FRICTION TEST REPORT

Client: **Klohn Crippen**
Project: **Kennecott Greens Creek Mining Co.**
Test Date: 8/07-8/09/05

TRI Log#: E2201-83-06
Test Method: ASTM D 5321

Tested Interface: SI Corp. non-woven Geotextile (1001), burnished side vs. Huitex 80 mil Textured HDPE Geomembrane



Upper Box: SI Corp. non-woven geotextile (burnished side)

Lower Box: Huitex 80 mil textured HDPE geomembrane

Interface Conditioning: Interface soaked and loading applied for a minimum of 1 hours prior to shear

Box Dimension: 12"x12"x4"

Test Condition: Wet

Shearing Rate: 0.2 inches/minute

Trial Number	1	2	3
Bearing Slide Resistance (lbs)	29	90	172
Normal Stress (psf)	2160	8640	17280
Maximum Shear Stress (psf)	1285	4077	9361
Corrected Shear Stress (psf)	1256	3987	9189
Secant Angle (degrees)	30.2	24.8	28.0

RESULTS: Maximum Friction Angle and Y-intercept

Regression Friction Angle (degrees):		**27.4**		
Y-intercept or Regression Adhesion (psf):		**0**		
Regression Line:	Y=	0.519	* X +	0
Regression Coefficient (r squared):		0.993		

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Note: The regression line includes the origin.

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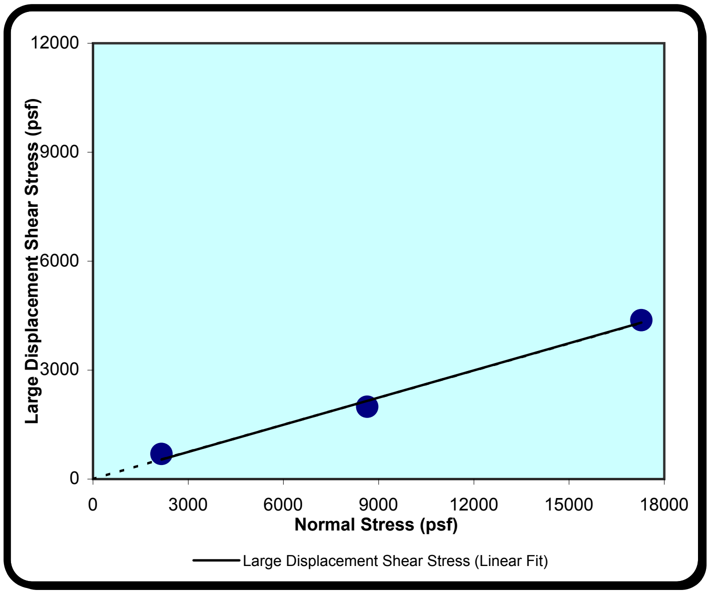
INTERFACE FRICTION TEST REPORT

Client:	**Klohn Crippen**	TRI Log#:	E2201-83-06
Project:	**Kennecott Greens Creek Mining Co.**	Test Method:	ASTM D 5321
Test Date:	8/07-8/09/05		

Tested Interface: SI Corp. non-woven Geotextile (1001), burnished side vs. Huitex 80 mil Textured HDPE Geomembrane



Upper Box: SI Corp. non-woven geotextile (burnished side)

Lower Box: Huitex 80 mil textured HDPE geomembrane

Interface Conditioning: Interface soaked and loading applied for a minimum of 1 hours prior to shear

Box Dimension: 12"x12"x4"

Test Condition: Wet

Shearing Rate: 0.2 inches/minute

Trial Number	1	2	3
Bearing Slide Resistance (lbs)	29	90	172
Normal Stress (psf)	2160	8640	17280
Large Displacment Shear Stress (psf)	721	2080	4545
Corrected Shear Stress (psf)	692	1990	4373
Secant Angle (degrees)	17.8	13.0	14.2

RESULTS: Large Displacement Friction Angle and Y-intercept at 3.3-in. of Displacement

Regression Friction Angle (degrees):		**13.8**		
Y-intercept or Regression Adhesion (psf):		**57**		
Regression Line:	Y=	0.245	* X +	57
Regression Coefficient (r squared):		0.993		

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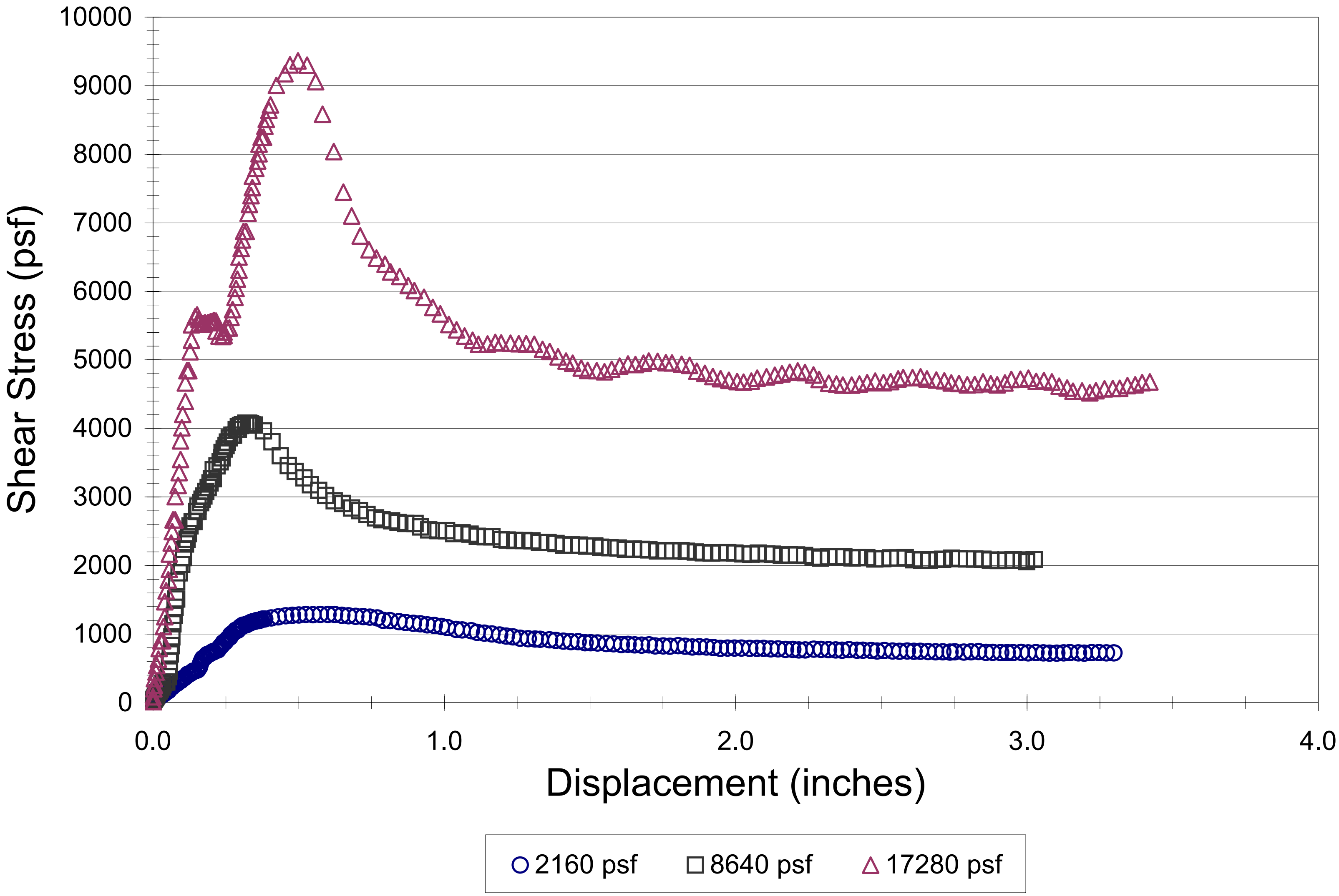
Klohn Crippen INTERFACE FRICTION TEST
SI Corp. non-woven Geotextile (1001), burnished side vs. Huitex 80 mil Textured HDPE Geomembrane



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TRI Log No. E2201-83-06

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INTERFACE FRICTION TEST REPORT

Client: **Klohn Crippen** TRI Log#: E2201-83-06
Project: **Kennecott Greens Creek Mining Co.** Test Method: ASTM D 5321
Test Date: 8/07-8/09/05

Tested Interface: SI Corp. non-woven Geotextile (1001), burnished side vs. Huitex 80 mil Textured HDPE Geomembrane

Figure 1 Interface as tested and removed from the direct shear box

Figure 2 Non-woven geotextile (burnished side) turned over. There was no tearing of geotextile under any of the normal compressive loads

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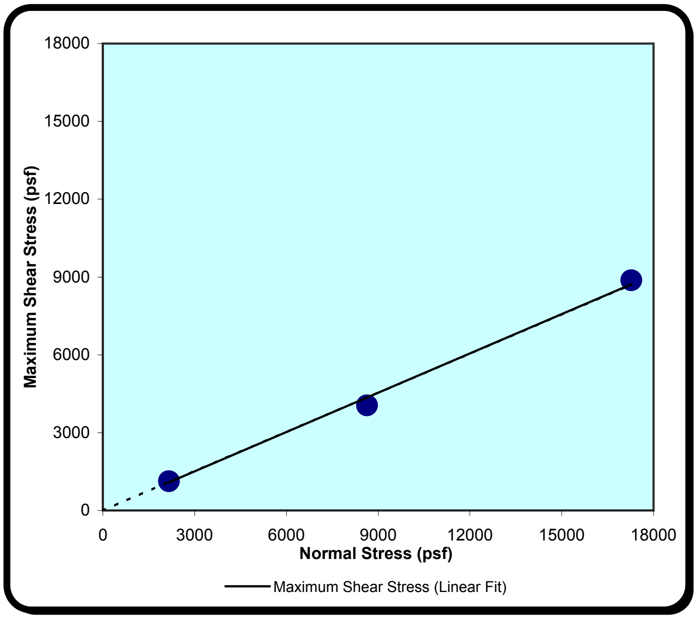
INTERFACE FRICTION TEST REPORT

Client: **Klohn Crippen** TRI Log#: E2201-83-06
Project: **Kennecott Greens Creek Mining Co.** Test Method: ASTM D 5321
Test Date: 8/07-8/09/05

Tested Interface: SI Corp. non-woven Geotextile (1001), non-burnished side vs. Huitex 80 mil Textured HDPE Geomembrane



Upper Box: SI Corp. non-woven geotextile (non-burnished side)

Lower Box: Huitex 80 mil textured HDPE geomembrane

Interface Conditioning: Interface soaked and loading applied for a minimum of 1 hours prior to shear

Box Dimension: 12"x12"x4"

Test Condition: Wet

Shearing Rate: 0.2 inches/minute

Trial Number	1	2	3
Bearing Slide Resistance (lbs)	29	90	172
Normal Stress (psf)	2160	8640	17280
Maximum Shear Stress (psf)	1151	4131	9044
Corrected Shear Stress (psf)	1122	4041	8872
Secant Angle (degrees)	27.5	25.1	27.2

RESULTS: Maximum Friction Angle and Y-intercept

Regression Friction Angle (degrees): **26.8**
Y-intercept or Regression Adhesion (psf): **0**
Regression Line: Y= 0.504 * X + 0
Regression Coefficient (r squared): 0.997

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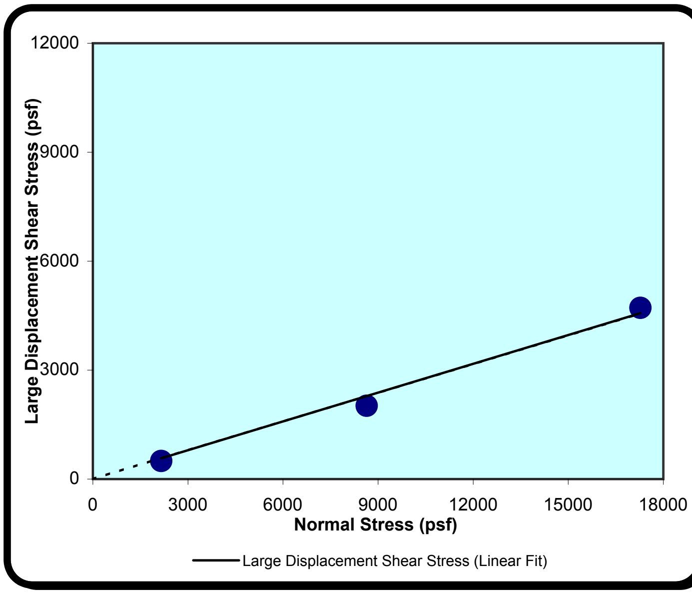
Note: The regression line includes the origin.

INTERFACE FRICTION TEST REPORT

Client:	**Klohn Crippen**	TRI Log#:	E2201-83-06
Project:	**Kennecott Greens Creek Mining Co.**	Test Method:	ASTM D 5321
Test Date:	8/07-8/09/05		

Tested Interface: SI Corp. non-woven Geotextile (1001), non-burnished side vs. Huitex 80 mil Textured HDPE Geomembrane



Upper Box: SI Corp. non-woven geotextile (non-burnished side)

Lower Box: Huitex 80 mil textured HDPE geomembrane

Interface Conditioning: Interface soaked and loading applied for a minimum of 1 hours prior to shear

Box Dimension: 12"x12"x4"

Test Condition: Wet

Shearing Rate: 0.2 inches/minute

Trial Number	1	2	3
Bearing Slide Resistance (lbs)	29	90	172
Normal Stress (psf)	2160	8640	17280
Large Displacment Shear Stress (psf)	528	2106	4885
Corrected Shear Stress (psf)	499	2016	4713
Secant Angle (degrees)	13.0	13.1	15.3

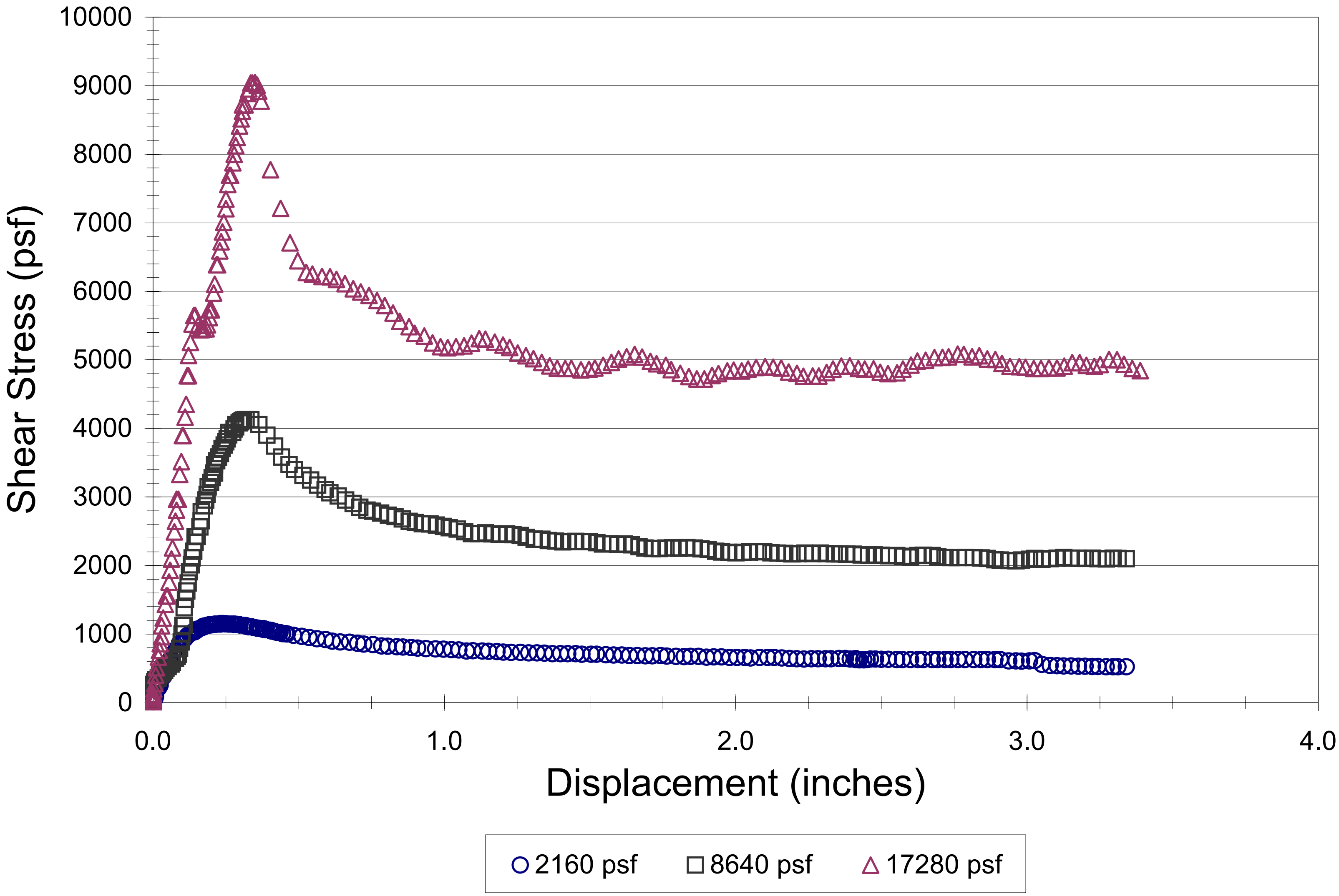
RESULTS: Large Displacement Friction Angle and Y-intercept at 3.3-in. of Displacement

Regression Friction Angle (degrees):		**14.8**		
Y-intercept or Regression Adhesion (psf):		**0**		
Regression Line:	Y=	0.264	* X +	0
Regression Coefficient (r squared):		0.989		

Note: The regression line includes the origin.

John M. Allen, E.I.T, 08/09/05
Quality Review/Date

Klohn Crippen INTERFACE FRICTION TEST
SI Corp. non-woven Geotextile (1001), non-burnished side vs. Huitex 80 mil Textured HDPE Geomembrane



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INTERFACE FRICTION TEST REPORT

Client:	**Klohn Crippen**	TRI Log#:	E2201-83-06
Project:	**Kennecott Greens Creek Mining Co.**	Test Method:	ASTM D 5321
Test Date:	8/07-8/09/05		

Tested Interface: SI Corp. non-woven Geotextile (1001), non-burnished side vs. Huitex 80 mil Textured HDPE Geomembrane

Figure 1 Interface as tested and removed from the direct shear box

Figure 2 Non-woven geotextile (no-burnished side) turned over. There was no tearing of geotextile under any of the normal compressive loads

APPENDIX XI

Liquefaction of Fine Grained Soils

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APPENDIX XI
Liquefaction of Fine-Grained Soils

XI-1. EVALUATION OF LIQUEFACTION OF FINE GRAINED SOILS

The tailings generally contain more than 80% fines (see Appendix VII). Liquefaction susceptibility of fine-grained soils (silts and silty clays) has been assessed using the Modified Chinese liquefaction criteria (Finn, et al., 1994), Andrews and Martin criteria (2000), and Bray, et al. (2004) as screening tools criteria. These assessments are described in the following sections; the results of these assessments are given in Section 9.3.3.

The grain size and Atterberg limit data collected during the 1997, 2002, and 2005 site investigations are shown on Table 1. These data are plotted in Figure 1 to Figure 3 with distinction made between old (pre-1996) and new (post-1996) tailings.

Table 1 Available Tailings Index Test Results

No.	Borehole	Elevation (ft)	Depth (ft)	Water Content W (%)	Liquid Limit LL (%)	Plastic Limit PL (%)	Wc/LL	PI	% Passing No. 200 Sieve	%Passing 0.005mm Sieve	%Passing 0.002mm Sieve
1	EastTail		0.0	12	20	15	0.61	5	83	10	6
2	WestTail		0.0	20	21	17	0.96	4	84	16	8
3	DH-02-04	175.9	60.8	21	21	16	0.98	5	96	14	9
4	DH-02-05	216.5	10.8	15	17	14	0.88	3	85	15	9
5	DH-02-06	172.4	10.8	19	19	15	1.02	4	87	12	6
6	DH-02-08	179.1	15.8	21	21	14	1.01	7	86	12	7
7	DH05-08	213.3	20.0	15	14	12	1.04	2	84.1	13	10
8	DH05-08	193.3	40.0	23	23	16	1.00	7	93.6	15	8
9	DH05-08	173.3	60.0	26	21	15	1.22	6	95.7	20	11
10	DH05-08	153.3	80.0	21	22	16	0.95	6			
11	DH05-08	163.3	70.0	27	26	20	1.02	6	96.1	17	10
12	DH05-09	213.9	19.0	15	16	16	0.96	0	77.1	10	7
13	DH05-09	192.9	40.0	16	19	17	0.83	2	84.4	10	5
14	DH05-09	172.9	60.0	15	17	15	0.89	2	89.7	8	4
15	DH05-09	152.9	80.0	21	18	16	1.17	2	93	10	6
16	DH05-09	142.9	90.0	20	21	17	0.93	4	95.8	18	9
17	DH05-10	192.4	21.6	13	15	14	0.85	1	83.8	10	6
18	DH05-10	174.3	39.7	10	16	14	0.61	2	70.8	16	9
19	DH05-10	159.3	54.7	15	19	16	0.79	3	77.8	14	9
20	DH05-11	189.1	24.3	8	20	16	0.39	4	63.8	13	8
21	DH05-11	174.7	38.7	17	21	19	0.83	2	86.4	12	6
22	DH05-11	159.4	54.0	15	20	16	0.74	4	81.5	13	8
23	DH05-11	144.1	69.3	15	21	15	0.72	6	91.1	12	7
24	DH05-12	153.4	4.6	14	16	13	0.88	3	88.5	10	5
25	DH05-13	148.9	9.3	13	16	15	0.81	NP	88.7	10	5

XI-2. CHINESE CRITERIA

The Modified Chinese criteria (Finn, et al., 1994) indicates that a soil is considered to be potentially liquefiable if the following three criteria are met:

- Clay content (percent finer than 0.005 mm) <15%;
- Liquid limit (LL) <35%; and
- Water content (W) >0.9 x LL. Water content is defined as weight of water divided by weight of solids.

Seventeen tailings samples satisfy the first criterion and are plotted on Figure 1.

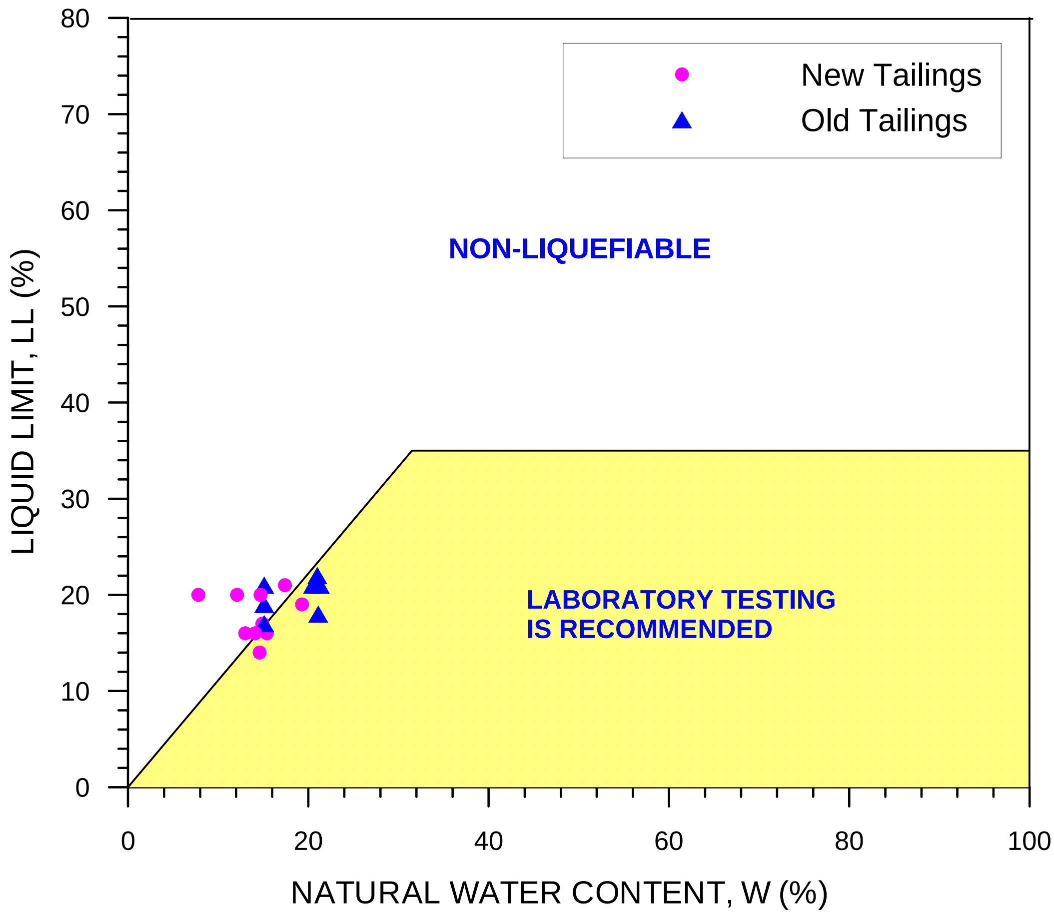


Figure 1 Modified Chinese Criteria

XI-3. ANDREWS AND MARTIN CRITERIA

The criteria proposed by Andrews and Martin (2000) for fine grained soils are as follows:

- Clay content (percent finer than 0.002 mm) < 10%; and
- Liquid limit < 32 %.

Both criteria need to be satisfied for the soil to be considered potentially liquefiable. If only one of the two criteria is satisfied, further testing is recommended by Andrews and Martin (2000). These criteria are illustrated on Figure 2.

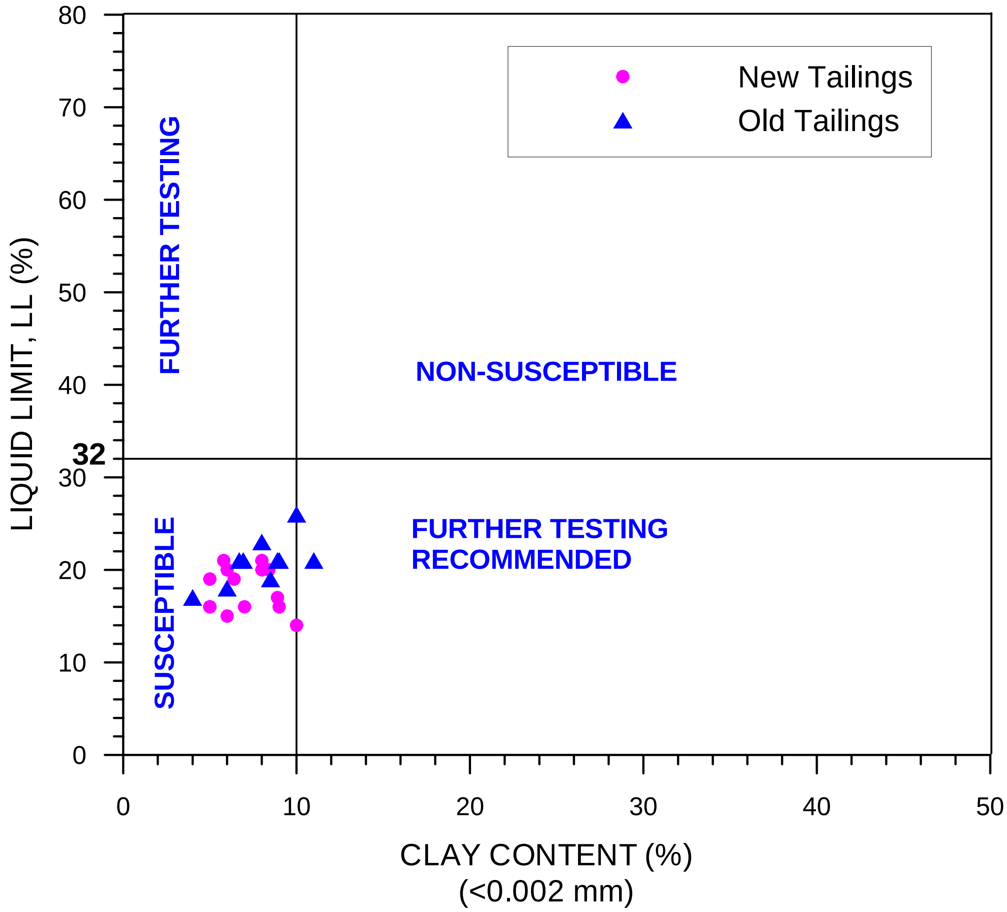


Figure 2 Andrews and Martin Criteria

XI-4. BRAY, ET AL. CRITERIA

Recently, Bray, et al. (2004) proposed a new criterion to evaluate the liquefaction susceptibility of fine-grained soils. They noted that it is not the amount of clay-size particles in the soil, but rather the amount of clay minerals that best indicate the susceptibility to liquefaction. Hence, they used the soils plasticity index and the ratio of water content to the liquid limit in their proposed criteria. Their criteria are illustrated on Figure 3.

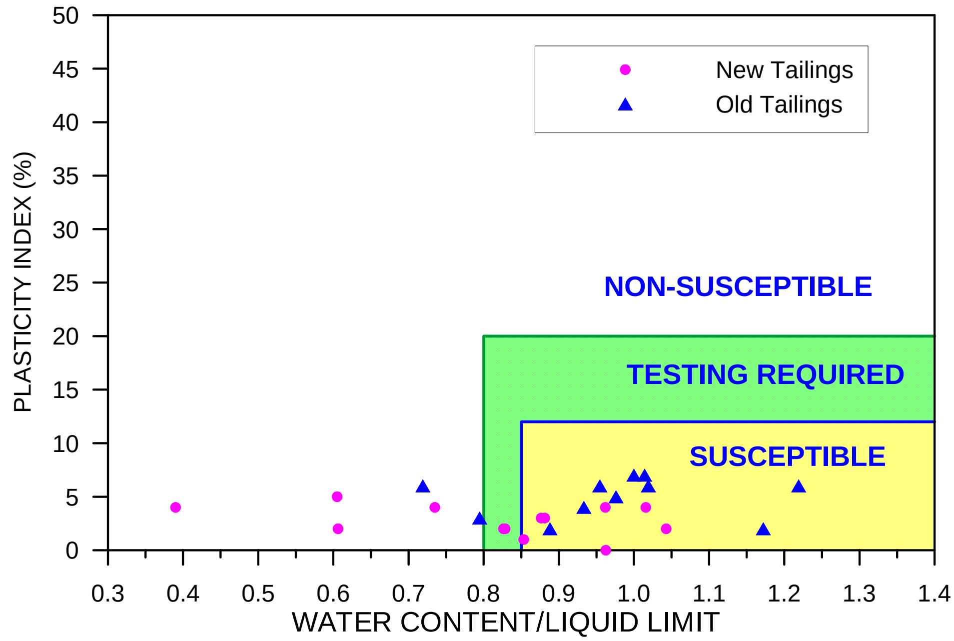


Figure 3 Bray, et al. Criteria

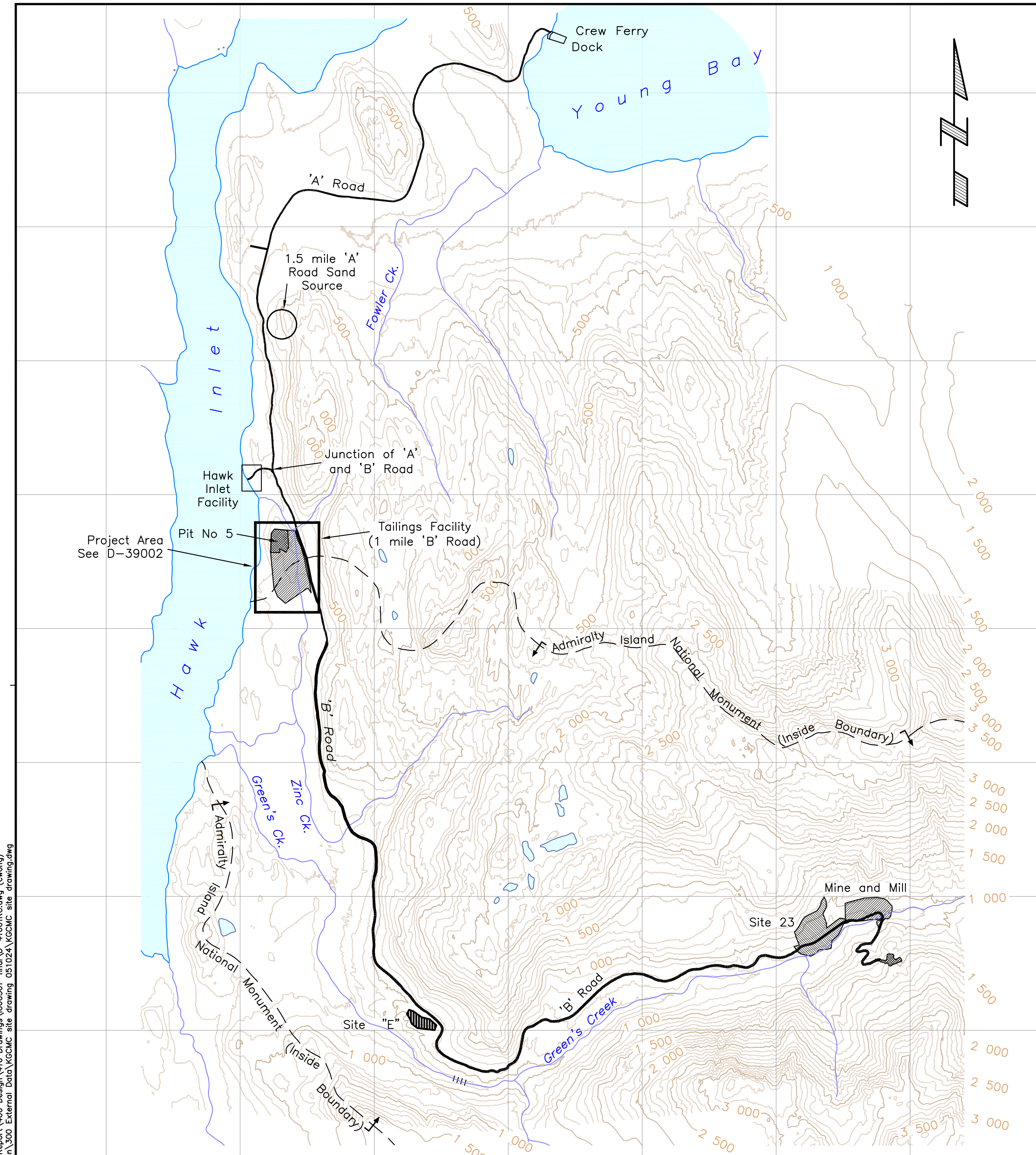
XI-5. RESULTS OF ASSESSMENT

Figure 1 indicates the liquefaction assessment of the samples using the Chinese Criteria. The majority of the new tailings are non-liquefiable, and additional laboratory testing is recommended for the old tailings to confirm the liquefaction potential. The Andrews and Martin criteria (Figure 2) indicates that both old and new tailings are susceptible to liquefaction. According to the criteria by Bray, et al. (Figure 3) about half of the new tailings samples are susceptible to liquefaction, and all but two of the old tailings samples are susceptible.

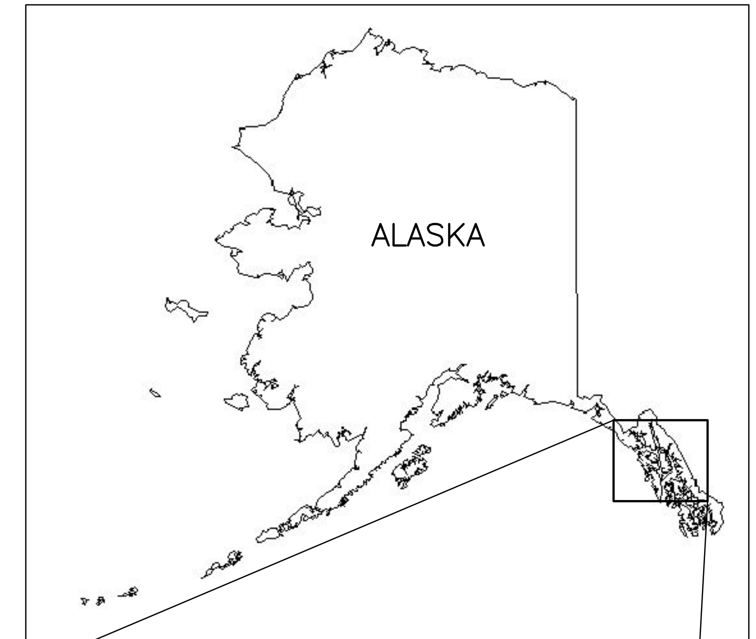
Note, these methods makes no reference to the cyclic stress, density or saturation levels but simply provides a screening tool to help with a decision on whether to proceed with more analyses.

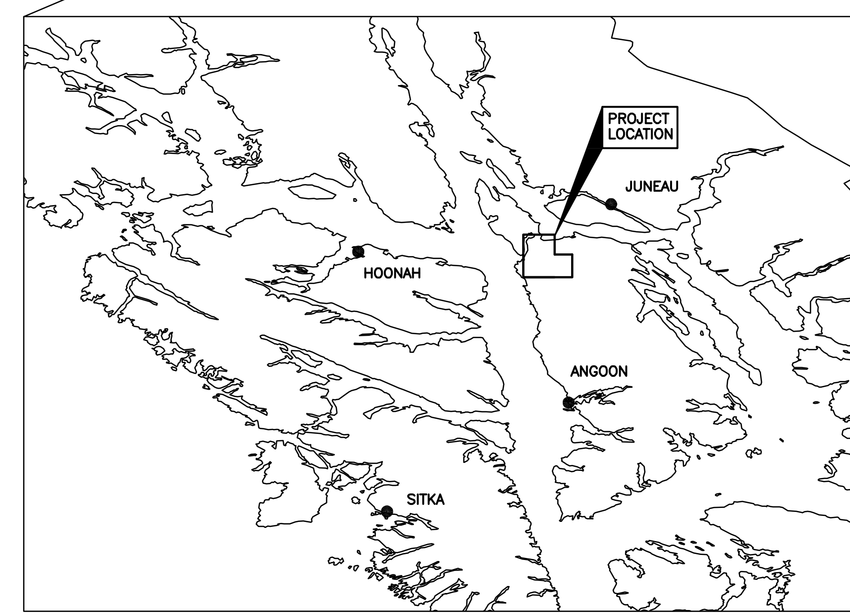
DRAWINGS

D-41001	**General Arrangement**
D-41002	**General Arrangement of Stage 2 Expansion**
D-41003	**Tailings Facility Plan showing all geotechnical holes**
D-41004	**Tailings Facility Plan showing all geotechnical holes by Klohn Crippen**
D-41005	**Final Tailings Pile Geometry Plan with Stability Sections**
D-41006	**Interpreted Thickness of Sand and Gravel (Unit 5) Layer**



PLAN
SCALE A





KEY PLAN
SCALE: NTS

NOTES:

1. BASE PLAN PROVIDED BY KGCMC, OCTOBER 2005.
2. CONTOUR INTERVAL IS 100 FT.
3. ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE. COORDINATES AND ELEVATIONS ARE REFERENCED TO MINE DATUM.

SCALE A: 2500 0 2500 FT

NOT FOR CONSTRUCTION

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
C	MAR. 1, 2006	ISSUED FOR FINAL				
B	JAN. 24, 2006	FOR CLIENT REVIEW				
A	SEPT 30 2005	FOR CLIENT REVIEW				

DRAWING NO.	REFERENCE DRAWING

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CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: GENERAL ARRANGEMENT

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802 A41	D-41001	C

CANCEL PRINTS BEARING PREVIOUS REVISION

KCC-D

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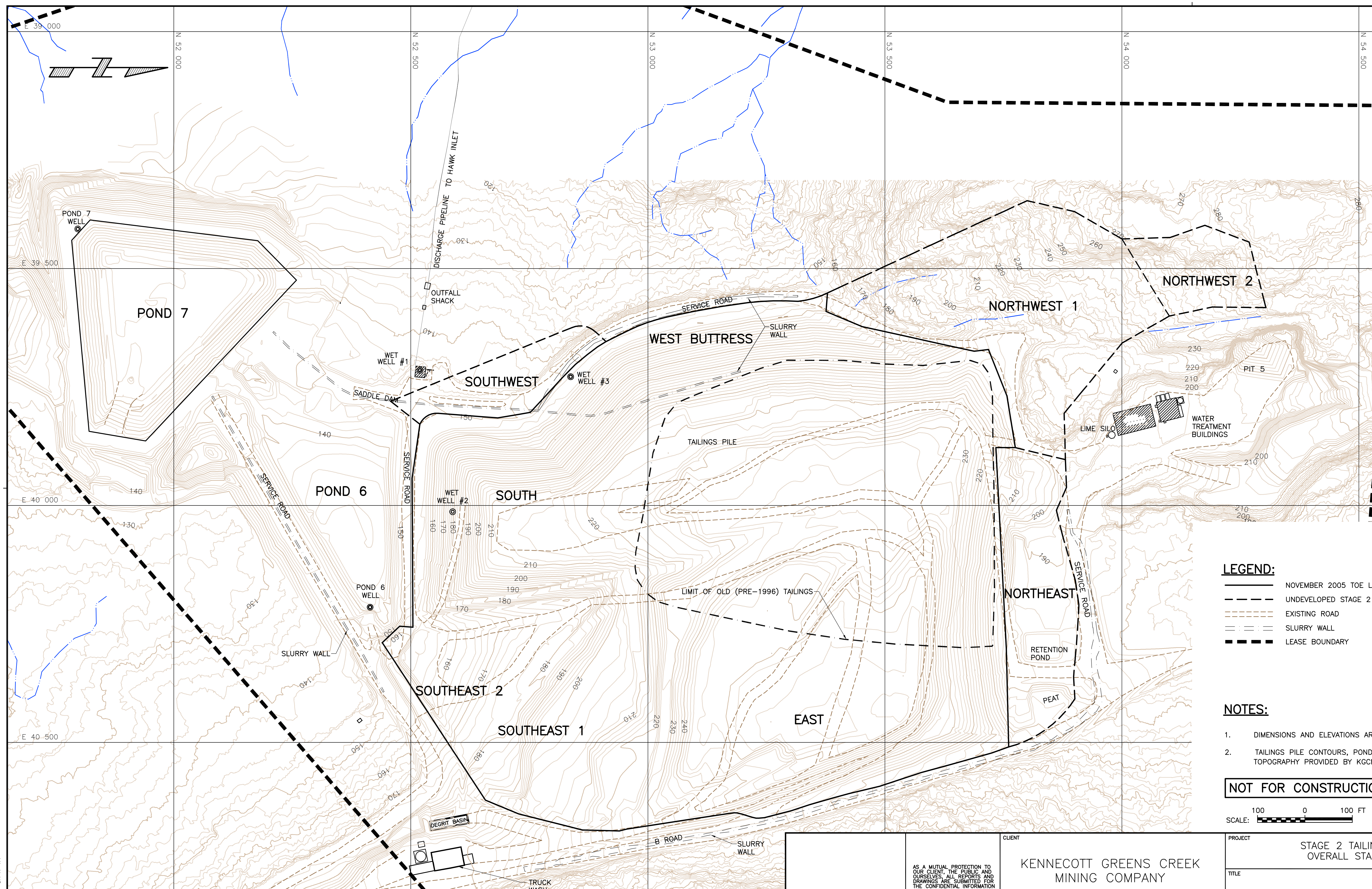

LEGEND:

- NOVEMBER 2005 TOE LINE OF TAILINGS FACILITY
- UNDEVELOPED STAGE 2 TAILINGS AREA
- EXISTING ROAD
- SLURRY WALL
- LEASE BOUNDARY

NOTES:

1. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.
2. TAILINGS PILE CONTOURS, POND 7 CONTOURS AND BASE TOPOGRAPHY PROVIDED BY KGCMC (OCTOBER 2005).

NOT FOR CONSTRUCTION

SCALE: 100 0 100 FT

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
C	MAR. 1, 2006	ISSUED FOR FINAL				
B	JAN. 24, 2006	FOR CLIENT REVIEW				
A	SEPT. 30, 2005	FOR CLIENT REVIEW				

DRAWING NO.	REFERENCE DRAWING

CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION
OVERALL STABILITY UPDATE

TITLE
GENERAL ARRANGEMENT OF STAGE 2 EXPANSION

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802 A41	D-41002	C

CANCEL PRINTS BEARING PREVIOUS REVISION

KCC-D

Date: 3/1/2006
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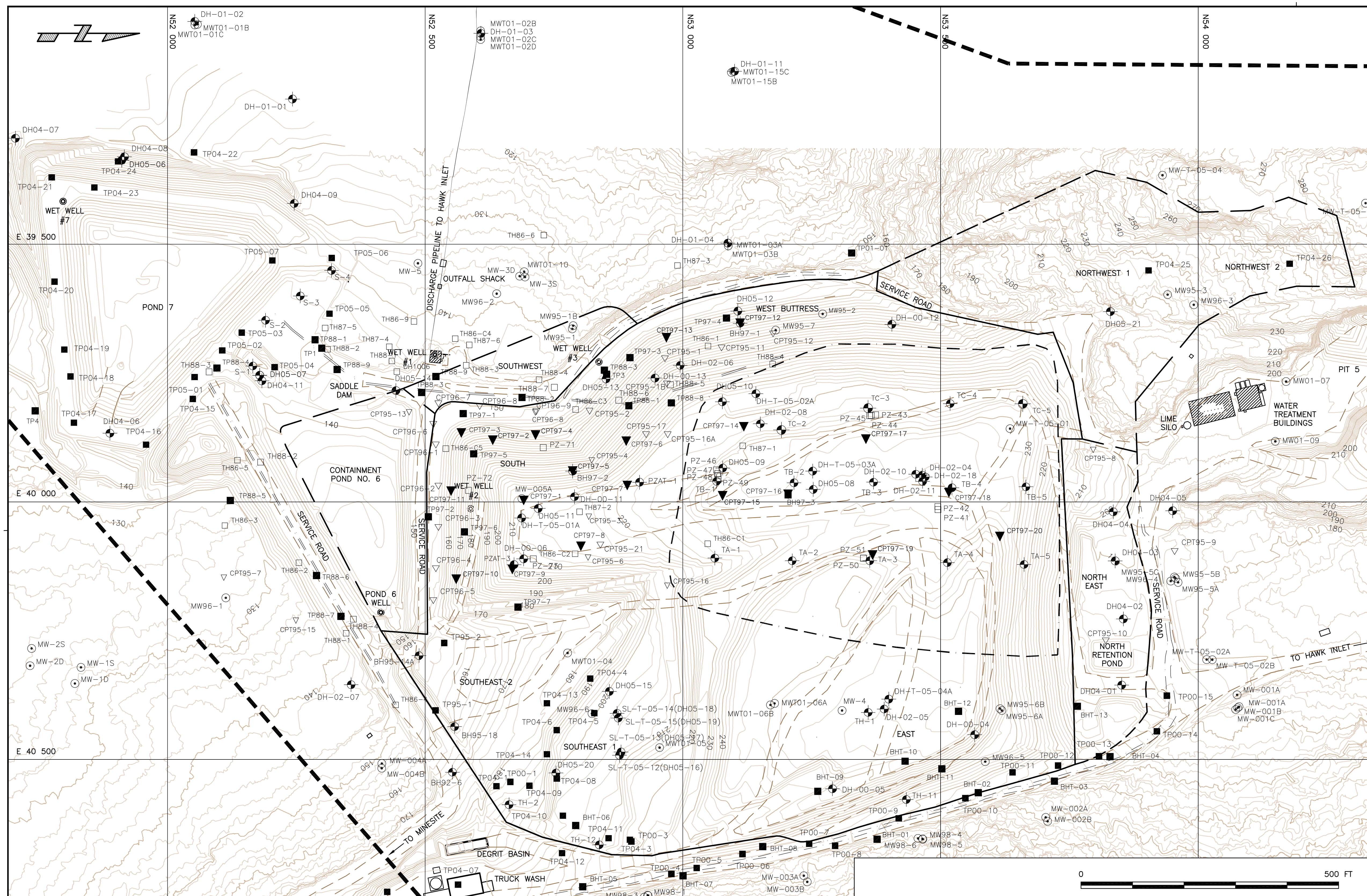
Xrefs: Pond 7contours-GA R-1A



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).

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DRAWING NO.	REFERENCE DRAWING	NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
		C	MAR. 1, 2006	ISSUED FOR FINAL				
		B	JAN. 24, 2006	FOR CLIENT REVIEW				
		A	SEPT 30 2005	FOR CLIENT REVIEW				

CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE
TAILINGS FACILITY PLAN
ALL GEOTECHNICAL INVESTIGATIONS

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802 A41	D-41003	C

CANCEL PRINTS BEARING PREVIOUS REVISION

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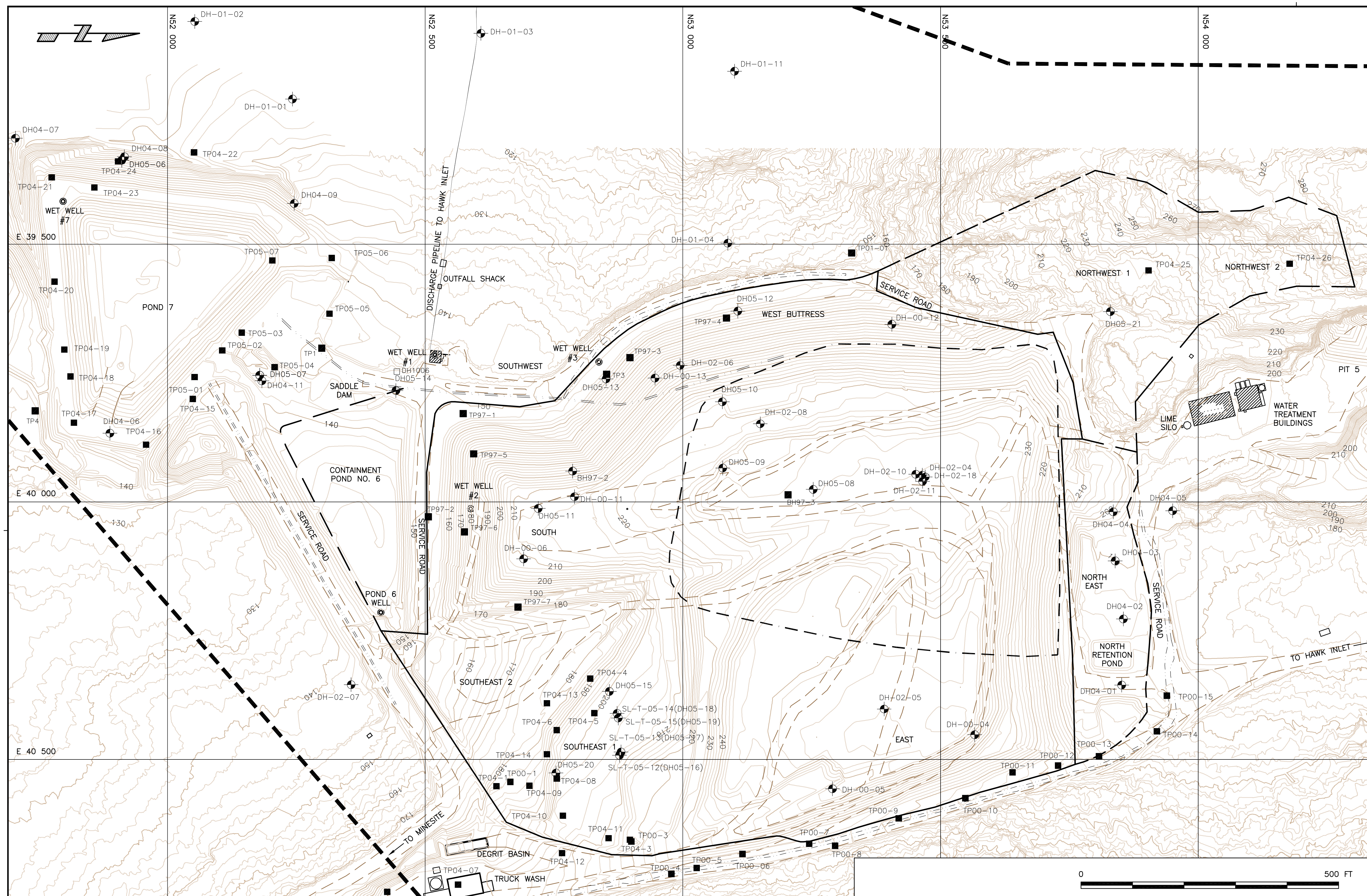
Xrefs: Pond 7contours-GA R-1A



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
- BH97-1 — DRILL HOLE, KLOHN CRIPPEN 1997
- TP97-1 — TEST PIT, KLOHN CRIPPEN 1997
- TP00-11 — TEST PIT, KLOHN CRIPPEN 2000
- DH-00-04 — DRILL HOLE, KLOHN CRIPPEN 2000
- 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

NOTES

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

NOT FOR CONSTRUCTION

DRAWING NO.	REFERENCE DRAWING	NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
		C	MAR. 1, 2006	ISSUED FOR FINAL				
		B	JAN. 24, 2006	FOR CLIENT REVIEW				
		A	SEPT 30 2005	FOR CLIENT REVIEW				

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: TAILINGS FACILITY PLAN GEOTECHNICAL INVESTIGATIONS BY KLOHN CRIPPEN

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802 A41	D-41004	C

CANCEL PRINTS BEARING PREVIOUS REVISION

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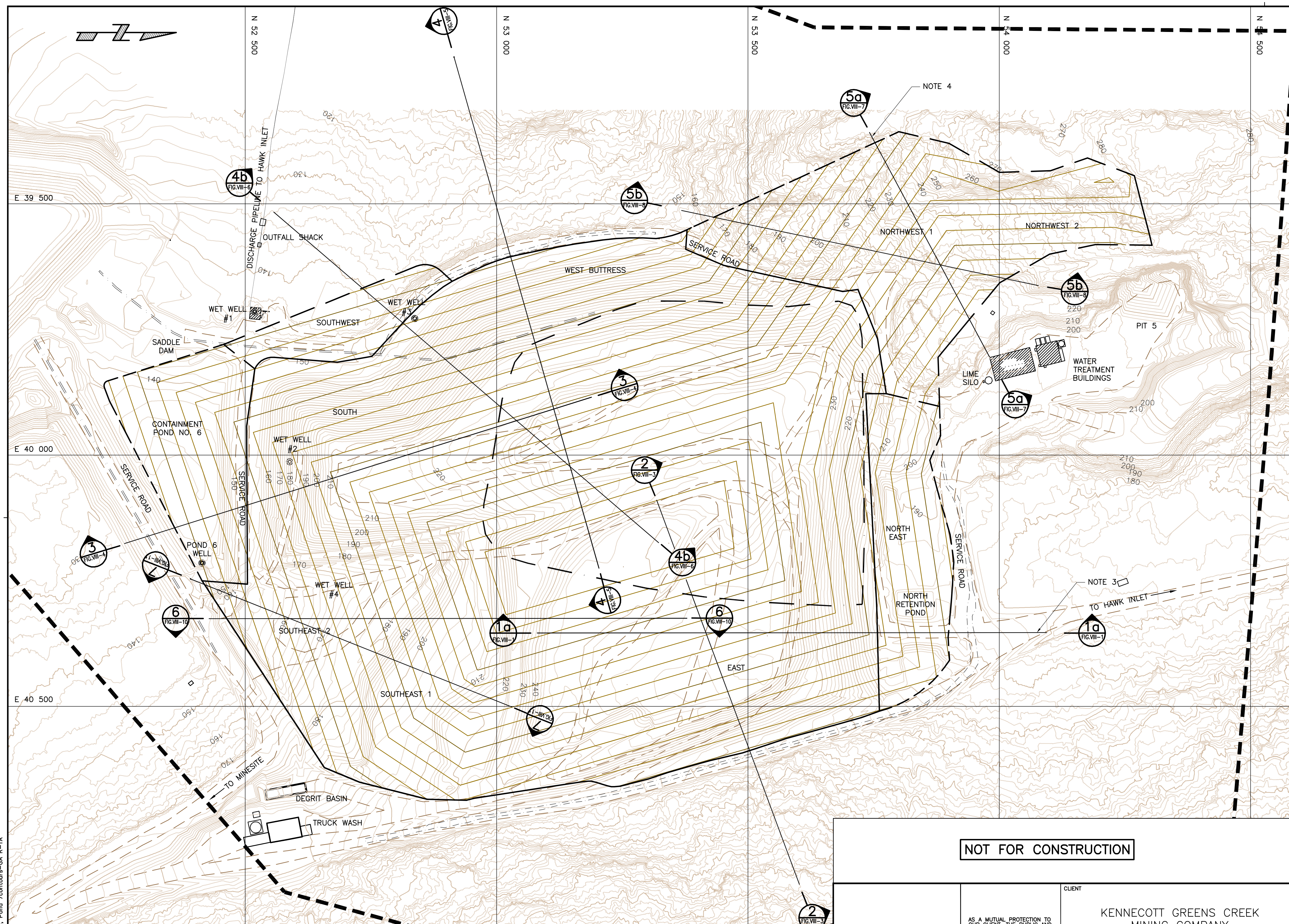

LEGEND

- OLD TAILINGS FACILITY BOUNDARY
- CURRENT TOE LINE OF TAILINGS FACILITY
- STAGE 2 TAILINGS TOE LINE
- EXISTING ROAD
- SLURRY WALL
- LEASE BOUNDARY

NOTES

1. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.
2. TAILINGS PILE CONTOURS AND BASE TOPOGRAPHY PROVIDED BY KGCMC (OCTOBER 2005).
3. SECTION 1b HAS THE SAME STRATIGRAPHY AS SECTION 1, WITH THE TAILINGS SLOPE EXTENDED TO ULTIMATE EL. 330 FT.
4. SECTION 5c IS BASED ON SECTION 5a STRATIGRAPHY, WITH THE EAST SLOPE INCREASED TO 3H:1V FROM THE TOE TO THE ULTIMATE PILE ELEVATION 280 ft.

0 500 FT

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NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
C	MAR. 1, 2006	ISSUED FOR FINAL				
B	JAN. 24, 2006	FOR CLIENT REVIEW				
A	SEPT 30 2005	FOR CLIENT REVIEW				

DRAWING NO. | REFERENCE DRAWING

CLIENT: KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT: STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE: FINAL TAILINGS PILE GEOMETRY PLAN WITH STABILITY SECTIONS

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802 A41	D-41005	C

CANCEL PRINTS BEARING PREVIOUS REVISION

Time: 12:1:33
Date: 3/1/2006
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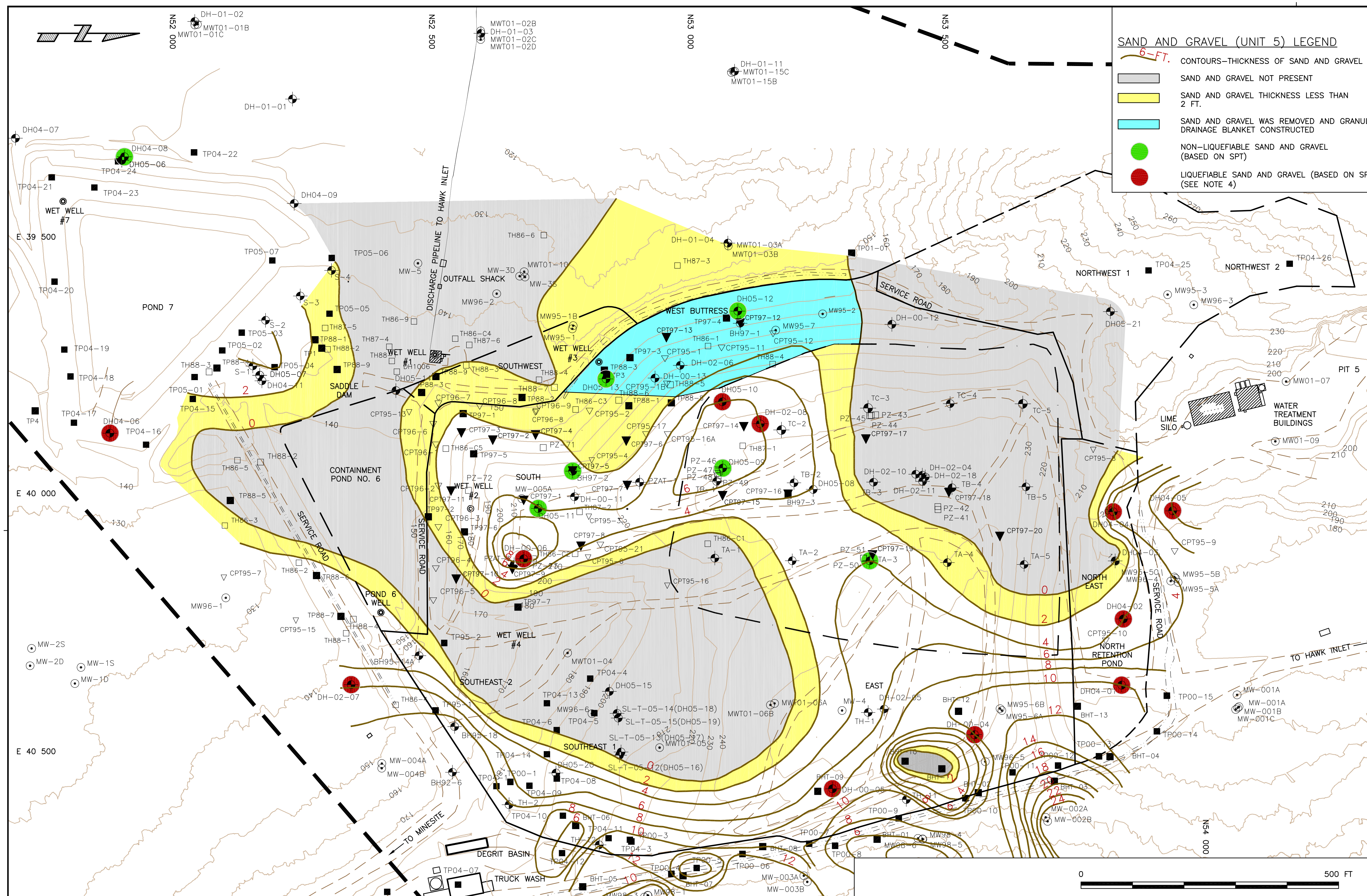
Xrefs: Tails-Final-Contour-RB, Pond 7contours-GA R-1A



SAND AND GRAVEL (UNIT 5) LEGEND

- 6-FT. CONTOURS–THICKNESS OF SAND AND GRAVEL
- SAND AND GRAVEL NOT PRESENT
- SAND AND GRAVEL THICKNESS LESS THAN 2 FT.
- SAND AND GRAVEL WAS REMOVED AND GRANULAR DRAINAGE BLANKET CONSTRUCTED
- NON–LIQUEFIABLE SAND AND GRAVEL (BASED ON SPT)
- LIQUEFIABLE SAND AND GRAVEL (BASED ON SPT) (SEE NOTE 4)

LEGEND

Symbol	Description
	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
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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
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TP00–11	TEST PIT, KLOHN CRIPPEN 2000
DH–00–04	DRILL HOLE, KLOHN CRIPPEN 2000
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MWT01–03B	MONITORING WELL, ENVIRONMENTAL DESIGN ENGINEERING 2001
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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

NOTES

1. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS NOTED OTHERWISE.
2. TAILINGS PILE CONTOURS AND BASE TOPOGRAPHY PROVIDED BY KGCMC (OCTOBER 2005).
3. LOOSE SAND AND GRAVEL WAS EXCAVATED DURING POND 7 INSTALLATION AND SE1 AND 2 CONSTRUCTION.
4. LIQUEFIABLE SAND HAS FOS<1.1 IN SPT LIQUEFACTION ASSESSMENT UNDER THE MDE.

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DRAWING NO.	REFERENCE DRAWING	NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
		C	MAR. 1, 2006	ISSUED FOR FINAL				
		B	JAN. 24, 2006	FOR CLIENT REVIEW				
		A	SEPT 30 2005	FOR CLIENT REVIEW				

CLIENT
KENNECOTT GREENS CREEK MINING COMPANY

KLOHN CRIPPEN

PROJECT
STAGE 2 TAILINGS EXPANSION OVERALL STABILITY UPDATE

TITLE
INTERPRETED THICKNESS OF SAND AND GRAVEL (UNIT 5) LAYER

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07802 A41	D–41006	C

CANCEL PRINTS BEARING PREVIOUS REVISION

Time: 12:3:42
Date: 3/1/2006
Scale: 1=1(PS)
Drawing File: M:\M07802\A41 – Overall Design Report\400 Design\410 Drawings\060301–final\D–41006RC.dwg (cwong)
Xrefs: Pond 7contours–GA R–1A