MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT



STABILITY ASSESSMENT REPORT

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MONTANA RESOURCES, LLP
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EXECUTIVE SUMMARY

Montana Resources, LLP is in the process of preparing a permit amendment application for continued use of the Yankee Doodle Tailings Impoundment (YDTI) to provide for continued mining beyond 2020. The proposed amendment considers the YDTI with embankments constructed to a crest elevation of 6,450 ft. The amendment will provide for approximately 12 years of additional mine life. Knight Piésold Ltd. has assessed the slope stability of the embankment at its design crest elevation by completing limit equilibrium and earthquake-induced deformation analyses.

The facility comprises a valley-fill style impoundment created by a continuous embankment with three limbs: North-South, East-West, and West. Originally constructed in 1963 using rockfill from the Berkeley Pit, the YDTI has been continuously raised to Elevation (EL.) 6,400 ft using rockfill first from the Berkeley Pit (ending 1982) and thereafter from the Continental Pit (beginning in 1986). The YDTI will continue to be constructed with similar techniques and methods that have been used in past raises. The embankments will continue to be raised or newly constructed as free-draining structures with rockfill from the Continental Pit. Seepage water is collected downstream of the embankment in Horseshoe Bend where it is conveyed for reuse in processing. Tailings were historically continuously discharged from a single point at the southern extent of the YDTI and the tailings beach is extensive and well drained in the top 50 to 80 ft with a current length in excess of 5,000 ft. The design contemplates multiple tailings discharge points to develop extensive drained tailings beaches adjacent to all three embankments. The changes to the tailings distribution system were made between 2016 and 2017. Eight discharge locations are now presently available.

The key performance factors for stability of the embankment are developing large drained tailings beaches that maintain the supernatant pond remote from the embankments and reduces the pore pressures in the tailings beach adjacent to the upstream face in the long-term. Selective and strategic placement of rockfill to further improve embankment stability and to support reclamation objectives should be considered while evaluating options for storage of excess rockfill produced during mining of the Continental Pit.

The slope stability assessment was carried out on four representative sections through the embankment. Critical rotational slip surfaces were found for both downstream and upstream configurations. Static analyses considered base case material properties for normal operating conditions and liquefied tailings properties for post-earthquake conditions. Material properties were selected based on past site investigations. Mohr-Coulomb strength was assigned to the bedrock and overburden while the Leps Angular Sand function was used for the embankment rockfill units. The tailings were assigned Mohr-Coulomb properties for the base case and a depth-related undrained shear strength ratio for the liquefied case. Pseudo-static analyses were completed to determine the seismic yield acceleration required for estimating crest settlement and deformation along the critical slip surface with seismic loading from the Maximum Credible Earthquake (MCE).

The governing scenario for all sections analyzed is a downstream slip surface through the embankment rockfill. The YDTI embankments are stable with a factor of safety (FS) of 2.0 or greater, which exceeds the legislated requirement of 1.5 for normal operating conditions. The FS values for the upstream cases are typically greater than 5 for normal operating conditions.

Tailings materials in the beach and beneath the rockfill surcharge layer are predominantly near to the boundary between potentially contractive and potentially dilative behavior, and the rockfill surcharge tends to increase the potential for dilative behavior. The cyclic liquefaction assessment indicates that the upper saturated tailings within the YDTI and outside the rockfill surcharge area are potentially liquefiable with seismic loading from the MCE. However, the top 50 ft to 80 ft or more of tailings beach adjacent to the embankment and beneath the rockfill surcharge is currently unsaturated or partially saturated and is therefore not subject to liquefaction. The liquefaction assessment demonstrated that the rockfill surcharge effectively mitigates the potential for cyclic liquefaction of the surcharged tailings zone during operations. Localized liquefaction of the tailings underlying the rockfill surcharge may occur in closure with seismic loading from the 84^{th}-percentile MCE while the tailings remain saturated.

The largest impact of the earthquake-induced strength loss is in the upstream scenarios where the FS could be reduced by more than 50%; however, factors of safety exceed 3.0 under these conservative conditions. The analyses indicate that the proposed facility meets the legislative requirements for static stability for post-earthquake conditions while conservatively considering undrained strength analyses for the saturated, potentially contractive tailings material. The analyses also indicate that the embankment will remain stable even if lower bound undrained strengths were triggered along a continuous layer of saturated overburden and rockfill in the base of the embankment. The favorable orientation of the embankment fabric and geotechnical investigations indicates that although weaker and stronger zones exist, a continuous weaker layer is not credible.

The static and seismic stability of the East-West Embankment was improved between 2014 and 2017 by constructing the rockfill surcharge. The tailings discharge point was also relocated further out into the impoundment due to the construction of the rockfill surcharge, which has resulted in reduced pore pressures in the tailings near the embankment and increased the depth of the unsaturated zone in the tailings below the surcharge. The continued filling of the EL. 6,450 ft lift of the YDTI can be completed while achieving FS values that do not decrease below those computed for 2014 conditions prior to surcharging. Thus the impoundment stability for the higher EL. 6,450 ft lift is equivalent or improved as compared to the 2014 conditions.

The earthquake deformation analysis indicates that the maximum estimated earthquake-induced deformation will be within design tolerances for the median MCE during operations and the 84^{th}-Percentile MCE during closure. The maximum combined displacement during operations was estimated to be 3.4 ft for the East-West Embankment compared to 5 ft of minimum freeboard. The maximum combined displacement during closure was estimated to be 23 ft compared to 50 ft of tolerable combined displacement.

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ABBREVIATIONS

ACC	Anaconda Copper Company
BQM	Butte Quartz Monzonite
CDSS	cyclic direct simple shear
CIUC	consolidated isotropically undrained compression
CPT	cone penetration testing
CRR	cyclic resistance ratio
CSR	cyclic stress ratio
CU	consolidated undrained
DSS	direct simple shear
EL	elevation
FS	factor of safety
ft	feet
HLA	Harding Lawson Associates
HsB	Horseshoe Bend
IDF	Inflow Design Flood
IECO	International Engineering Company
KP	Knight Piésold Ltd.
kPa	kilopascal
LE	limit equilibrium
MR	Montana Resources, LLP
MCA	Montana Code Annotated
MCE	maximum credible earthquake
NCS	Normalized Crest Settlement
pcf	pounds per cubic foot
PGA	peak ground acceleration
PMF	probable maximum flood
psf	pounds per square foot
psi	pounds per square inch
SI	site investigation
SOL	setting out line
TAC	The Anaconda Company
TSF	tailings storage facility
TX	triaxial
WED	West Embankment Drain
YDTI	Yankee Doodle Tailings Impoundment

1 – INTRODUCTION

1.1 PROJECT DESCRIPTION

Montana Resources, LLP (MR) operates an open pit copper and molybdenum mine located adjacent to Butte, Montana. The operation includes a daily mill throughput of roughly 50,000 short tons of ore, a small-scale leach operation, and the Yankee Doodle Tailings Impoundment (YDTI). The YDTI comprises a valley-fill style impoundment created by a continuous embankment that, for descriptive purposes, is divided into three limbs (Figure 1.1):

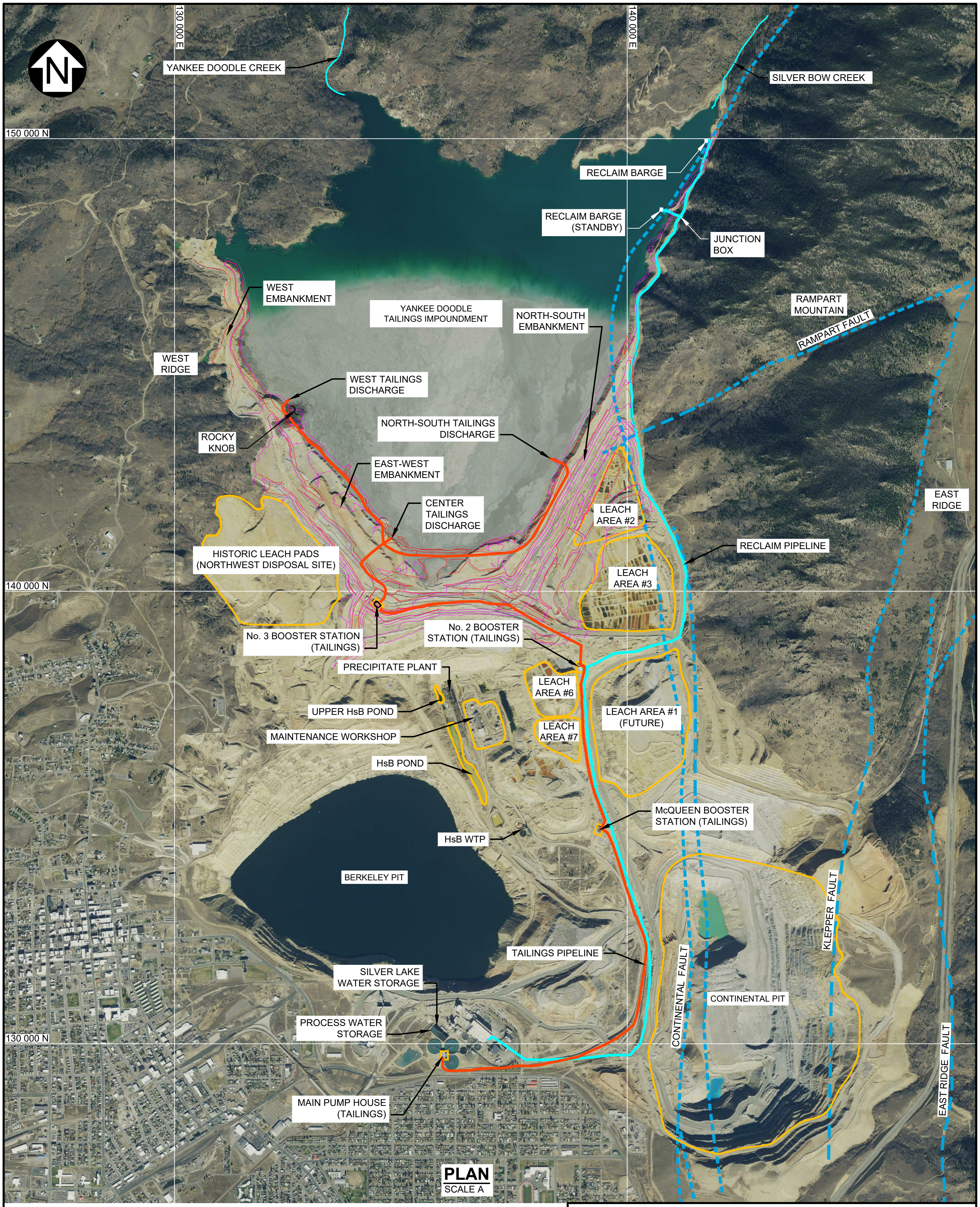
- The North-South Embankment forms the eastern to southeastern limb, abutting the base of Rampart Mountain, and forming the eastern battery limit of the MR mine site.
- The East-West Embankment forms the southwestern limb, running approximately east to west in orientation and is constructed upstream of Horseshoe Bend (HsB) and the Berkeley Pit.
- The West Embankment forms the western limb of the YDTI and runs approximately north to south in orientation. The West Embankment is constructed along the side of the West Ridge and forms the western battery limit of the facility.

Originally constructed in 1963 using rockfill from the Berkeley Pit, the YDTI has been continuously raised to Elevation (EL.) 6,400 ft using rockfill first from the Berkeley Pit (ending 1982) and thereafter from the Continental Pit (beginning in 1986). Historically, the YDTI has been constructed by progressively placing rockfill to form free-draining embankments. The rockfill comprises pit-run material, end-dumped in 30 to 100-foot thick lifts, and traffic compacted with the mine haul fleet. The embankment design also incorporates a zone of fine-grained material (alluvium) placed on the upstream face of the embankment to limit tailings migration into the rockfill.

Tailings were historically discharged into the YDTI at a single location at the southern point of the impoundment near Station 8+00W on the East-West Embankment. Supernatant water is reclaimed for re-use in the mill process from the northeast end of the YDTI using two floating barges. The design contemplates multiple tailings discharge points to develop extensive drained tailings beaches adjacent to all three embankments. The changes to the tailings distribution system were made between 2016 and 2017. Three discharge locations were operational as of March 2017 as shown on Figure 1.1, and eight discharge locations are now presently available.

Seepage water flows through the free-draining rockfill embankments and discharges as a number of small seeps along the downstream toe of the East-West Embankment. Smaller flows of perched seepage (Seep 10) discharge at approximately EL. 5,925 ft. Flows at Seep 10 are inferred to be from lateral drainage from the tailings into the more permeable rockfill, and ultimately follow a historic mine haul ramp alignment. Flow began in approximately 1989, and the seepage flow rate has been relatively constant since it began. The acidic seepage flows are collected and conveyed to the Precipitation Plant for processing to recover copper. The processed seepage is collected in the ponds of HsB, located immediately downstream of the East-West Embankment (Figure 1.1), and then treated in a water treatment plant before it is incorporated into the process water system.

The project site has both active and decommissioned leach areas (Figure 1.1). Active leach pads, no longer loaded, are located downstream from the North-South Embankment and are used to recover copper from weakly-mineralized rock. Decommissioned leach pads are located northwest of the HsB Pond and have been capped as a rockfill storage area downstream of the East-West Embankment.



NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. IMAGERY FROM 2015 AIR PHOTO PROVIDED BY MONTANA RESOURCES.
3. EMBANKMENT TOPOGRAPHY 2017 PROVIDED BY MONTANA RESOURCES ON MARCH 19, 2017.

LEGEND:

- - - - FAULT LOCATIONS FROM WITKIND (1975) SMEDES (1966,1962) AND OTHERS.

SCALE A: 1500 0 1500 3000 4500 6000 7500 ft

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
1	14AUG'17	ISSUED WITH REPORT	DDF	MJC	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

SITE LAYOUT (MARCH 2017)

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA101-126/12	2

FIGURE 1.1 REV 1

1.2 PURPOSE AND SCOPE

MR is in the process of preparing a permit amendment application for continued use of the YDTI to provide for continued mining beyond 2020. The proposed amendment considers the YDTI with embankments constructed to a crest elevation of 6,450 ft and commencing operation of the West Embankment Drain (WED). The amendment will provide for approximately 12 years of additional mine life.

This report, prepared by Knight Piésold Ltd. (KP), summarizes the slope stability assessment of the YDTI embankment at its proposed design crest elevation. Specifically, this assessment includes liquefaction assessment, limit equilibrium slope stability, and earthquake-induced deformation analyses. It does not consider other hypothetical failure modes such as internal erosion and flood overtopping, which are discussed in a separate report (KP, 2018).

1.3 DESIGN STANDARDS AND CRITERIA

The laws governing tailings storage facility (TSF) design, operation and reclamation are contained within sections of State law as described by Montana Code Annotated (MCA) Title 82 Chapter 4 Part 3. The governing legislation for a TSF design document supporting a permit application is MCA 82-4-376, which requires either an analysis showing that the proposed design meets the minimum design requirements for a new TSF or an analysis showing that the proposed design does not reduce the TSF's original design factors of safety and seismic event design criteria. The design basis for the YDTI considers the requirements for a new facility including the following for the slope stability objectives:

- The tailings, embankment, and foundation materials controlling slope stability are not susceptible to liquefaction or to significant strain-weakening under the anticipated static or cyclic loading conditions, to the extent that the amount of estimated deformation under the loading conditions would result in loss of containment.
- For a new TSF, minimum design factors of safety (FS) against slope instability are required to be:
    - 1.5 for static loading under normal operating conditions.
    - 1.2 for post-earthquake, static loading conditions.
    - The analysis of normal operating and post-earthquake static loading conditions must consider (if relevant to the loading condition) appropriate use of undrained shear strength analysis for saturated, contractive materials.
    - Reduced factors of safety or seismic design criteria are acceptable if the Independent Review Panel agrees that site-specific conditions justify the design to the specified requirements of factors of safety or seismic design criteria is not necessary.
- For a new TSF, an analysis showing that the seismic response of the TSF does not result in the uncontrolled release of impounded materials or other undesirable consequences when subject to the ground motion associated with the 1-in-10,000-year event, or the maximum credible earthquake (MCE), whichever is larger. Any numerical analysis of the seismic response must be calculated for the normal maximum loading condition with steady-state seepage. The analysis must consider:
    - Anticipated ground motion frequency content, fundamental period, and dynamic response
    - Potential liquefaction
    - Loss of material strength

  - o Settlement, ground displacement, deformation, and
  - o Potential secondary failure modes.
- For a pseudo-static stability analysis, a justification for the use of the method with respect to the anticipated response to cyclic loading of the tailings facility structure and constituent materials, accompanied by a description of the assumptions used in deriving the seismic coefficient.

The design must store or otherwise manage the probable maximum flood (PMF) event with sufficient freeboard for wave action in addition to the maximum normal operating water level of the facility or must demonstrate that the design does not reduce the ability to store or otherwise manage the original facility design storm or flood events.

1.4 COORDINATE SYSTEM

The design of the YDTI references the site coordinate system known as the ‘Anaconda Mine Grid’ established by The Anaconda Company (TAC) in 1957. The Anaconda Mine Grid is based on the Anaconda Copper Company (ACC) Datum established in 1915. All elevations are stated in Anaconda Mine Grid coordinates with respect to the ACC Vertical Datum unless specifically indicated otherwise. The Montana Resources GPS Site Coordinate System is based on the ‘Anaconda Mine Grid’ and utilizes International Feet.

1.5 LEGACY CROSS SECTION CONVENTION

The stability assessment references a series of legacy cross section locations that have been historically used for the project. The cross sections most likely align with a historical setting out line for the embankments that is no longer consistent with the current design; however the legacy cross sections are used for the design in the interest of consistency for as-built drawings and annual reporting. The convention begins with Station 0+00 at the interface between the North-South and East-West Embankments, and increases in station in both directions. The stationing convention uses a directional suffix (e.g. N or W) to describe the location of the cross section (e.g. 8+00 N for 800 ft along the North-South Embankment). The actual stationing measured along the current setting out line (SOL) will not be equal to the stationing as referenced on the cross section due to differences in the historical and current setting out lines.

2 – SITE CONDITIONS

2.1 GENERAL

A comprehensive summary of the site characteristics is provided in the Site Characterization Report (KP, 2017b). A summary of the information relevant to this report is provided in the following sections.

2.2 GEOLOGIC SETTING

The YDTI lies in the northern end of the upper Silver Bow Creek Basin, at the confluence of Yankee Doodle Creek and Silver Bow Creek. The basin, 3.5 miles wide and 7 miles long, includes the North-South trending Continental Fault zone, which lies along the East Ridge (Figure 1.1). The YDTI is bordered on all sides by mountainous terrain except the south.

The basin has a thin soil mantle and is underlain in places by thick alluvial material derived from weathering and erosion of the surrounding mountains. The valley slopes are formed by granitic bedrock covered with a layer of residual soils up to 10 ft thick.

The basin and surrounding mountains lie entirely within the intrusive rock for the Boulder Batholith and have been subjected to intense faulting. Butte Quartz Monzonite (BQM) is the dominant rock type. The bedrock consists of two distinct subunits: an upper weathered or leached zone, grading into less weathered bedrock with depth, overlying the competent zone.

Fault structures are found near the eastern boundary in the vicinity of the project area (Figure 1.1). The most critical structures identified in the seismic hazard assessment are the North-South trending Continental Fault and Northeast-Southwest trending Rampart Fault (Al Atik and Gregor, 2016). Both structures intersect the YDTI and underlie the North-South Embankment and potentially also the East-West Embankment. Other prominent faults nearby are the North-South trending Klepper and East Ridge Faults, located east of Rampart Mountain.

2.3 SEISMICITY AND DESIGN EARTHQUAKES

Site-specific probabilistic and deterministic seismic hazard analyses were performed for the YDTI, which lies within a region characterized by late-Quaternary Basin and Range normal faulting as well as historical seismicity. Two fault sources are located in close proximity to the site: the Continental Fault, which intersects the site (Figure 1.1), and the Rocker Fault, which is located more than 8.5 km southwest of the site. At the probabilistic 10,000-year return period, the Continental Fault was found to be the significant contributor. The median and 84^{th}-percentile response spectra for the deterministic analysis for the Continental Fault were significantly larger than for the Rocker Fault. The deterministic MCE spectra exceeded those for the probabilistically-derived 1-in-10,000-year event. As a result, the MCE selected as the design earthquake entails a magnitude 6.5 event with a rupture distance of 0.1 km. Table 2.1 summarizes the results of the analyses. The detailed assessment was prepared by Al Atik and Gregor (2016), and is included in Appendix B of the Site Characterization Report (KP, 2017b).

Table 2.1 Seismic Design Parameters

Condition	Design Earthquake	PGA (g)	Moment Magnitude
Maximum Normal Operating	MCE with Rrup of 0.1 km Median	0.45	6.5
Long-term Closure	MCE with Rrup of 0.1 km 84th Percentile	0.84	6.5

NOTES:
1. PEAK GROUND ACCELERATIONS ARE FOR ROCK SITE CONDITIONS (Vs30 – 760 m/s).

Appropriate earthquake (horizontal acceleration) time-history records have been selected as input ground motions for the seismic response analysis. Earthquake records representative of the design events were selected to the extent possible by Al Atik and Gregor (2016). Five representative earthquake time histories were provided for each earthquake event with characteristics as shown in Table 2.2. The scaled and baseline corrected earthquake time-history records were derived for bedrock with a shear wave velocity of 760 m/s.

Table 2.2 Parameters of Input Time Series for MCE Design Events

Earthquake	Station	Moment Magnitude	Mechanism	Rupture Distance (km)
Helena, Montana-01	Carroll College	6.0	Strike-slip	2.86
San Fernando	Pacoima Dam	6.6	Reverse	1.81
Imperial Valley-06	El Centro Array #5	6.5	Strike-slip	3.95
Niigata, Japan	NIG020	6.6	Reverse	8.47
L’Aquila, Italy	L’Aquila - Parking	6.3	Normal	5.38

The influence of nearby fault structures and the interaction between the Rocker and Continental Faults were also examined. The probability of rupture propagation across both the Continental and Rocker Faults was found to be very low (Al Atik and Gregor, 2016). The deterministic spectra for the median and 84th-percentile scenarios on the Continental Fault are larger than those for the other three faults (Rampart, Klepper, and East Ridge).

The median and 84th-percentile values for the average fault displacement on the Continental Fault were estimated to be 1.7 ft and 4.7 ft, respectively.

2.4 EMBANKMENT DESIGN SUMMARY

The YDTI will continue to be constructed with similar techniques and methods that have been employed in past raises. Embankment construction will be completed in staged lifts as a continuous activity when rockfill becomes available. Several of the embankment construction techniques that are good practices for the YDTI are as follows:

- The front slope of the embankment was designed with an overall slope angle much flatter than 37 degrees to generate a lighter loaded zone along the toe that was destined to constitute a buttress. This lighter loaded zone, referred to as the downstream dike, was constructed in the early 1960s and has not subsequently been altered in any substantial way. The lower stresses in the downstream dike will be less influenced by stress-induced degradation effects and was not an active leach area.

- The embankment is built up in layers instead of dumping at full height. The different lifts, with local angle-of-repose slopes of around 37 degrees, include intermediate benches and occasional larger benches.
- The dumping sequence, particularly in the central pedestal area, has resulted in a heterogeneous dump fabric that provides greater resistance to possible downstream slip surfaces exiting the toe of the embankment (Applied Geologic Services, 2017).

The rate of storage depends on the tailings production, variability of the tailings density, final beach slopes, and the supernatant pond area and volume. Tailings will be deposited at multiple locations during continued use of the facility to develop extensive drained tailings beaches adjacent to the embankments. The filling of the YDTI will be monitored throughout operations and construction sequencing will be evaluated periodically to confirm agreement with the design assumptions.

The North-South Embankment will be constructed using the downstream method. A typical section of the layout criteria is illustrated in Figure 2.1. The structural portion of the embankment is comprised of a crest width of 230 ft, measured perpendicular from the SOL towards the impoundment. The upstream slope will be at an angle of repose (1.3H:1V) and the downstream slope will be 2H:1V or flatter. The zone of embankment fill generated by these layout criteria comprises the structural portion of the embankment. A rock disposal site will be progressively developed over the existing leach pad areas located at the downstream toe of the North-South Embankment. The buttressing effects provided by the rock disposal site have been conservatively ignored in the stability assessment.

The East-West Embankment will be constructed using the centerline method (Figure 2.1). The structural portion consists of a 230-ft crest width, measured perpendicular from the SOL, with an angle-of-repose (1.3H:1V) upstream slope and a downstream slope of 2H:1V or flatter (or 2.5H:1V in some areas). For the non-structural portion of the embankment, additional rockfill will be placed upstream to provide a surcharge to the tailings mass adjacent to the embankment.

The West Embankment is new construction and will be built in stages using the downstream method. The upstream slope will be at an angle of repose (1.3H:1V) and the downstream slope will be 3H:1V or flatter. The minimum embankment crest width will be 230 ft measured perpendicular from the downstream edge of the crest towards the impoundment at each stage. A typical section is illustrated in Figure 2.1. The typical section exceeds the minimum layout criteria described above at an embankment crest elevation of 6,450 ft.

The embankment raises will be comprised of the following rockfill zones:

- Zone U rockfill is intended to promote free-draining behavior. The material will be hauled from the Continental Pit and end-dumped by 240-ton trucks. Segregation is expected to occur as finer-grained materials tend to accumulate near the top of the lifts while cobbles and boulders roll down the slope and accumulate at the toe.
- Zone F embankment earthfill will be placed to construct a separation zone between the tailings and the Zone U rockfill along the upstream face of the embankment. Zone F material will consist of variable alluvium, consistent with current practice, to limit tailings migration into the rockfill.
- Zone D1 rockfill will be used to construct the downstream zone of the West Embankment. Its design function is to act as an impediment to potential horizontal migration of perched seepage towards the downstream face of the embankment and to encourage free draining behavior in Zone U such that seepage flows are ultimately collected in the West Embankment Drain.

- Zone D2 embankment earthfill will be placed to provide a capping layer on the downstream slope of the embankment to promote runoff of meteoric water. Zone D2 material will consist of non-acid generating alluvium.

2.4.1 Storm Storage Freeboard

The YDTI relies on storm storage capacity to manage the Inflow Design Flood (IDF) during operations. The IDF for the design will be the PMF. The PMF is theoretically the largest flood resulting from a combination of the most severe meteorological and hydrologic conditions that could conceivably occur in a given area. The intent of adopting the PMF as the IDF for determining storm storage freeboard is to provide a design storm volume that is so great that it will not be exceeded, but not so great as to require excessive contingency storage capacity.

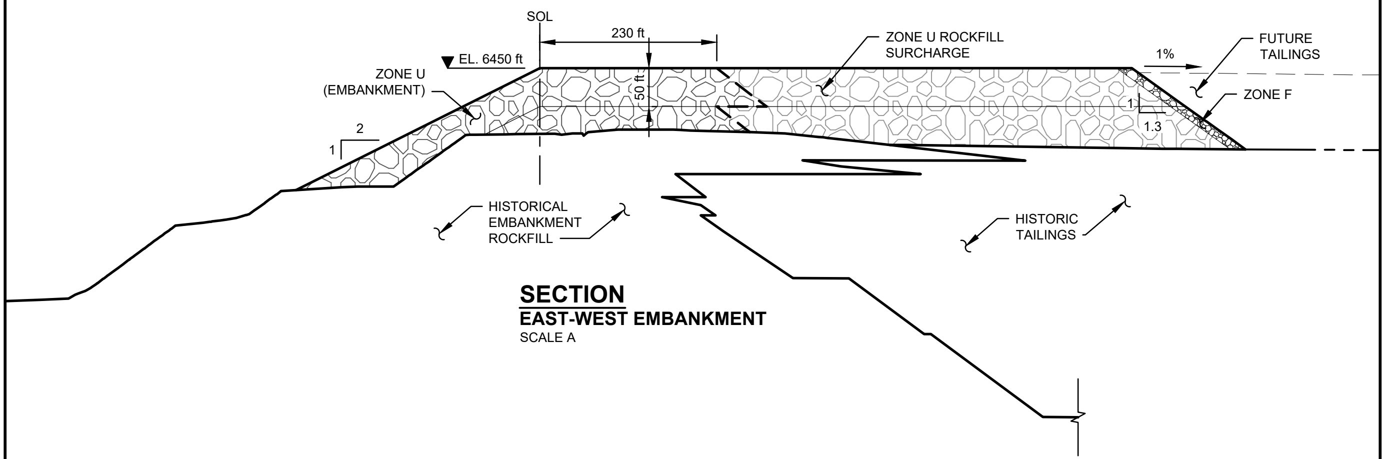
The selected design storm event is a combination of the 24 hour probable maximum precipitation combined with complete melt of the 1 in 100 year snowpack, and assuming full failure of the upstream Moulton Reservoirs. The PMF runoff volume was determined to be 19,000 acre-ft. Storage of the PMF runoff volume requires approximately 20 ft of storm storage freeboard above the supernatant pond elevation.

2.4.2 Minimum Freeboard

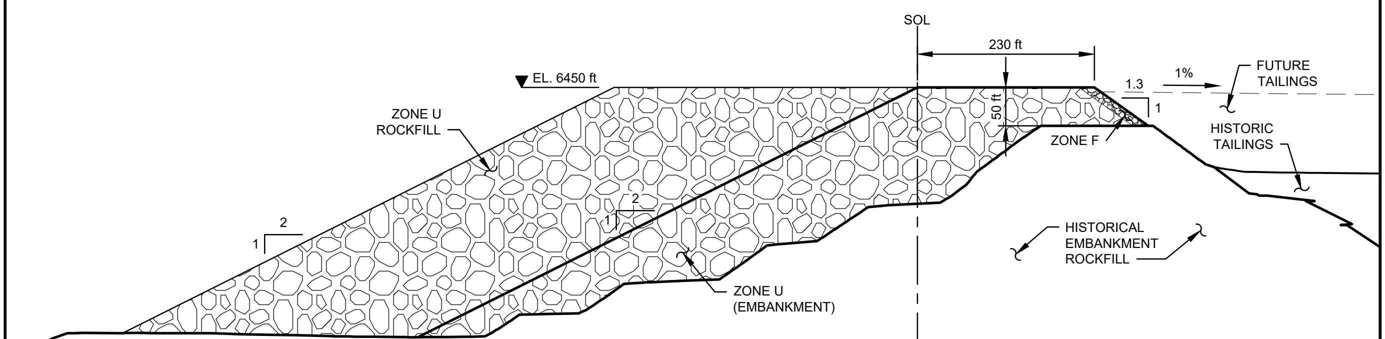
A minimum freeboard requirement of 5 ft will be incorporated in the YDTI design for wave run-up above and beyond the storm storage freeboard. The 5 ft of minimum freeboard is included in the design by limiting the tailings discharge elevation to an elevation that is 5 ft below the embankment crest at all times.

The minimum freeboard creates additional capacity in excess of 6,500 acre-ft. Embankment construction will be completed in staged lifts, and therefore the total actual freeboard will tend to be larger than the design freeboard until just before operations cease.

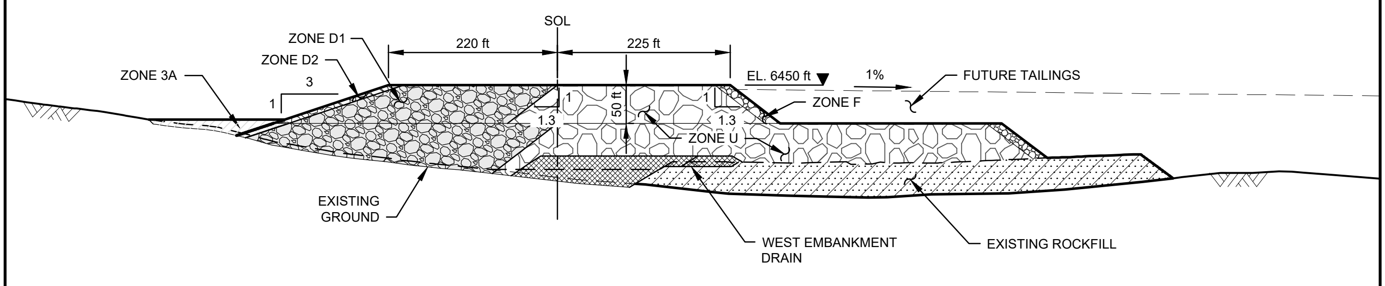
The total design freeboard is comprised of storm storage freeboard and additional minimum freeboard for wave run-up. The tailings beach and supernatant pond during maximum normal operating conditions will be below the embankment crest by at least 5 ft and 22 ft, respectively. These freeboard requirements are considered further in the definition of potential failure modes in Section 4.



SECTION
EAST-WEST EMBANKMENT
SCALE A



SECTION
NORTH-SOUTH EMBANKMENT
SCALE A



SECTION
WEST EMBANKMENT
SCALE A

LEGEND:

- ZONE F - EARTHFILL
- ZONE U - ROCKFILL SURCHARGE
- ZONE U - ROCKFILL
- ZONE U - ROCKFILL (EMBANKMENT)
- ZONE 3A - ROCKFILL 2
- ZONE D1 - ROCKFILL
- ZONE D2 - ROCKFILL
- DRAIN POD / EXTRACTION BASIN / WEST EMBANKMENT DRAIN
- SECONDARY SEEPAGE COLLECTION DRAIN
- EXISTING ROCKFILL
- SOL SETTING OUT LINE

NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON MINE GRID.

125 0 125 250 375 500 625 ft
SCALE A

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

TYPICAL EMBANKMENT SECTIONS

Knight Piésold CONSULTING

P/A NO. VA101-126/12 | REF NO. 2

FIGURE 2.1 | REV 2

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
2	12MAR'18	ISSUED WITH REPORT, LEGEND UPDATED	DDF	MJC/WAL	KJB

3 – TAILINGS LIQUEFACTION ASSESSMENT

3.1 TAILINGS STATE CHARACTERIZATION

3.1.1 Methodology

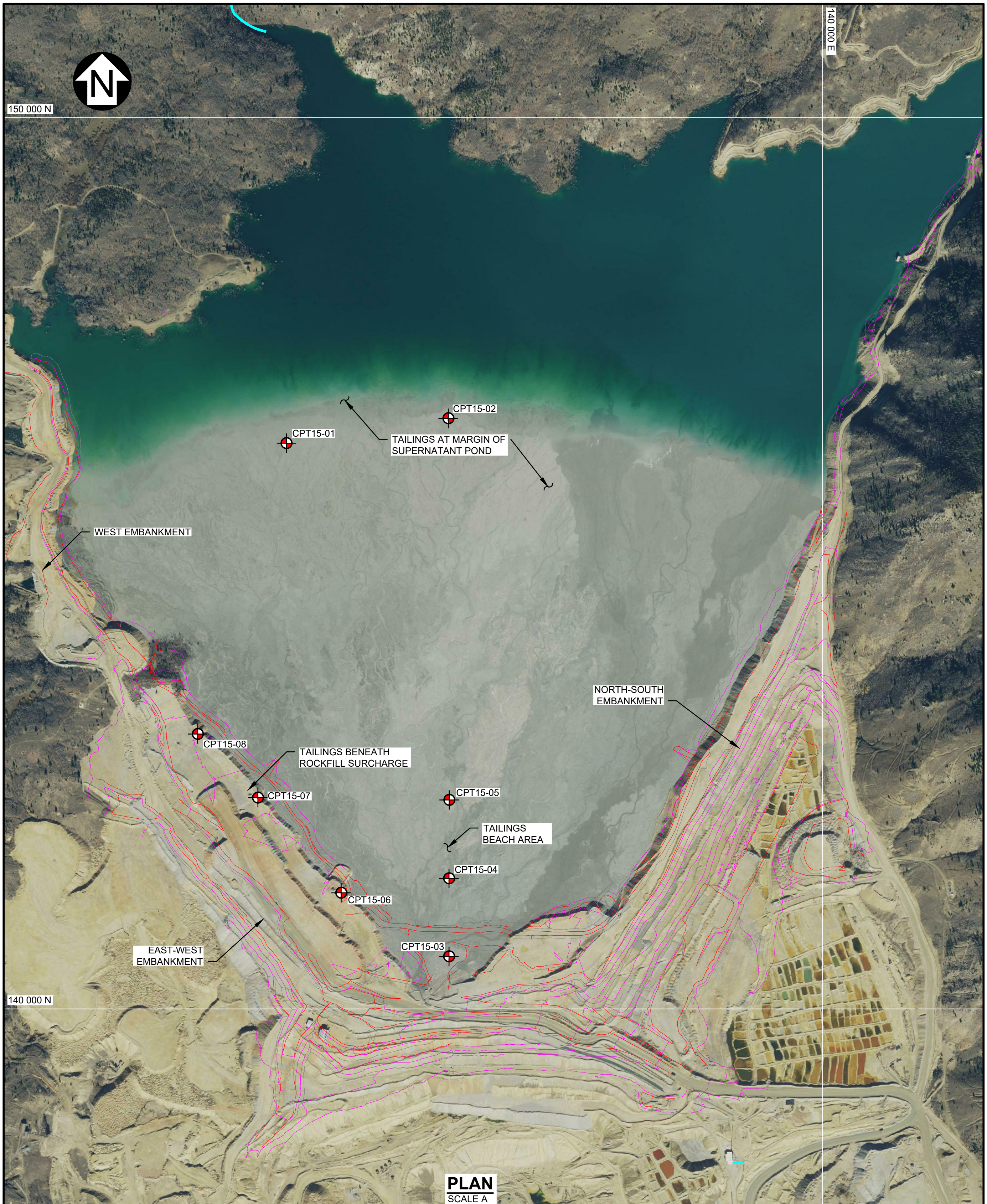
Tailings state characterization was completed to evaluate liquefaction susceptibility of tailings materials within YDTI. The preliminary analysis relies on data collected during piezocone penetration testing (CPTu) to classify tailings material as either potentially dilative or potentially contractive based on CPTu-based soil behavior and excess pore pressure conditions measured during each CPT sounding. Analyses of state parameter and normalized pore pressure difference were completed to highlight locations where potentially contractive soil behavior and excess pore pressure conditions are indicative of potentially liquefiable tailings materials within the YDTI. The analyses were completed for the following three regions of the YDTI, as shown on Figure 3.1, using data from eight CPT soundings completed during 2015 (listed in parentheses):

- Tailings beach (CPT15-03, CPT15-04 and CPT15-05)
- Tailings beneath rockfill surcharge (CPT15-06, CPT15-07 and CPT15-08), and
- Tailings at the southern margin of the supernatant pond (CPT15- 01 and CPT15-02).

The state parameter can be used to differentiate between dense soils that behave in a dilative manner and have a low potential of liquefaction, and loose saturated soils that behave in a contractive manner and are more susceptible to liquefaction. The state parameter is the difference between the current void ratio of the tailings and the critical state void ratio. Positive values of the state parameter indicate that the tailings are loose of critical state and have potential to contract (behave in an undrained manner). Negative state parameters indicate that the tailings are denser than critical state and are likely to dilate. Dilative soils are strain hardening (i.e. they get stronger with strain) and are thus not considered liquefiable.

A state parameter (y) analysis was completed to delineate regions where potentially contractive tailings material types exist. In general, negative state parameters (y <0) are indicative of potentially dilative material types and positive state parameters (y >0) of potentially contractive material types (Winckler et. al., 2014). Jefferies and Been (2006) proposed y >-0.05 be used as division between potentially contractive and dilative materials specifically for tailings liquefaction analyses, rather than y >0. State parameter profile plots were used to graphically identify potentially contractive tailings materials by comparison of the calculated state parameter values with both the y > 0 and y > -0.05 criteria.

The state parameter assessment was supplemented with a pore pressure-based analysis to highlight regions where excess pore pressures are present along with potentially contractive material types. Normalized pore pressure difference (u) profiles were developed for each sounding by calculating the difference between measured dynamic and static pore pressures and dividing by the effective vertical stress. Materials exhibiting a positive u (dynamic pressures exceeding static pressures) are potentially contractive while those with a negative u (static pressures exceeding dynamic pressures) indicate potentially dilative material behavior (Winckler et. al, 2014). Tailings materials with a positive state parameter and positive normalized pore pressure difference were highlighted as regions with higher susceptibility to liquefaction.



PLAN
SCALE A

NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. IMAGERY FROM 2015 AIR PHOTO PROVIDED BY MONTANA RESOURCES.
3. EMBANKMENT TOPOGRAPHY 2017 PROVIDED BY MONTANA RESOURCES ON MARCH 19, 2017.

SCALE A 64 0 64 128 192 256 320 ft

LEGEND:

DRILLHOLE AND CPTS - KNIGHT PIESOLD 2015

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YANKEE DOODLE TAILINGS IMPOUNDMENT

TAILINGS STATE CHARACTERIZATION REGIONS AND 2015 CPT SOUNDINGS

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA101-126/12	2

FIGURE 3.1 REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	14AUG'17	ISSUED WITH REPORT	DDF	PP	KJB

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XREF FILE(S): IMAGE FILE(S): 2015_airphoto

3.1.2 Discussion of Results

Results from the tailings state characterization analyses are shown on the figures in Appendix A1. CPTs within the tailings beach generally have state parameters on the boundary of potentially dilative and contractive behavior depending on whether the y< -0.05 or y< 0 is used. State parameter plots suggest that potentially contractive material types exist in CPT15-03 and CPT15-04 between EL. 6,275 and 6,200 ft; however, tailings at these elevations do not exhibit excess pore pressures. Predominantly negative normalized pore pressure differences (u) are present throughout the tailings profile suggesting that although the materials are on the boundary of potentially contractive or dilative based on state parameter, the excess pore pressures required for potentially contractive behavior are generally not present.

Tailings beneath the rockfill surcharge show the lowest potential susceptibility to liquefaction of the eight CPT locations. Similar to the tailings beach soundings, state parameters beneath the surcharge predominantly range from y < -0.05 and y < 0 and are near the boundary between dilative/contractive behavior depending on which state parameter criterion axis is used. A significant portion of the data satisfy the y < -0.05 criterion. CPT15-06 and CPT15-07 show some potentially contractive materials near the bottom of the soundings (EL. 6,225 – 6,150 ft) based on both state parameter and normalized pore pressure difference analysis. In general, normalized pore pressure differences become increasingly positive with increasing distance from the tailings discharge as materials become finer. The combined analyses show that although the materials are near to the boundary of potentially contractive or dilative based on state parameter; the excess pore pressures required for potentially contractive behavior are generally not present.

Both the normalized pore pressure difference and state parameter analyses indicate that tailings along the southern margin of the supernatant pond are potentially contractive. The combined analyses indicate the existence of tailings materials with both a state parameter of y>-0.05 and excess pore pressures present. This indicates potentially contractive materials and therefore potential for liquefaction during shearing.

3.1.3 Summary

Drainage of the tailings in much of the facility has resulted in tailings densities that are higher than would otherwise be achieved under hydrostatic effective stresses. Dense tailings are typically less susceptible to liquefaction and loss of strength than loose tailings. Based on the normalized pore pressure difference, state parameter and the combined analyses the following general conclusions are made:

- Tailings materials in the tailings beach and beneath the rockfill surcharge are predominantly near to the boundary between potentially contractive and potentially dilative behavior. The excess pore pressures required for potentially contractive behavior during shearing are generally not present under existing conditions.
- The rockfill surcharge tends to increase the potential for dilative behavior in the underlying tailings.

Tailings further north, along the southern margin of the supernatant pond, have consolidated over time under near hydrostatic conditions and exhibit potentially contractive behavior. The tailings in the vicinity of the supernatant pond therefore have a higher susceptibility to loss of strength during shearing.

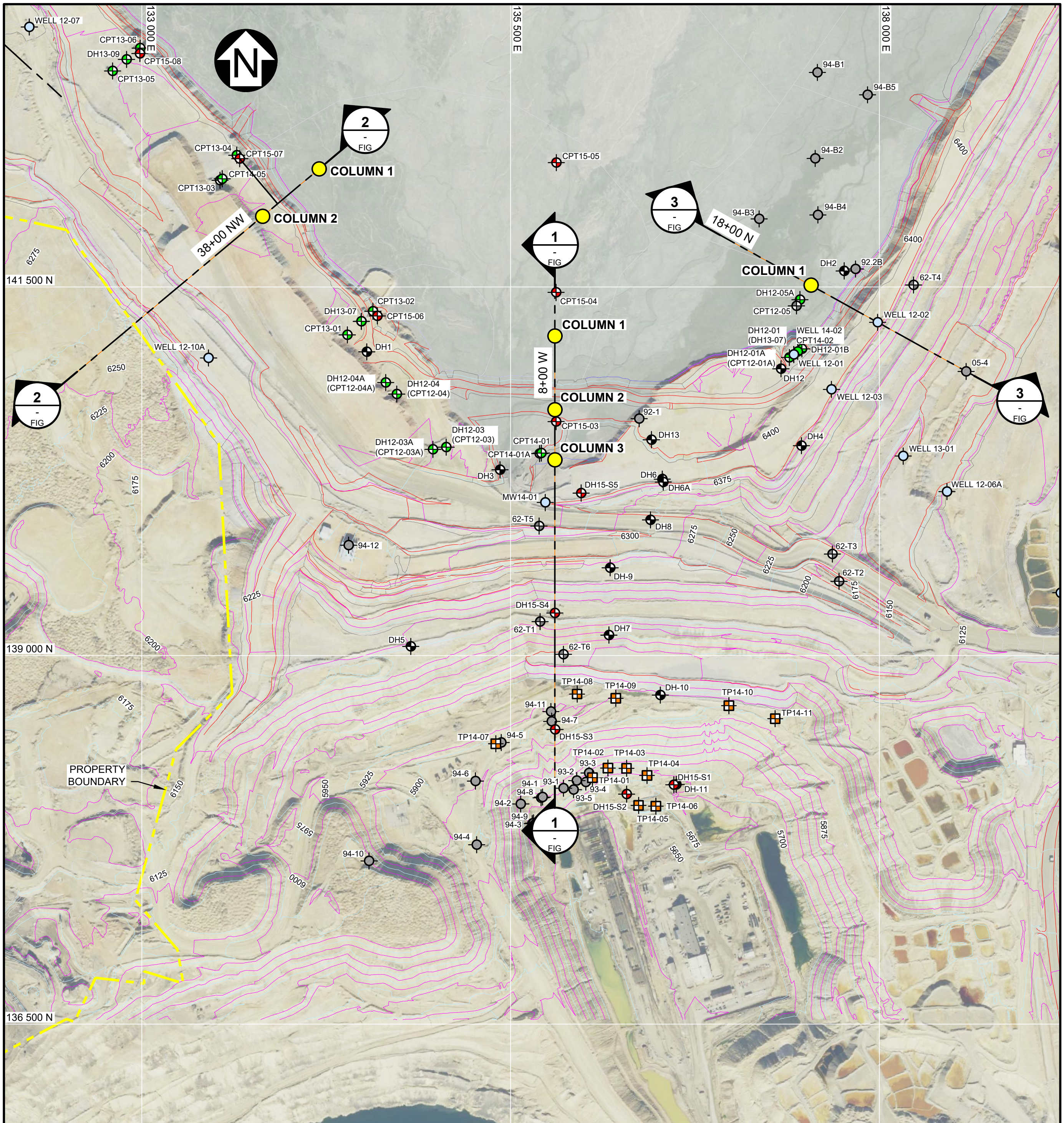
3.2 CYCLIC LIQUEFACTION ASSESSMENT

3.2.1 Methodology

An assessment was completed to determine the cyclic liquefaction potential of the tailings, as a result of the design earthquake loading, at the current (as of the CPT testing in November 2015) and the future embankment configuration up to EL. 6,450 ft. The liquefaction assessment was carried out by determining the predicted cyclic stress ratios (CSR) induced by the design earthquakes and comparing them to the estimated cyclic resistance ratios (CRR) of the tailings. Saturated tailings sand are predicted to liquefy and lose strength when the CSR exceeds the CRR.

The CSR induced by the median and 84^{th}-percentile earthquakes were predicted along six stratigraphic columns taken at three sections through the East-West (8+00W and 38+00W) and North-South (18+00N) Embankments. The location of each stratigraphic column is shown on Figure 3.2. Dynamic response analyses were carried out using SHAKE2000, which simulates the effect of one-dimensional seismic shear wave propagation from bedrock through the overlying materials. The input ground motions for the assessment were presented previously in Table 2.2. The maximum CSR value at any depth from each of the five earthquake motions was used to develop a conservative CSR profile that effectively represents an upper bound envelope for the purposes of the assessment.

The estimated CRR were determined from cone penetration test (CPT) data. The computer software CLiq (Geologismiki, 2015) was used to process the CPT penetration data to derive CRR profiles for the analysis using the methods of Idriss and Boulanger (2014) and Robertson (2009). The results from the CLiq analyses are included as Appendix A2 and A3, respectively. It is important to note that the CSR values calculated by CLiq were not used for the liquefaction assessment. The CRR was also derived from the shear wave velocity of the tailings using the method of Kayen et al. (2013). All three methods of deriving the CRR were compared to the CSR estimates and are presented on the figures in the liquefaction assessment included as Appendix A4. A design line with a CRR of 0.07 was selected based on the lowest CRR resulting from the various methods. The CRR design line functions as a conservative lower bound envelope for the assessment.



NOTES:

1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.
4. IMAGERY FROM 2015 AIR PHOTO PROVIDED BY MONTANA RESOURCES.
5. EMBANKMENT TOPOGRAPHY FROM MARCH 2017 PROVIDED BY MONTANA RESOURCES.

SCALE A 500 0 500 1000 1500 2000 2500 ft

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
ANALYZED CROSS SECTIONS

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA101-126/12	2

FIGURE 3.2 REV 1

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
1	14AUG'17	ISSUED WITH REPORT	DDF	PP	DAY

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XREF FILE(S): Topo 5 ft 2013, X-C-PROP-BNDY-2015, dam topo 201703 IMAGE FILE(S): 2015_NAIP_Silver_Bow

3.2.2 Summary of Results for Operations Period

The analysis compares an upper bound envelope curve for CSR with a lower bound envelope CRR design line. The potential for liquefaction of tailings is predicted during operations where the tailings are saturated and the CSR for the median MCE case exceeds the CRR.

The liquefaction assessment indicates that saturated tailings within the YDTI and without the rockfill surcharge are potentially liquefiable under the median MCE during the operations period. Column 1 in each of the sections is representative of this prediction both for current conditions and future embankment conditions. The potentially liquefiable tailings extend to approximately 200 ft depth in the central impoundment area; however, the top 50 to 80 ft or more of tailings beach adjacent to the embankment is unsaturated or partially saturated and is therefore not likely to liquefy.

Tailings below the surcharged areas have very low liquefaction potential for operating conditions with seismic loading from the median MCE. The analysis demonstrates that the rockfill surcharge effectively mitigates the potential for cyclic liquefaction of the surcharged tailings zone during operations. The potential for liquefaction is reduced due to the loading from the overlying rockfill mass. The top 50 ft or more of the tailings are also unsaturated or partially saturated and are not subject to liquefaction. The tailings discharge point has also been relocated further out into the impoundment due to the construction of the rockfill surcharge, which has reduced pore pressures in the tailings near the embankment and increased the depth of the unsaturated zone in the tailings below the surcharge.

3.2.3 Summary of Results for Closure Period

The potential for liquefaction of tailings is predicted during closure if the CSR for the 84th-Percentile MCE exceeds the CRR. The liquefaction assessment indicates that the upper saturated tailings within the YDTI and outside the rockfill surcharge area are potentially liquefiable to approximately 275 ft depth with seismic loading from the 84th-percentile MCE during the closure period. Localized liquefaction of the tailings underlying the rockfill surcharge may also while the tailings remain saturated. The potentially liquefiable tailings will be contained by non-liquefiable foundation soil, bedrock, and free draining embankment rockfill. The top 50 ft to 80 ft or more of tailings beach adjacent to the embankment and beneath the rockfill surcharge is currently unsaturated or partially saturated and is therefore not likely to liquefy. The tailings pore water will drain down and the thickness of this unsaturated zone will increase following closure, which progressively reduces the liquefaction potential of the tailings in the vicinity of the embankments.

Two dimensional limit equilibrium slope stability analysis was recommended to check the post-earthquake stability of the embankment assuming liquefaction will occur in the tailings at the identified locations. The earthquake-induced deformation potential has also been estimated. The results of the post-earthquake stability analyses and estimates of earthquake-induced deformation are described in subsequent sections of this report.

4 – DEFINITION OF POTENTIAL FAILURE MODES

4.1 GENERAL

The following sections provide a general description of the failure modes analyzed in this report that would have the potential to initiate a loss of containment leading to uncontrolled release of impounded mine water and tailings. This stability assessment considers the potential for the following failure modes:

- Foundation and embankment slope instability under static normal operating conditions and static post-earthquake operating conditions, and
- Earthquake induced embankment deformation leading to slope instability or loss of freeboard.

The failure mode description provides the necessary context to understand the conservatism in the definition of the failure modes, the stability model construction, and the material parameters used in the analyses.

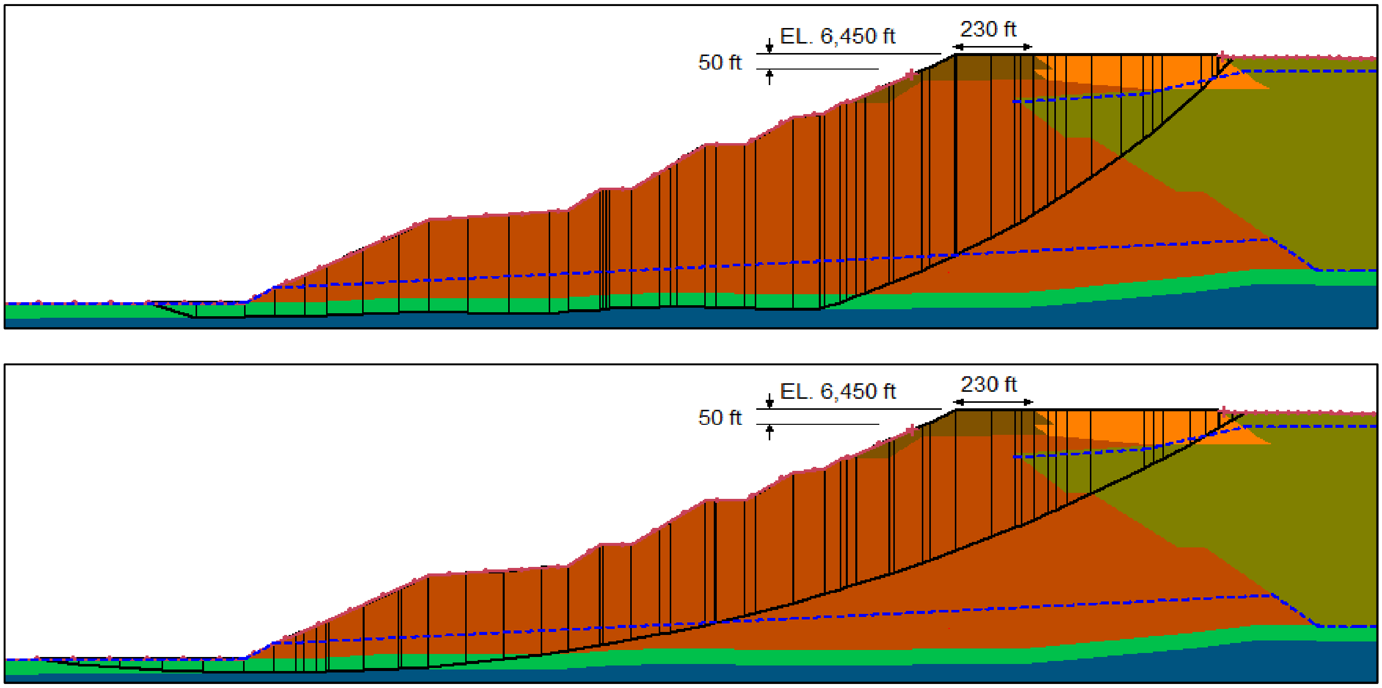
4.2 FOUNDATION AND EMBANKMENT SLOPE INSTABILITY

The static instability mode through the foundation and the embankment is examined by considering both downstream and upstream slip surfaces. Material strength properties are consistent with the case being analyzed. The piezometric lines are simplified and conservative representations of recently measured pore pressure conditions.

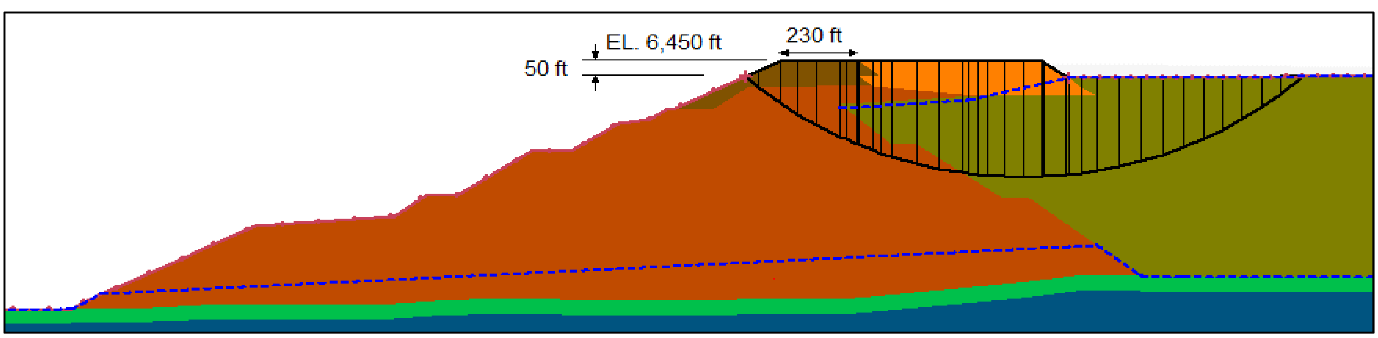
Downstream instability is assessed by considering scenarios involving slip surfaces that propagate through the embankment crest and intersect the tailings beach (including the rockfill surcharge for the East-West Embankment). The downstream scenarios consider maximum normal loading during operating conditions just prior to closure with the tailings surface adjacent to the embankment at 5 ft (minimum freeboard) below the crest. Field data suggests that the upper 50 ft to 80 ft or more of tailings is not saturated and has little flow potential since the supernatant pond is far from the embankments. A slip surface of this kind may not produce a loss of containment; however, it was conservatively assumed to constitute a potential failure mode.

Upstream instability is assessed with a single scenario for normal operating conditions. The modelled instability represents a combination of the maximum driving force against the minimum resisting force. The lifts of the embankments will be completed as a continuous activity when rockfill is available from mining operations. The upstream maximum driving force is represented by an instantaneous raise of a 50-ft embankment lift. The minimum resisting force is represented by the maximum elevation difference (50 ft) between the top of the tailings and the embankment crest. The slip surfaces intersect the downstream slope a minimum of 50 ft vertical distance below the downstream crest to simulate a residual embankment crest elevation that is below the tailings elevation considered in the analysis. This simulates a potential for loss of containment and uncontrolled release of the impounded material. The upstream instability is akin to bearing capacity failure, requiring displacement of embankment rockfill into the long tailings beach and tailings subsequently flowing over the displaced rockfill. The piezometric line is assumed at the top of the tailings, although field data suggests that the upper 50 ft or more is not saturated and has little or no flow potential.

TYPICAL CRITICAL DOWNSTREAM SLIP SURFACE SHAPE AND EXTENTS



TYPICAL CRITICAL UPSTREAM SLIP SURFACE SHAPE AND EXTENTS



LEGEND:

- STRUCTURAL EMBANKMENT ROCKFILL
- HISTORIC EMBANKMENT ROCKFILL
- ROCKFILL SURCHARGE
- TAILINGS
- OVERBURDEN
- BEDROCK

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

STATIC SLOPE INSTABILITY MODES

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P/A NO. VA101-126/12

REF. NO. 2

FIGURE 4.1

REV 1

1	14AUG'17	ISSUED WITH REPORT	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	RVW'D

4.3 EARTHQUAKE INDUCED EMBANKMENT DEFORMATION

The dynamic instability mode considers earthquake induced embankment deformation leading to slope instability or a loss of freeboard. An earthquake can induce ground displacement, deformations and settlement of the embankment crest, and movement along a hypothetical slip surface within the embankment. The deformation analysis examines an extreme scenario that includes the combined displacements relating to the following assumptions:

- The embankment displaces downward relative to and independent of the tailings contained within the YDTI and supernatant pond due to displacement on the Continental Fault. The median and 84th-percentile values for the average displacement on the Continental Fault were estimated to be 1.7 ft and 4.7 ft, respectively, as described in Section 2.3.
- The embankment deforms and the crest settles due to the seismic loading of the embankment independent of the tailings contained in the YDTI. The crest settlement was determined using a mathematical formula, developed based on statistical analysis of empirical data, which relates Normalized Crest Settlement (NCS) to PGA and earthquake Magnitude (Swaisgood, 2014).
- Seismic loading induces slope displacement in the embankment relative to and independent of the tailings. The slope displacement and the probability of non-tolerable displacement was estimated using a simplified probabilistic procedure (Bray and Travasarou, 2007).

A combined displacement of greater than 5 ft (minimum freeboard) during the maximum normal operating conditions would potentially expose the tailings beach. A combined displacement greater than 22 ft (total freeboard) would be required to position the embankment crest below the elevation of the supernatant pond and theoretically create the geometric conditions necessary to allow for surface water or seepage to flow from the supernatant pond towards and beyond the embankment. The tailings in the vicinity of the embankment are not saturated in the top 50 to 80 ft, and therefore would not be prone to flow unless the displacement occurred in the vicinity of the active tailings discharge point. Therefore, it is conservatively assumed that the displacement would occur in the vicinity of the active tailings discharge area and that a combined displacement greater than 5 ft during operations constitutes a potential failure mode. Seismic loading for the operations case considers the median MCE.

A combined displacement greater than 50 ft would be required to position the crest of the embankment below the phreatic surface in the tailings immediately following closure. The supernatant pond elevation will reduce following closure as the pond area shrinks at the far end of the facility eliminating the potential for the supernatant pond to flow towards and beyond the embankment. The analysis considers piezometric conditions that are simplified and conservative representations of current measured pore pressure conditions. Active tailings deposition will have ceased and the tailings beach would be capped and reclaimed. The tailings pore water will drain down and the thickness of the unsaturated tailings zone adjacent to the embankments will increase following closure, which may prevent an uncontrolled release even if this large amount of displacement occurred. Seismic loading for the post-closure case considers the 84th percentile MCE.

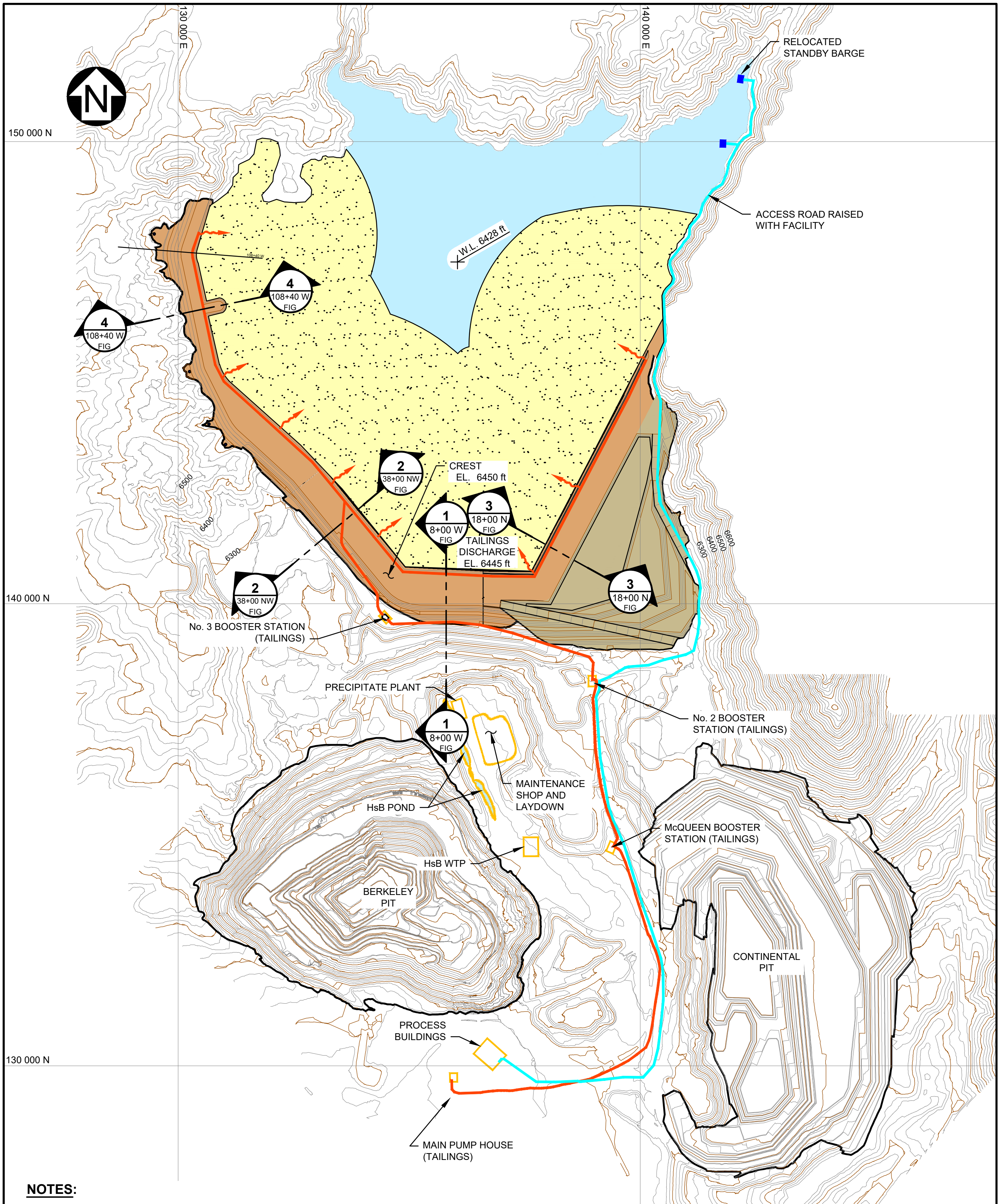
5 – EMBANKMENT SLOPE STABILITY ANALYSES

5.1 GENERAL

The stability analyses considered four representative cross-sections of the embankment (8+00 W, 38+00 NW, 18+00 N, 108+40 W) as shown on Figure 5.1. The first three sections coincide with the sections analyzed in the tailings liquefaction assessment as described previously. These sections were selected to be representative of the limbs of the continuous embankment based on the height of the embankment, historic tailings deposition limits, and foundation conditions. Section 8+00 W is representative of the maximum embankment section. The design drawings are included in Appendix D of the Design Basis Report (KP, 2017a), and include the geometry of all additional sections for comparison with the selected sections analyzed in the slope stability assessment.

The stability assessment was completed through a series of static limit equilibrium (LE) analyses (Morgenstern-Price method with SLOPE/W by GEO-SLOPE, 2014). The potential instability scenarios were selected to satisfy the legislative requirements (Section 1.3). Static loading conditions were examined for ascertaining factors of safety against hypothetical rotational slip surfaces. The critical slip surface was determined by using the ‘Entry and Exit’ method in SLOPE/W. This search routine analyzes several thousand potential slip surfaces and determines the surfaces with the lowest factors of safety.

Pseudostatic analyses were completed to determine the seismic yield acceleration required for estimating earthquake-induced slope displacements along the slip surface. The geometry of each section was used in the determination of earthquake-induced embankment crest settlement.



NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON MINE GRID.

LEGEND:

	TAILINGS BEACH		TAILINGS DEPOSITION
	EMBANKMENT FILL		TAILINGS PIPELINE
	ROCK DISPOSAL SITE		RECLAIM PIPELINE
	MINE WATER		RECLAIM BARGE

SCALE A 1500 0 1500 3000 4500 6000 7500 ft

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	14AUG'17	ISSUED WITH REPORT	DDF	MJC	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LAYOUT OF SLOPE STABILITY SECTIONS EMBANKMENT EL. 6450 (FULL)

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA101-126/12	2

FIGURE 5.1 REV 0

5.2 MATERIAL PROPERTIES

5.2.1 Summary

The material properties assigned in the stability models were determined by evaluating the site investigation work and laboratory testing that was completed between 1962 and 2015. Six material units have been defined with properties as summarized in Table 5.1.

Table 5.1 Slope Stability Material Properties

Material	γ (pcf) [1]	Base Case Analyses		Sensitivity Analyses	
		Drained		Undrained	Liquefied
		c' (psf)	ϕ' (°)	s_u/σ'_v	s_u/σ'_v
Bedrock	165	Impenetrable			
Overburden	135	0	27	0.23	N/A
Historic Rockfill	140	0	Leps Angular Sand	0.27 / 0.37	N/A
Zone U Rockfill	140	0	Leps Lower Boundary	N/A	N/A
Zone D1 Rockfill	145	0		N/A	N/A
Tailings Sand	120	0	32	0.20	0.05

NOTES:
1. UNIT WEIGHT, γ, IS THE TOTAL SATURATED UNIT WEIGHT FOR EACH MATERIAL.

The modelled foundation materials consist of an overburden layer and the underlying bedrock. The bedrock can be conceptually sub-divided into two sub-units: an upper weathered horizon overlying the lower competent BQM. The overburden unit in the slope stability models conceptually represents the alluvial and colluvial deposits as well as the completely and highly weathered bedrock sub-units.

The embankment materials have been separated into zones that depend on the source and relative age of the rockfill. The oldest two zones are historic rockfill with the bottommost from the Berkeley Pit underlying the topmost historic rockfill from the Continental Pit. Due to the age and design of the facility, the lower unit was expected to be more degraded by weathering and leaching relative to the overlying unit. However, embankment drilling investigations have provided little evidence of a distinguishable contact or a difference in material strength properties between these two units. The tailings beach in the vicinity of the embankment sections analyzed is considered as a single unit due to the extensive beaches and relatively consistent nature of the tailings material throughout the impoundment.

The following sections described the selection of material properties for each unit. Additional detailed information on the historic and recent geotechnical drilling and testing investigating these units is found in Site Characterization Report (KP, 2017b).

5.2.2 Bedrock

The YDTI valley slopes are formed by BQM bedrock, which is part of the Boulder Batholith. Bedrock outcrops rise above the general topography as scattered rounded boulders that are the result of spheroidal weathering of fractured quartz monzonite.

The BQM in the west ridge area was observed during the 2015 site investigation work and shows a typical meteoric weathering profile that grades from completely weathered (residual soil) to highly weathered to moderately weathered. The profile of weathering within the bedrock can be distinguished based on characteristics of rock fracture spacing, intactness, and discolouration observed in the test pits and drill core.

Completely weathered bedrock is decomposed to soil with little original rock texture or fabric preserved. Highly weathered bedrock is the middle weathered horizon where the rock is discoloured predominantly along joint faces and more than half of the rock material is decomposed. Highly weathered bedrock is weak and requires moderate force to crumble. The thickness of the horizon varied in the test pits and drillholes from 0 ft (not present) to 50 ft thick.

Moderately weathered bedrock is the bottom weathered horizon where less than half the rock is decomposed or disintegrated and the original rock texture or fabric is preserved. Fresh rock fragments are typically present as blocks or boulders that fit together. The excavator bucket generally refused to advance on this horizon during the test pit program. Test pits located on the topographic high points encountered shallow moderately weathered bedrock and little weathering profile. Slightly weathered to fresh BQM bedrock is classified as competent bedrock with little to no weakness or weathering noted. Competent BQM is described as dark grey and medium to coarse grained. Laboratory testing of competent BQM bedrock indicates a wet density typically ranging from 163 to 169 pcf (KP, 2017b).

Historically, Mohr-Coulomb properties were applied to the bedrock unit for the stability assessment and were taken from IECO (1981) and MR (1999). A high friction angle (45 degrees) and high cohesion value (10,000 psf) consistent with the strength of massive BQM were applied to bedrock, which had the effect of limiting critical slip surfaces to the embankment and overburden. A similar effect is achieved by applying the impenetrable bedrock material model in Slope/W.

5.2.3 Overburden

The surficial materials in the vicinity of the YDTI have historically been referred to generally as alluvium. A description of the nature, variability and condition of these surficial materials prior to substantial development of the YDTI was provided by Dames and Moore (1963). Recent stream deposits within developed channels were filled with dark brown well-sorted and moderately-loose sands and gravels with occasional isolated lenses of silt. The stream deposits were approximately 800 ft wide and up to 45 ft deep (Dames and Moore, 1963). A broad band of alluvial and outwash material is located directly east of the historic Silver Bow Creek channel and forms a moderately-sloping plain adjacent to the East Ridge.

Direct shear tests were performed on undisturbed samples of natural soils by Dames and Moore (Dames and Moore, 1962 & 1963). Samples were typically tested at two surcharge pressures, and the results were plotted to illustrate the relationship between normal pressure and shearing strength.

The results showed a wide range of variability, but the material was generally assumed to be non-cohesive with angles of internal friction in the range of 16 to 42 degrees (MR, 1999).

Golder Associates (Golder, 1980) conducted a site investigation including three drillholes on the alluvium deposits south of the Berkeley Pit and completed laboratory test work on select samples to determine geotechnical strength characteristics of the alluvial material for pit wall stability evaluations. The triaxial testing by Golder indicated that under consolidated drained testing conditions the angle of internal friction for alluvium recovered in the drilling ranged from approximately 25 to 35 degrees. The statistical variation in the laboratory test results indicated a mean friction angle of 30.3 degrees with a standard deviation of 1.3 degrees. The original purpose of the investigation was to provide shear strength characteristics for the alluvium for the evaluation of pit wall stability for a proposed expansion of the Berkeley Pit. The embankment stability analyses since 1980 (by IECO, HLA, and MR) have adopted a friction angle of 34 to 35 degrees for alluvium, using a value at the higher end of the range reported by Golder.

The recent stream deposits described by Dames and Moore generally only underlie the existing maximum section of the East-West Embankment, which is represented in the stability analyses by Section 8+00 W. The outwash materials generally underlie the existing North-South Embankment. KP encountered the alluvium below the embankment fill during drilling on Section 8+00 W at depths down hole of between 288 ft and 717 ft (between EL. 5,623 ft and 5,656 ft). The alluvium was generally a dark brown to medium grey, dense to very dense, silty sand with gravel. The material grades through approximately 10 ft of alluvium into completely weathered bedrock that is very similar to the overlying alluvium, and then terminates in more competent grey BQM. The grain size distribution of the alluvium is approximately 2% gravel, 80% sand, 15% silt, and 3% clay.

Two sets of consolidated undrained (CU) triaxial tests were completed on overburden samples remolded at dry densities between 110 and 122 pcf (targeting a saturated density of approximately 135 pcf). The triaxial tests were completed at confining stresses ranging between 50 and 400 psi (344 and 2761 kPa) and indicate an effective friction angle (φ') between 27 and 29 degrees. The CU triaxial testing results also indicate the potential for contractive behavior if material is exposed to undrained conditions and rapid shearing. The laboratory tests indicate an undrained strength ratio (S_u/σ'_v) with a range between 0.33 to 0.50 for the two sets of isotopically consolidated undrained compression (CIUC) tests.

The stability assessment adopts a drained response and base case friction angle (φ') of 27 degrees for overburden (with cohesion conservatively ignored), based on the lower bound of this recent CU triaxial testing. The analysis using a drained response for future loading conditions is reasonable and appropriate given the long history of loading and anticipated rate of future construction. Undrained conditions would only be relevant under static normal operating conditions in the event of rapid construction causing shear induced pore pressures within the foundation. No new construction over saturated overburden is planned. The surficial materials in the vicinity of the West Embankment have been stripped from the foundation and stockpiled for reclamation purposes.

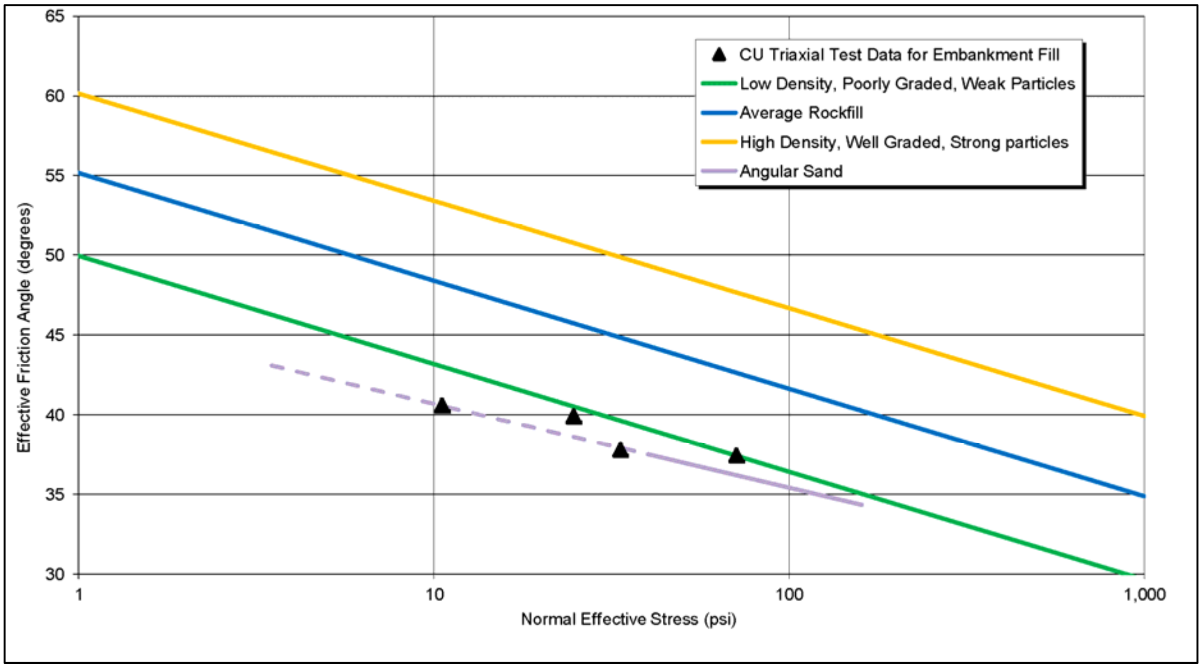
5.2.4 Rockfill

The shear strength characterization of the rockfill for engineering stability analyses historically adopted a uniform drained friction angle between 35 and 38 degrees. The friction angle used in the analysis was supported by direct shear testing (Dames and Moore, 1962), triaxial shear testing (IECO, 1981), and field observations of the constructed embankments (MR, 1999).

Rockfill strength parameters have been assessed for this stability assessment based on research by Thomas Leps (Leps, 1970). Leps compiled and analyzed published data for individual large scale triaxial tests on gravels and rockfill, and for comparison included his own research data on the shearing strength of sands. Leps developed a series of relationships from his analysis to show friction angle as a function of normal pressure. The Leps non-linear shear strength functions recognize that rockfill (and sand) can maintain a higher angle of friction at a lower confining pressure and lower friction angle at higher pressures. Several non-linear shear strength functions resulted from the analysis by Leps and the general trends in the data. Leps established a strength function for *Average Rockfill*, which represents his judgement of the median strength of all the tests analyzed. Relationships were provided for an *Upper Boundary* for well graded, high density rockfill with strong particles and a *Lower Boundary* for poorly graded, low density rockfill with weak particles. An *Angular Sand* function was also determined based on his research data.

A suite of single stage CU triaxial compression tests were carried out in 2014 to investigated the shear strength of the YDTI rockfill over a range of confining stresses. A composite surface sample from test pitting was prepared with particles greater than 1-inch replaced with finer gravel to facilitate standard proctor and triaxial shear testing in a 6-inch cell. The sample specimen selected for testing specifically targeted an area of poor quality embankment fill near the base of the East-West Embankment. The rockfill material selected was well-graded sand and gravel with many cobbles and boulders, some silt, and trace clay. The fine fraction of the embankment fill was typically slightly to medium plastic, which is likely the result of alteration due to exposure to acidic drainage. A standard proctor test indicated a maximum dry density of 136 pcf at 8% moisture content. The samples were then prepared to a dry density of 123 pcf (targeting a saturated density of approximately 140 pcf) for the triaxial testing based on in situ densities reported by MR (MR, 1999), which corresponded to approximately 90% standard proctor maximum dry density. The tests were performed at effective confining stresses of between 14 and 116 psi (100 and 800 kPa) covering the range of stresses encountered at the toe of the embankment. The test results indicate an average effective friction angle of 37 degrees with zero cohesion (KP, 2016a).

The strength characteristics of the embankment fill over the stress range have been examined and compared to published information provided by Leps (1970) on Figure 5.2. A trend of decreasing strength (effective friction angle) at increasing stresses is evident in the results. This trend is typical for granular materials where the strength is higher at low stresses due to the dilatant behaviour of the material under shearing, while a lower strength may occur at high stresses due to particle crushing and reduced dilation of the material.



NOTES:

1. RELATIONSHIP BETWEEN NORMAL STRESS ACROSS THE FAILURE PLANE AND FRICTION ANGLE FOR 1) HIGH DENSITY, WELL GRADED, STRONG PARTICLES, 2) AVERAGE ROCKFILL, 3) LOW DENSITY, POORLY GRADED, WEAK PARTICLES, AND 4) ANGULAR SAND AFTER LEPS (1970).
2. RELATIONSHIP FOR ANGULAR SAND IS EXTRAPOLATED TO NORMAL EFFECTIVE STRESSES BELOW 40 PSI.
3. CU TRIAXIAL TEST DATA POINTS PLOTTED AT ABOVE ARE BASED ON TESTS COMPLETED ON YDTI EMBANKMENT FILL.

Figure 5.2 Rockfill Triaxial Compression Test Results Compared with Strength Functions based on Leps, 1970

The stress paths from CU triaxial testing results indicate the potential for contractive behavior if material is subjected to undrained conditions and rapid shearing. The stress paths do not demonstrate strain softening behavior and indicate that the rockfill is not susceptible to static liquefaction. The test results indicate an undrained strength ratio (S_u/σ'_v) of approximately 0.5 for the CIUC tests.

The stability assessment adopts a drained response and the weakest Leps (1970) relation of *Angular Sand* was assigned as the base case material strength to all historic rockfill material for the stability analyses. The selection of this strength function was made considering the results of testing indicated on Figure 5.2 and in recognition of the potential for site wide variability and long-term degradation of the rockfill material in closure. Use of this function may be somewhat conservative, particularly where historically the most durable rockfill materials have been placed. New rockfill material (Zone D1 and Zone U) from the Continental Pit for future raises of the embankment were assigned the slightly stronger *Lower Boundary* relationship. The higher unit weight of Zone D1 represents denser material as a result of increased compaction of the lifts.

An analysis using a drained response for future static normal operating conditions is reasonable and appropriate given that the majority of the embankment is unsaturated. Perched zones of saturation within the embankment are surrounded by unsaturated embankment fill. A saturated zone along the base of the highest section of the East-West Embankment has a long history of saturation throughout loading and the slow rate of construction has not been observed to influence pore pressure conditions in the rockfill materials. The *Angular Sand* relationship already includes lower strengths at higher confining stresses while also considering the high strength of the material at lower confining stresses.

5.2.5 Tailings

Properties for the tailings were estimated from CPT data collected from 2012 through 2015. The tailings unit for the slope stability assessment is defined by the sand fraction due to the extensive beach length (> 5,000 ft). The tailings are variable both laterally and vertically in terms of grain-size distribution, water content, and stage of consolidation due to the size of the facility and the previous use of a single discharge point. The variability evident in the soundings indicates the behaviour type can range from clean sand to clay in the upper 65 ft. The behaviour type can be constrained to a narrower range between the sand and silt mixtures below 65 ft.

The saturated density of 120 pcf for tailings was determined as the average density from an analysis of the CPT data collected between 2012 and 2015 in the upper 300 ft of the tailings. This saturated density is corroborated by Shelby tube samples collected during the CPT program in 2015. The filling schedule for the YDTI uses an initial settled dry density of tailings of 85 pcf, which is equivalent to a saturated density of 115 pcf. This slightly lower density is consistent with dry density testing on near surface brass tube samples of the tailings completed in 2014 (KP, 2016a). It is noted that a lower density used in the filling schedule provides a reasonably conservative estimate of storage capacity and subsequent filling rate of the YDTI (KP, 2017a).

The base case frictional strength for the tailings sand unit represents the 30th-percentile value of the data set. A comparison of the pre-2015 and 2015 SI data sets yielded no difference in either the bulk unit weight or the friction angle.

Interpretation of the CPT data suggests the tailings deposit is stratified with inter-bedded layers of sand-silt mixtures. Index testing prior to the 2015 SI indicates that the tailings are non-plastic and therefore, the tailings have been assumed to exhibit sand-like behaviour.

The strength loss of the tailings sands was determined from an analysis of the CPT data at the 84th percentile MCE event. The post-earthquake strength of the tailings unit is based on the lower bound ratio of undrained shear strength to effective overburden pressure. A lower bound value (S_u/σ'_v) of 0.05 has been delineated from the CPT data set and was utilized in the stability assessment.

The results of cyclic direct simple shear (CDSS) laboratory testing on tailings in 2014 (Wijewickreme, D., 2014) indicated a residual undrained strength ratio (S_u/σ'_v) ranging from 0.09 to 0.18 with a mean value of 0.13 at shear strains limited to 10%. Higher strengths are determined with shear strains greater than 10%. It is typical that the laboratory residual strength values are higher than CPT interpreted results. The results corroborate the conservatism in using the lower bound undrained strength ratio (S_u/σ'_v) of 0.05 for post-earthquake stability assessment.

5.3 PIEZOMETRIC CHARACTERIZATION

Piezometric conditions for the stability assessment were determined from the evaluation of data collected by KP since 2012, historic standpipe monitoring records, and previous seepage analyses

(Hydrometrics, 1997). The field data includes dissipation tests from CPT, data from monitoring wells and vibrating wire piezometers, and detailed observations presented in the drillhole logs. The disparity between the permeability of the embankment rockfill and tailings, and the anisotropy resulting from the method of embankment construction creates complex piezometric conditions that are difficult to incorporate in the LE stability models. The piezometric lines used in the stability assessment are simplified and conservative representations of the pore pressure conditions determined during the 2015 SI and are corroborated by historical measurements and interpretations.

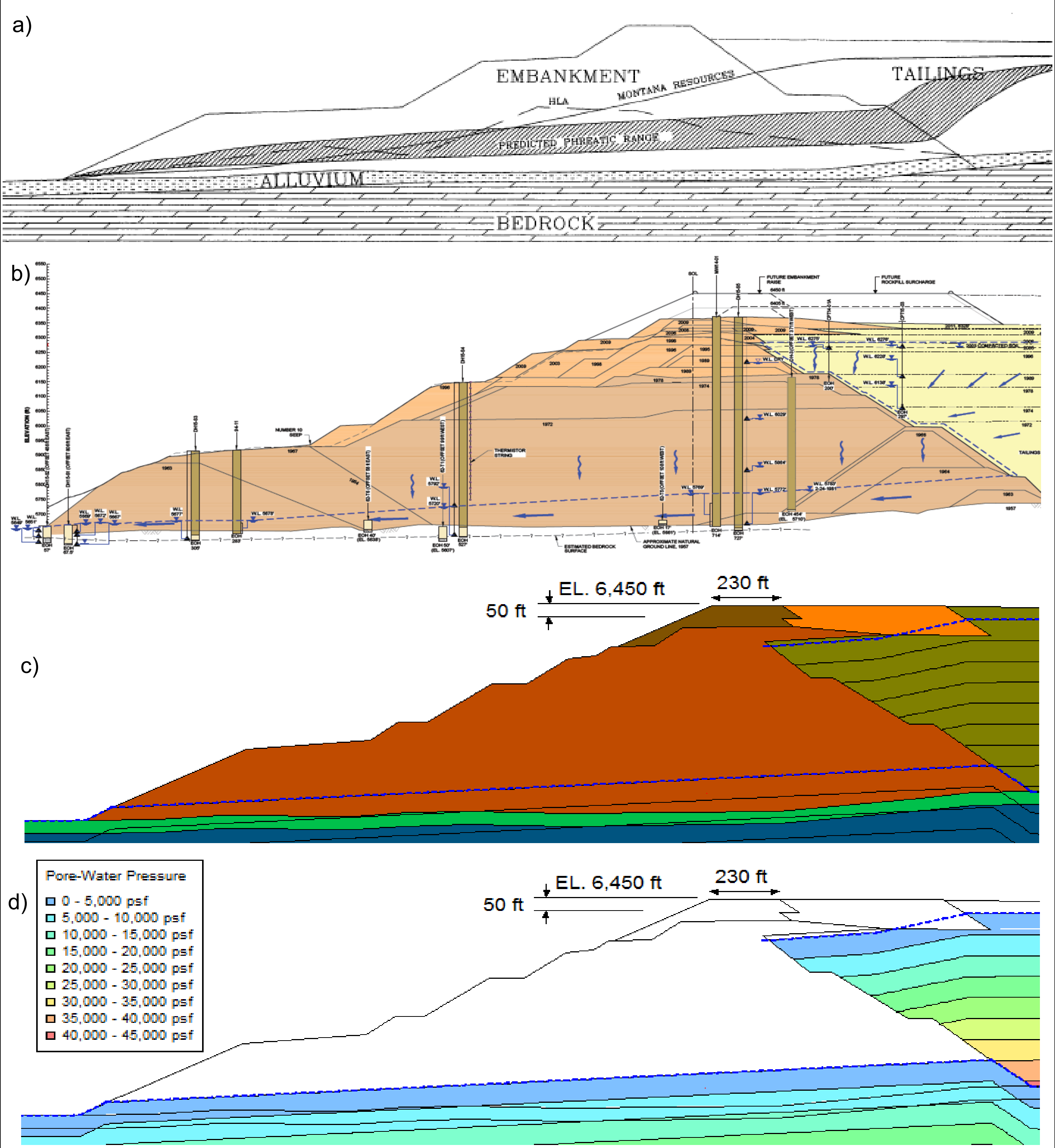
Hydrometrics (1997) defined a zone over which the phreatic surface could be located in a seepage analysis that considered both isotropic and anisotropic permeability. The conditions defined by Hydrometrics are shown on Figure 5.3(a). Seepage flow through the facility was found to be dominated by the large difference (about two orders of magnitude) between the permeability of the embankment and the tailings impoundment. As a result, the embankment acts as a drain while the tailings provide, by comparison, a seepage impediment.

The interpretation by Hydrometrics is corroborated by field data collected in 2015 as shown on Figure 5.3(b). The data shows a shallow phreatic surface (about 30 ft above original ground) and a vertical hydraulic gradient that is approximately hydrostatic near the downstream toe of the embankment. The thickness of the saturated zone below the phreatic surface increases to approximately 100 ft at the center of the embankment. The pore pressures measured in the center of the embankment show perched saturated conditions at higher elevations than the downstream toe with a strong downward gradient. Perched zones of saturation within the embankment are surrounded by unsaturated embankment fill based on observations in the drill core. The water table in the tailings adjacent to the embankment was determined to be at approximately 70 ft below the top of the tailings beach. The elevation of the phreatic surface within the tailings beach gradually increases towards the supernatant pond. Flow within the tailings beach is downwards and towards the embankment. The data indicate that the vertical hydraulic gradient is downward in the embankment and tailings beach areas.

The piezometric lines used in the stability assessment are shown on Figure 5.3(c). The analysis uses two piezometric lines to best represent pore pressure conditions in the embankment and tailings. One piezometric line is associated with the tailings and applies hydrostatic conditions to the tailings below the piezometric line. This representation is conservative for the tailings material because measured conditions during piezocone advancement and subsequent vibrating wire piezometer measurements indicate less than hydrostatic pore pressure development below the phreatic surface. This approximates the pore pressure conditions at the tailings discharge point, and is conservatively extrapolated through the remainder of the impoundment. The second piezometric line is associated with the rockfill, overburden and bedrock materials and applies hydrostatic conditions to these materials existing below the piezometric line.

The pore water pressure conditions resulting from this representation using two piezometric lines are shown as contours on Figure 5.3(d). The embankment fill and tailings materials below the piezometric lines are modelled as saturated, and the contours roughly represent 80 ft of hydrostatic pore pressure development. Unsaturated conditions are modelled to the left and above the piezometric lines. This representation is consistent with hydrogeological conditions described in Section 4.8 of the Site Characterization Report (KP, 2017b). The differences between the modelled and measured conditions below the piezometric lines are conservative.

The conditions within the embankment have not substantially changed since the 1980s. The saturated zone in the based on the embankment existed in the 1980s and was represented in the stability analyses at that time, and recent pore pressure measurements indicate that the phreatic surface may actually be lower now in the embankment than it was during the early 1980s.



NOTES:

a) PREVIOUS STUDIES (HYDROMETRICS, 1997)

b) PIEZOMETRIC CONDITIONS BASED ON MONITORING INSTRUMENTS

c) PIEZOMETRIC LINE LOCATIONS IN SLOPE/W

d) PIEZOMETRIC CONDITIONS APPLIED IN SLOPE/W

REV	DATE	DESCRIPTION	PREP'D	RVW'D
1	14AUG'17	ISSUED WITH REPORT	DDF	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

PIEZOMETRIC CHARACTERIZATION

Knight Piésold CONSULTING

P/A NO. VA101-126/12

REF. NO. 2

FIGURE 5.3

REV 1

5.4 STATIC STABILITY

5.4.1 Normal Operating Conditions

The static slope stability was assessed by determining the FS for the selected representative sections of the East-West, North-South, and West Embankments. Downstream and upstream slip surfaces for static normal operating conditions were evaluated. The legislative requirement for static maximum normal operating loading conditions (FS ≥ 1.5) is considered at each embankment section with LE models involving base case material properties and the full height of the embankment raise. The analyses indicate that the proposed facility design meets the legislative requirements for static stability under normal operating conditions. The results are tabulated below in Table 5.2 and illustrations of the critical slip surfaces are included in Appendix B1.

Table 5.2 Factors of Safety for Static Normal Operating Conditions

Embankment Section	Material Properties	Downstream FS [1]	Upstream FS [1]
08+00 W	Base Case	2.0	> 5
38+00 NW	Base Case	4.2	> 5
18+00 N	Base Case	2.0	> 5
108+40 W	Base Case	7.2	> 5

NOTES:

1. "FS" DENOTES FACTOR OF SAFETY, AND THE VALUE PRESENTED REPRESENTS THE LOWEST FACTOR OF SAFETY DETERMINED FROM THE EXAMINATION OF SEVERAL THOUSAND SLIP SURFACES.

It is important to note that the material strength properties used to determine the factors of safety presented above are more conservative than past studies performed for the YDTI (Dames and Moore, 1962; IECO, 1981; HLA, 1993; MR, 1999). In particular, the material strength used for rockfill has been reduced in recognition of the potential nature and variability of the gravel, cobbles, and boulders present within a silty-sand or sandy-silt matrix. For example, the lowest downstream FS determined for Section 8+00 W would be approximately 2.3 using the material strength properties adopted by IECO in 1981. Rockfill was modelled with a uniform friction angle of 35 degrees in this particular sensitivity analysis.

5.4.2 Post-Earthquake Conditions

The legislative requirements for static post-earthquake (FS ≥ 1.2) loading conditions are considered in the LE models through a series of conservative sensitivity analyses using undrained and liquefied material properties for tailings. These models also incorporate an embankment crest at its proposed elevation (EL. 6,450 ft) and saturated, hydrostatic conditions modelled below the piezometric lines.

The liquefaction assessment indicated that saturated tailings within the YDTI and without the rockfill surcharge are potentially liquefiable under the median MCE during the operations period and the 84^{th}-Percentile MCE in closure. The potentially liquefiable tailings are assumed to extend to approximately 275 ft depth in the central impoundment area; however, the top 50 to 80 ft or more of tailings beach adjacent to the embankment is unsaturated or partially saturated and is therefore not likely to liquefy. However, the entire tailings mass was conservatively assigned the undrained and liquefied material strength properties for the purpose of conservatively analyzing static, post-earthquake conditions.

The first set of sensitivity analyses consider base case material properties for all materials except the tailings. The tailings were modelled with two undrained strength ratios to investigate the sensitivity of the FS to a variation in tailings material strength. Tailings are assigned an undrained strength ratio (S_u/σ'_v) of 0.20 in one set of analyses and a liquefied undrained strength ratio (S_u/σ'_v) of 0.05 in the second set. The results are tabulated below in Table 5.3 and illustrations of the critical slip surfaces are included in Appendix B2.

Table 5.3 Factors of Safety for Static Post-Earthquake Conditions

Embankment Section	Material Properties [1]	Downstream FS [2]	Yield Acceleration (g)	Upstream FS [2]	Yield Acceleration (g)
08+00 W	Tailings $S_u/\sigma'_v = 0.20$	1.9	0.24	> 5	0.28
	Tailings $S_u/\sigma'_v = 0.05$	1.9	0.24	3.1	0.08
38+00 NW	Tailings $S_u/\sigma'_v = 0.20$	3.9	0.21	4.6	0.32
	Tailings $S_u/\sigma'_v = 0.05$	3.8	0.20	3.2	0.10
18+00 N	Tailings $S_u/\sigma'_v = 0.20$	2.0	0.31	> 5	0.24
	Tailings $S_u/\sigma'_v = 0.05$	2.0	0.30	> 5	0.19
108+40 W	Tailings $S_u/\sigma'_v = 0.20$	7.2	0.37	> 5	0.35
	Tailings $S_u/\sigma'_v = 0.05$	7.1	0.35	> 5	0.35

NOTES:
1. MATERIAL PROPERTIES FOR MATERIALS OTHER THAN TAILINGS ARE CONSISTENT WITH BASE CASE PROPERTIES THAT WERE ANALYZED FOR STATIC NORMAL OPERATING CONDITIONS.
2. "FS" DENOTES FACTOR OF SAFETY, AND THE VALUE PRESENTED REPRESENTS THE LOWEST FACTOR OF SAFETY DETERMINED FROM THE EXAMINATION OF SEVERAL THOUSAND SLIP SURFACES.

The tailings strength has very little impact on the predicted factor of safety for downstream slip surfaces. The largest impact of the earthquake-induced strength loss is in the upstream scenarios where the FS could be reduced by more than 50%. The actual calculated factors of safety for the upstream case, which are shown in Table 5.2 to be greater than 5, are included on the corresponding figures in Appendix B2. The upstream slip involves a large tailings mass. Smaller upstream slip surfaces may have lower FS, such as surficial sloughing, but will not affect the overall integrity of the impoundment. The post-earthquake strength of the tailings leads to a substantial reduction in the FS; however, the FS still exceeds the minimum design requirements. The analyses indicate that the proposed facility design meets the legislative requirements for static stability for post-earthquake conditions while conservatively considering undrained strength analyses for the saturated, potentially contractive tailings material.

5.4.3 Sensitivity Analyses for Undrained Response

Antecedent experience within the mining industry has documented the potential for undrained loading in saturated foundation soils and fill materials to be significant contributors to past instabilities in waste rock dumps and tailings facilities. It is prudent practice to consider whether such conditions are relevant for the YDTI.

Static collapse of saturated or nearly saturated sandy-gravel layers have resulted in flow slides within a number of coal mine waste dumps in British Columbia (Dawson et al., 1998). The runout distances in these examples were likely exacerbated by the relatively steep foundation slopes, which were in excess of 15 degrees and sometimes up to 30 degrees, and also by the dumping sequence that was from the top down at angle of repose. Back analysis of several of these failures inferred a position for a fine grained sandy-gravel zone that potential may have behaved in an undrained manner. A series of analyses were then used to show that conventional friction angles based on the angle of repose of the bulk material may have over predicted the factor of safety. Laboratory triaxial testing on selected samples of material representing the finer grained zone showed clearly contractive behavior with an observed peak strength, strain softening, and high strain steady-state strength. The LE approach analyses adopted a trigger and collapse based assessment approach using a peak strength for the trigger analysis and steady-state undrained strength ratio for the collapse analysis. The authors concluded that the dumps were marginally stable at peak strength and only a small trigger was required to initiate collapse. The collapse analyses showed factors of safety considerably less than 1.0 illustrating that once movement was initiated, the strain-weakening behavior of the continuous saturated material and steep foundation slopes would allow for a rapidly moving flowslide for these waste dumps.

There is other recent experience with the design and operation of very high waste rock dumps in Chile where the geotechnical characteristics of waste rock subject to high confining pressures and in situ leaching were tested in a large scale triaxial apparatus under both drained and undrained conditions (Valenzuela et al., 2008 & 2011). The waste dumps under consideration will reach final heights in excess of 1,600 ft (500 m) in mountainous environments where the only available space is often in relatively narrow and steep valleys. The authors found that high pressures associated with the weight of materials stockpiled in waste rock dumps with heights over 330 ft (100 m) induce significant particle crushing and lead to an increase in the amount of sands and fines in the granular matrix. The high stress conditions modify the void ratio and consequently change the permeability and density. The testing identified that the compressibility of the material increased significantly for confining pressures over 21,000 psf (1 MPa). The high stress, large scale triaxial tests show a contractive response in the tested stress range up to approximately 50,000 psf (2.5 MPa), but did not demonstrate strain-softening behavior. The interpreted undrained strength ratio (S_u/σ'_v) for these tests at high stresses was approximately 0.28 to 0.29.

Both of these examples, while containing minor similarities, are markedly different as compared to the conditions prevailing at the YDTI. Perhaps the most applicable aspects of the Chilean studies to the YDTI are the defensive design considerations recommended by the authors to reduce potential instabilities (Valenzuela et al., 2008). These good practices, among others, have been incorporated in the embankment construction for the YDTI. Several indicators of stability and the corresponding differences in construction practices between the BC and Chile dumps and the YDTI embankments are described in Table 5.4.

Table 5.4 Indicators of Stability and Construction Practices

Indicator	High Dumps (BC Coal, Chile)	YDTI Embankments
Foundation Slope	Steep, greater than 15 degrees	Flat
Constructed Slopes	Angle of repose, dumped from top down for full height	Angle of repose 50 ft lifts, with benches to produce smooth global angle flatter than 2H:1V
Fabric Orientation	Oriented unfavorably compared to critical slip surface direction	Oriented favorably, providing greater resistance to downstream slip surfaces exiting the toe

NOTES:
1. THE DUMPING SEQUENCE FOR THE CENTRAL PEDESTAL AREA IS BASED ON AVAILABLE AERIAL SURVEYS, AERIAL AND OBLIQUE PHOTOGRAPHS, PLANIMETRIC PROGRESS MAPS, AND ENGINEERINGING NOTES AND MEMORANDA DATING BACK TO 1954 (APPLIED GEOLOGIC SERVICES, 2017).

It is important to note several distinctions:

(1) Saturation does not mean a material will be subject to undrained loading, it only indicates that the possibility exists. A trigger is required to initiate undrained loading. In the failure precedents described above, that trigger was likely static in nature making it relevant to consider for normal operating conditions. The dumps were only marginally stable under drained conditions prior to failure due to the orientation of the dumping surface and continuous angle of repose slopes combined with bedded fine grained zones along that surface that retain water.

(2) Undrained loading under static conditions is expected to occur locally at some point in the life of the facility and pore pressures will dissipate over time. The favorable orientation of the embankment fabric and geotechnical investigations indicates that although weaker and stronger zones exist, a continuous weaker layer is not credible.

(3) **The YDTI embankments are stable with a FS of 2.0 or greater.**

Seismic loading is the only credible potential trigger to initiate an undrained response for the YDTI at a scale more than local. The mechanism for the rise in pore pressures could be earthquake-induced deformation along a slip surface. The actual likelihood of loss of material strength during seismic loading is uncertain and would only be a rare and localized occurrence. An extreme sensitivity analysis was performed using the LE slope stability model on Section 8+00W using very low undrained strengths for overburden and rockfill applied to the zone of saturation in the base of the embankment. Strength reductions and corrections were applied to the laboratory undrained strength ratio for overburden and rockfill to estimate the undrained strength ratio under anisotropic (K_o) field conditions with a direct simple shear (DSS) slip surface where appropriate.

- The interpreted S_u/σ'_v ratio for overburden has a range between 0.23 and 0.30. The sensitivity analyses for post-earthquake conditions adopts a conservative lower bound strength ratio of S_u/σ'_v equal to 0.23 for saturated overburden in the maximum section (Section 8+00 W). The adopted undrained strength ratio of 0.23 is corroborated by the theoretical equation $(S_u/\sigma'_v)_{DSS}$ equal to 0.5 sin (φ') for a φ' of 27 degrees adopted from the triaxial compression testing. This lower bound material strength effectively applies to the alluvium a typical strength for a normally consolidated clay for the post-earthquake sensitivity analyses.

- The sensitivity analyses for post-earthquake conditions adopts a conservative lower bound undrained strength ratio of S_u/σ'_v equal to 0.37 for sloping portions of the slip surface and 0.27 in DSS for saturated rockfill in the maximum section (Section 8+00 W). The adopted DSS undrained strength ratio of 0.27 is corroborated by the theoretical equation $(S_u/\sigma'_v)_{DSS}$ equal to 0.5 sin (φ') for a φ' of 33 degrees.

The sensitivity analyses consider an undrained response in the saturated overburden and rockfill independently, and also in a combined analysis that includes a relatively flat slip surface in overburden and sloping portion of the slip surface in saturated rockfill. These analyses allow for geotechnical uncertainties in the embankment fill and foundation response to shaking and deformation. An additional analysis was performed analyzing the sensitivity of the FS to a perched zone of fine-grained material higher up in the embankment behaving in an undrained manner. The slip surface for this final case is fully specified within hypothetical materials behaving in an undrained manner. The lowest calculated FS for each analysis is shown on Table 5.5 and the critical slip surfaces are shown in Appendix B3.

Table 5.5 Factors of Safety for Undrained Strength Sensitivity Analyses

Embankment Section	Material Properties [1]	Downstream FS [2]	Yield Acceleration (g)
08+00 W	Overburden S_u/σ'_v = 0.23 (DSS)	1.3	0.04
	Saturated Rockfill S_u/σ'_v = 0.27 (DSS)	1.3	0.07
	Overburden S_u/σ'_v = 0.23 (DSS) & Saturated Rockfill S_u/σ'_v = 0.37 (TX)	1.2	0.04
08+00 W (Hypothetical Slip Surface at EL. 5,925 ft near Seep 10)	Perched Saturated Rockfill S_u/σ'_v = 0.27 (DSS) & S_u/σ'_v = 0.37 (TX)	1.4	0.11

NOTES:
1. MATERIAL PROPERTIES FOR TAILINGS ARE CONSISTENT WITH THE LIQUIFIED UNDRAINED STENGTH RATIO (S_u/σ'_v = 0.05) THAT WAS ANALYZED FOR STATIC POST-EARTHQUAKE OPERATING CONDITIONS. ALL OTHER MATERIAL PROPERTIES ARE CONSISTENT WITH BASE CASE PROPERTIES THAT WERE ANALYZED FOR STATIC NORMAL OPERATING CONDITIONS UNLESS OTHERWISE NOTED.
2. "TX" DENOTES STRENGTH VALUE USED FOR TRIAXIAL COMPRESSION ORIENTATION TO THE SLIP SURFACE; "DSS" DENOTES STRENGTH VALUE USED FOR DIRECT SIMPLE SHEAR ORIENTATION TO THE SLIP SURFACE.
3. "FS" DENOTES FACTOR OF SAFETY, AND THE VALUE PRESENTED REPRESENTS THE LOWEST FACTOR OF SAFETY DETERMINED FROM THE EXAMINATION OF SEVERAL THOUSAND SLIP SURFACES.

When compared with the evaluation performed for the BC coal dumps (Dawson et al., 1998) another comparative distinction is readily apparent. The evaluation for the coal dumps predicted FS values well below 1.0 for the lower bound undrained strength steady-state condition, which indicated that if undrained loading was triggered then the dump would fail rapidly.

The extreme sensitivity analyses performed for the YDTI indicate that the embankment will remain stable even if lower bound undrained strengths were triggered. The legislative requirements for static post-earthquake (FS ≥ 1.2) loading conditions are satisfied even in these extreme sensitivity analyses. Establishing this extreme condition as the base case for static normal operating conditions for the YDTI, while it would lead to a more conservative design, is not a reasonable and appropriate use of undrained strength analysis. However, selective and strategic placement of rockfill to further mitigate the very low risk associated with this condition should be considered while evaluating options for storage of excess rockfill produced during mining of the Continental Pit.

5.4.4 Sensitivity Analyses showing Staged Factors of Safety

A sensitivity analysis was performed to evaluate progression of the factor of safety for the maximum section (8+00 W) over time. The analysis included an evaluation of the current (2017) embankment conditions and an analysis of embankment conditions in approximately 2014 prior to commencing construction of the rockfill surcharge adjacent to the East-West Embankment. The lowest FS computed for normal operating conditions and post-earthquake conditions at each stage is shown on Figure 5.4. The critical slip surfaces and corresponding FS values in Appendix B4 are shown for normal operating conditions, post-earthquake conditions, and the extreme sensitivity analysis using an undrained response in saturated rockfill and overburden. **The analysis shows that the YDTI is stable under current and future conditions with a FS well above the legislated requirements of 1.5 for normal operating conditions and 1.2 for post-earthquake conditions. A factor of safety of 1.2 is also achieved for the extreme sensitivity analysis for undrained response.**

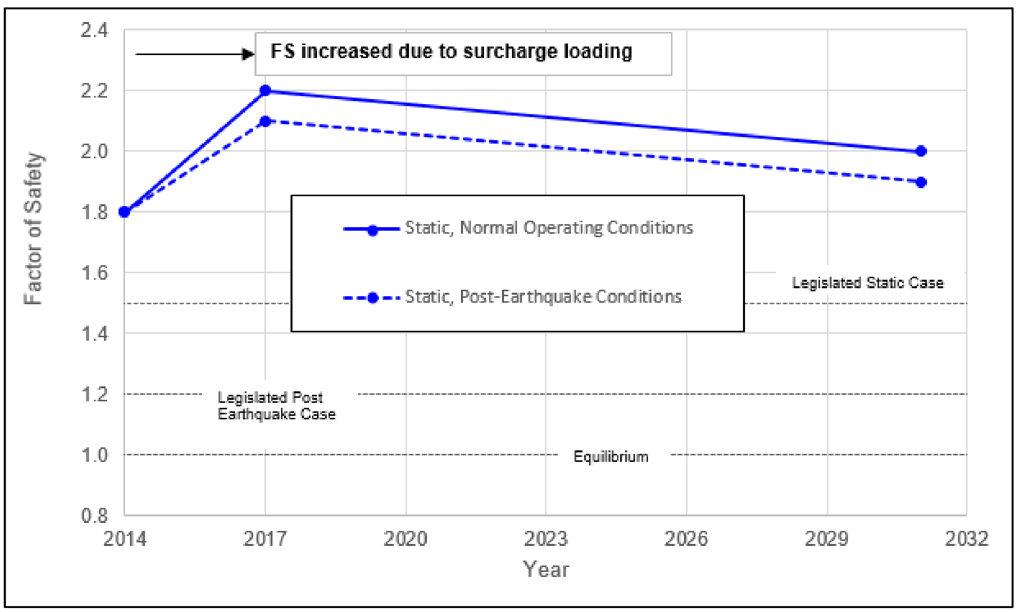


Figure 5.4 Staged Factor of Safety

The static and seismic stability of the East-West Embankment has been improved by constructing the rockfill surcharge. Tailings materials in the tailings beach and beneath the rockfill surcharge are predominantly near to the boundary between potentially contractive and potentially dilative behavior, and the rockfill surcharge tends to increase the potential for dilative behavior. The liquefaction assessment demonstrated that the rockfill surcharge effectively mitigates the potential for cyclic liquefaction of the surcharged tailings zone during operations. Drainage of the tailings in much of the facility has resulted in tailings densities that are higher than would otherwise be achieved under hydrostatic effective stresses. Dense tailings are less susceptible to loss of strength than loose tailings. The potential for liquefaction is reduced due to the pressure of the overlying rockfill mass, which also increases the density of the tailings. The tailings discharge point has been relocated further out into the impoundment due to the construction of the rockfill surcharge, which has reduced pore pressures in the tailings near the embankment and increased the depth of the unsaturated zone in the tailings below the surcharge.

The FS was increased between 2014 and 2017 by constructing the rockfill surcharge as shown on Figure 5.4. This improvement to the stability of the impoundment allows for continued filling of the EL. 6,450 ft lift of the YDTI with a FS greater than or equivalent to those computed for 2014 conditions prior to surcharging, including consideration of expected conditions and the extreme sensitivity analyses evaluating undrained response in saturated rockfill and overburden.

6 – EARTHQUAKE-INDUCED EMBANKMENT DEFORMATION

6.1 GENERAL

The dynamic instability mode was assessed by estimating the maximum earthquake-induced deformations along the critical slip surface and settlement of the crest at each embankment section. The yield acceleration required to reduce the FS to unity was determined for each section. An earthquake is assumed to induce ground displacement, deformations and settlement of the embankment crest, and movement along a hypothetical slip surface within the embankment. The deformation analysis examines an extreme scenario that includes the following assumptions:

- The embankment displaces downward relative to and independent of the tailings contained within the YDTI and supernatant pond due to displacement on the Continental Fault. The median and 84th-percentile values for the average displacement on the Continental Fault were estimated to be 1.7 ft and 4.7 ft, respectively, as described in Section 2.3.
- The embankment deforms and the crest settles due to the seismic loading of the embankment independent of the tailings contained in the YDTI. The crest settlement was determined using a mathematical formula, developed based on statistical analysis of empirical data, which relates Normalized Crest Settlement (NCS) to PGA and earthquake Magnitude (Swaisgood, 2014).
- Seismic loading induces slope displacement in the embankment relative to and independent of the tailings. The slope displacement and the probability of non-tolerable displacement was estimated using a probabilistic simplified procedure (Bray and Travasarou, 2007).

6.2 OPERATING CONDITIONS

Earthquake-induced embankment deformations were evaluated for operating conditions while considering seismic loading from the median MCE with a design PGA of 0.45 g and compared with freeboard requirements described in Section 4. It is conservatively assumed that the displacement would occur in the vicinity of the active tailings discharge area and that a combined displacement greater than 5 ft during operations constitutes a potential failure mode. The total tolerable displacement was reduced by predicted fault displacement and crest settlement to estimate a tolerable displacement along the critical slip surface for each section as shown in Table 6.1.

Table 6.1 Estimated Tolerable Slip Surface Displacement during Operations

Section	Total Tolerable Displacement	Average Estimated Fault Displacement	Maximum Estimated Crest Settlement	Estimated Tolerable Displacement along Slip Surface
08+00 W	5 ft	1.7 ft	1.7 ft	**1.6 ft**
38+00 NW	5 ft	1.7 ft	0.5 ft	**2.8 ft**
18+00 N	5 ft	1.7 ft	0.7 ft	**2.6 ft**
108+40 W	5 ft	1.7 ft	0.3 ft	**3.0 ft**

Displacements along the critical upstream and downstream slip surfaces were then estimated to check that predicted displacements were within the tolerable limits. The probabilistic simplified procedure used for the analysis allows for several concurrent estimates to be made including an estimate of the slip surface displacement at any selected probability of exceedance and the probability of displacement greater than tolerable (if an estimate of tolerable displacement is known). The median and 99th percentile (1% probability of exceedance) displacement levels were selected for this analysis and are included in Table 6.2.

Table 6.2 Maximum Slip Surface Displacement during Operations

Section	Slip Location	Yield Acceleration, k_y (g)	Estimated Median Displacement	Estimated 99th Percentile Displacement	Probability of Displacement Greater than Tolerable
08+00 W	Downstream	0.24 [1]	Negligible	Negligible	0 %
		0.04 [2]	0.3 ft	2.0 ft	2 %
	Upstream	0.08 [1]	< 0.1 ft	0.3 ft	0 %
38+00 NW	Downstream	0.20 [1]	0.1 ft	0.5 ft	0 %
	Upstream	0.10 [1]	0.4 ft	1.9 ft	0 %
18+00 N	Downstream	0.30 [1]	Negligible	Negligible	0 %
	Upstream	0.19 [1]	Negligible	0.1 ft	0 %
108+40 W	Downstream	0.35 [1]	Negligible	Negligible	0 %
	Upstream	0.35 [1]	Negligible	0.1 ft	0 %

NOTES:

1. YIELD ACCELERATION FROM TABLE 5.3. MATERIAL PROPERTIES FOR MATERIALS OTHER THAN TAILINGS ARE CONSISTENT WITH BASE CASE PROPERTIES THAT WERE ANALYZED FOR STATIC NORMAL OPERATING CONDITIONS.
2. YIELD ACCELERATION FROM TABLE 5.5. MATERIAL PROPERTIES CONSISTENT WITH EXTREME SENSITIVITY ANALYSIS FOR LOWER BOUND UNDRAINED STRENGTH CONDITIONS.

The analysis demonstrates that the maximum estimated displacement for the median MCE during operations will be tolerable for all sections at the 99th-percentile displacement estimate level. The analysis further indicates that the maximum slip surface deformation is estimated to be negligible in many cases, with earthquake-induced deformation to the embankment controlled primarily by crest settlement.

The exception to this finding is the extreme sensitivity case considering undrained strengths in overburden and rockfill. The estimate for this lower bound case is that the maximum slip surface displacement could exceed the tolerable displacement by approximately 0.4 ft. The results are acceptable for this case due to the collective conservatism in the definition of tolerable displacement and the material strength conditions imposed in this sensitivity analysis. This residual risk can be mitigated by maintaining an additional 0.5 ft or more of minimum freeboard.

6.3 CLOSURE CONDITIONS

Earthquake-induced embankment deformations were evaluated for closure conditions while considering seismic loading from the 84th-Percentile MCE with a design PGA of 0.84g and compared with freeboard requirements described in Section 4. A combined displacement greater than 50 ft would be required to position the crest of the embankment below the phreatic surface in the tailings immediately following closure. The tailings pore water will drain down and the thickness of the unsaturated tailings zone adjacent to the embankments will increase following closure. The total tolerable displacement was reduced by predicted fault displacement and crest settlement for closure to estimate a tolerable displacement along the critical slip surface for each section as shown in Table 6.3.

Table 6.3 Estimated Tolerable Slip Surface Displacement after Closure

Section	Total Tolerable Displacement	Average Estimated Fault Displacement	Maximum Estimated Crest Settlement	Estimated Tolerable Displacement along Slip Surface
08+00 W	50 ft	4.7 ft	18.3 ft	**27 ft**
38+00 NW	50 ft	4.7 ft	4.3 ft	**41 ft**
18+00 N	50 ft	4.7 ft	6.2 ft	**39 ft**
108+40 W	50 ft	4.7 ft	2.8 ft	**42 ft**

Displacements along the critical upstream and downstream slip surfaces were then estimated to confirm that maximum estimated displacements were tolerable. The median and 99th percentile (1% probability of exceedance) displacement levels were selected for this analysis and are included in Table 6.4.

Table 6.4 Maximum Slip Surface Displacement after Closure

Section	Slip Location	Yield Acceleration, k_y (g)	Estimated Median Displacement	Estimated 99th Percentile Displacement	Probability of Displacement Greater than Tolerable
08+00 W	Downstream	0.24 [1]	Negligible	0.1 ft	0 %
		0.04 [2]	2.1 ft	10 ft	0 %
	Upstream	0.08 [1]	0.7 ft	3.1 ft	0 %
38+00 NW	Downstream	0.20 [1]	0.5 ft	2.5 ft	0 %
	Upstream	0.10 [1]	1.6 ft	7.2 ft	0 %
18+00 N	Downstream	0.30 [1]	< 0.1 ft	0.5 ft	0 %
	Upstream	0.19 [1]	0.3 ft	1.3 ft	0 %
108+40 W	Downstream	0.35 [1]	0.2 ft	0.7 ft	0 %
	Upstream	0.35 [1]	0.2 ft	1.1 ft	0 %

NOTES:
1. YIELD ACCELERATION FROM TABLE 5.3. MATERIAL PROPERTIES FOR MATERIALS OTHER THAN TAILINGS ARE CONSISTENT WITH BASE CASE PROPERTIES THAT WERE ANALYZED FOR STATIC NORMAL OPERATING CONDITIONS.
2. YIELD ACCELERATION FROM TABLE 5.5. MATERIAL PROPERTIES CONSISTENT WITH EXTREME SENSITIVITY ANALYSIS FOR LOWER BOUND UNDRAINED STRENGTH CONDITIONS.

6.4 SUMMARY

The results of the earthquake-induced deformation estimates are summarized as follows:

- The analysis demonstrates that the maximum estimated earthquake-induced deformation will be tolerable for the median MCE during operations and the 84th-Percentile MCE during closure.
- The maximum combined displacement during operations (median MCE) was estimated to be 3.4 ft at Section 8+00 W compared to 5 ft of minimum freeboard.
  - An upper bound estimate of 5.4 ft of total combined displacement was estimated for operations while considering the undrained strength lower bound sensitivity analysis from Section 5.4.3. The residual risk of this condition can be mitigated by maintaining an extra 0.5 ft or more of minimum freeboard during operations.
- The maximum combined displacement during closure (84th-Percentile MCE) was estimated to be 23 ft at Section 8+00 W compared to 50 ft of tolerable combined displacement.
  - An upper bound estimate of 33 ft of total combined displacement was estimated for closure while considering the undrained strength lower bound sensitivity case compared to 50 ft of tolerable combined displacement. No additional mitigation is required for closure conditions.

7 – CONCLUSIONS AND RECOMMENDATIONS

The key performance factors for stability of the embankment are developing large drained tailings beaches that maintain the supernatant pond remote from the embankments and reduces the pore pressures in the tailings beach adjacent to the upstream face in the long-term. Selective and strategic placement of rockfill to further improve embankment stability and to support reclamation objectives should be considered while evaluating options for storage of excess rockfill produced during mining of the Continental Pit.

The governing scenario for all sections analyzed is a downstream slip surface through the embankment rockfill. The YDTI embankments are stable with a factor of safety (FS) of 2.0 or greater, which exceeds the legislated requirement of 1.5 for normal operating conditions. The FS values for the upstream cases are typically greater than 5 for normal operating conditions.

Tailings materials in the beach and beneath the rockfill surcharge are predominantly near to the boundary between potentially contractive and potentially dilative behavior, and the rockfill surcharge tends to compress the tailings and increase the potential for dilative behavior. The cyclic liquefaction assessment indicates that the upper saturated tailings within the YDTI and outside the rockfill surcharge area are potentially liquefiable with seismic loading from the MCE. However, the top 50 ft to 80 ft or more of tailings beach adjacent to the embankment and beneath the rockfill surcharge is currently unsaturated or partially saturated and is therefore not susceptible to liquefaction. The liquefaction assessment demonstrated that the rockfill surcharge effectively mitigates the potential for cyclic liquefaction of the surcharged tailings zone during operations. Localized liquefaction of the tailings underlying the rockfill surcharge may occur in closure with seismic loading from the 84^{th}-percentile MCE while the tailings remain saturated.

The largest impact of the earthquake-induced strength loss is in the upstream scenarios where the FS could be reduced by more than 50%; however, factors of safety exceed 3.0 under these conservative conditions. The analyses indicate that the proposed facility design meets the legislative requirements for static stability for post-earthquake conditions while conservatively considering undrained strength analyses for the saturated, potentially contractive tailings material. The analyses also indicate that the embankment will remain stable even if lower bound undrained strengths were triggered in a continuous layer of saturated overburden and rockfill in the base of the embankment. The favorable orientation of the embankment fabric and geotechnical investigations indicates that although weaker and stronger zones exist, a continuous weaker layer is not credible. Undrained loading under static conditions is expected to occur locally at some point in the life of the facility and pore pressures will dissipate over time.

The static and seismic stability of the East-West Embankment was improved between 2014 and 2017 by constructing the rockfill surcharge. The tailings discharge point was also relocated further out into the impoundment due to the construction of the rockfill surcharge, which has resulted in reduced pore pressures in the tailings near the embankment and increased the depth of the unsaturated zone in the tailings below the surcharge. The continued filling of the EL. 6,450 ft lift of the YDTI can be completed while achieving FS values that do not decrease below those computed for 2014 conditions prior to surcharging. Thus the impoundment stability for the higher EL. 6,450 ft lift is equivalent or improved as compared to the 2014 conditions.

The earthquake deformation analysis indicates that the maximum estimated earthquake-induced deformation will be within design tolerances for the median MCE during operations and the 84^{th}-Percentile MCE during closure. The maximum combined displacement during operations was estimated to be 3.4 ft for the East-West Embankment compared to 5 ft of minimum freeboard. The maximum combined displacement during closure was estimated to be 23 ft compared to 50 ft of tolerable combined displacement.

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9 – CERTIFICATION

This report was prepared and reviewed by the undersigned.

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
March 13, 2018
D. D. FONTAINE
36208

Prepared: ______________________________

Daniel Fontaine, P.Eng.
Senior Civil Engineer | Associate

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Mar 14, 2018
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PROFESSIONAL ENGINEER

Reviewed: ______________________________

Ken Brouwer, P.E.
President, Engineer of Record

Approval that this document adheres to Knight Piésold Quality Systems: KJB

APPENDIX A

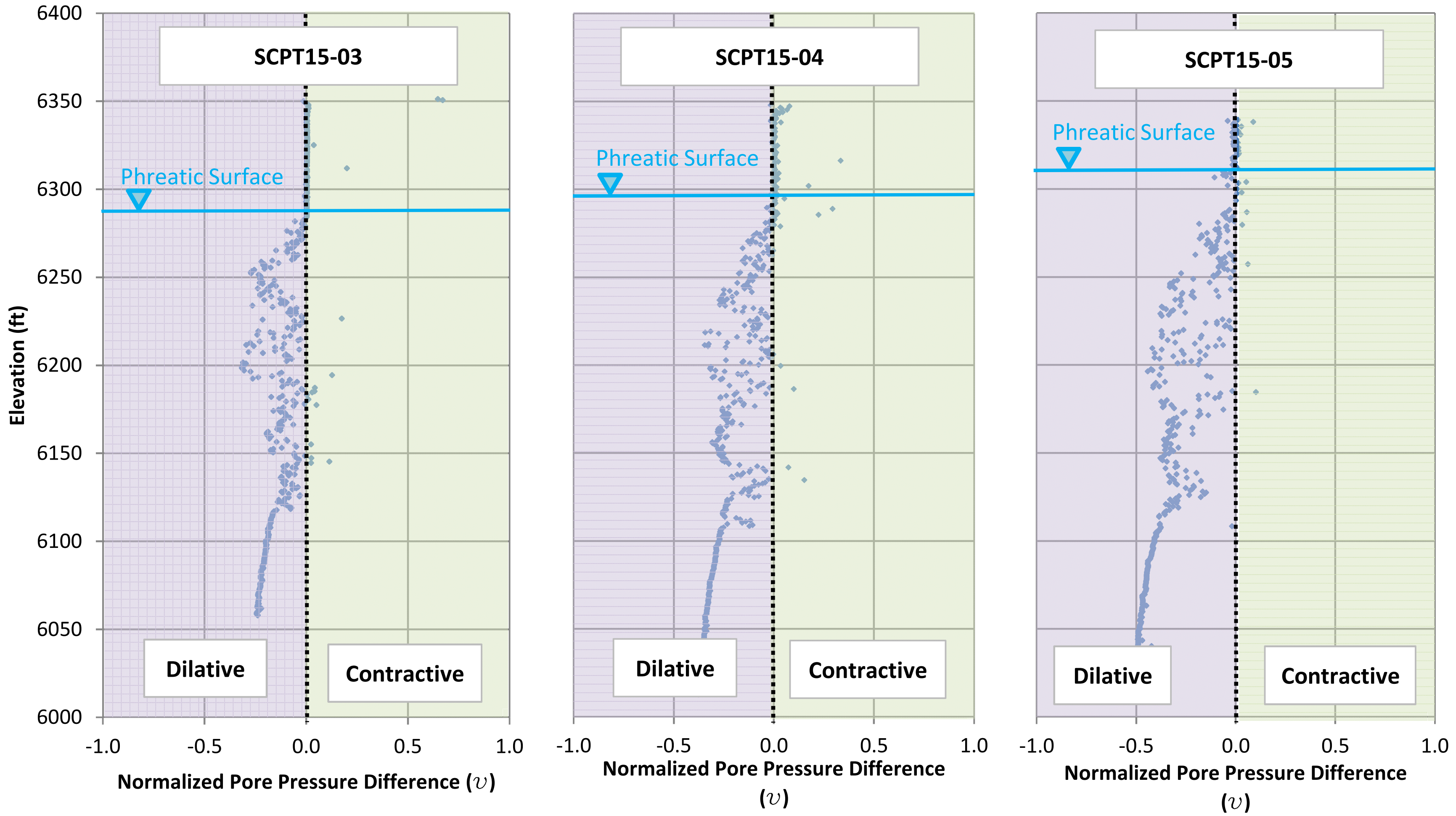
LIQUEFACTION ASSESSMENT

Appendix A1	Tailings State Characterization
Appendix A2	Select Results from CLiq using Idriss and Boulanger (2014) Method
Appendix A3	Select Results from CLiq Using Robertson (2009) Method
Appendix A4	Liquefaction Assessment for the Yankee Doodle Tailings Impoundment

APPENDIX A1

TAILINGS STATE CHARACTERIZATION

(Pages A1-1 to A1-9)



NOTES:
1. ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. NORMALIZED PORE PRESSURE DIFFERENCE (υ) IS CALCULATED AS THE DIFFERENCE BETWEEN DYNAMIC AND STATIC PORE PRESSURES DIVIDED BY THE EFFECTIVE VERTICAL STRESS.
3. MATERIALS WITH A NEGATIVE υ ARE POTENTIALLY DILATIVE AND THOSE WITH A POSITIVE υ ARE POTENTIALLY CONTRACTIVE.
4. THE PHREATIC SURFACE SHOWN FOR EACH SOUNDING IS FROM NOVEMBER, 2015.

0	21JUL'17	ISSUED WITH REPORT	KTD	DDF
REV	DATE	DESCRIPTION	PREP'D	RVW'D

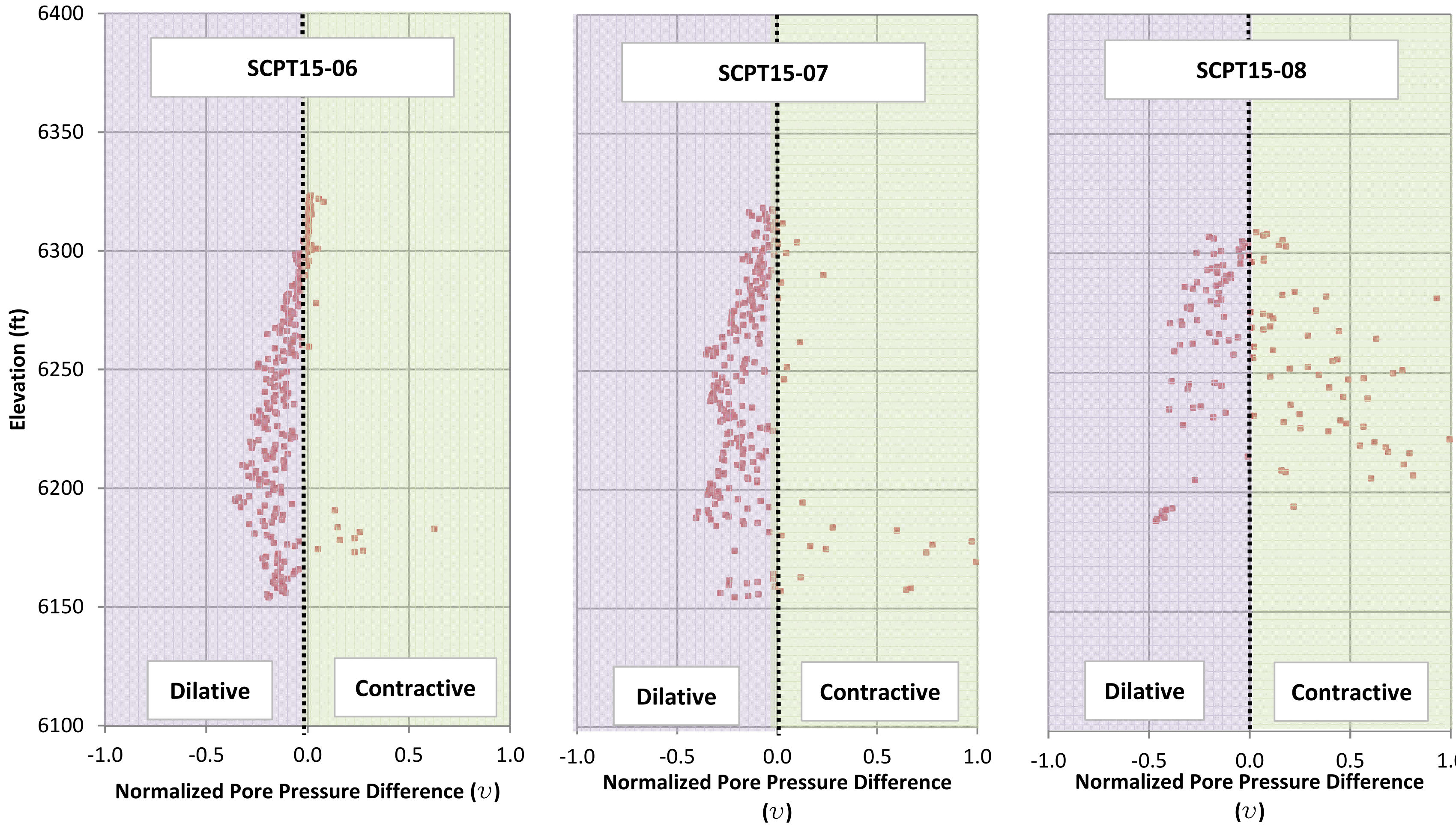
MONTANA RESOURCES, LLP.
YANKEE DOODLE TAILINGS IMPOUNDMENT

NORMALIZED PORE PRESSURE DIFFERENCE
TAILINGS BEACH

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/12	2

FIGURE A1.1 REV 0



NOTES:

1. ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. NORMALIZED PORE PRESSURE DIFFERENCE (υ) IS CALCULATED AS THE DIFFERENCE BETWEEN DYNAMIC AND STATIC PORE PRESSURES DIVIDED BY THE EFFECTIVE VERTICAL STRESS.
3. MATERIALS WITH A NEGATIVE υ ARE POTENTIALLY DILATIVE AND THOSE WITH A POSITIVE υ ARE POTENTIALLY CONTRACTIVE.

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YANKEE DOODLE TAILINGS IMPOUNDMENT

NORMALIZED PORE PRESSURE DIFFERENCE
TAILINGS BENEATH WASTE ROCK SURCHARGE

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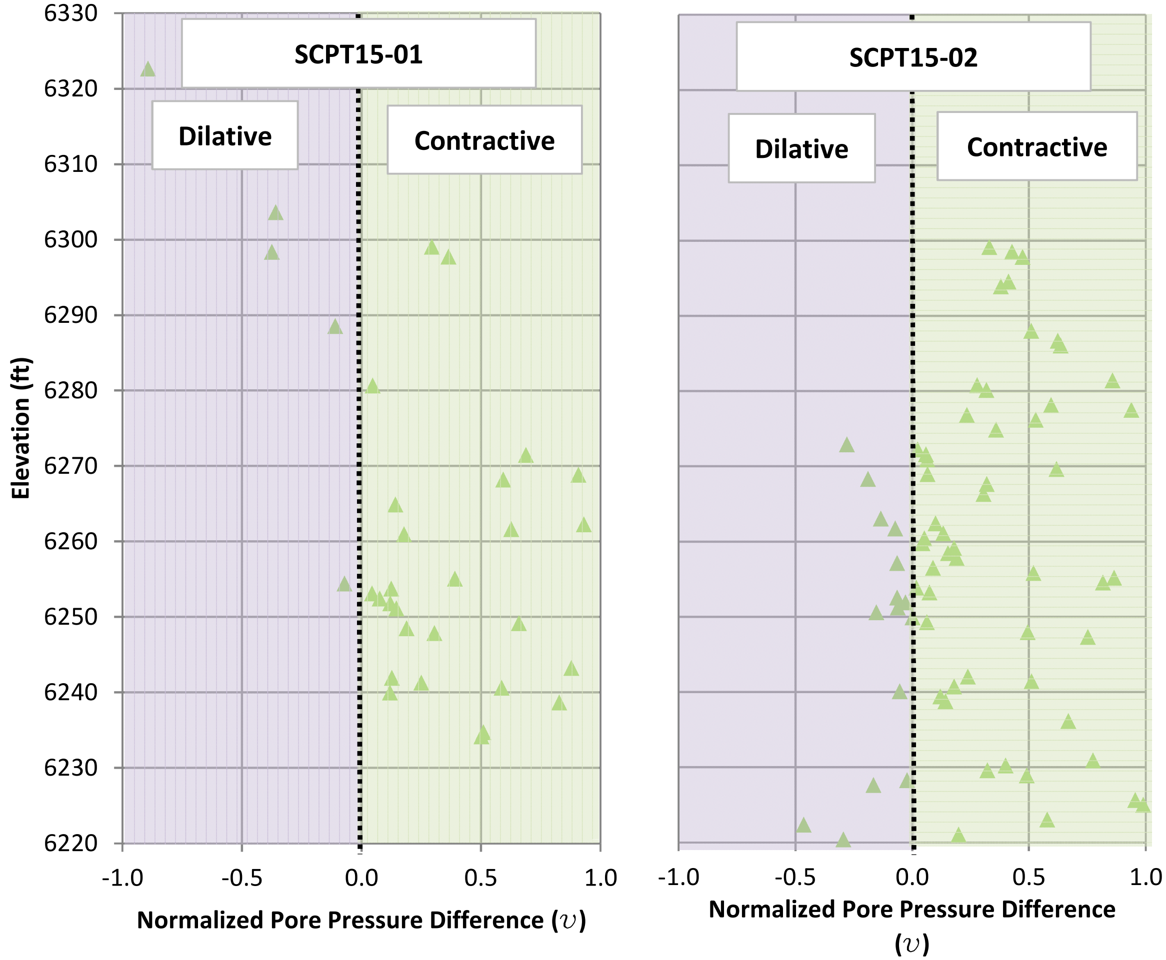
REF. NO. 2

FIGURE A1.2

REV 0



NOTES:

1. ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. NORMALIZED PORE PRESSURE DIFFERENCE (υ) IS CALCULATED AS THE DIFFERENCE BETWEEN DYNAMIC AND STATIC PORE PRESSURES DIVIDED BY THE EFFECTIVE VERTICAL STRESS.
3. MATERIALS WITH A NEGATIVE υ ARE POTENTIALLY DILATIVE AND THOSE WITH A POSITIVE υ ARE POTENTIALLY CONTRACTIVE.

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YANKEE DOODLE TAILINGS IMPOUNDMENT

NORMALIZED PORE PRESSURE DIFFERENCE SOUTHERN MARGIN OF TAILINGS POND

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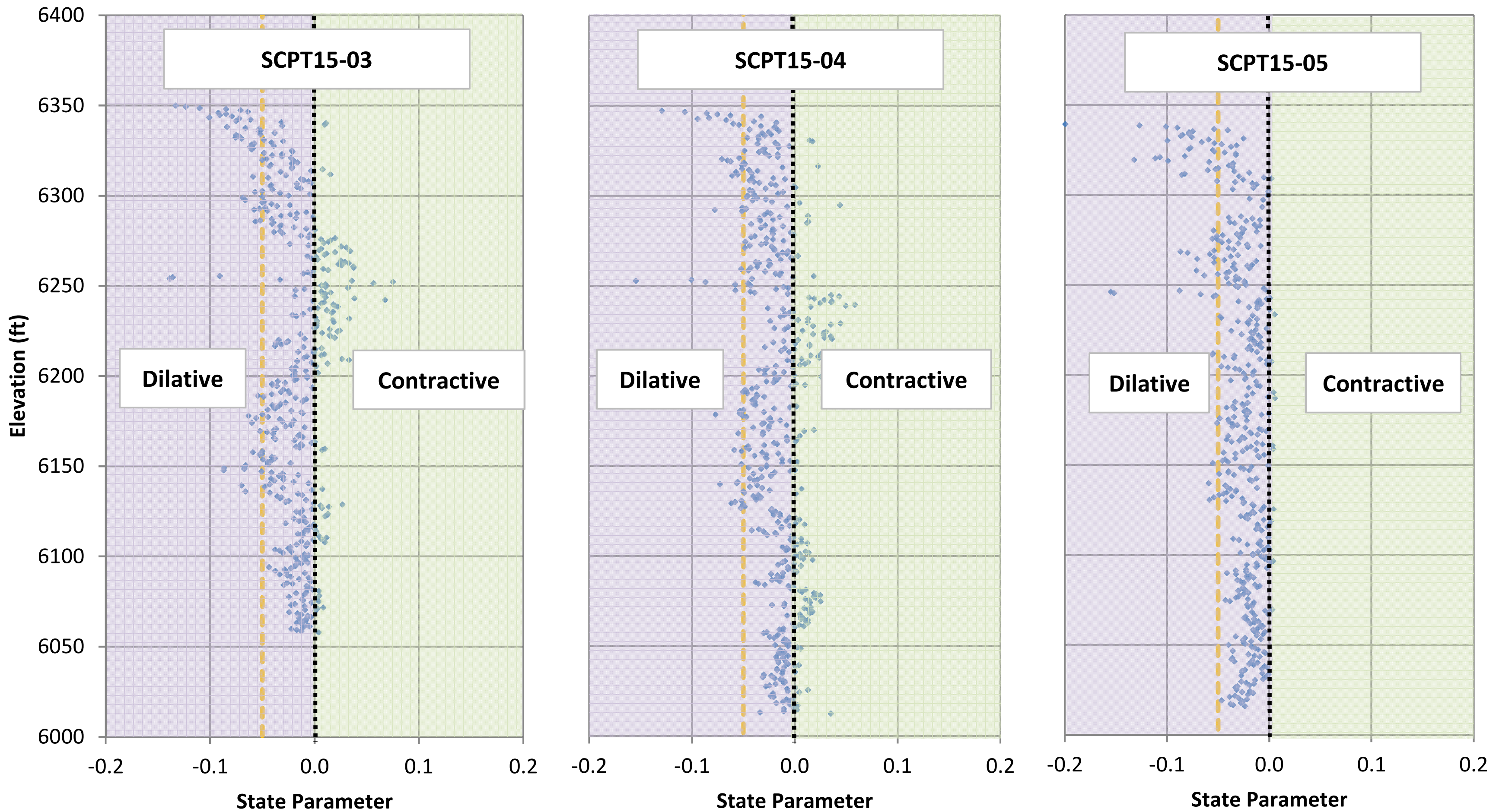
P/A NO. VA101-126/12

REF. NO. 2

FIGURE A1.3

REV 0



NOTES:

1. ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. STATE PARAMETER IS CALCULATED AS THE DIFFERENCE BETWEEN IN-SITU VOID RATIO AND VOID RATIO AT CRITICAL STATE.
3. MATERIALS WITH A NEGATIVE STATE PARAMETER ARE POTENTIALLY DILATIVE AND THOSE WITH A POSITIVE STATE PARAMETER ARE POTENTIALLY CONTRACTIVE.
4. THE YELLOW-DASHED LINE SHOWS AN ALTERNATIVE AXIS DIVISION (ψ= - 0.05) BETWEEN POTENTIALLY DILATIVE AND CONTRACTIVE MATERIALS AS PRESENTED IN

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MONTANA RESOURCES, LLP.

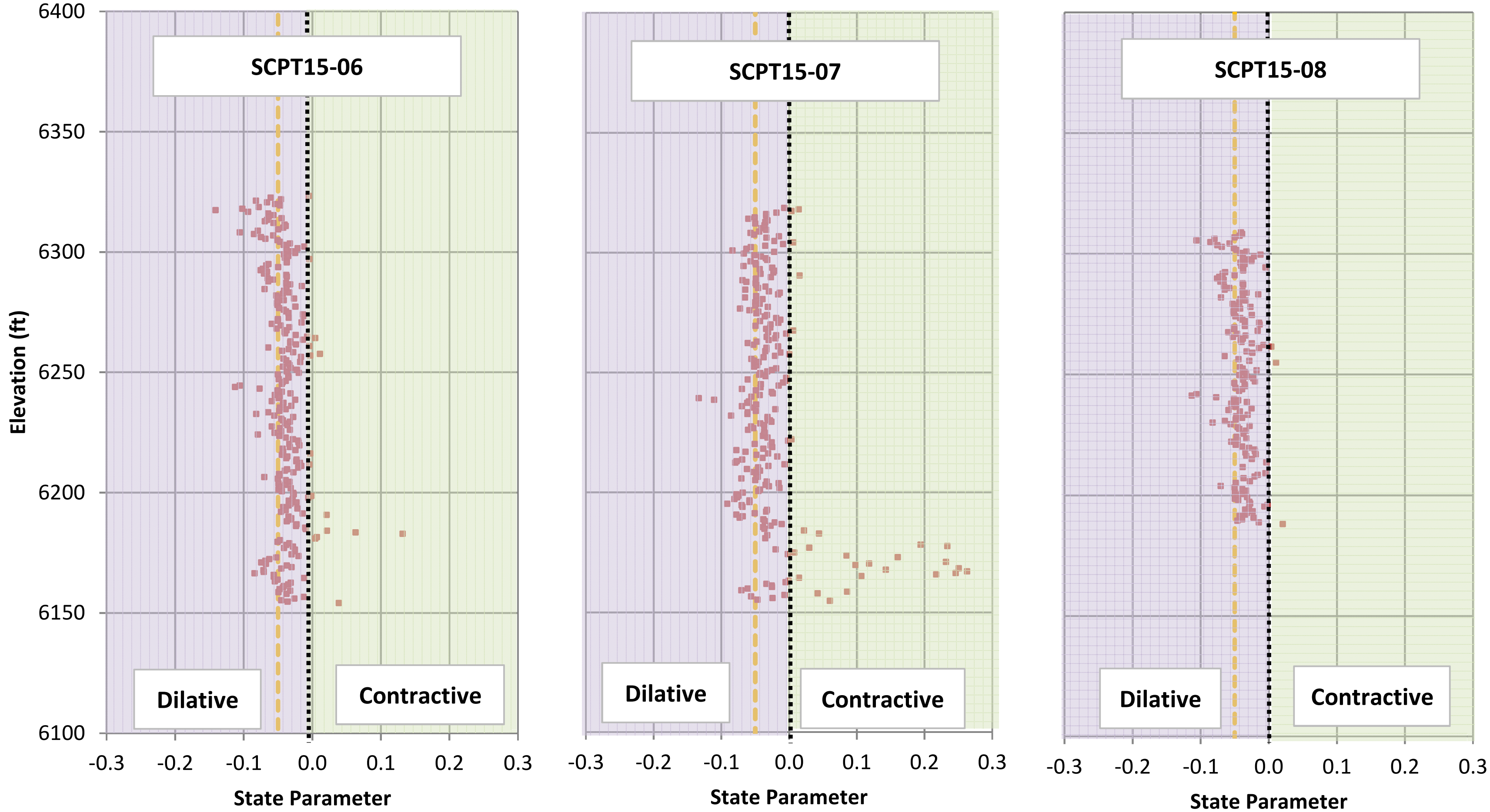
YANKEE DOODLE TAILINGS IMPOUNDMENT

STATE PARAMETER
TAILINGS BEACH

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/12	2

FIGURE A1.4 REV 0



NOTES:

1. ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. STATE PARAMETER IS CALCULATED AS THE DIFFERENCE BETWEEN IN-SITU VOID RATIO AND VOID RATIO AT CRITICAL STATE.
3. MATERIALS WITH A NEGATIVE STATE PARAMETER ARE POTENTIALLY DILATIVE AND THOSE WITH A POSITIVE STATE PARAMETER ARE POTENTIALLY CONTRACTIVE.
4. THE YELLOW-DASHED LINE SHOWS AN ALTERNATIVE AXIS DIVISION (ψ = - 0.05) BETWEEN POTENTIALLY DILATIVE AND CONTRACTIVE MATERIALS AS PRESENTED IN WINCKLER ET. AL. (2014)

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MONTANA RESOURCES, LLP.
YANKEE DOODLE TAILINGS IMPOUNDMENT

STATE PARAMETER
TAILINGS BENEATH WASTE ROCK SURCHARGE

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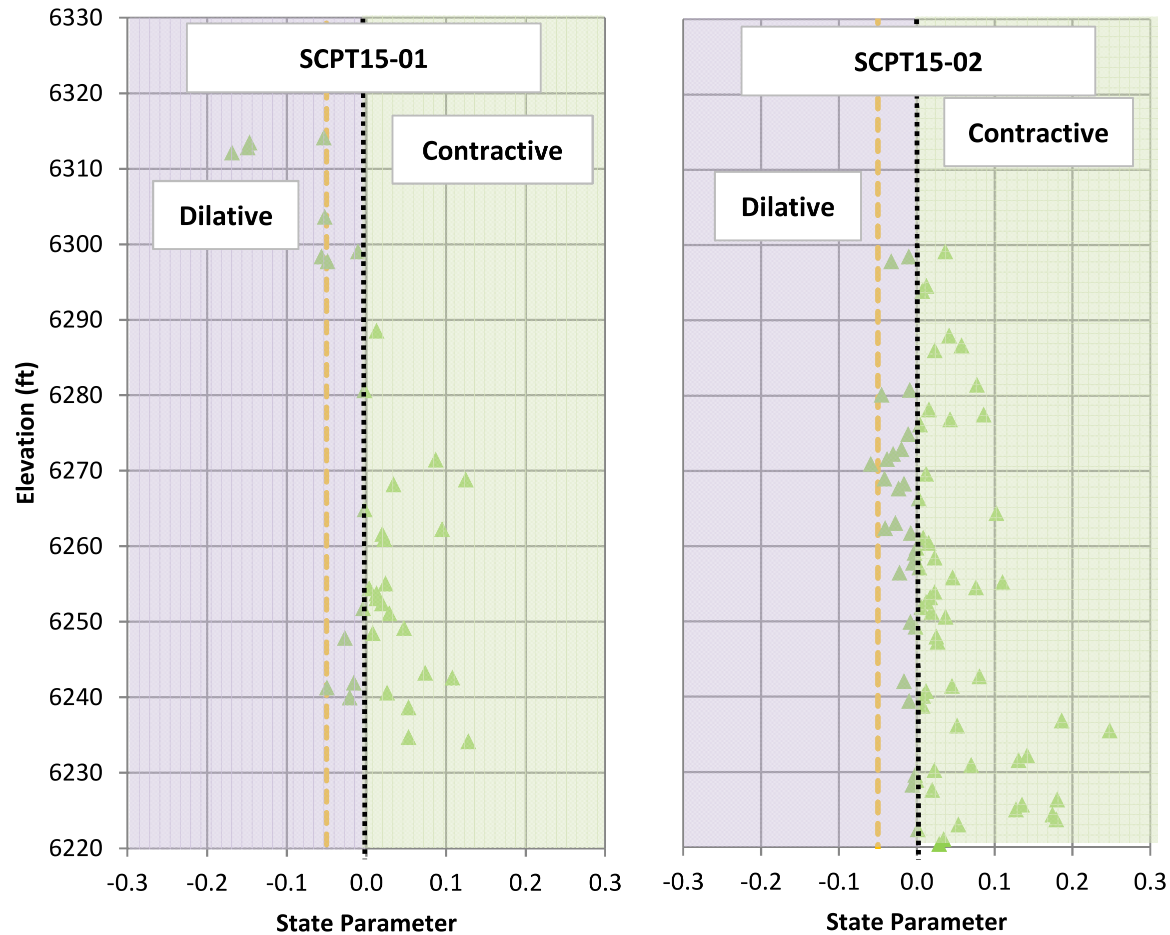
P/A NO. VA101-126/12
REF. NO. 2

FIGURE A1.5
REV 0



NOTES:

1. ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. STATE PARAMETER IS CALCULATED AS THE DIFFERENCE BETWEEN IN-SITU VOID RATIO AND VOID RATIO AT CRITICAL STATE.
3. MATERIALS WITH A NEGATIVE STATE PARAMETER ARE POTENTIALLY DILATIVE AND THOSE WITH A POSITIVE STATE PARAMETER ARE POTENTIALLY CONTRACTIVE.
4. THE YELLOW-DASHED LINE SHOWS AN ALTERNATIVE AXIS DIVISION (ψ= - 0.05) BETWEEN POTENTIALLY DILATIVE AND CONTRACTIVE MATERIALS AS PRESENTED IN WINCKLER ET. AL. (2014)

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MONTANA RESOURCES, LLP.

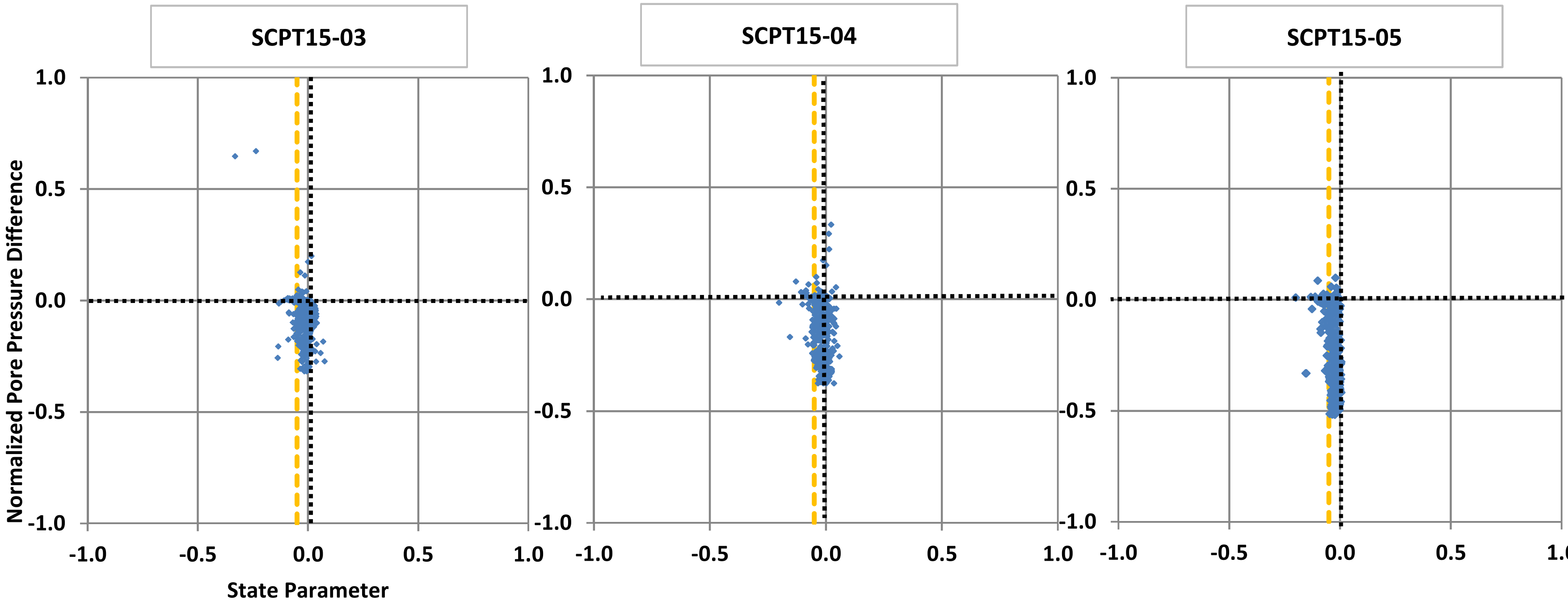
YANKEE DOODLE TAILINGS IMPOUNDMENT

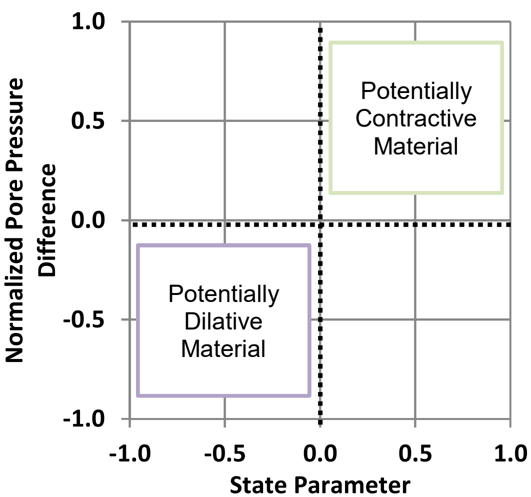
STATE PARAMETER
SOUTHERN MARGIN OF TAILINGS POND

Knight Piésold CONSULTING

P/A NO. VA101-126/12 | REF. NO. 2

FIGURE A1.6 | REV 0





NOTES:

1. MATERIALS WITH A POSITIVE STATE PARAMETER AND POSITIVE NORMALIZED PORE PRESSURE DIFFERENCE VALUES PLOT ARE POTENTIALLY CONTRACTIVE. MATERIALS WITH A NEGATIVE STATE PARAMETER AND NEGATIVE NORMALIZED PORE PRESSURE DIFFERENCE ARE POTENTIALLY DILATIVE.
2. THE YELLOW-DASHED LINE SHOWS AN ALTERNATIVE AXIS DIVISION (y= - 0.05) BETWEEN POTENTIALLY DILATIVE AND CONTRACTIVE MATERIALS AS PRESENTED IN WINCKLER ET. AL. (2014).

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YANKEE DOODLE TAILINGS IMPOUNDMENT

STATE PARAMETER VS. NORMALIZED PORE PRESSURE DIFFERENCE TAILINGS BEACH

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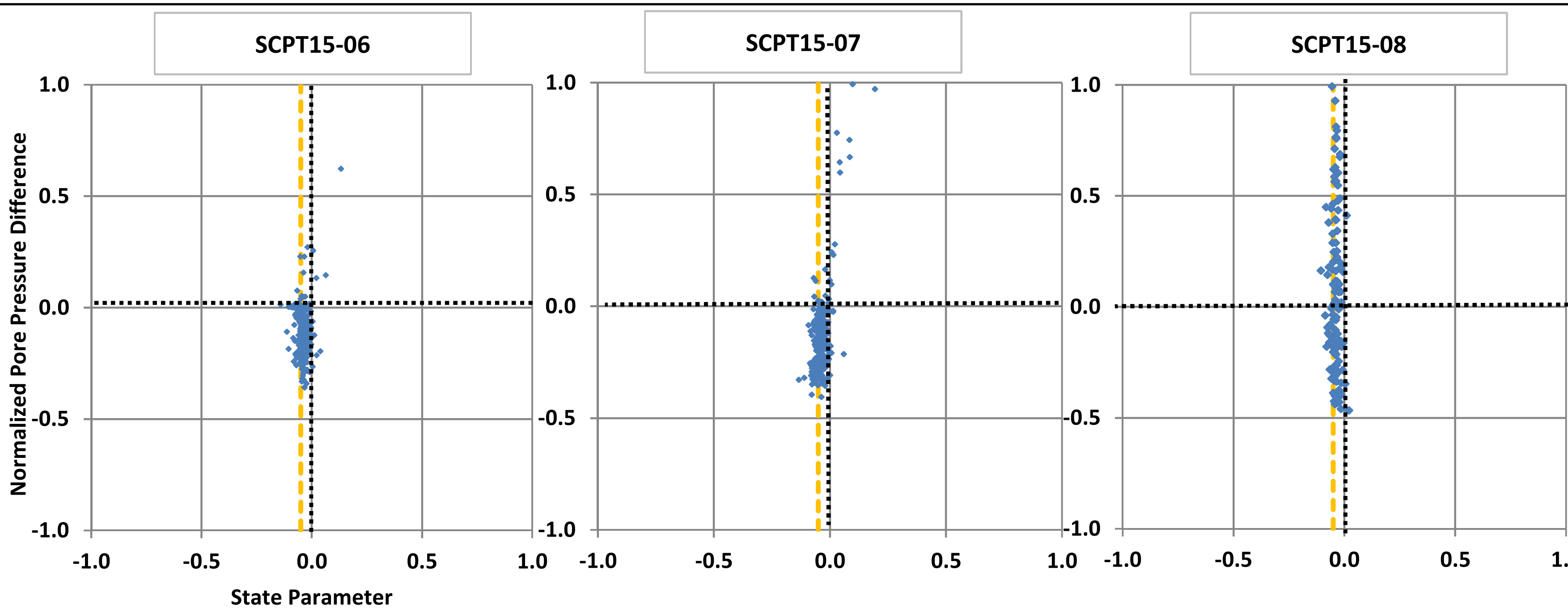
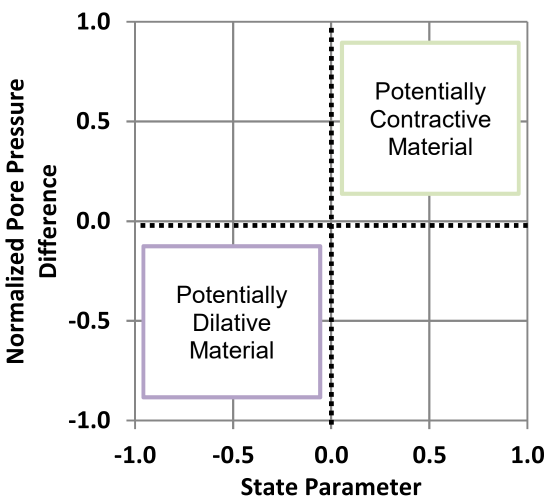
P/A NO.	REF. NO.
VA101-126/12	2

FIGURE A1.7 REV 0

0	21JUL'17	ISSUED WITH REPORT	KTD	DDF
REV	DATE	DESCRIPTION	PREP'D	RVW'D





NOTES:

1. MATERIALS WITH A POSITIVE STATE PARAMETER AND POSITIVE NORMALIZED PORE PRESSURE DIFFERENCE VALUES PLOT ARE POTENTIALLY CONTRACTIVE. MATERIALS WITH A NEGATIVE STATE PARAMETER AND NEGATIVE NORMALIZED PORE PRESSURE DIFFERENCE ARE POTENTIALLY DILATIVE.
2. THE YELLOW-DASHED LINE SHOWS AN ALTERNATIVE AXIS DIVISION (y= - 0.05) BETWEEN POTENTIALLY DILATIVE AND CONTRACTIVE MATERIALS AS PRESENTED IN WINCKLER ET. AL. (2014).

0	21JUL'17	ISSUED WITH REPORT	KTD	DDF
REV	DATE	DESCRIPTION	PREP'D	RVW'D

MONTANA RESOURCES, LLP.

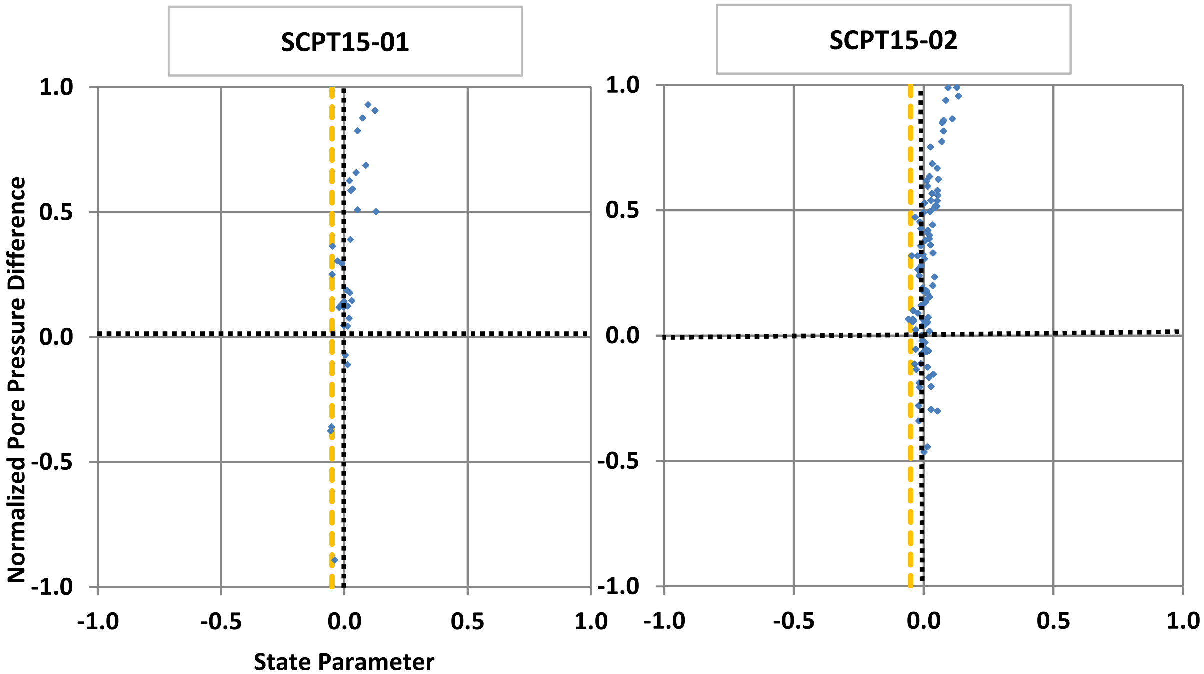
YANKEE DOODLE TAILINGS IMPOUNDMENT

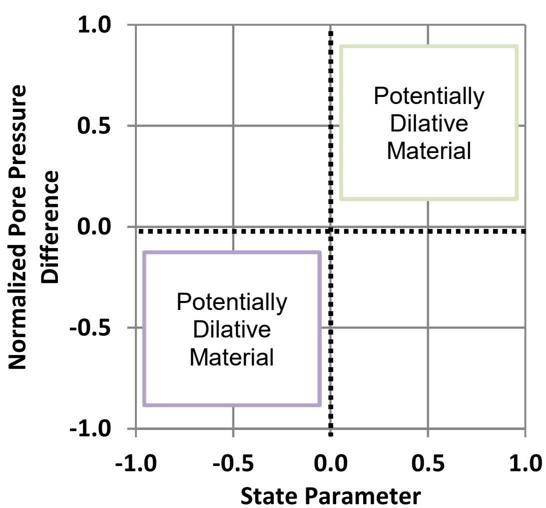
STATE PARAMETER VS. NORMALIZED PORE PRESSURE DIFFERENCE
TAILINGS BENEATH WASTE ROCK SURCHARGE

Knight Piésold CONSULTING

P/A NO. VA101-126/12 | REF. NO. 2

FIGURE A1.8 | REV 0





NOTES:

1. MATERIALS WITH A POSITIVE STATE PARAMETER AND POSITIVE NORMALIZED PORE PRESSURE DIFFERENCE VALUES PLOT ARE POTENTIALLY CONTRACTIVE. MATERIALS WITH A NEGATIVE STATE PARAMETER AND NEGATIVE NORMALIZED PORE PRESSURE DIFFERENCE ARE POTENTIALLY DILATIVE.
2. THE YELLOW-DASHED LINE SHOWS AN ALTERNATIVE AXIS DIVISION (y= - 0.05) BETWEEN POTENTIALLY DILATIVE AND CONTRACTIVE MATERIALS AS PRESENTED IN WINCKLER ET. AL. (2014).

0	21JUL'17	ISSUED WITH REPORT	KTD	DDF
REV	DATE	DESCRIPTION	PREP'D	RVW'D

MONTANA RESOURCES, LLP.

YANKEE DOODLE TAILINGS IMPOUNDMENT

STATE PARAMETER VS. NORMALIZED PORE PRESSURE DIFFERENCE
SOUTHERN MARGIN OF TAILINGS POND

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P/A NO. VA101-126/12

REF. NO. 2

FIGURE A1.9

REV 0

APPENDIX A2

SELECT RESULTS FROM CLIQ USING IDRISS AND BOULANGER (2014) METHOD

(Pages A2-1 to A2-10)

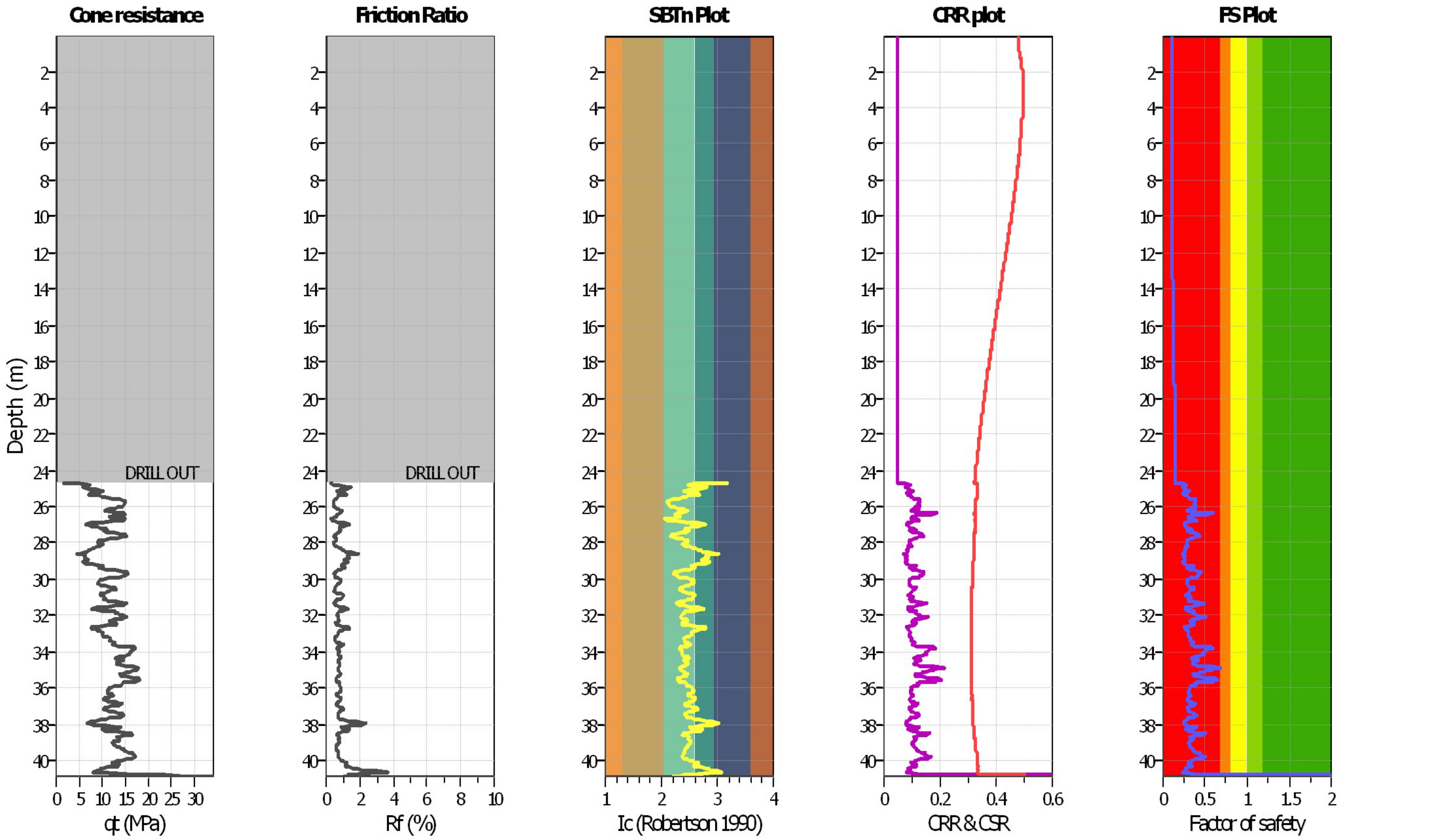
LIQUEFACTION ANALYSIS REPORT

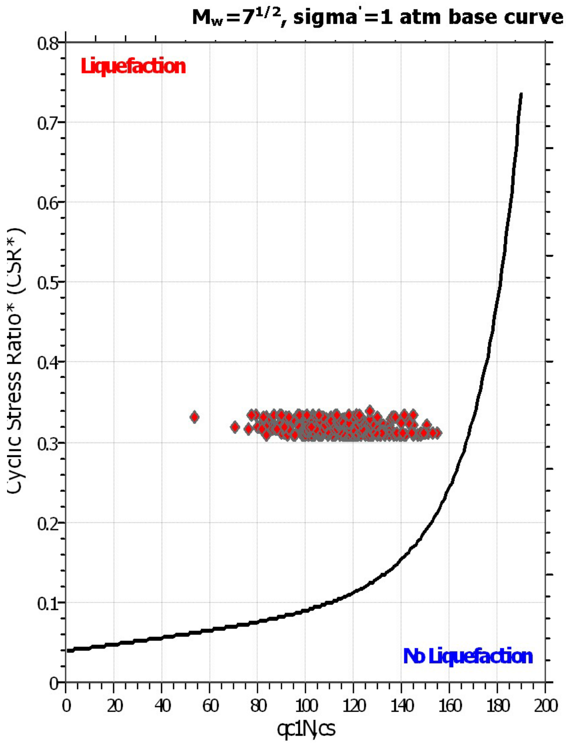
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

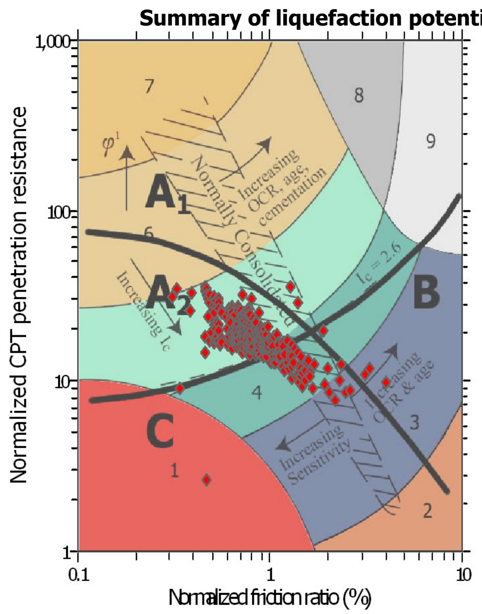
CPT file : CPT12-01A

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

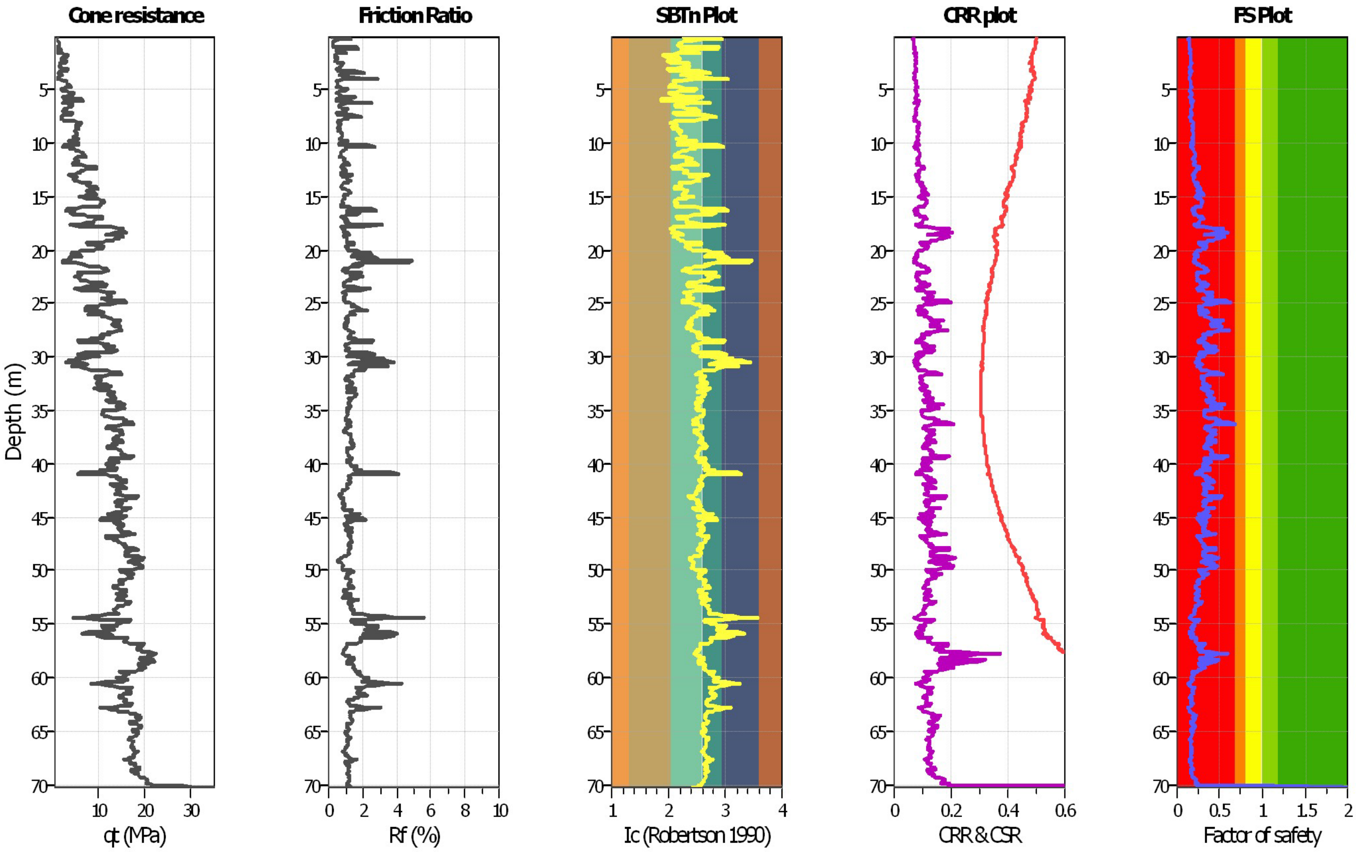
LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

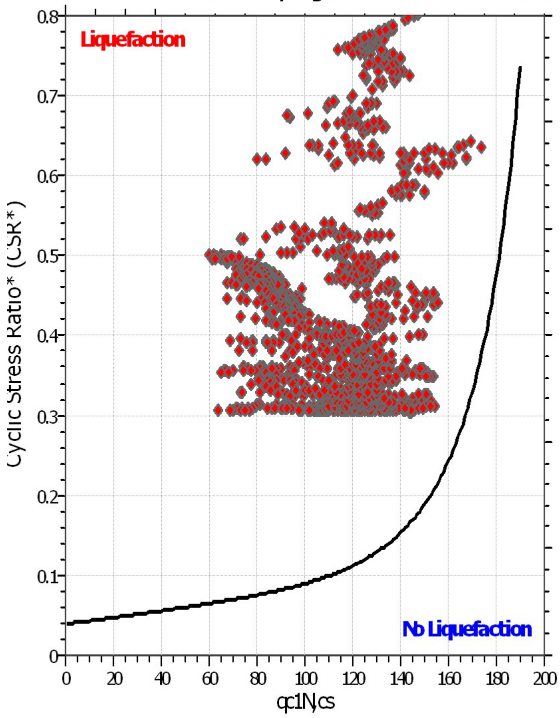
CPT file : CPT12-05

Input parameters and analysis data

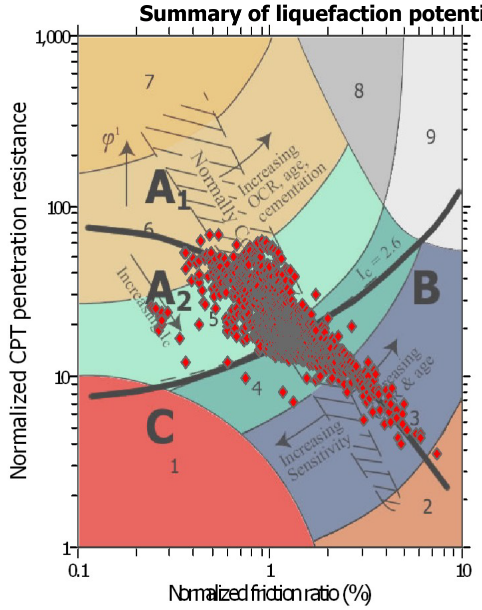
Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

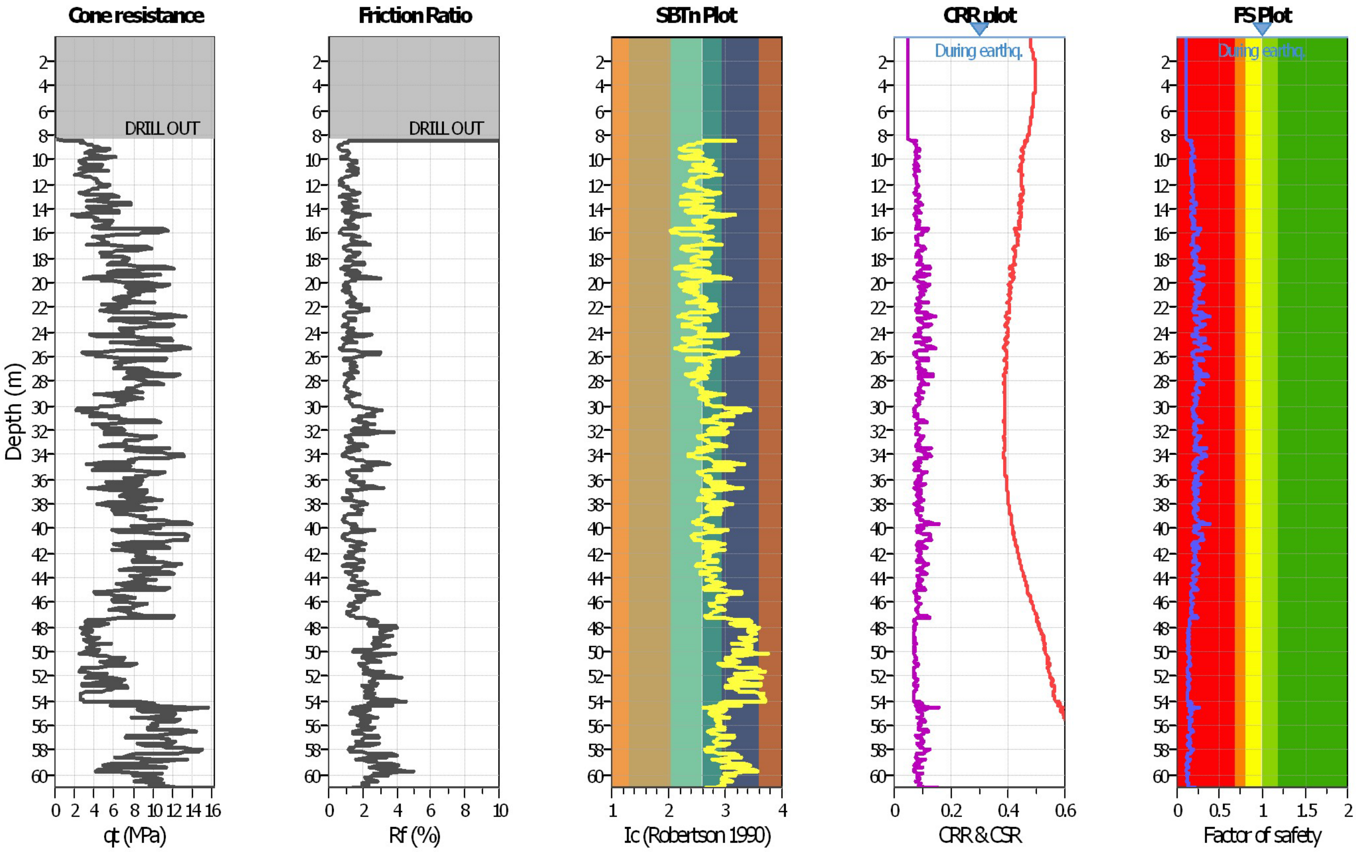
LIQUEFACTION ANALYSIS REPORT

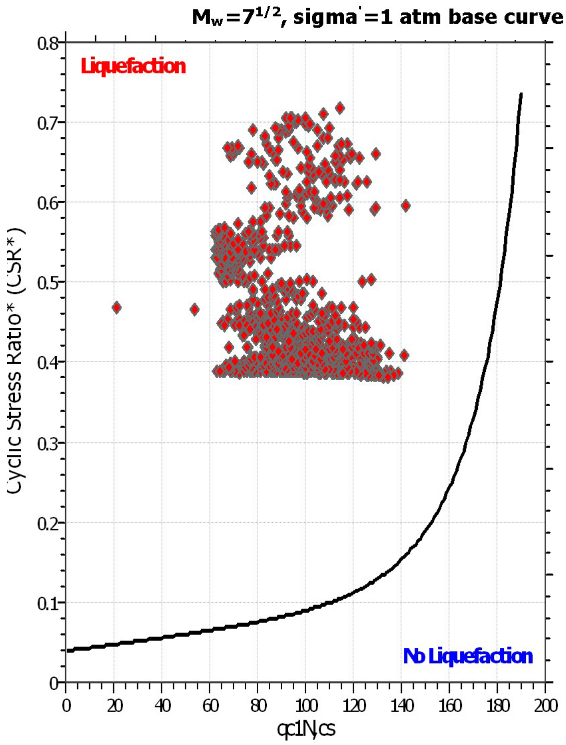
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

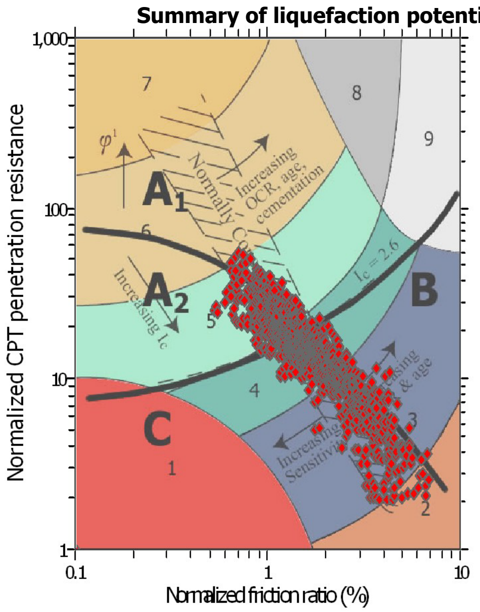
CPT file : CPT13-03

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

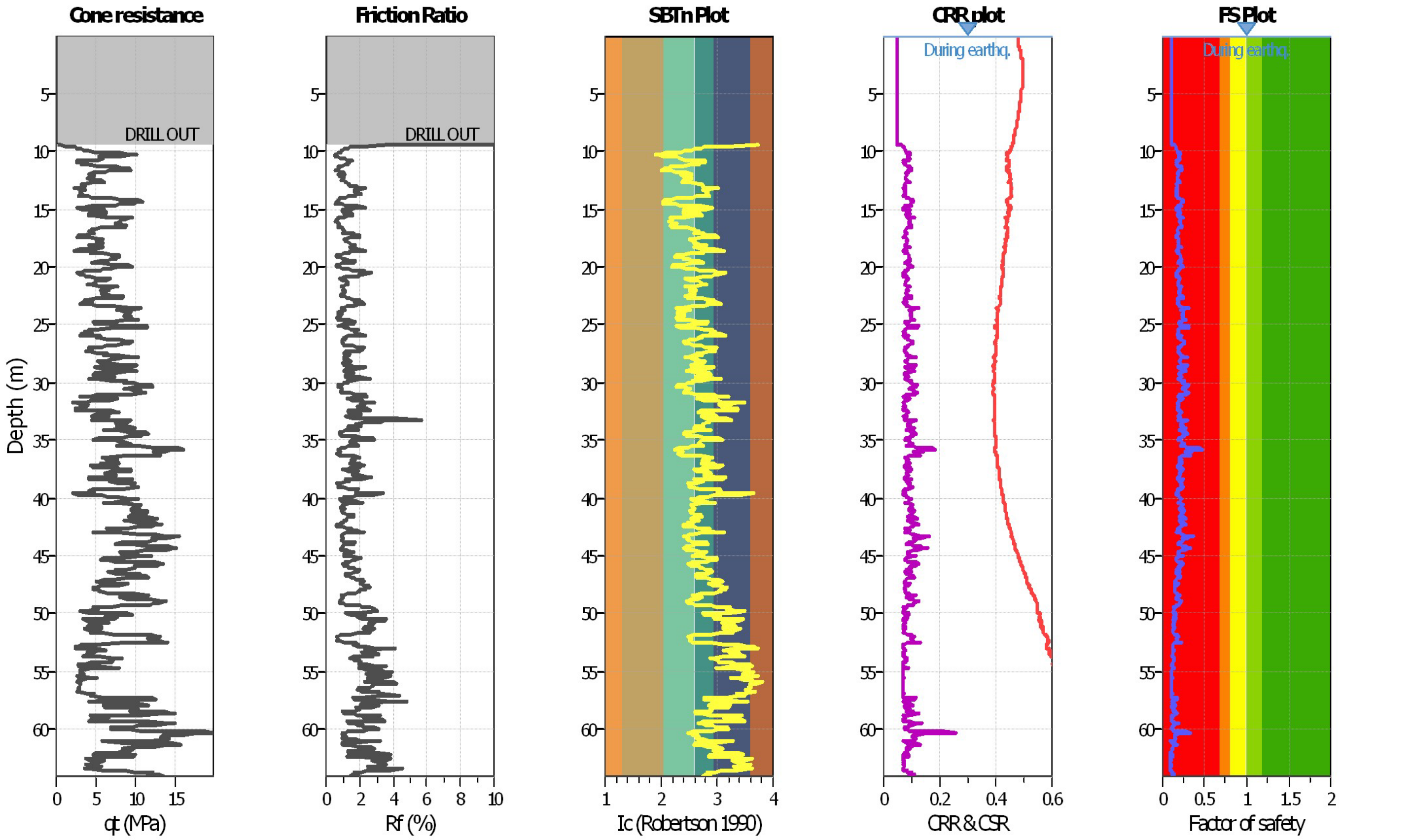
LIQUEFACTION ANALYSIS REPORT

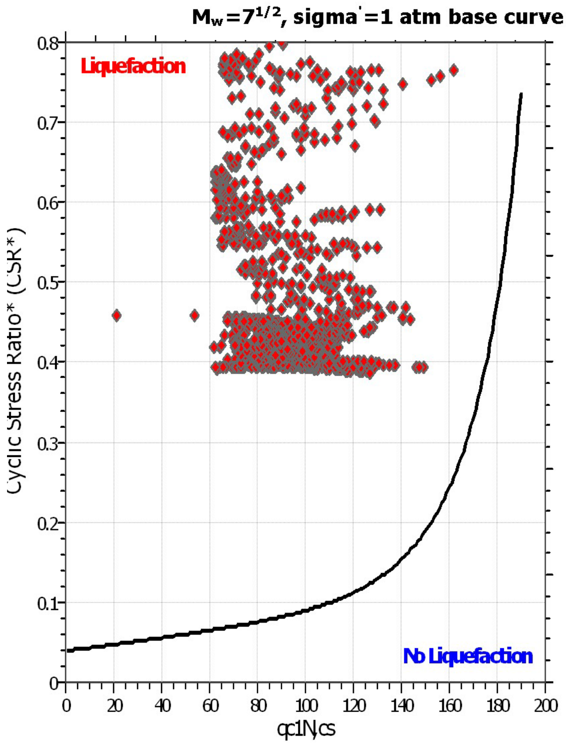
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

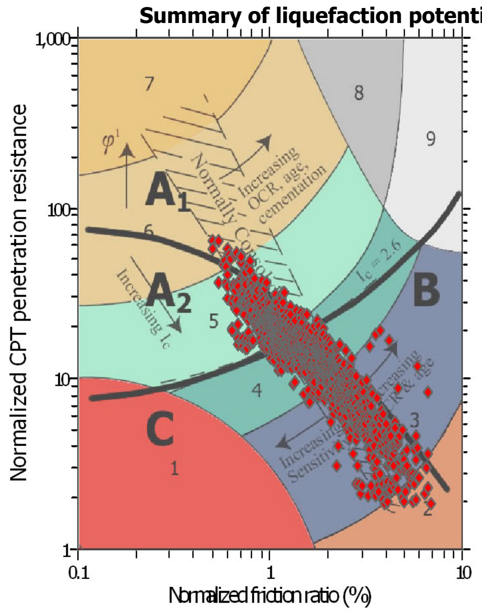
CPT file : CPT13-04

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

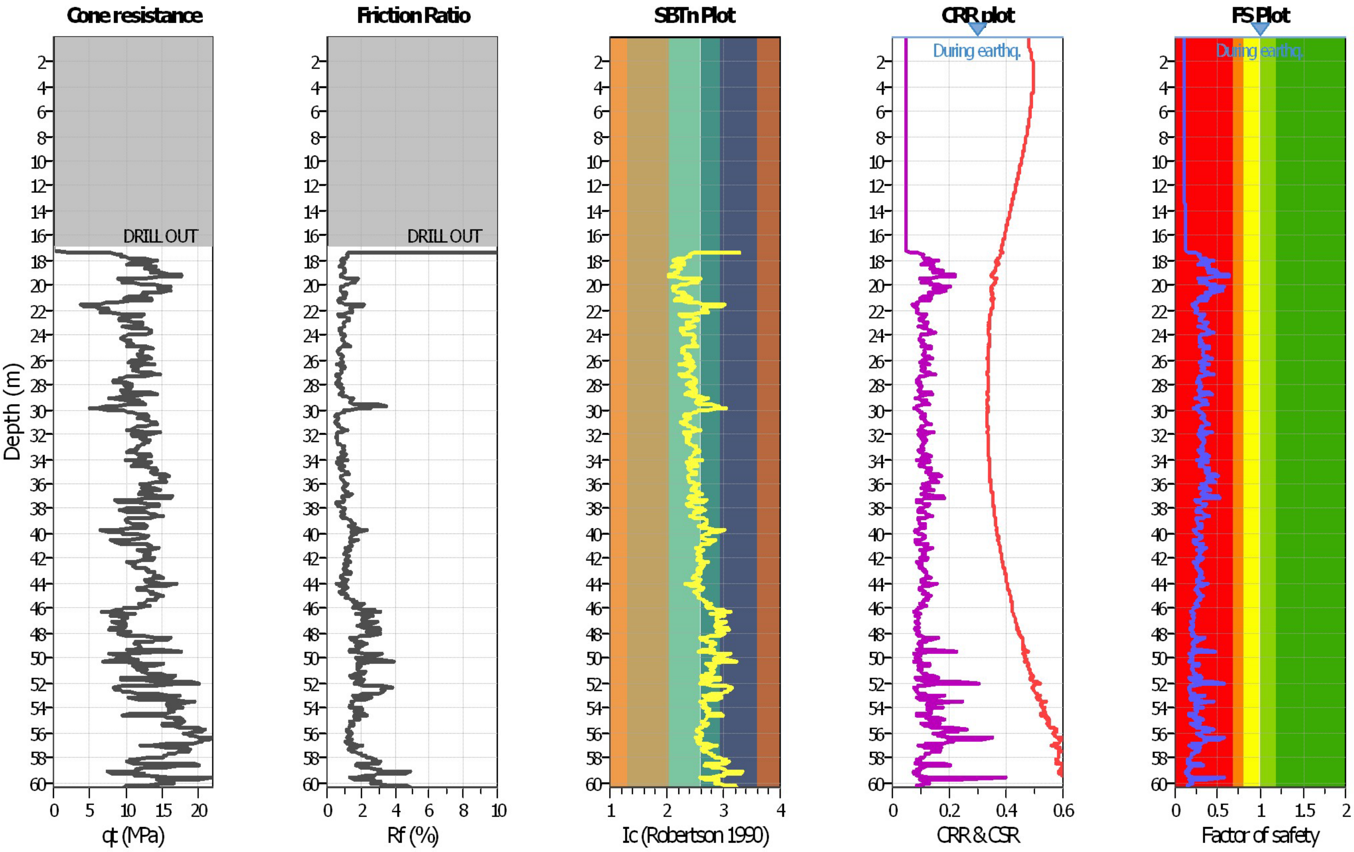
LIQUEFACTION ANALYSIS REPORT

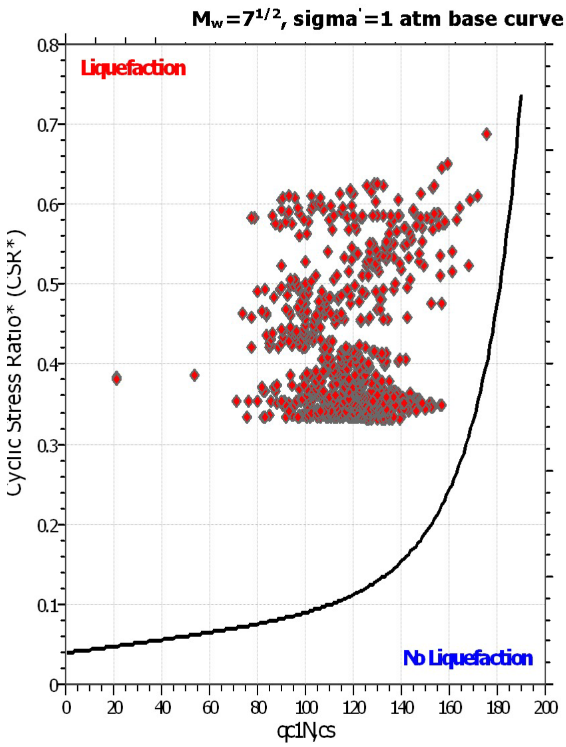
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

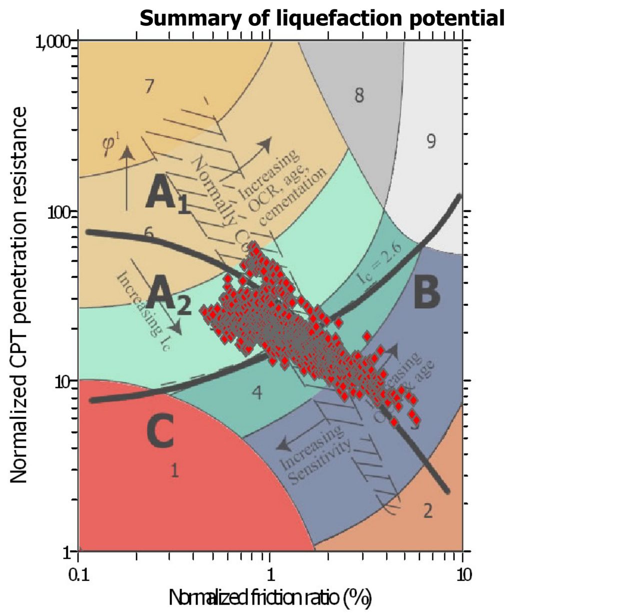
CPT file : CPT14-01A

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

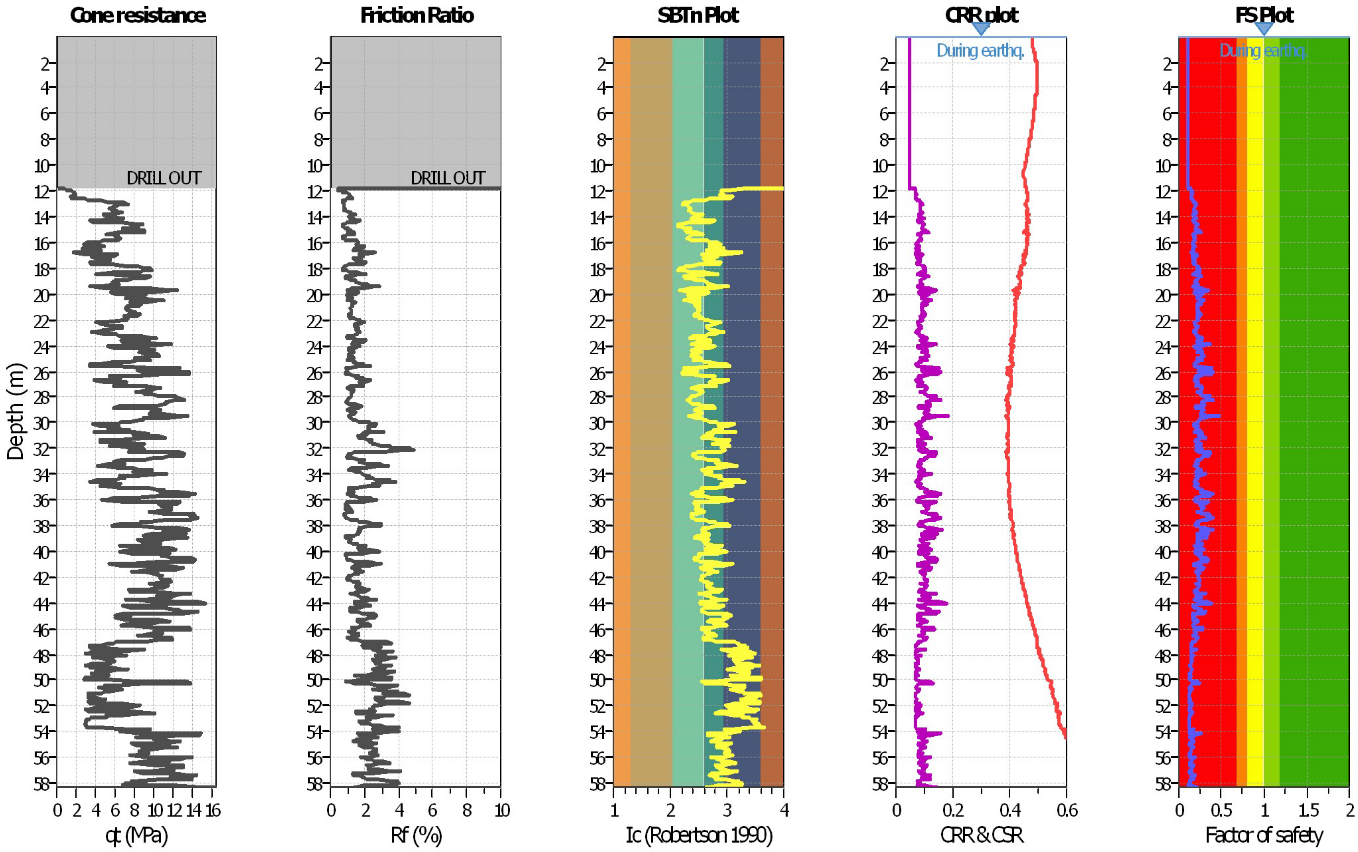
LIQUEFACTION ANALYSIS REPORT

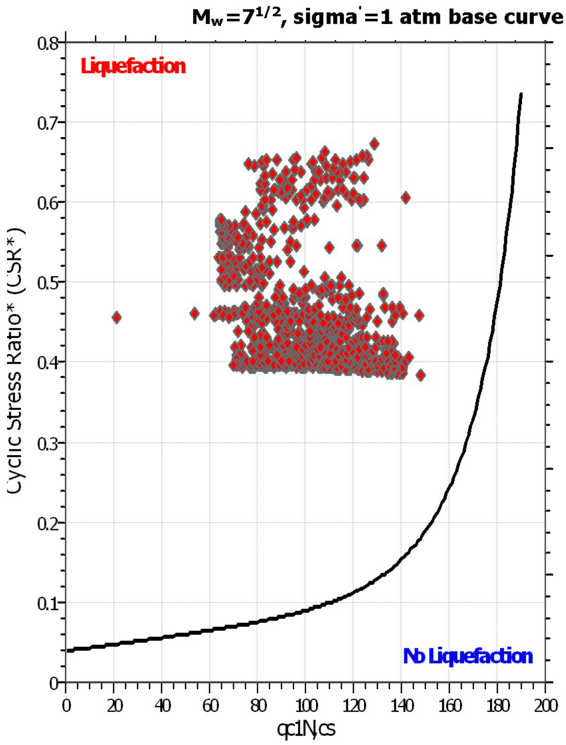
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

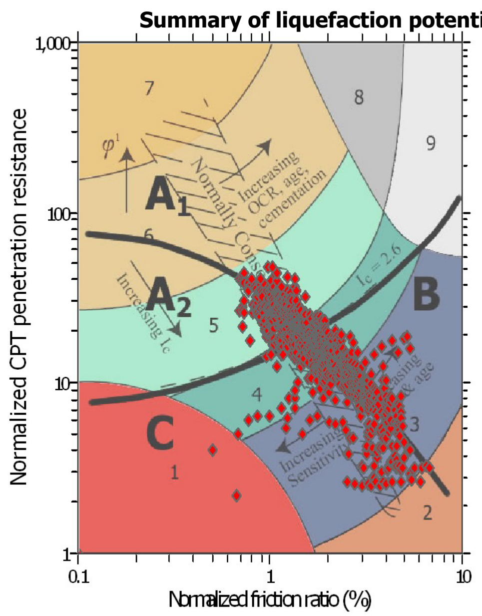
CPT file : CPT14-05

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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www.knightpiesold.com

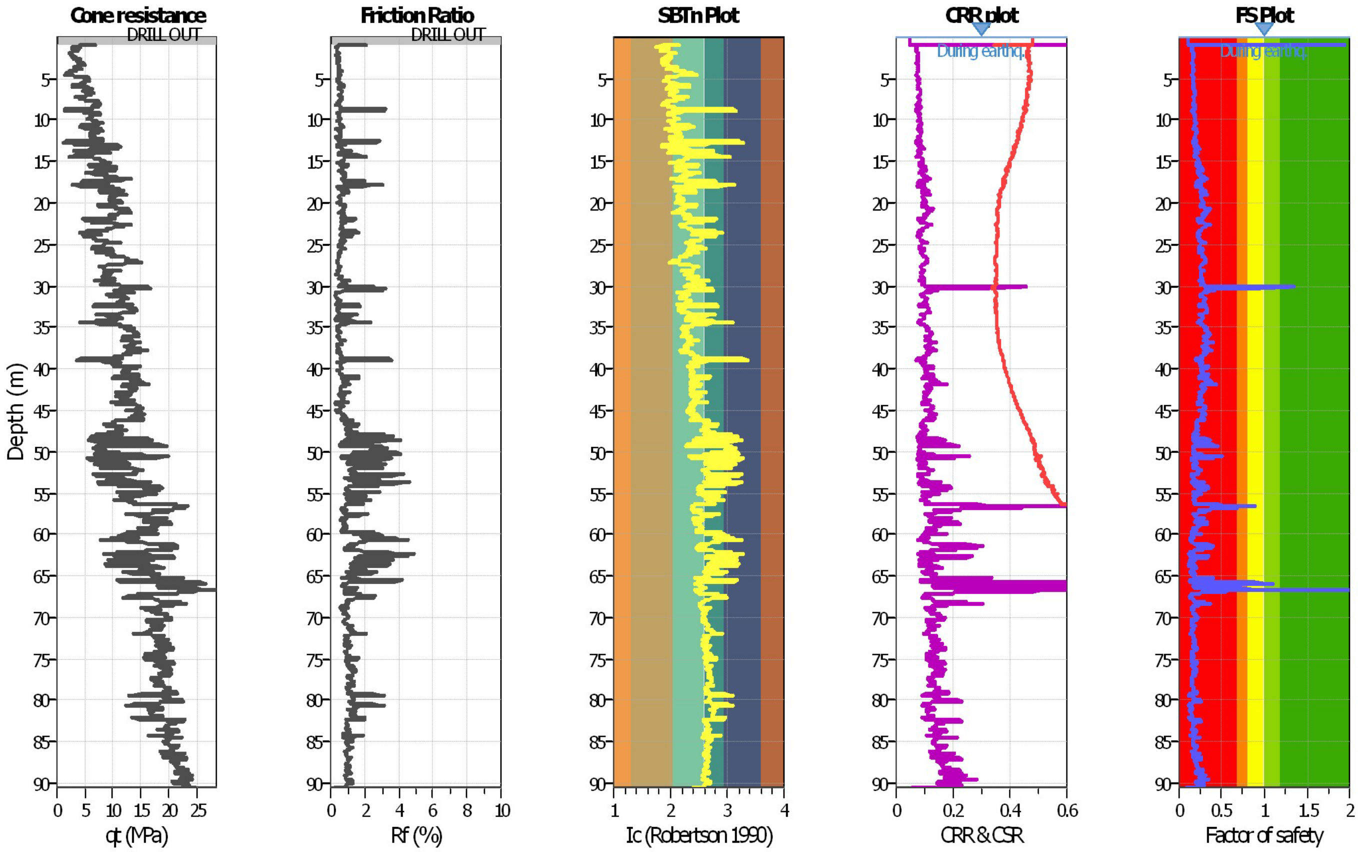
LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

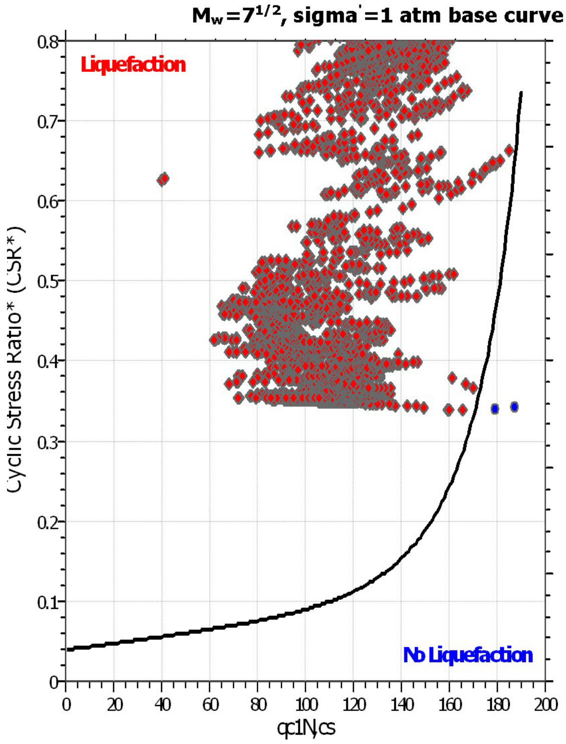
CPT file : CPT15-03

Input parameters and analysis data

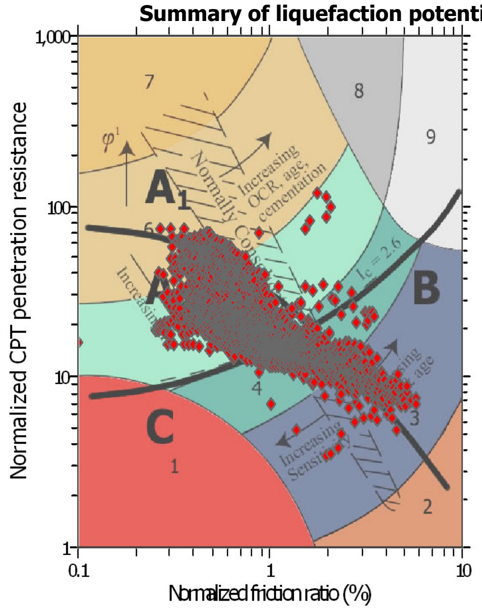
Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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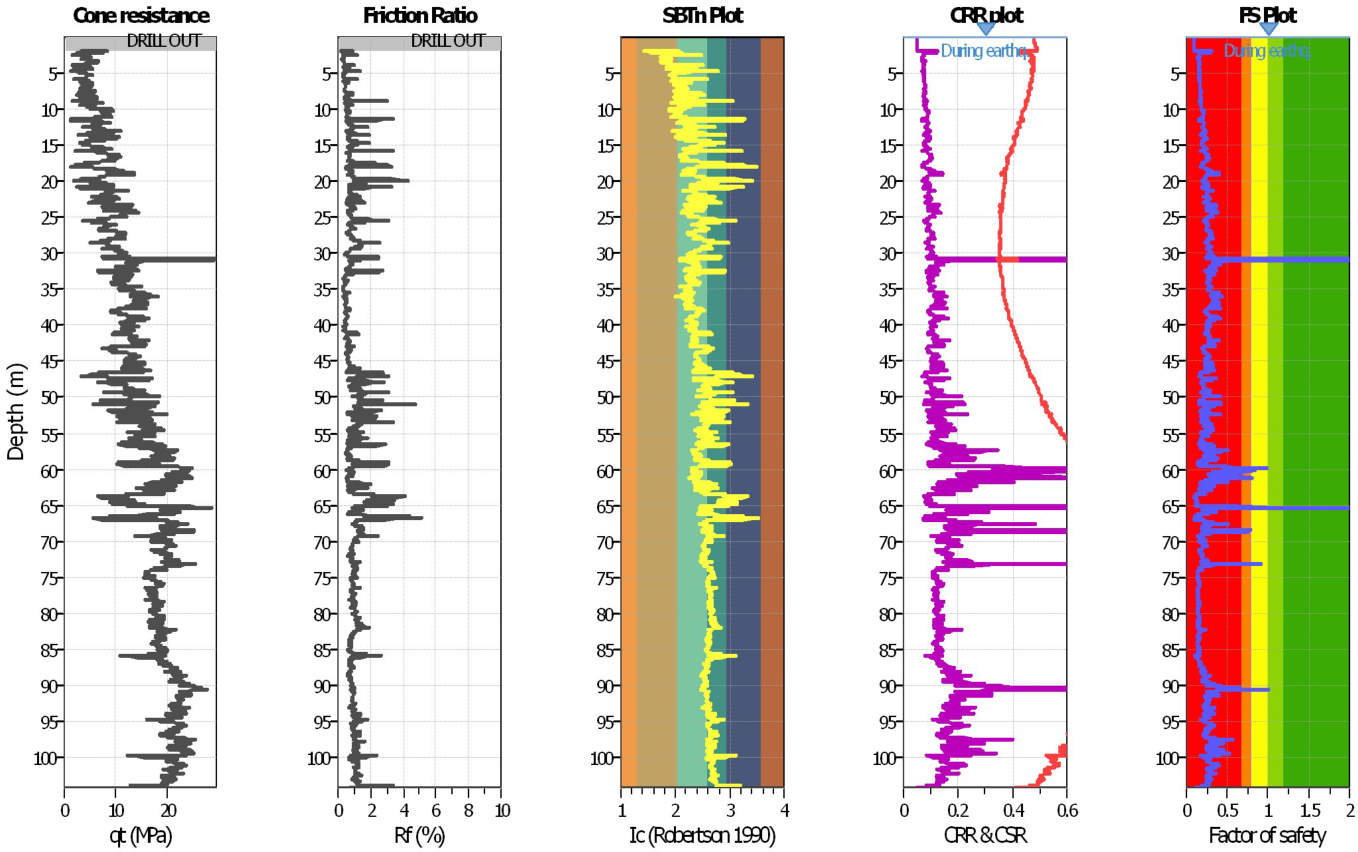
LIQUEFACTION ANALYSIS REPORT

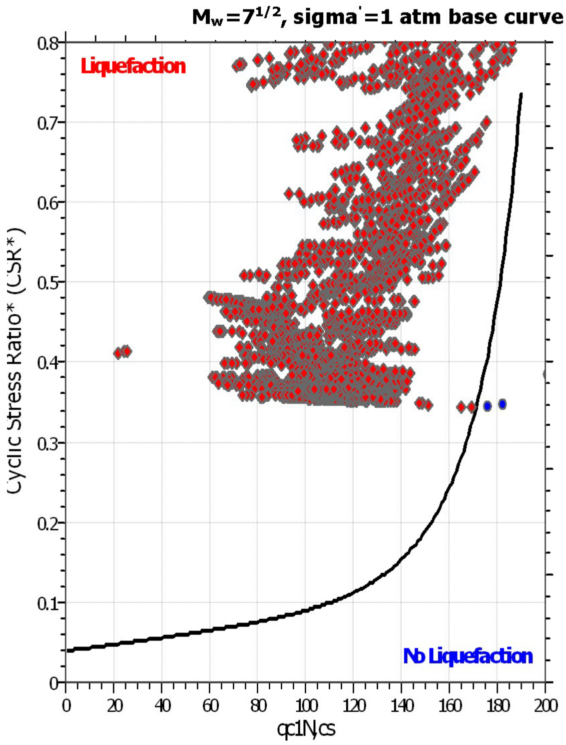
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

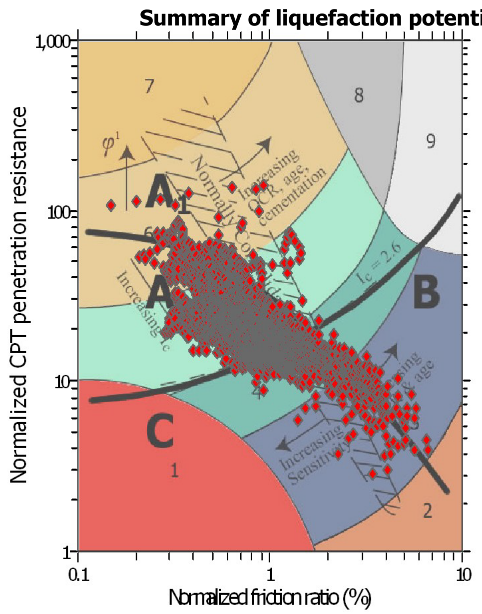
CPT file : CPT15-04

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

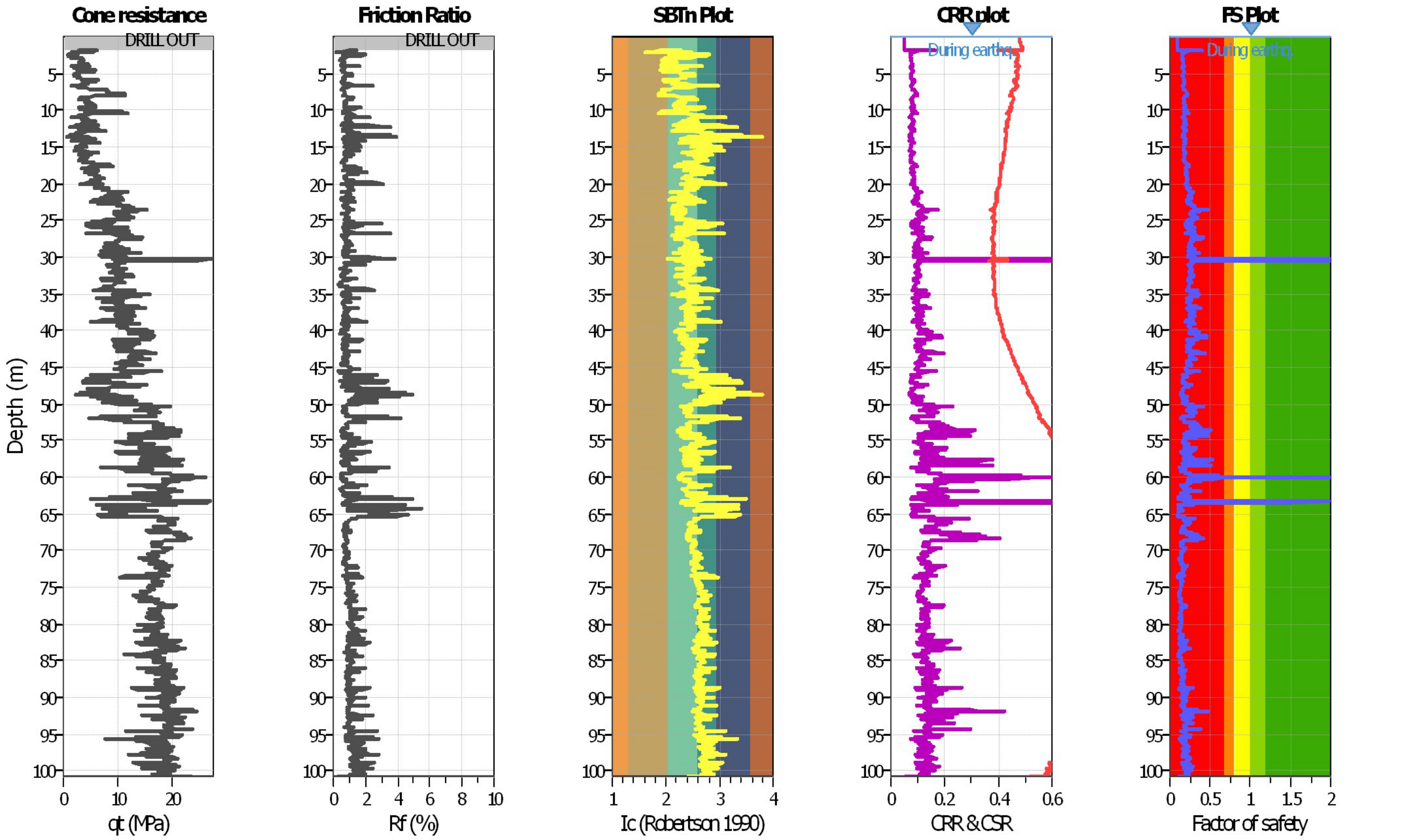
LIQUEFACTION ANALYSIS REPORT

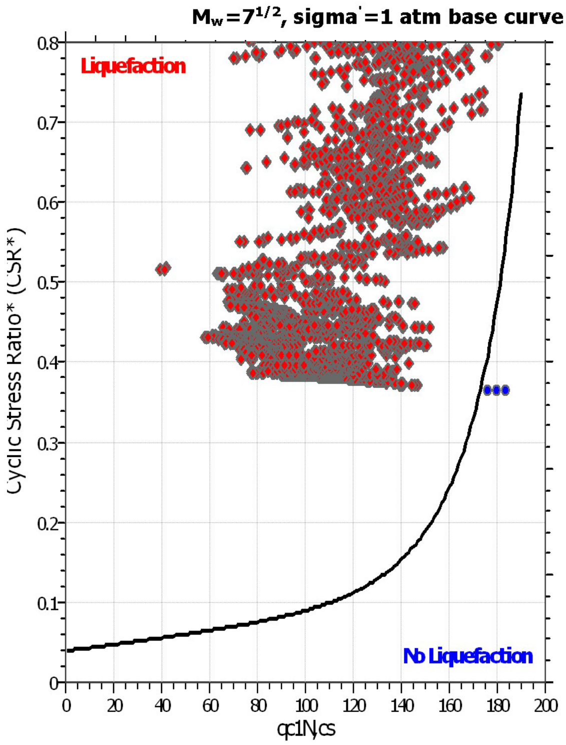
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

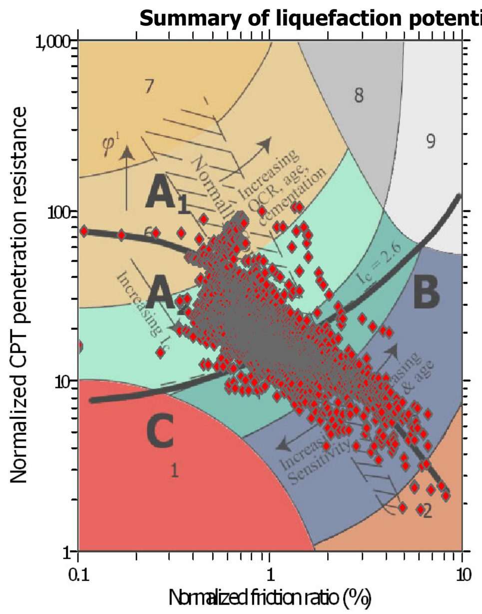
CPT file : CPT15-05

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

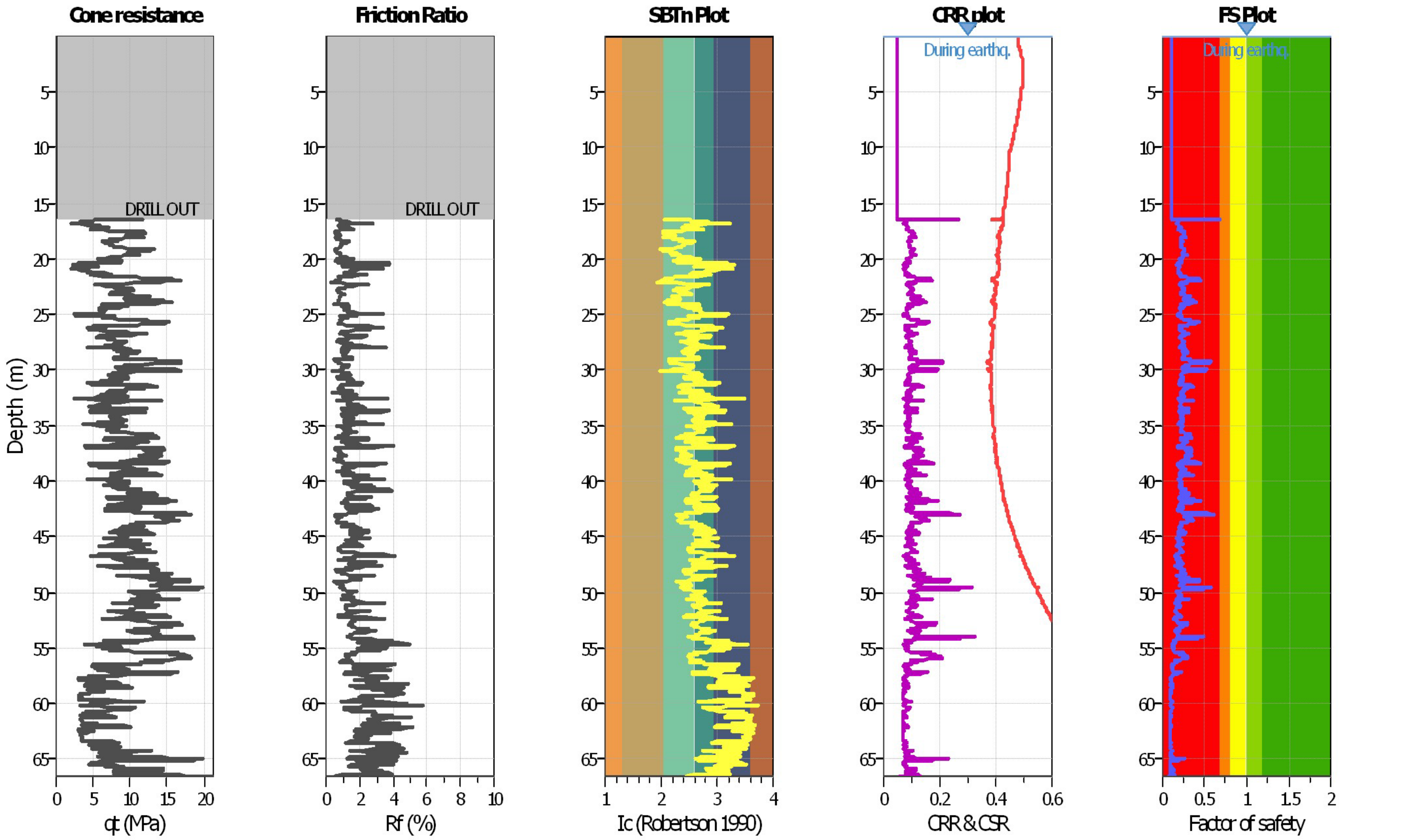
LIQUEFACTION ANALYSIS REPORT

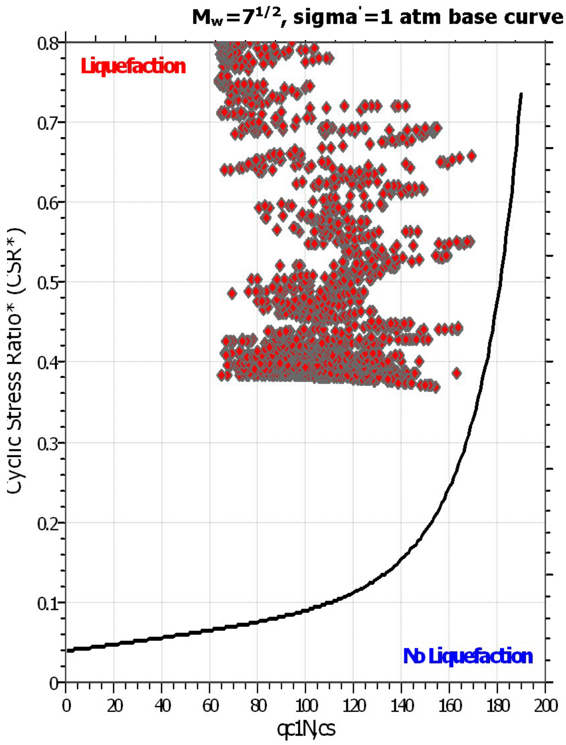
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

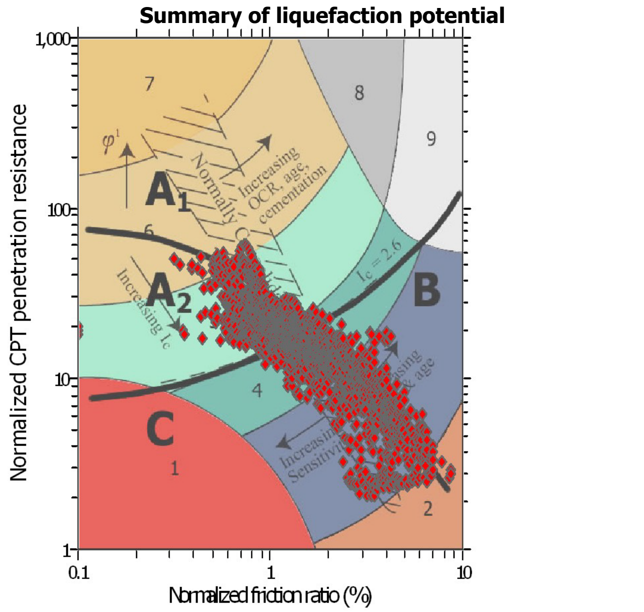
CPT file : CPT15-07

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

APPENDIX A3

SELECT RESULTS FROM CLIQ USING ROBERTSON (2009) METHOD

(Pages A3-1 to A3-10)

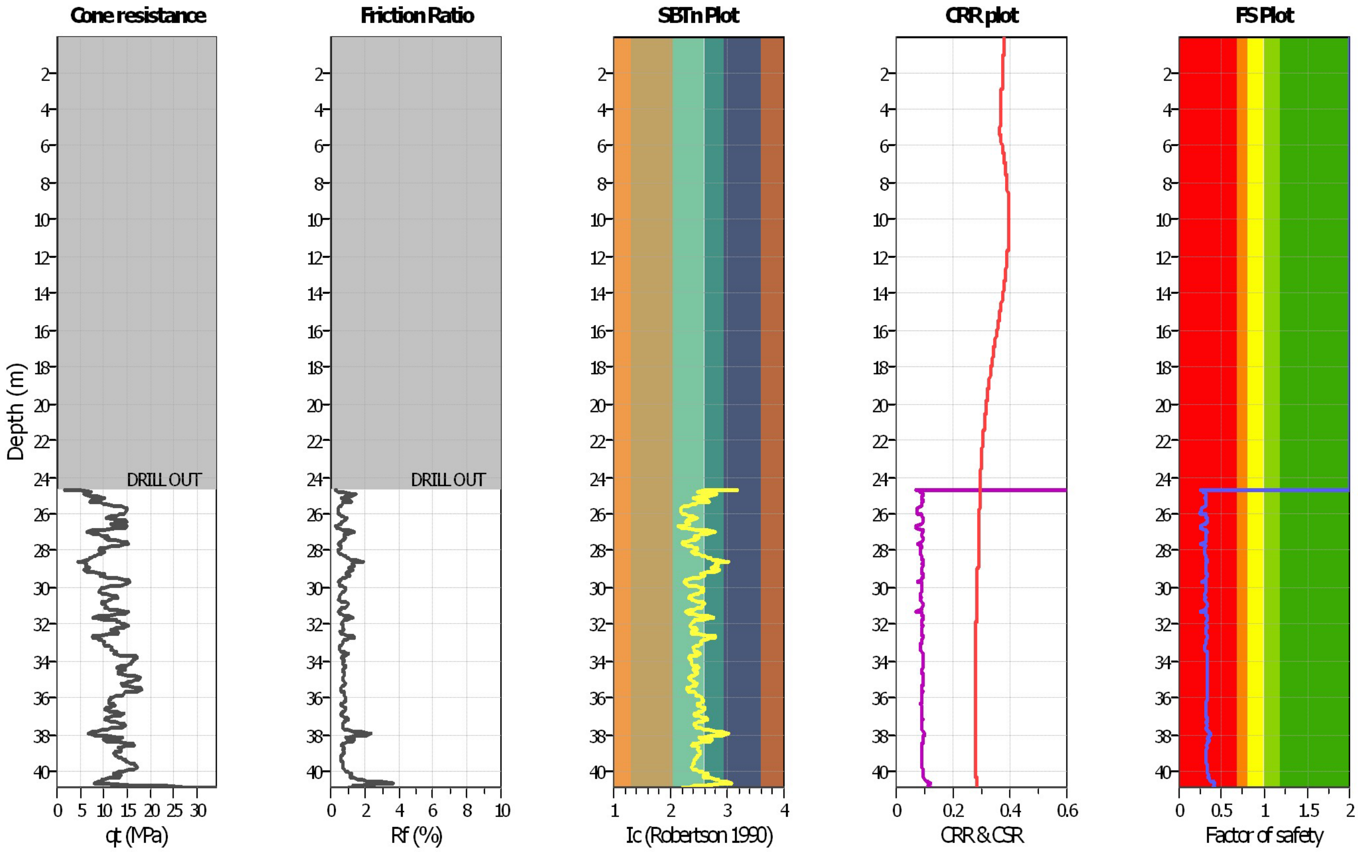
LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

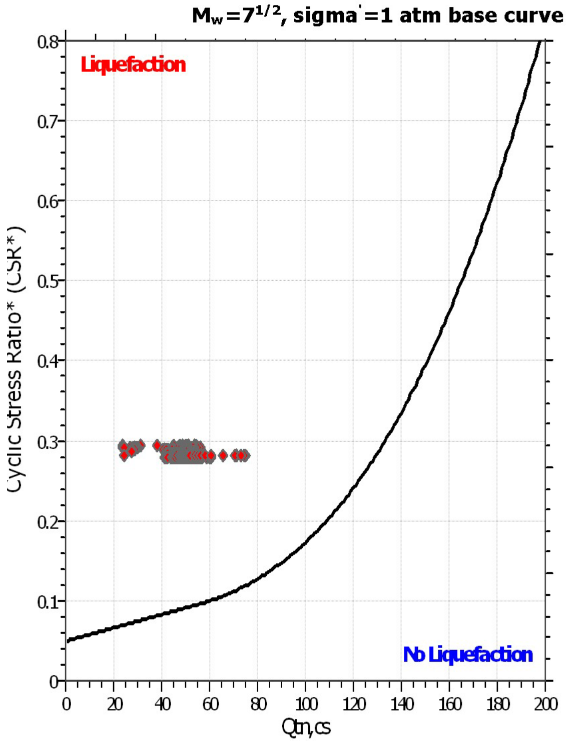
CPT file : CPT12-01A

Input parameters and analysis data

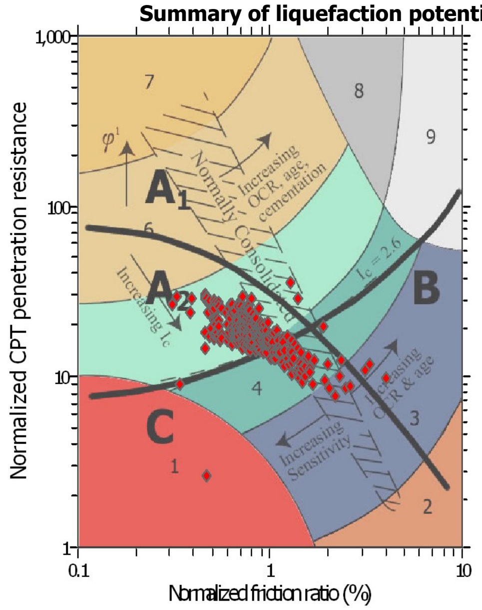
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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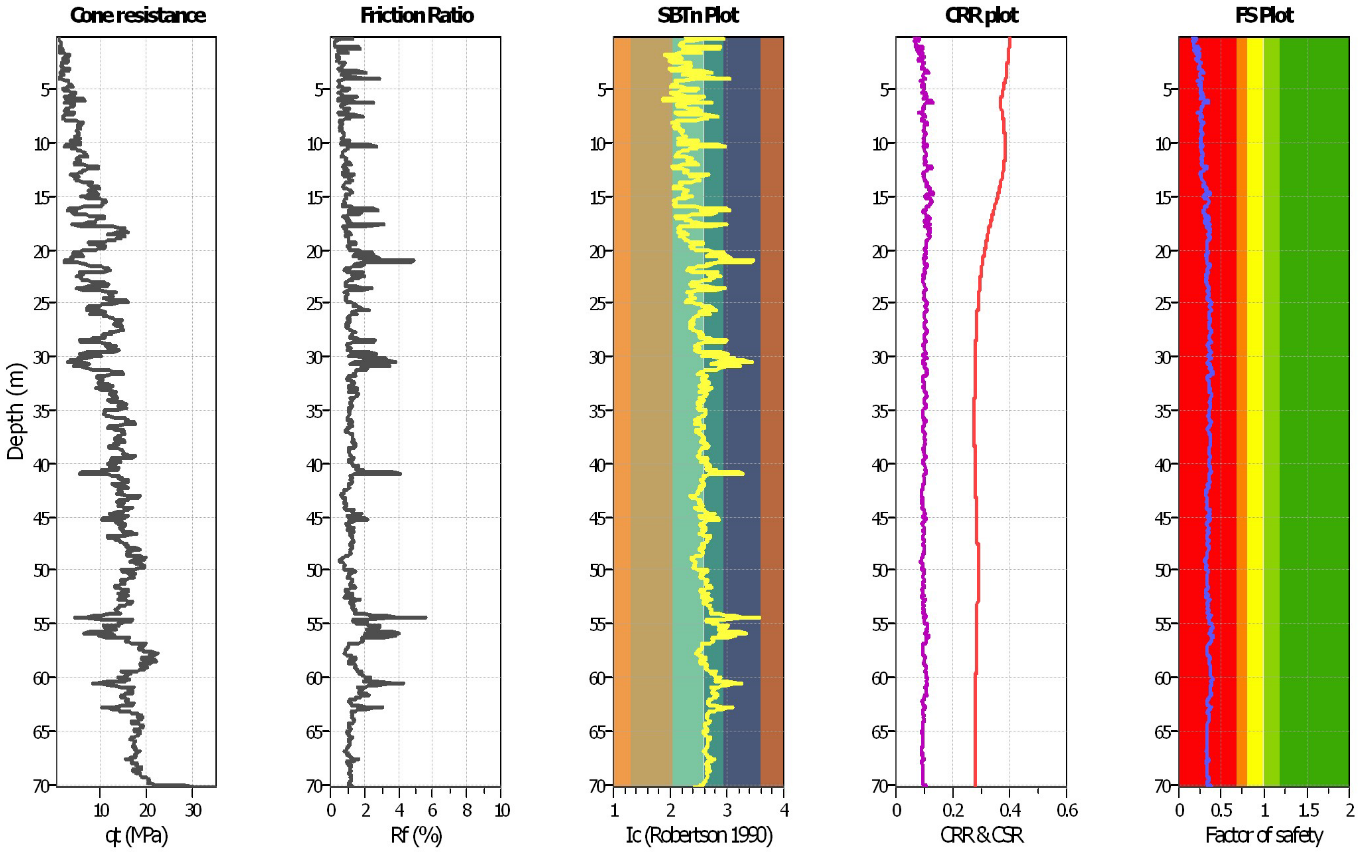
LIQUEFACTION ANALYSIS REPORT

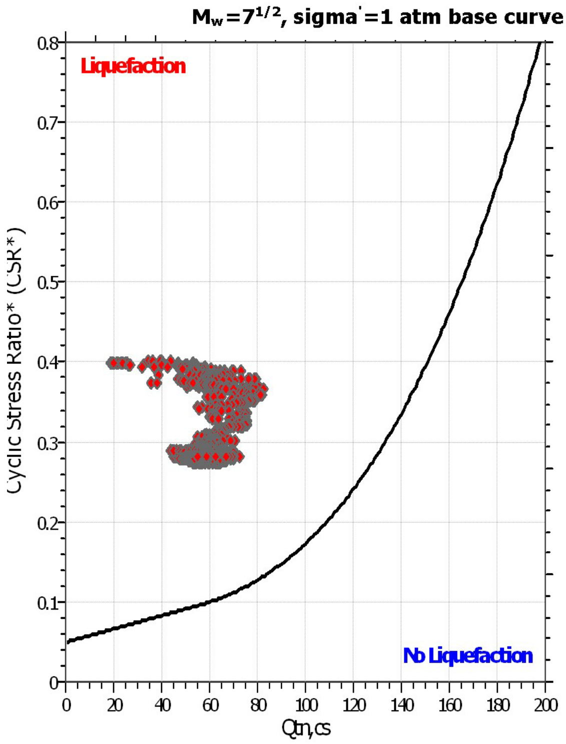
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

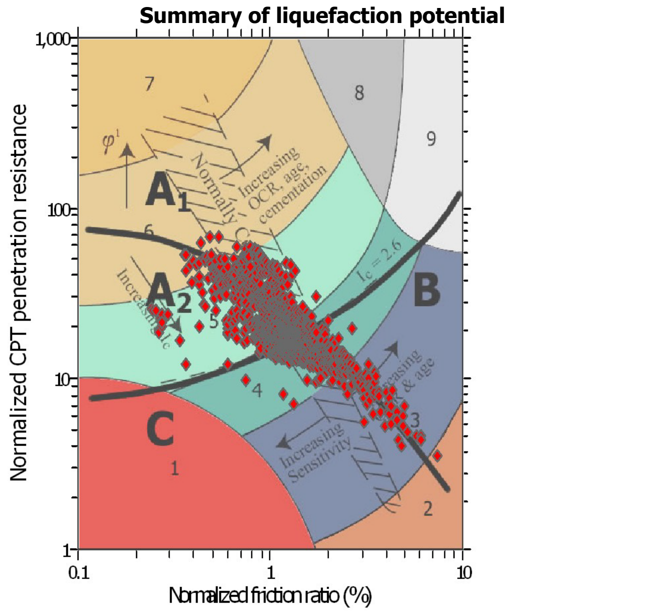
CPT file : CPT12-05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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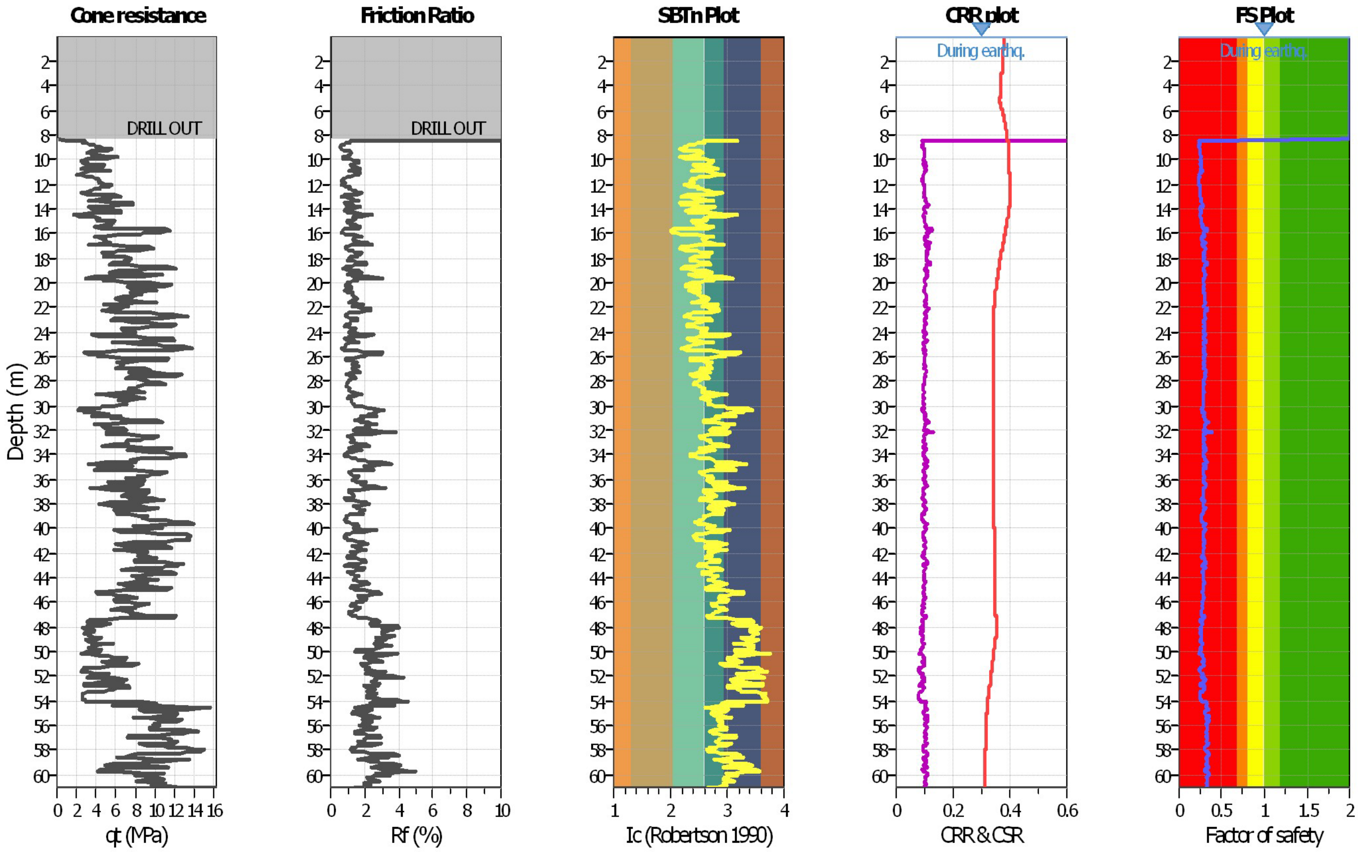
LIQUEFACTION ANALYSIS REPORT

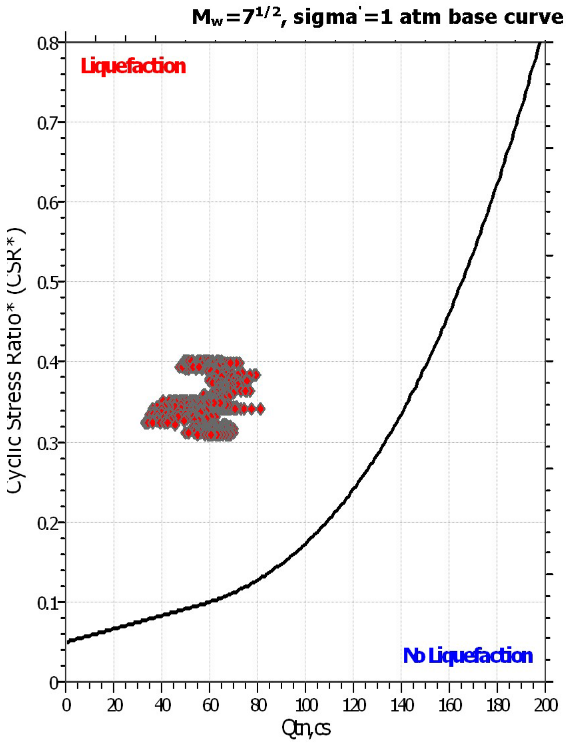
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

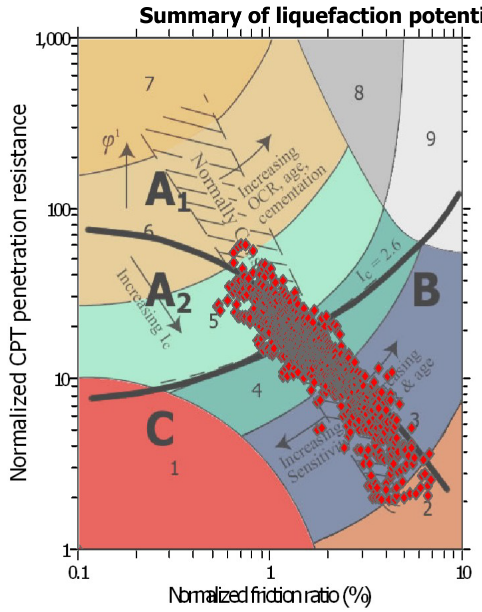
CPT file : CPT13-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

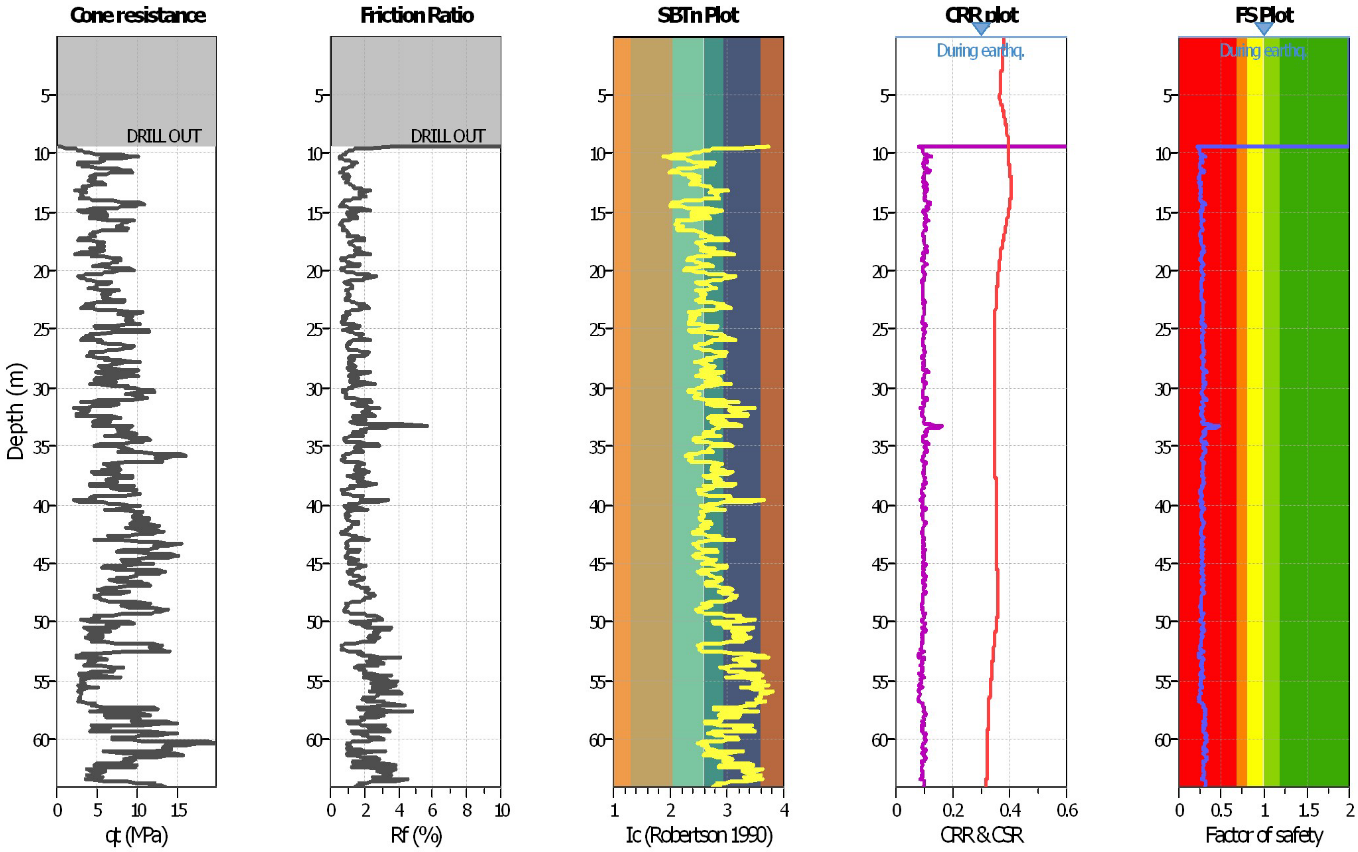
LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

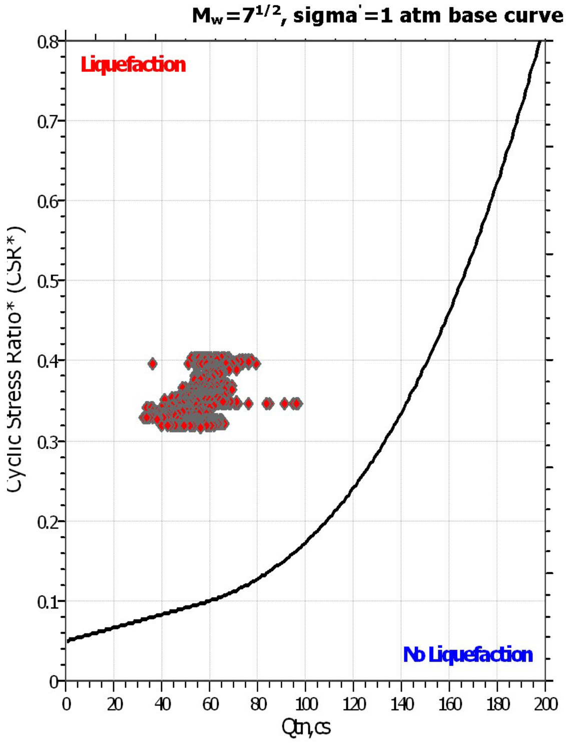
CPT file : CPT13-04

Input parameters and analysis data

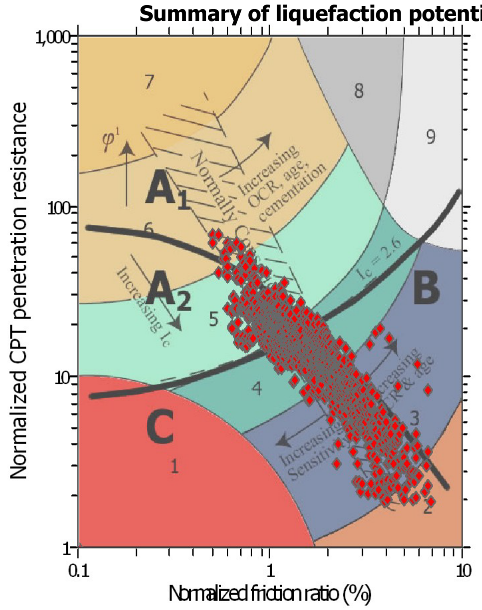
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

CPT file : CPT14-01A

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
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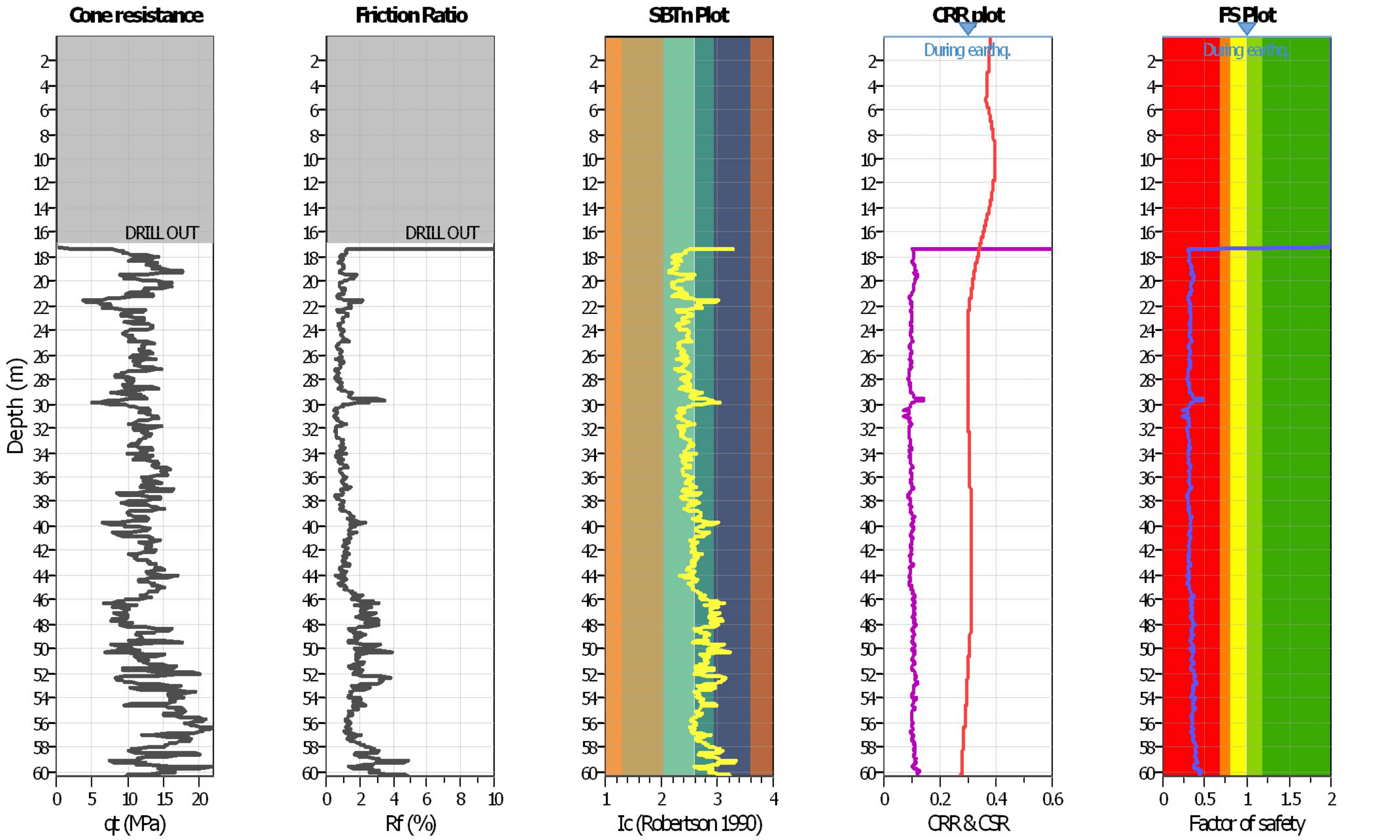
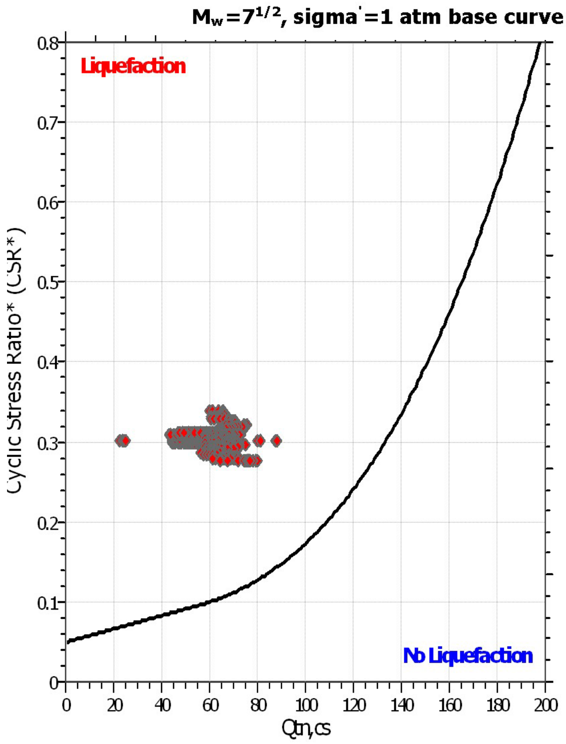
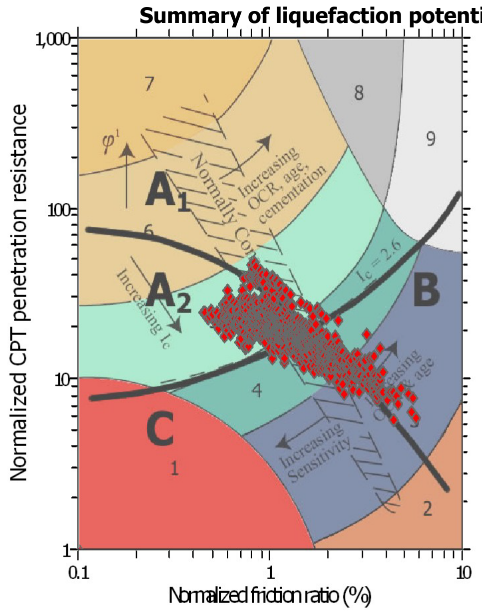
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

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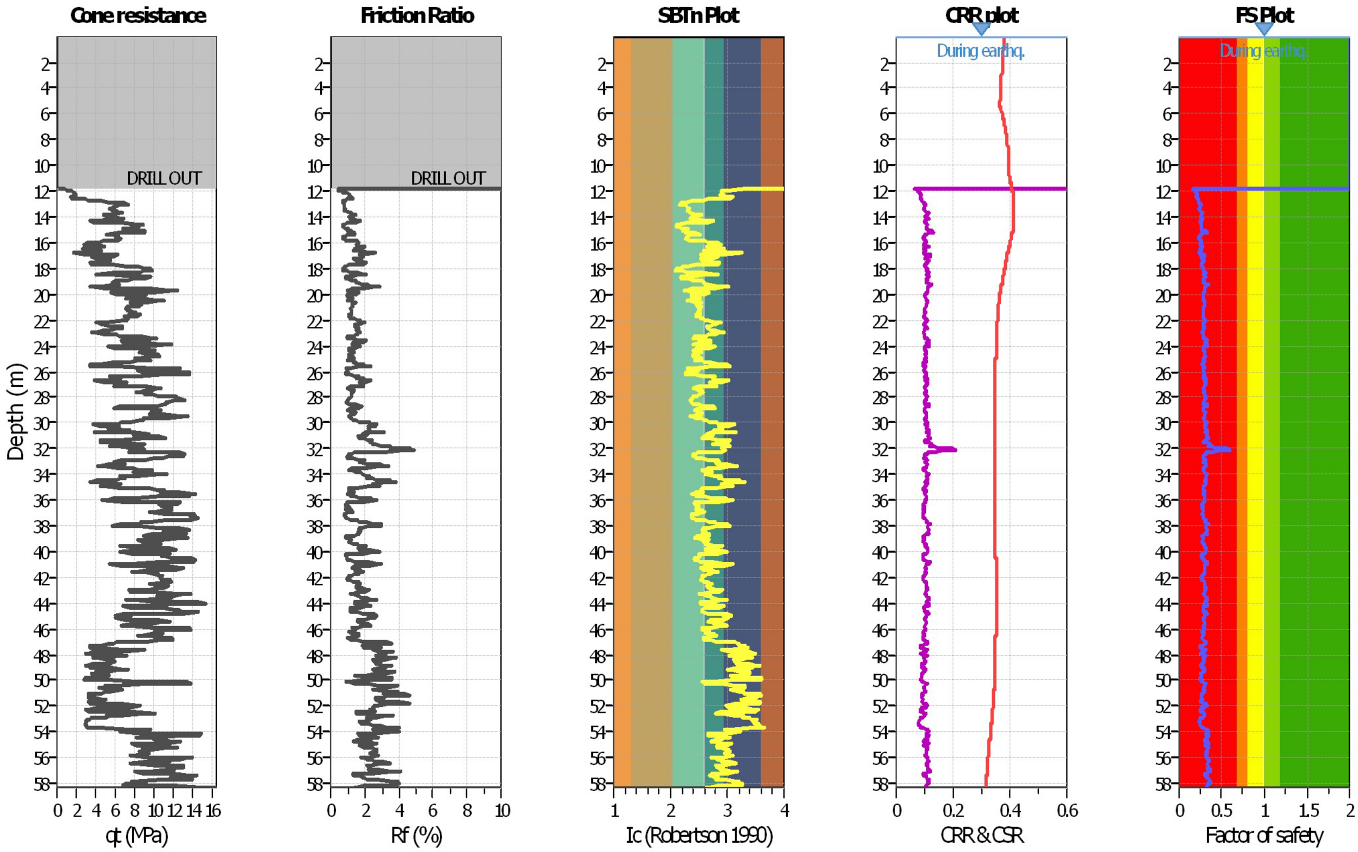
LIQUEFACTION ANALYSIS REPORT

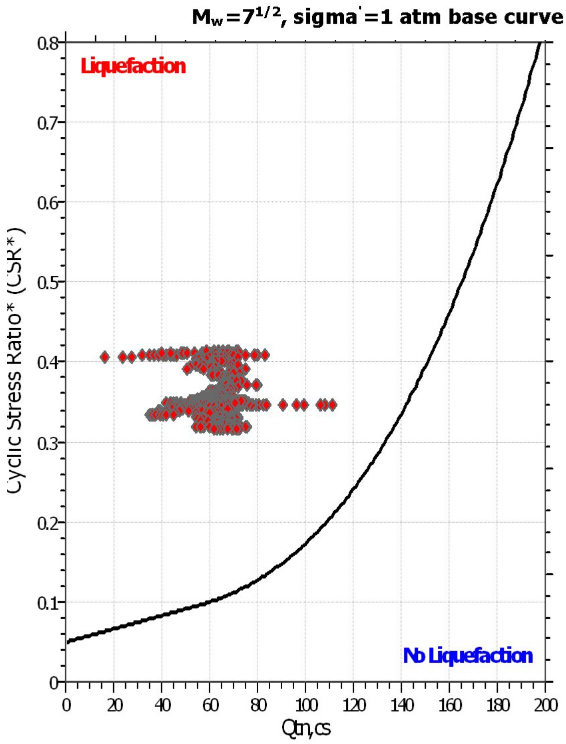
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

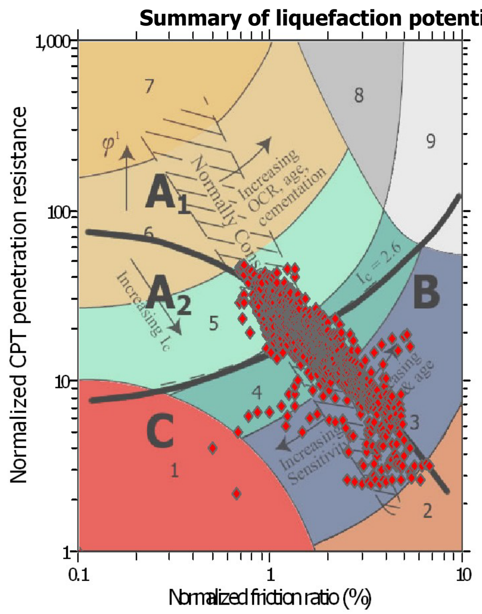
CPT file : CPT14-05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
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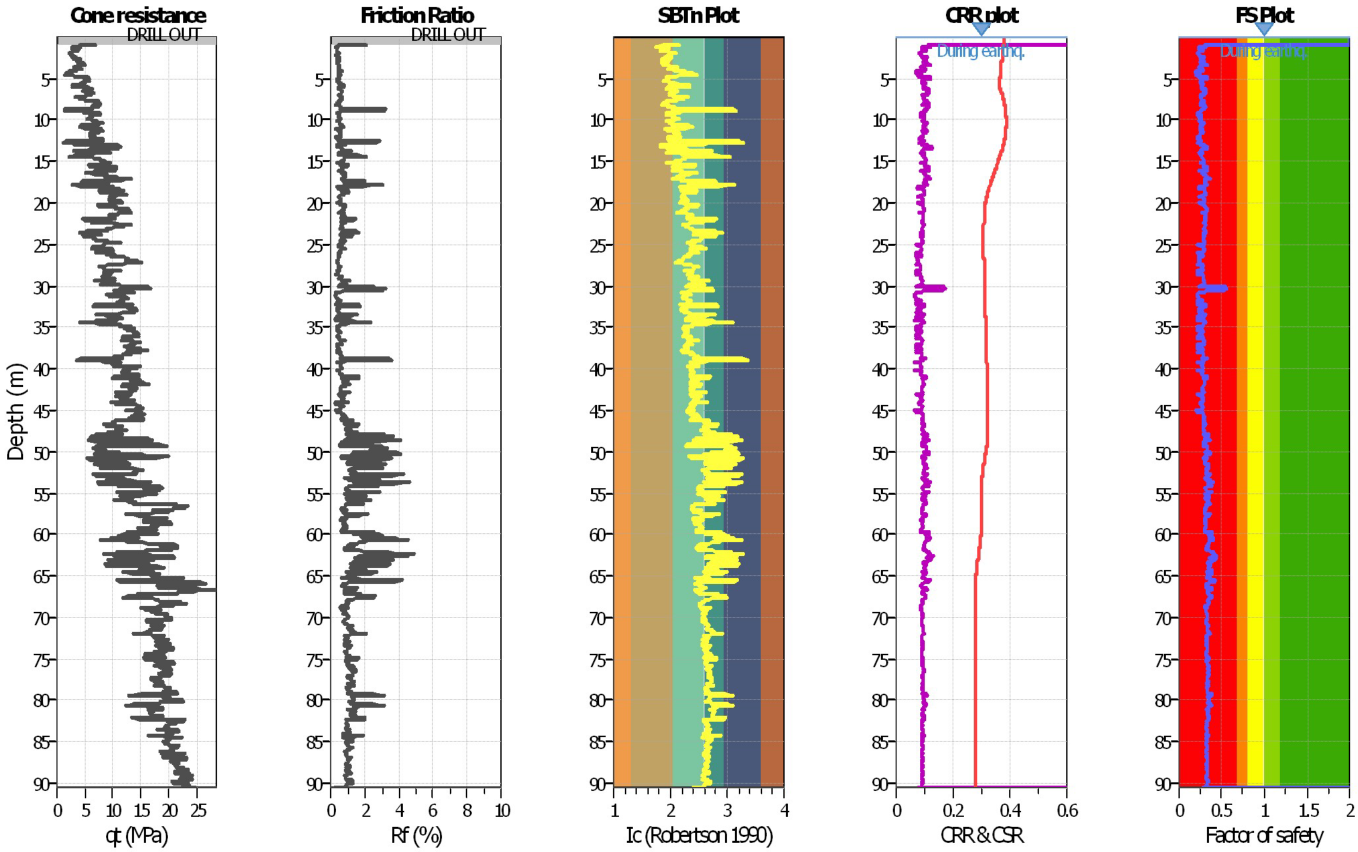
LIQUEFACTION ANALYSIS REPORT

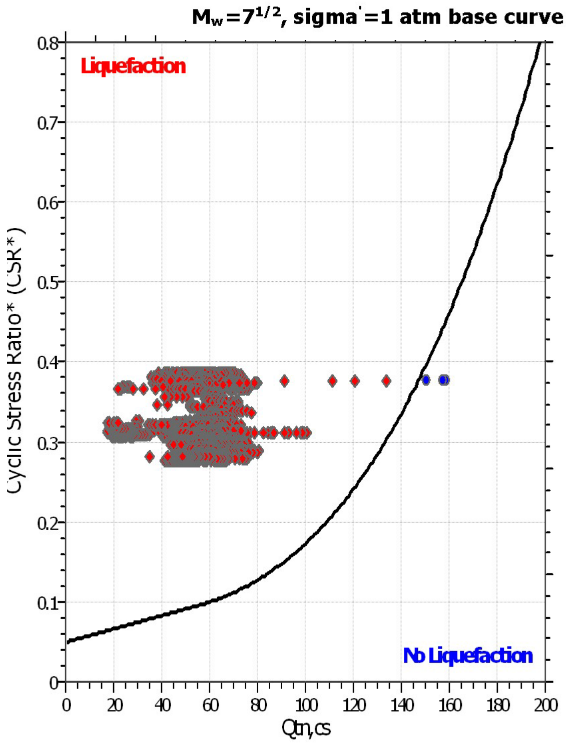
Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

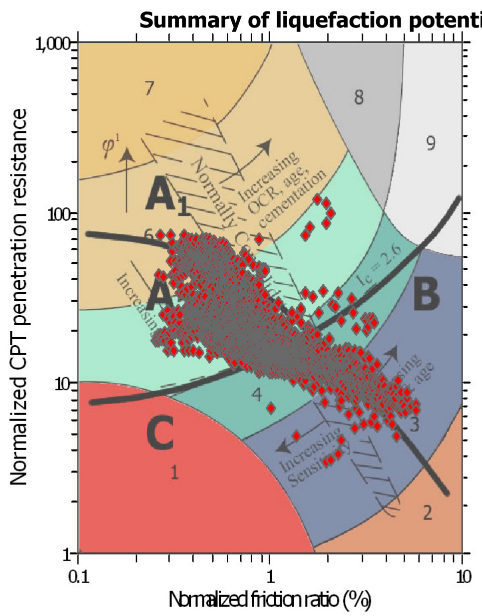
CPT file : CPT15-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

CPT file : CPT15-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
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Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
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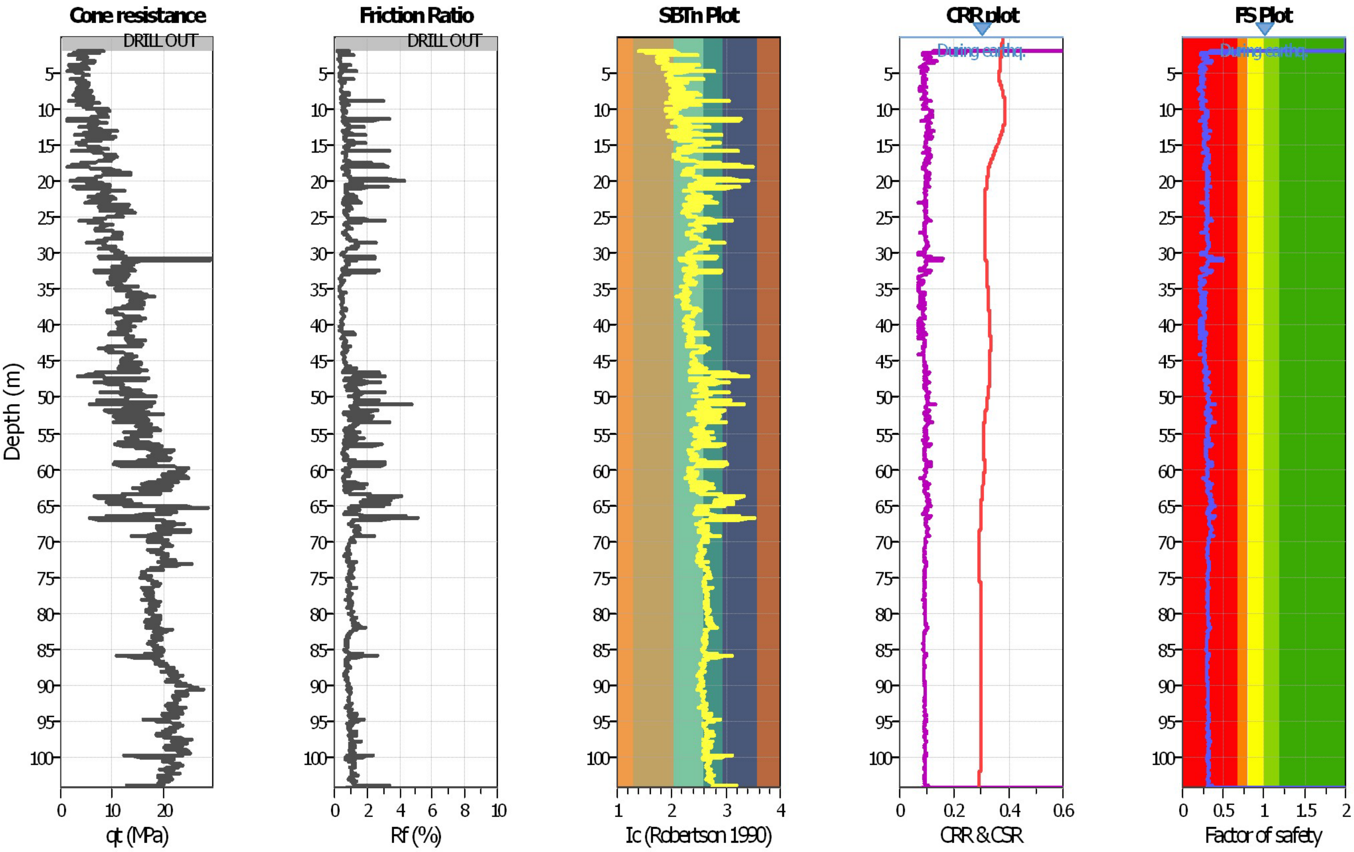
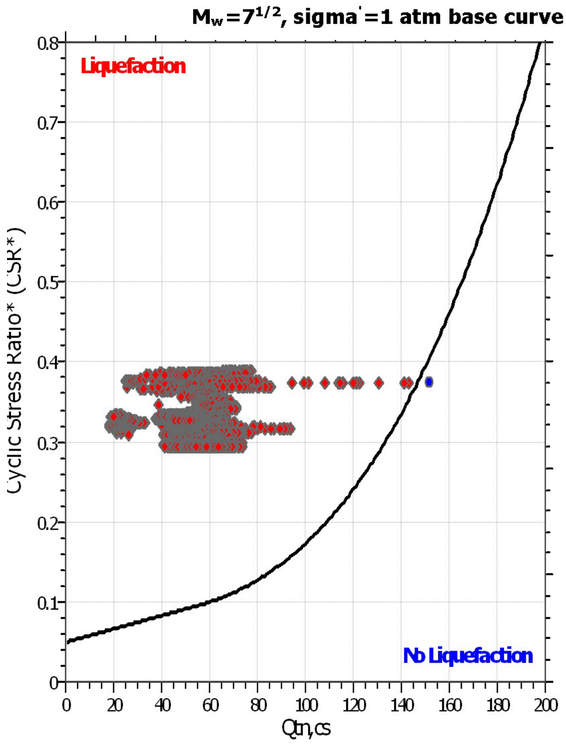
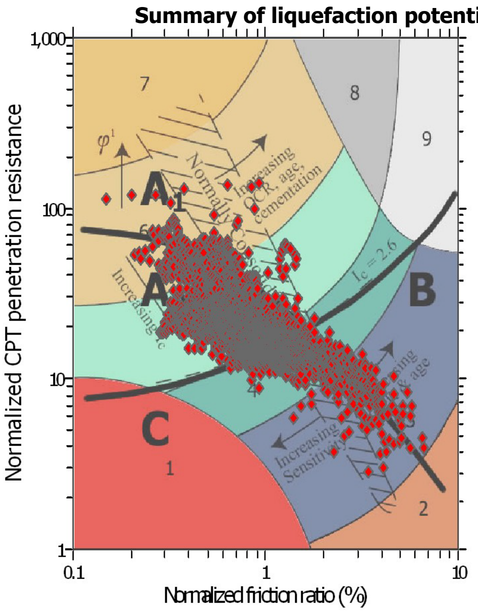
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

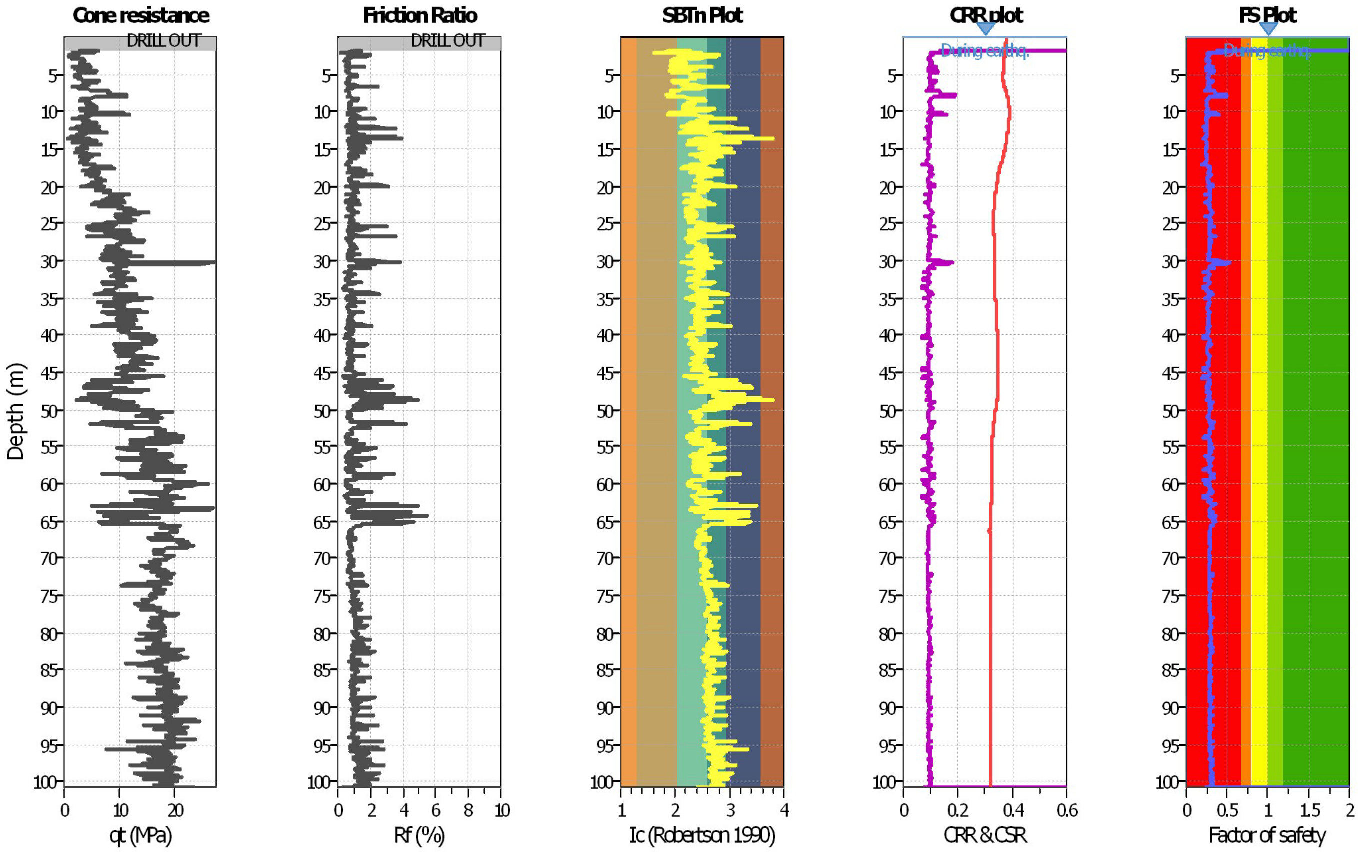
LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

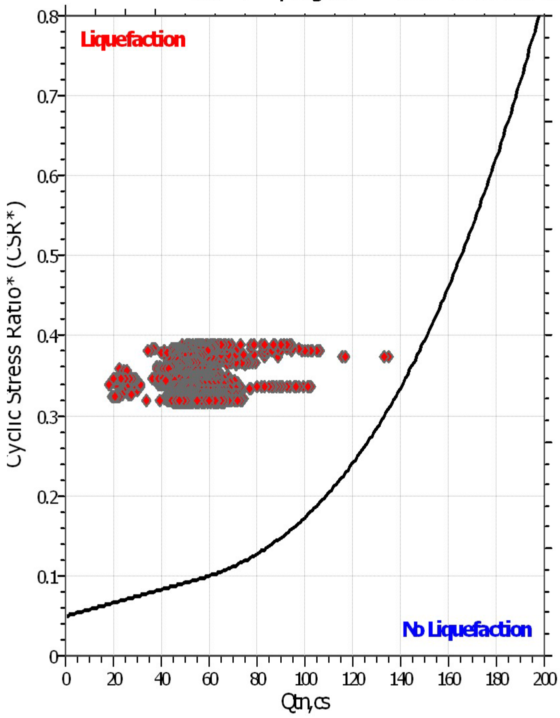
CPT file : CPT15-05

Input parameters and analysis data

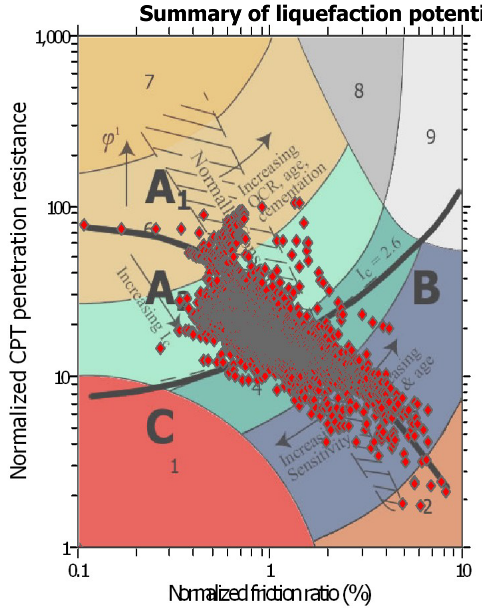
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

LIQUEFACTION ANALYSIS REPORT

Project title : Montana Resources **Location : Yankee Doodle Tailings Facility - Main Embankment**

CPT file : CPT15-07

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	0.00 m	Use fill:	No	Clay like behavior applied:	Sands only
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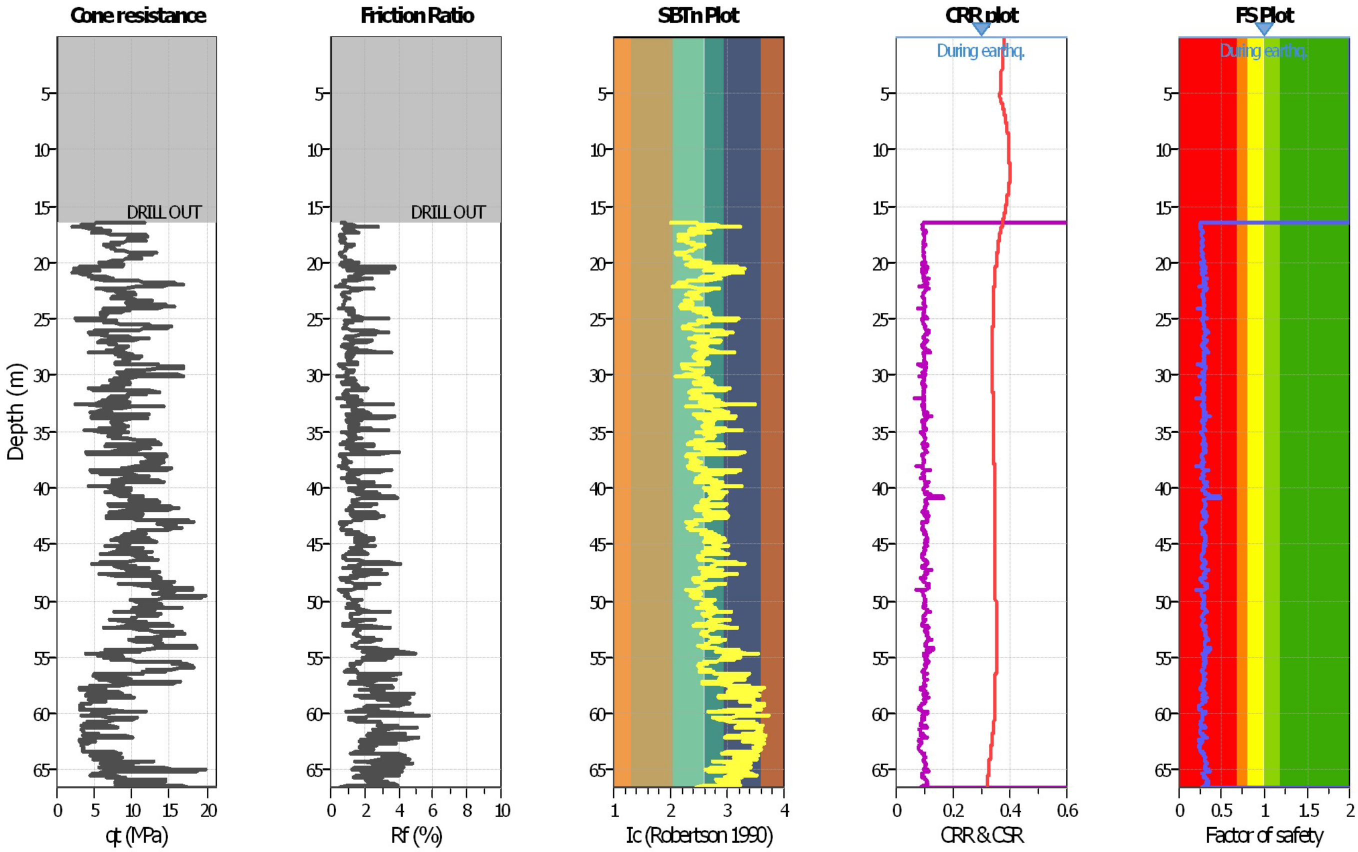
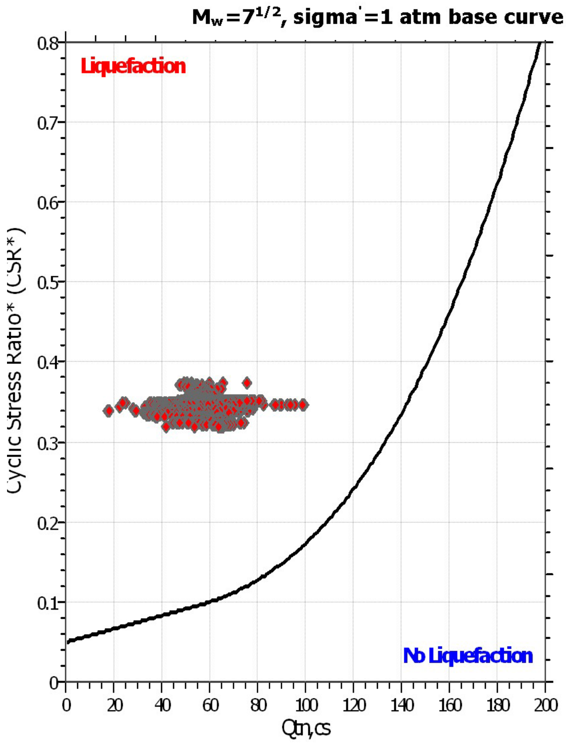
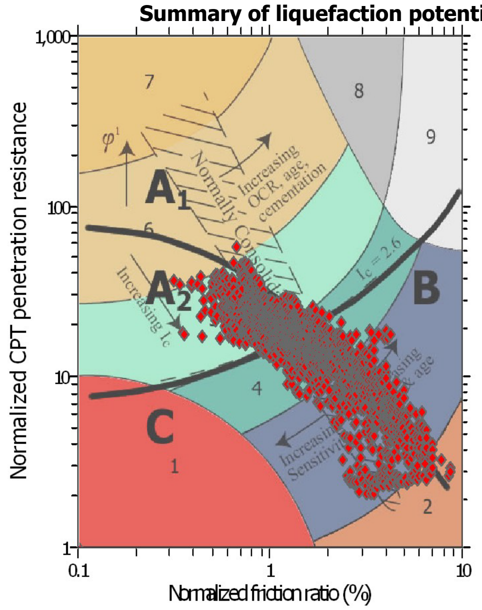
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	5.00	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		





Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

APPENDIX A4

LIQUEFACTION ASSESSMENT FOR THE YANKEE DOODLE TAILINGS IMPOUNDMENT

(Pages A1-1 to A1-31)

August 14, 2017

File No.:VA101-00126/16-A.01
Cont. No.:VA17-01098

Mr. Ken Brouwer
President
Knight Piésold Ltd.
Suite 1400 - 750 West Pender
Vancouver, British Columbia
Canada, V6C 2T8

Dear Ken,

Re: Montana Resources – Liquefaction Assessment for the Yankee Doodle Tailings Impoundment

1 – INTRODUCTION

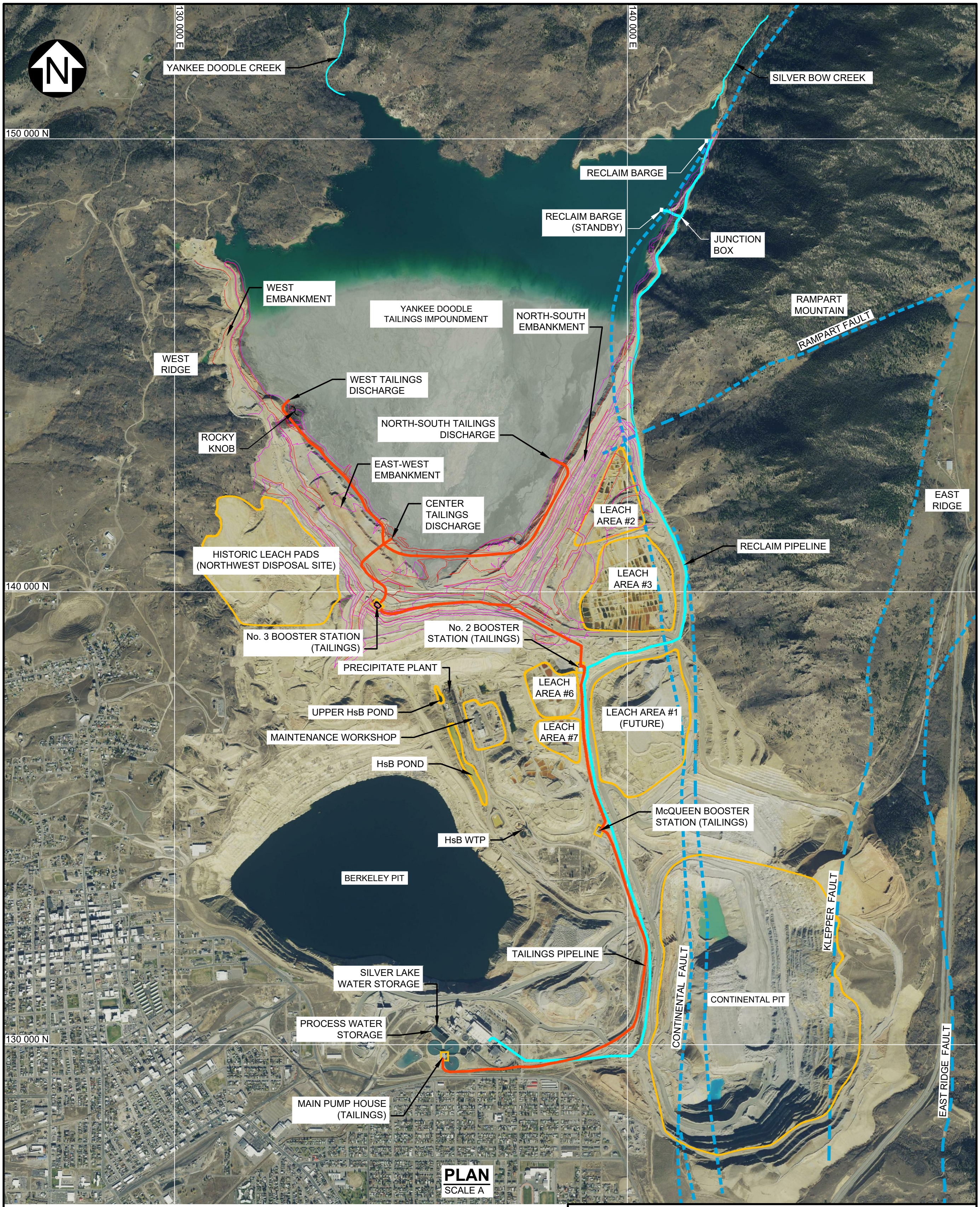
Montana Resources, LLP (MR) currently operates a copper-molybdenum mine northeast of Butte, Montana. All tailings are currently discharged into the Yankee Doodle Tailings Impoundment (YDTI) from multiple discharge points situated at the crest of the embankment. The YDTI comprises a valley-fill style impoundment created by a continuous rockfill embankment that for descriptive purposes is divided into three rockfill embankments according to the general geometry of each limb of the continuous embankment. These embankments are the:

- The North-South Embankment forms the eastern to southeastern limb, abutting the base of Rampart Mountain, and forming the eastern battery limit of the MR mine site.
- The East-West Embankment forms the southwestern limb, running approximately east to west in orientation and is constructed upstream of Horseshoe Bend (HsB) and the Berkeley Pit.
- The West Embankment forms the western limb of the YDTI and runs approximately north to south in orientation. The West Embankment is constructed along the side of the West Ridge and forms the western battery limit of the facility.

Originally constructed in 1963 using rockfill from the Berkeley Pit, the YDTI has been continuously raised to Elevation (EL.) 6,400 ft using rockfill first from the Berkeley Pit (ending 1982) and thereafter from the Continental Pit (beginning in 1986). The majority of the embankment crest is currently at an elevation (EL.) of approximately 6,400 ft (Anaconda Datum). MR is in the process of preparing a permit amendment application for continued use of the YDTI to provide for continued mining beyond 2020. Knight Piésold Ltd. (KP) prepared this liquefaction assessment in support of a slope stability assessment of the future YDTI configuration. Plan views showing the current facility and the conceptual layout of the future facility with an embankment crest of EL. 6,450 ft are shown on Figures 1 and 2, respectively.

Silty sand tailings generated from the milling process have been hydraulically emplaced from the rockfill embankment and have accumulated to form an extensive tailings beach. The tailings are highly stratified with inter-bedded layers of sand-silt mixtures and ranges from predominantly drained and consolidated sandy beach deposits immediately adjacent to the embankment to saturated silt deposits near the water reservoir (supernatant pond) to the north.

KP completed a liquefaction assessment to evaluate the potential for liquefaction of the tailings sands due to seismic loading from the design earthquake. The potential for liquefaction in the tailings deposit has been analyzed for the current conditions and the future configuration. This letter presents the findings of the liquefaction assessment.



NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. IMAGERY FROM 2015 AIR PHOTO PROVIDED BY MONTANA RESOURCES.
3. EMBANKMENT TOPOGRAPHY 2017 PROVIDED BY MONTANA RESOURCES ON MARCH 19, 2017.

LEGEND:

FAULT LOCATIONS FROM WITKIND (1975) SMEDES (1966,1962) AND OTHERS.

SCALE A: 1500 0 1500 3000 4500 6000 7500 ft

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

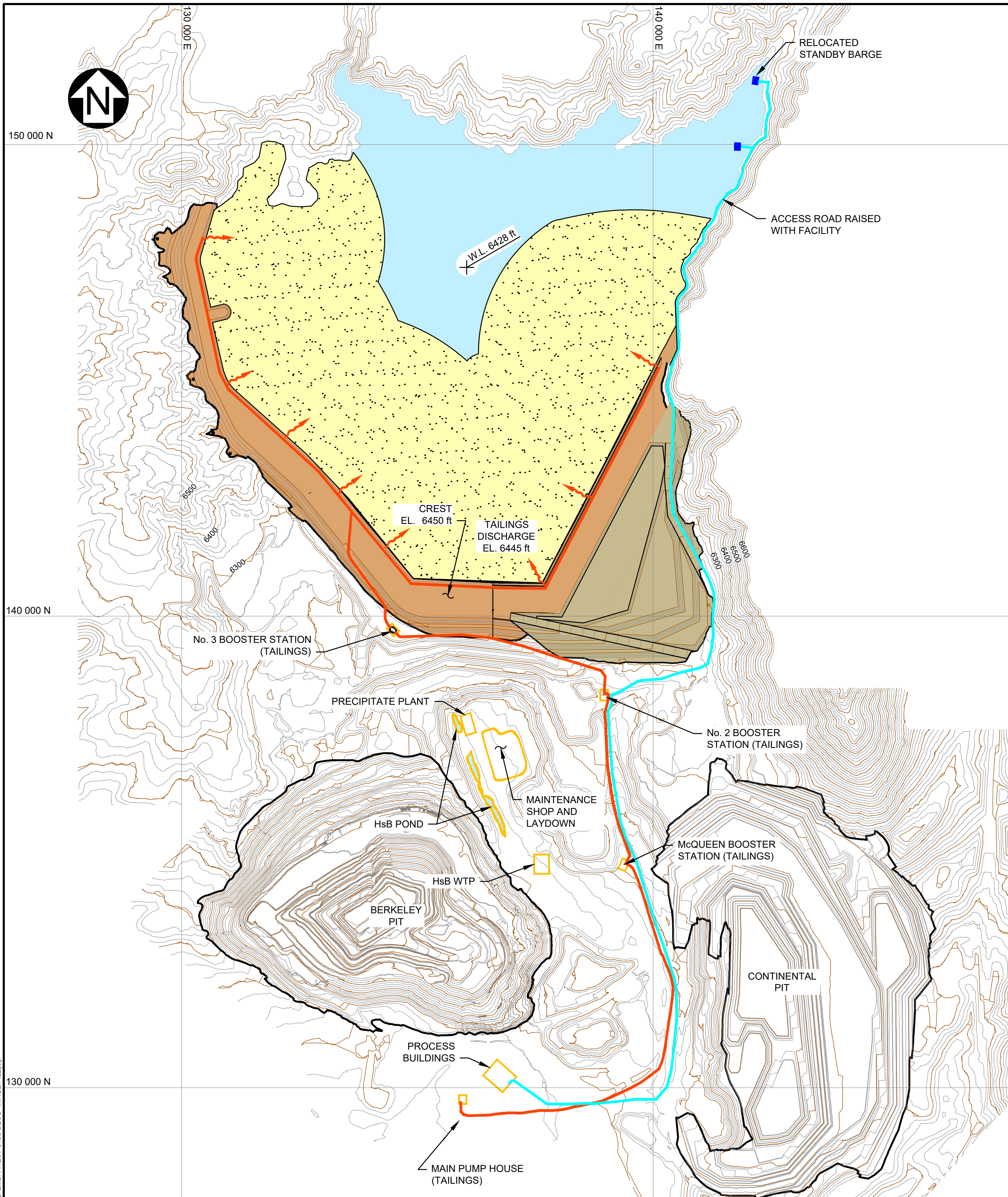
SITE LAYOUT (MARCH 2017)

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA101-126/16	VA17-01098

FIGURE 1 REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
1	14AUG'17	ISSUED WITH REPORT	DDF	MJC	KJB



NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON MINE GRID.

LEGEND:

TAILINGS BEACH	TAILINGS DEPOSITION
EMBANKMENT FILL	TAILINGS PIPELINE
ROCK DISPOSAL SITE	RECLAIM PIPELINE
MINE WATER	RECLAIM BARGE

1500 0 1500 3000 4500 6000 7500 ft
SCALE A

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

GENERAL ARRANGEMENT PLAN - 2031

Knight Piésold
CONSULTING

P/A NO.	REF NO.
VA101-126/16	VA17-01098

FIGURE 2 REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	14AUG'17	ISSUED WITH LETTER	DDF	KJM/MJC	KJB

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XREF FILE(S): 2030 Open Pit; EOY 2031 Topo; X-C-EMBK-EAST-6450-DESG; X-C-EMBK-WEST-6450-DESG IMAGE FILE(S):

2 – METHODOLOGY

The liquefaction assessment was carried out by determining the predicted cyclic stress ratios (CSR) induced by the design earthquakes and comparing them to the estimated cyclic resistance ratios (CRR) of the tailings. The CSR is defined as the shear stress imposed by the earthquake and is expressed as a ratio of the effective vertical stress on the soil. Saturated tailings sands are predicted to liquefy when the CSR exceeds the CRR.

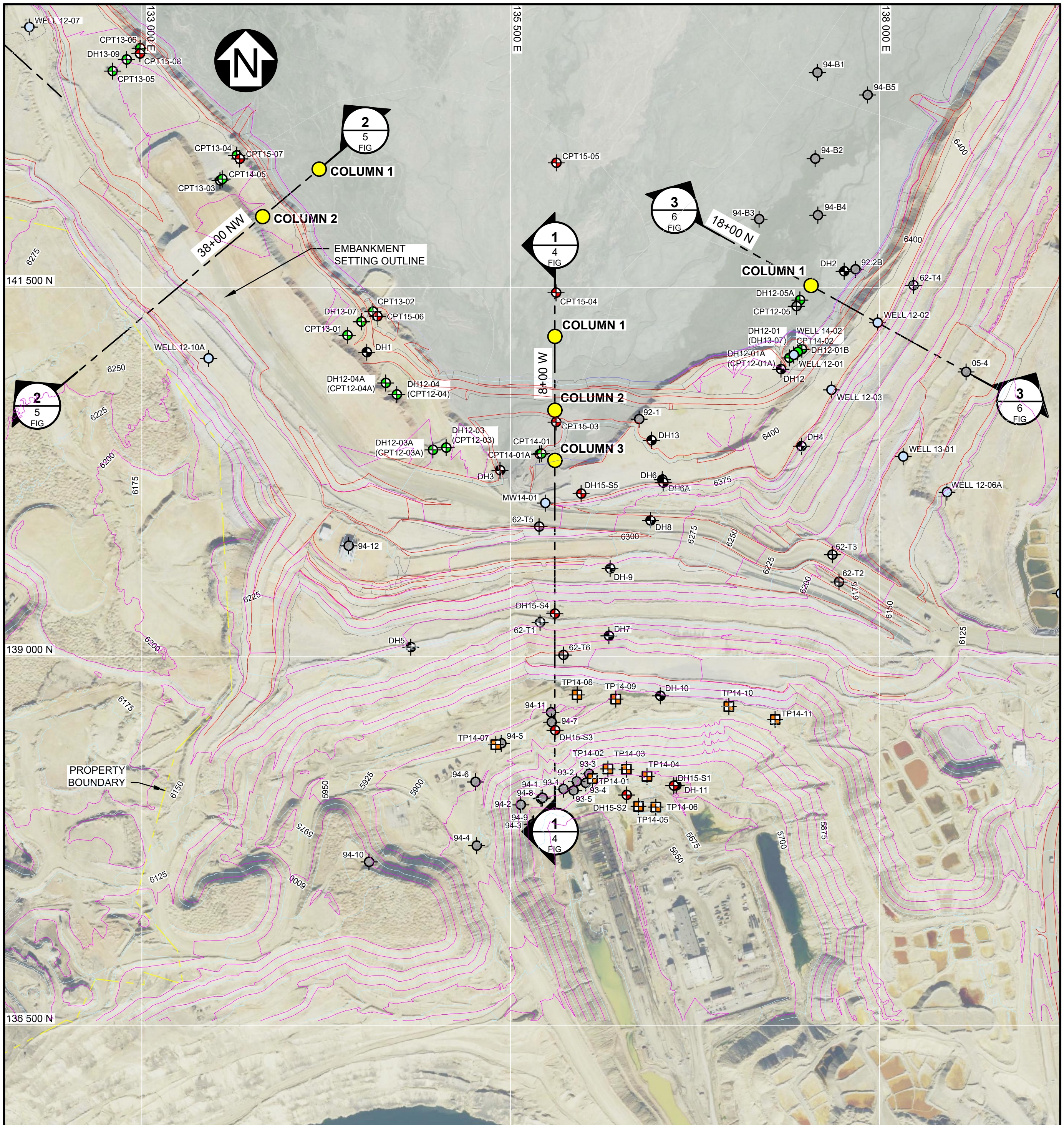
Dynamic response analyses were performed using the computer program SHAKE2000 (Ordonez, 2012) to estimate potential cyclic shear stresses within the tailings, embankment and foundation soils resulting from the design earthquake events. In addition to the earthquake loading, calculated CSRs are dependent on the magnitude of pore water pressures prior to a seismic event and the stiffness properties of the soil materials.

The CRR was calculated based on the cone penetration testing (CPT) data using the methods of Idriss and Boulanger (2014) and Robertson (2009). The CRR was also derived from the shear wave velocity of the tailings using the method by Kayen et al. (2013).

The analyses were completed for six stratigraphic profiles through the tailings storage facility, for both current conditions and future conditions coincident with a tailings embankment crest at EL. 6,450 ft. The analyses considered three representative sections (8+00W, 38+00NW, and 18+00N) as shown on Figure 3. Cross sections and analyzed columns for the current embankment geometry were based on the topographic survey of the facility from March 2017. Figure 3 shows the three cross sections analyzed, the locations of the columns analyzed for each section, and nearby geotechnical and hydrogeological information from the various site investigation programs.

Sections 8+00W, 38+00NW, and 18+00N are shown on Figures 4, 5 and 6, respectively. The figures show current and future conditions, locations of the analyzed columns, and relevant CPT locations used for the analysis. Three columns were analyzed on Section 8+00W, two columns on Section 38+00NW, and one column on Section 18+00N.

Column 1 on Sections 8+00W, 38+00NW and 18+00N represent conditions in the tailings facility where no rockfill surcharging has occurred on top of the tailings. The other column locations were selected to assess the potential for liquefaction of tailings adjacent to the embankment in current and future conditions with surcharge loading from rockfill.



NOTES:

1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.
4. IMAGERY FROM 2015 AIR PHOTO PROVIDED BY MONTANA RESOURCES.
5. EMBANKMENT TOPOGRAPHY FROM MARCH 2017 PROVIDED BY MONTANA RESOURCES.

SCALE A 500 0 500 1000 1500 2000 2500 ft

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
ANALYZED CROSS SECTIONS

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA101-126/16	VA17-01098

FIGURE 3 REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
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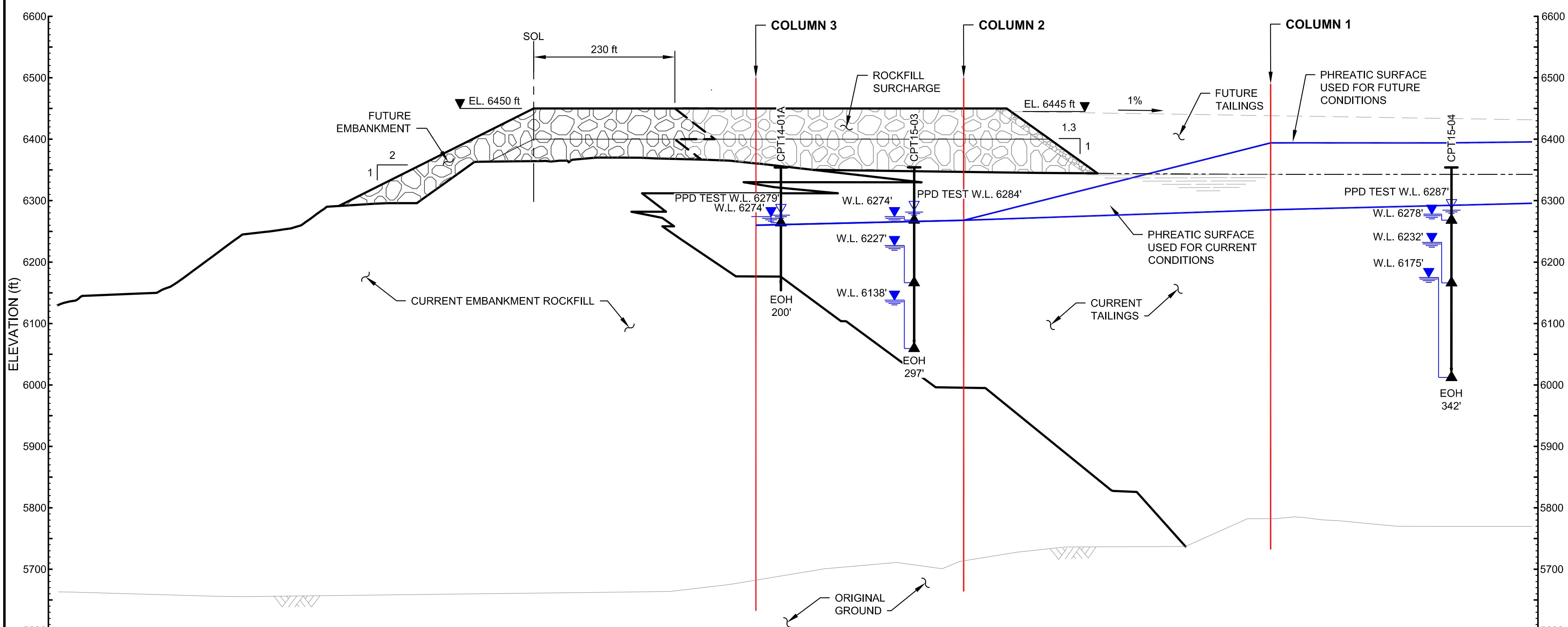
XREF FILE(S): IMAGE FILE(S): 2015_NAIP_Silver_Bow



1 SECTION
3 FIG
STATION 8+00 W
SCALE A

SCALE A 125 0 125 250 375 500 625 ft

NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.

LEGEND:

ZONE F - EARTHFILL
ZONE U - ROCKFILL (WASTE)
ZONE U - ROCKFILL (EMBANKMENT)
SOL SETTING OUT LINE
PPD PORE PRESSURE DISSIPATION
W.L. WATER LEVEL
▲ VIBRATING WIRE PIEZOMETER

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	14AUG'17	ISSUED WITH LETTER	DDF	RM	KJB

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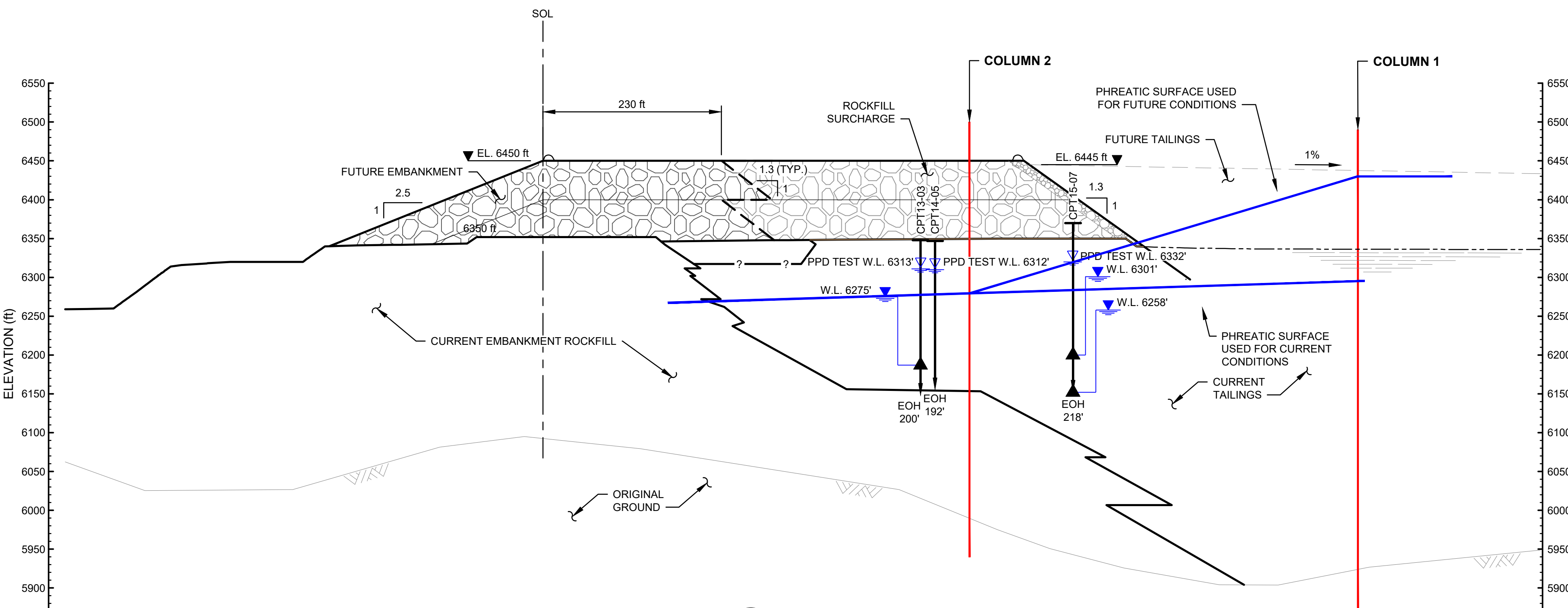
YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
SECTION 8+00 W
ANALYSIS COLUMNS AND CPT LOCATIONS

Knight Piésold CONSULTING

P/A NO.	REF NO.	
VA101-126/16	VA17-01098	
FIGURE 4		REV 0



2 / 3 FIG **SECTION**
STATION 38+00 NW
SCALE A

SCALE A 100 0 100 200 300 400 500 ft

NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. CPT13-04 NOT SHOWN ON SECTION FOR CLARITY.

LEGEND:

	ZONE F - EARTHFILL
	ZONE U - ROCKFILL (WASTE)
	ZONE U - ROCKFILL (EMBANKMENT)
SOL	SETTING OUT LINE
PPD	PORE PRESSURE DISSIPATION
W.L.	WATER LEVEL
▲	VIBRATING WIRE PIEZOMETER

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	14AUG'17	ISSUED WITH LETTER	DDF	RM	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
SECTION 38+00 NW
ANALYSIS COLUMNS AND CPT LOCATIONS

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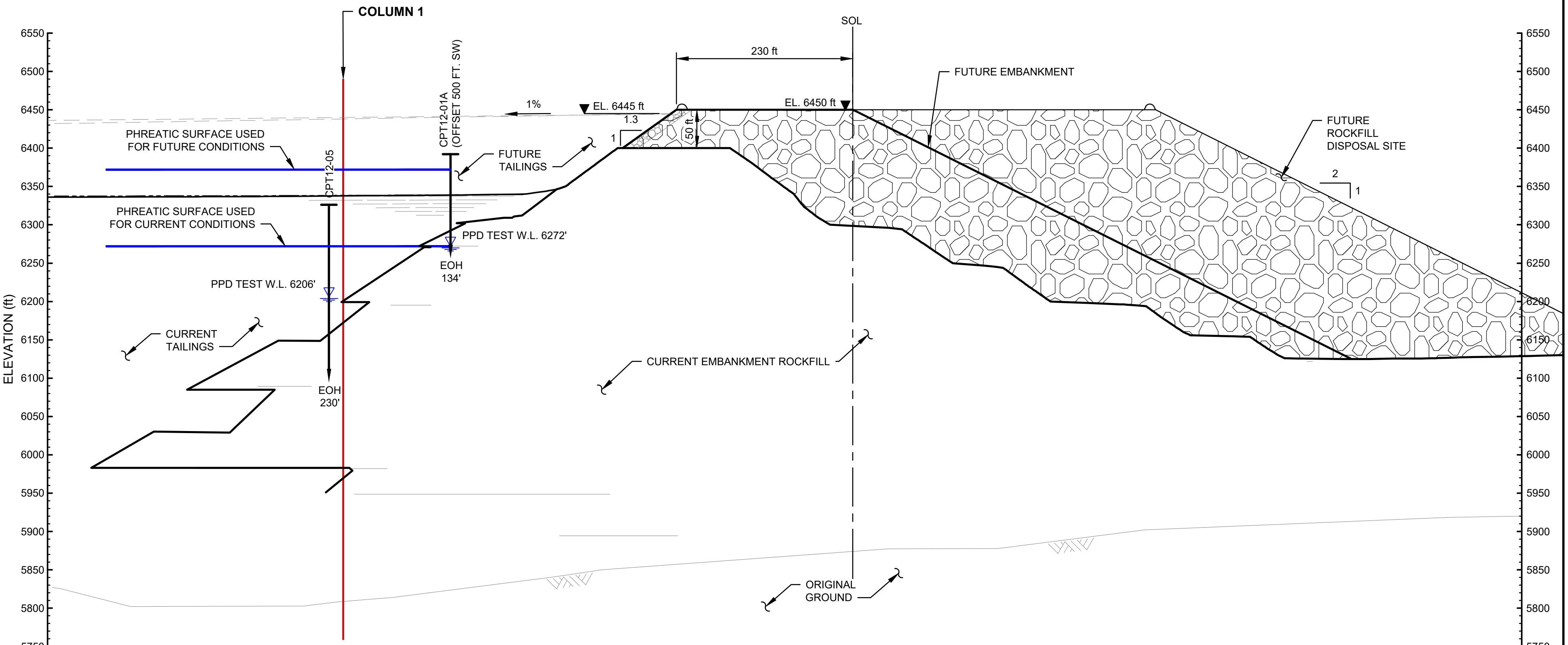
P/A NO.	REF NO.
VA101-126/16	VA17-01098

FIGURE 5 REV 0



3
3 FIG
SECTION
STATION 18+00 N
SCALE A

NOTES:

1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.

LEGEND:

ZONE F - EARTHFILL
ZONE U - ROCKFILL
SOL SETTING OUT LINE
PPD PORE PRESSURE DISSIPATION
W.L. WATER LEVEL

SCALE A 100 0 100 200 300 400 500 ft

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	14AUG'17	ISSUED WITH LETTER	DDF	RM	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

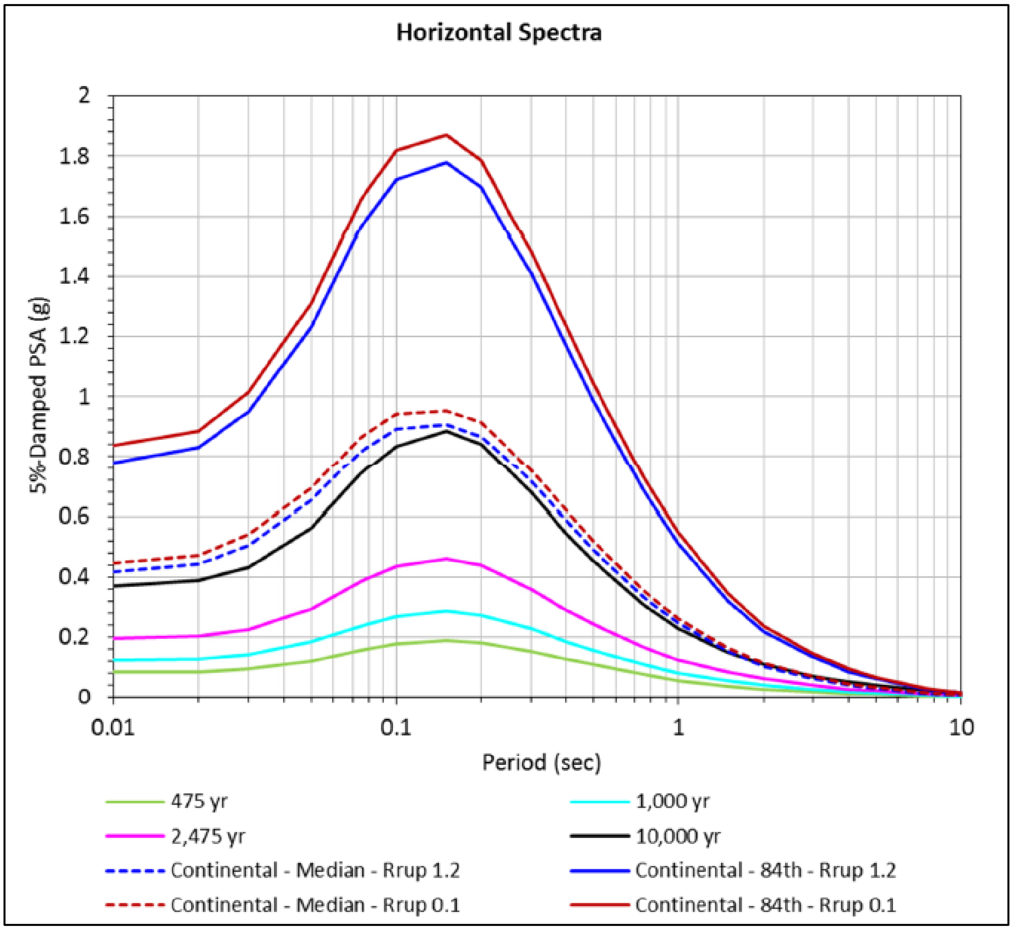
LIQUEFACTION ASSESSMENT
SECTION 18+00 N
ANALYSIS COLUMNS AND CPT LOCATIONS

Knight Piésold CONSULTING

P/A NO. VA101-126/16
REF NO. VA17-01098
FIGURE 6
REV 0

3 – DESIGN EARTHQUAKES

The design earthquakes were selected to meet the obligations as stipulated in MCA 82-4-376 (2), (m), (i) and (l) (MCA, 2015). The requirements from the legislation are satisfied if it can be demonstrated that the seismic response of the tailings storage facility does not result in the uncontrolled release of impounded materials when subject to the ground motion associated with the 1 in 10,000 year event, or the maximum credible earthquake (MCE), whichever is larger.



NOTES:

1. Source: Figure 6-2 of Al Atik and Gregor (2016).
2. PSA stands for Peak Spectral Acceleration.

Figure 7 Probabilistic and Deterministic Horizontal Design Spectra

A site-specific seismic hazard assessment was carried out by Al Atik and Gregor (2016). The seismic hazard assessment is included in the appendices of the Site Characterization Report (KP, 2017a). Seismic ground motion parameters (including maximum acceleration and earthquake magnitude) were determined from probabilistic and deterministic seismic hazard analyses. Figure 7 shows the horizontal design spectra for the YDTI. The deterministically derived MCE spectra exceed those for the probabilistically derived 1 in 10,000 year event. The MCE was therefore selected as the design earthquake. The MCE with a rupture distance of 0.1 km produces spectral accelerations that are greater than the MCE with a rupture distance of 1.2 km. Therefore, the MCE based on a rupture distance of 0.1 km was conservatively chosen as the design earthquake.

The 50th percentile (median) value was chosen as the design event for operating conditions, and the 84th percentile value was chosen as the design event for post-closure conditions (KP, 2017b). A moment magnitude 6.5 was adopted for the MCE earthquake, which is consistent with the ground motion characterization models of Al Atik and Gregor. The earthquake design events and their characteristics are summarized in Table 1.

Table 1 Seismic Design Parameters

Condition	Design Earthquake	PGA (g)	Moment Magnitude M
Maximum normal operating	MCE with R_{rup} of 0.1 km Median	0.45	6.5
Long-term closure	MCE with R_{rup} of 0.1 km 84th Percentile	0.84	6.5

NOTES:
1. Peak ground accelerations are for rock site conditions (Vs_{30} – 760 m/s).

4 – EARTHQUAKE TIME-HISTORY RECORDS

Appropriate earthquake (horizontal acceleration) time-history records have been selected as input ground motions for the seismic response analysis. Earthquake records representative of the design events were selected to the extent possible by Al Atik and Gregor. Five representative earthquake time histories were provided for each earthquake event with characteristics as shown in Table 2. The scaled and baseline corrected earthquake time-history records were derived for bedrock with a shear wave velocity of 760 m/s.

Table 2 Parameters of input time series for MCE design events

Earthquake	Station Name	Moment Magnitude	Mechanism	Rupture Distance (km)
Helena, Montana-01	Carroll College	6.0	Strike-slip	2.86
San Fernando	Pacoima Dam	6.6	Reverse	1.81
Imperial Valley-06	El Centro Array #5	6.5	Strike-slip	3.95
Niigata, Japan	NIG020	6.6	Reverse	8.47
L'Aquila, Italy	L'Aquila - Parking	6.3	Normal	5.38

NOTES:
1. Source: Table 8-2 of Al Atik and Gregor (2016).

5 – SOIL STIFFNESS AND DAMPING

The material parameters required for dynamic response analyses include the soil stiffness (defined by the maximum shear modulus) and shear modulus reduction versus shear strain and damping ratio versus shear strain relationships for each soil unit. The small strain shear modulus (G_{max}) of bedrock was calculated from the shear wave velocity, using the equation:

$$G_{max} = \frac{\gamma}{g} {v_s}^2$$

Where: γ = unit weight
g = acceleration of gravity
v_s = shear wave velocity

The stiffness of the overburden, embankment fill, and tailings materials are calculated using the following relationship by Seed et al. (1986):

$$G_{max} = 1000\ K_{2max}(\sigma'_m)^{1/2}$$

Where: K_{2max} = shear modulus factor
σ'_m = Mean Effective Stress (psf)

A K_{2max} value of 75 was adopted for the overburden soils based on material classifications and typical values for dense sands and gravelly soils. The results of cyclic triaxial testing by IECO (1981) were used to select a K_{2max} of 75 for embankment fill. In-situ shear wave velocity measurements were obtained from CPT in the tailings deposit and were used to calculate the G_{max}. A design K_{2max} was back-calculated for each section based on the G_{max} profile from CPTs in the vicinity of the cross-section. The K_{2max} for tailings is 36 for Section 8+00W and 18+00N, and 31 for 38+00NW.

The actual stiffness (shear modulus) and damping characteristics of the materials during shaking are related to the levels of shear strain developed. SHAKE2000 accounts for this by using shear modulus reduction versus shear strain and damping ratio versus shear strain relationships assigned to each soil type. Published shear modulus and damping versus shear strain relationships appropriate for gravely soils (Seed, et al, 1986) and rock (Schnabel, 1973) were used for the overburden and bedrock, respectively.

The modulus reduction curve and damping curve for embankment fill are selected to provide a good fit with test data from cyclic triaxial testing conducted by IECO in 1981. The test data and selected modulus and damping reduction curves are presented on Figure A.1 in Appendix A. The modulus reduction curve and damping curve for tailings are selected to provide a good fit with test data from Cyclic Direct Simple Shear testing (CDSS) conducted by UBC in 2014 (Wijewickreme, D., 2014). The test data and selected modulus and damping reduction curves are presented on Figure A.2.

Table 3 Material Parameters for Seismic Response Analysis

Material	Density (pcf)	Stiffness			Damping curve
		K2max	Gmax (psf)	Modulus reduction curve	
Tailing Sand	120	Varies per section	Calculated from K_{2max}	Sand (Seed and Idriss, 1970) Lower Bound	Sand (Seed and Idriss, 1970) Lower Bound
Overburden	135	75	Calculated from K_{2max}	Gravel (Seed et al., 1986) Average	Gravel (Seed et al., 1986) Average
Embankment Fill	140	75	Calculated from K_{2max}	Rockfill (Gazetas and Dakoulas, 1992)	Sand (Seed and Idriss, 1970) Upper Bound
Bedrock	165	Not used	32×10^6	Rock (Schnabel, 1973)	Rock (Schnabel, 1973)

6 – PORE WATER PRESSURE CONDITIONS

The phreatic surface used in the liquefaction assessment for the current conditions was modelled to be consistent with measured conditions during the site investigations completed between 2012 and 2015. The location of the phreatic surface was determined from the results of pore pressure dissipation tests conducted during the CPT program. Pore pressures within the tailings deposit were found to be approximately 70% of hydrostatic, with the elevation of the phreatic surface varying depending on the location in the impoundment. Lower pore pressures were observed at locations close to the free draining embankment fill and overburden. However, this was ignored for the CSR calculation, resulting in conservatively high values.

The following assumptions were used for pore pressure conditions for future conditions (EL. 6,450 ft):

- At locations where the tailings extending to surface, the current phreatic surface was raised by the thickness of tailings deposited between now and the ultimate elevation (approximately 100 ft). The pore pressures were modelled to be 70% of hydrostatic below the phreatic surface to be consistent with current conditions.

- At locations where embankment fill is present at the surface, the phreatic surface was estimated to be at the base of the fill. The embankment fill is relatively free draining material and will prevent mounding of the phreatic surface. The pore pressures were modelled to be 70% of hydrostatic below the phreatic surface to be consistent with current conditions.

Active tailings deposition was taking place during the site investigation programs. The pore pressure conditions derived from the site investigation programs are therefore representative of normal operating conditions. Tailings located above the phreatic surface have a greatly reduced potential for liquefaction, as these are typically unsaturated (or partially saturated) during operations. The pore pressures are expected to decrease after closure, when active deposition has ceased. The pore pressures used for the liquefaction assessment of the EL. 6,450 ft embankment are therefore conservative for long-term conditions.

7 – CYCLIC STRESS RATIO (CSR)

Dynamic response analyses were performed to estimate the seismically induced cyclic shear stresses from the MCE. The CSR is defined as the average cyclic shear stress divided by effective confining stress.

The CSR analyses were carried out using the computer program SHAKE2000. This one-dimensional modelling program simulates the effect of seismic shear waves from an earthquake propagating upward from bedrock through the overlying materials. Input motion and material stiffness and damping parameters were used as described in the preceding sections, for both the current conditions and expanded conditions. The maximum CSR profiles resulting from the five earthquake motions are presented in Figures B.1 to B.12 in Appendix B for both the median and 84th percentile MCE.

8 – CYCLIC (LIQUEFACTION) RESISTANCE RATIO (CRR)

The resistance of soils to cyclic loading and potential liquefaction is represented by the CRR. The CRR is defined as the cyclic stress ratio required to initiate liquefaction.

The CRRs were calculated from the data recorded during the geotechnical site investigation programs. Profiles of CRR were estimated from the CPT data using the empirical methods outlined by Idriss and Boulanger (2014) and Robertson (2009). The software CLiq (Geologismiki, 2015) was used to process the CPT data.

Soils can have behave like sands or clays in response to cyclic loading. Cyclic loading of loose saturated sands will ultimately lead to liquefaction, resulting in loss of most of its strength. Plastic clays typically exhibit less strength loss as a result of cyclic loading, but can still cause significant deformation as a result of cyclic softening. The tailings deposit is highly stratified with inter-bedded layers of sand-silt mixtures. The Soil Behavior Type index (I_c) resulting from the CPT analysis indicated that the finer tailings may exhibit clay-like behaviour and the coarser tailings sand-like behaviour. The Atterberg Limits testing on tailings samples from the Standard Penetration Tests (SPTs) in 2012, 2013 and 2014 indicated the tailings along the analyzed sections are non-plastic. All tailings considered in the analysis were therefore assumed to have sand-like behaviour.

The CRR profiles were also derived from shear wave velocity data from the SCPTs using the procedure by Kayen et al. (2013).

All CRR values have been corrected to account for the effects of the design earthquake magnitude (MSF) and confining stress (K_σ). The correction factors were calculated according to the method used to derive the CRR.

The CRR values are derived using data from recent geotechnical site investigation programs, and are therefore representative of current conditions. The same CRR values were adopted for analysis of the expanded embankment, conservatively assuming no strength gain due to loading of materials. The resulting CRR values are presented on Figures B.1 to B.12 in Appendix B. A design line with a CRR of 0.07 is included on these figures. The design line was visually estimated based on the lowest CRR resulting from the various methods.

9 – RESULTS

The results of liquefaction analysis are presented in the figures of Appendix B. Liquefaction is predicted to occur within the saturated tailings at points where the CSR exceeds the CRR.

Summary of Results for the Operations Period

The analysis compares an upper bound envelope curve for CSR with a lower bound envelope CRR design line. The potential for liquefaction of tailings is predicted during operations where the tailings are saturated and the CSR for the median MCE case exceeds the CRR.

The liquefaction assessment indicates that saturated tailings within the YDTI and without the rockfill surcharge are potentially liquefiable under the median MCE during the operations period. Column 1 in each of the sections is representative of this prediction both for current conditions and future embankment conditions. The potentially liquefiable tailings extend to approximately 200 ft depth in the central impoundment area; however, the top 50 to 80 ft or more of tailings beach adjacent to the embankment is unsaturated or partially saturated and is therefore not likely to liquefy.

Tailings below the surcharged areas have very low liquefaction potential for operating conditions with seismic loading from the median MCE. The analysis demonstrates that the rockfill surcharge effectively mitigates the potential for cyclic liquefaction of the surcharged tailings zone during operations. The potential for liquefaction is reduced due to the loading from the overlying rockfill mass. The top 50 ft or more of the tailings are also unsaturated or partially saturated and are not subject to liquefaction. The tailings discharge point has also been relocated further out into the impoundment due to the construction of the rockfill surcharge, which has reduced pore pressures in the tailings near the embankment and increased the depth of the unsaturated zone in the tailings below the surcharge.

The deep tailings below EL. 6,000 ft in section 18+00N are not expected to liquefy assuming the same CRR as for recently deposited tailings.

Summary of Results for the Closure Period

The potential for liquefaction of tailings is predicted during closure if the CSR for the 84th-Percentile MCE exceeds the CRR. The liquefaction assessment indicates that the upper saturated tailings within the YDTI and outside the rockfill surcharge area are potentially liquefiable to approximately 275 ft depth with seismic loading from the 84th-percentile MCE during the closure period. Localized liquefaction of the tailings underlying the rockfill surcharge may also occur while the tailings remain saturated. The potentially liquefiable tailings will be contained by non-liquefiable foundation soil, bedrock, and free draining embankment rockfill. The top 50 ft to 80 ft or more of tailings beach adjacent to the embankment and beneath the rockfill surcharge is currently unsaturated or partially saturated and is therefore not likely to liquefy. The tailings pore water will drain down and the thickness of this unsaturated zone will increase following closure, which progressively reduces the liquefaction potential of the tailings in the vicinity of the embankments.

The deep tailings (below 6,000 ft) in section 18+00N are not expected to liquefy for expansion conditions assuming the same CRR as for recently deposited tailings.

It should be noted that substantial conservatism is embedded in the liquefaction assessment for the expansion conditions. The most extreme earthquake loading conditions were applied for CSR determination. The lower bound of the CRR values is used to derive the design line without allowing for any increase in CRR due to consolidation of the tailings mass. The CSR lines have been adjusted to account for the pressure of the overlying mass, but the CRR was not. The pore pressure may also be lower than assumed due to on-going consolidation of the tailings deposit.

10 – CONCLUSIONS AND RECOMMENDATIONS

The liquefaction assessment indicates at the current and future conditions that tailings in the central impoundment area are potentially subject to liquefaction under maximum design earthquakes. The potentially liquefiable tailings extend to beyond 200 ft deep in the central impoundment area, but are limited or not subject to liquefaction in the areas surcharged by rockfill. The top 50 ft or more of tailings beach adjacent to the embankment is unsaturated or partially saturated, and is not subject to liquefaction. It should be noted that there has been no recorded evidence suggesting that liquefaction occurs to a depth of 200 ft due to the high confining pressure at depth.

There is inherent conservatism throughout the liquefaction assessment, and the likelihood of tailings liquefaction adjacent to the embankment in the areas surcharged by rockfill is low. Two dimensional limit equilibrium slope stability analysis should be performed to check the post-earthquake stability of the embankment assuming liquefaction will occur in the tailings at the identified locations. The earthquake-induced deformation potential should also been estimated.

Please contact the undersigned with any questions or comments.

Yours truly,
Knight Piésold Ltd.

PROFESSIONAL
PROVINCE OF BRITISH COLUMBIA
August 14, 2017
D. D. FONTAINE
36208
ENGINEER

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Project Engineering

Reviewed: Daniel Fontaine, P.Eng.
Senior Civil Engineer | Associate

Approval that this document adheres to Knight Piésold Quality Systems: DDF

Attachments:

Appendix A – Shear Modulus Reduction and Damping Ratio Versus Shear Strain Relationships
- Figure A.1 Rev 0 Shear Modulus Reduction and Damping Ratios – Embankment Fill Materials
- Figure A.2 Rev 0 Shear Modulus Reduction and Damping Ratios – Tailings

Appendix B – Liquefaction Assessment Figures
- Figure B.1 Rev 0 Liquefaction Assessment Current Conditions - Section 8+00W Column 1
- Figure B.2 Rev 0 Liquefaction Assessment Current Conditions - Section 8+00W Column 2
- Figure B.3 Rev 0 Liquefaction Assessment Current Conditions - Section 8+00W Column 3
- Figure B.4 Rev 0 Liquefaction Assessment Current Conditions - Section 38+00NW Column 1
- Figure B.5 Rev 0 Liquefaction Assessment Current Conditions - Section 38+00NW Column 2
- Figure B.6 Rev 0 Liquefaction Assessment Current Conditions - Section 18+00N Column 1
- Figure B.7 Rev 0 Liquefaction Assessment Future Conditions – EL. 6,450 FT Section 8+00W Column 1
- Figure B.8 Rev 0 Liquefaction Assessment Future Conditions – EL. 6,450 FT Section 8+00W Column 2
- Figure B.9 Rev 0 Liquefaction Assessment Future Conditions – EL. 6,450 FT Section 8+00W Column 3
- Figure B.10 Rev 0 Liquefaction Assessment Future Conditions – EL 6,450 FT Section 38+00NW Column 1
- Figure B.11 Rev 0 Liquefaction Assessment Future Conditions – EL. 6,450 FT Section 38+00NW Column 2
- Figure B.12 Rev 0 Liquefaction Assessment Future Conditions – EL. 6,450 FT Section 18+00N Column 1

/as

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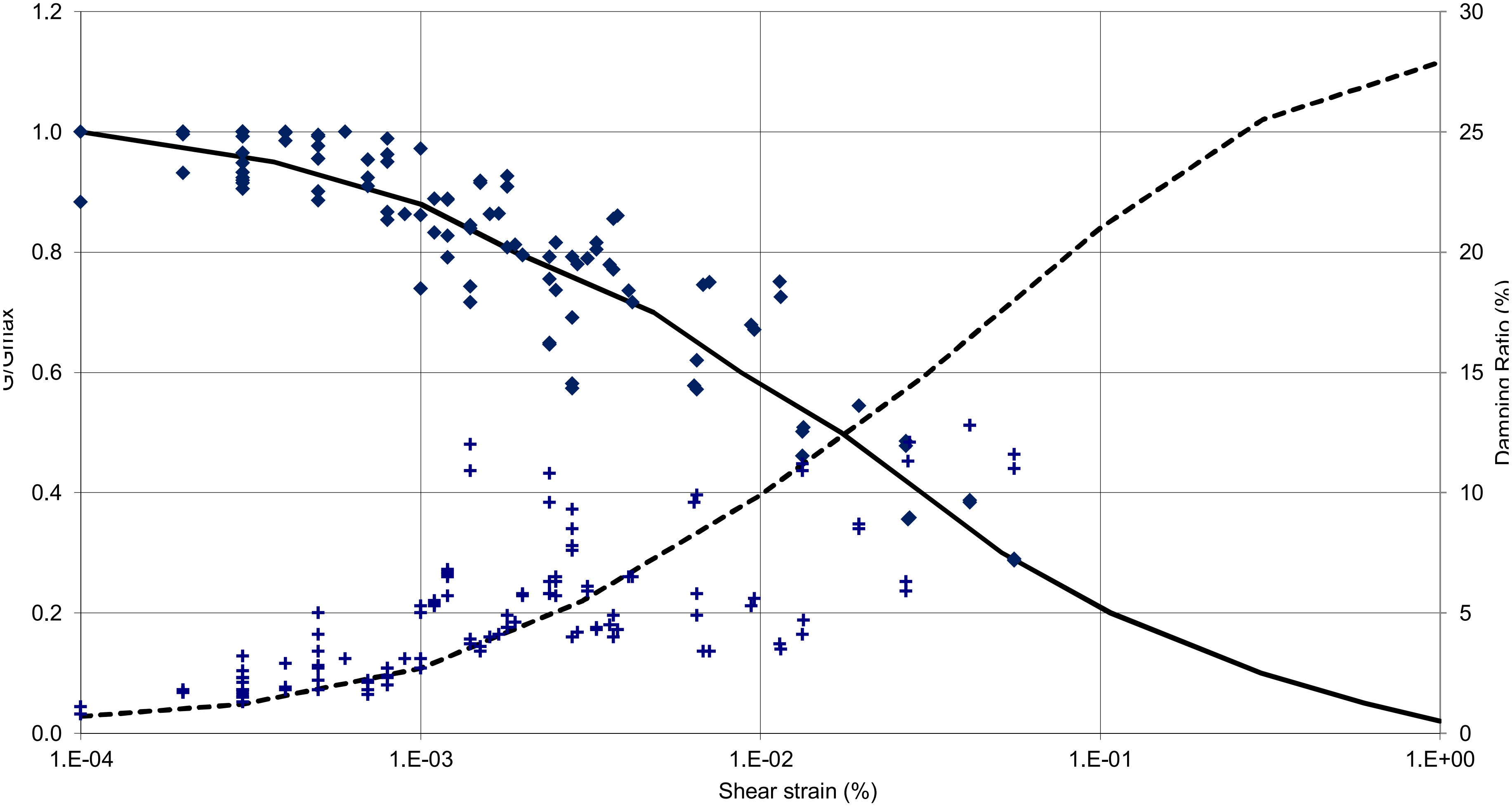
APPENDIX A

SHEAR MODULUS REDUCTION VERSUS SHEAR STRAIN AND DAMPING RATIO VERSUS SHEAR STRAIN RELATIONSHIPS

(Pages A-1 to A-2)



NOTES:
1. BASED ON CYCLIC TRIAXIAL TESTING COMPLETED BY INTERNATIONAL ENGINEERING COMPANY (IECO, 1981).

- G/Gmax
- G/Gmax - ROCKFILL (Gazetas 1986)
- Damping Ratio
- Damping - SAND, Upper Bound (Seed & Idriss 1970)

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	06JUL'16	ISSUED WITH LETTER	SB	DAY

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

SHEAR MODULUS REDUCTION AND DAMPING RATIOS EMBANKMENT FILL MATERIALS

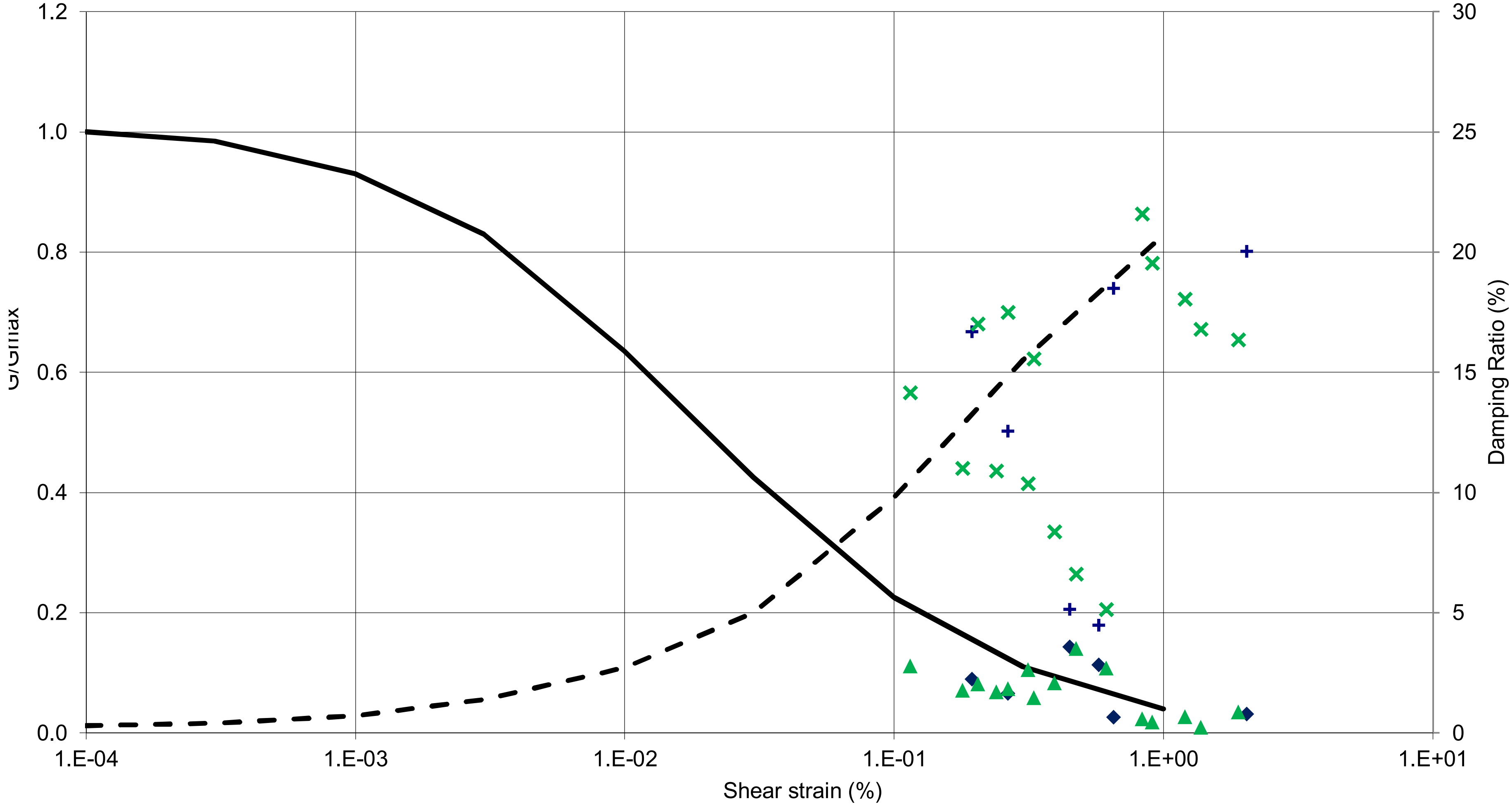
Knight Piésold CONSULTING

P/A NO. VA101-126/16

REF. NO. VA17-01098

FIGURE A.1

REV 0



NOTES:

1. BASED ON DATA FROM CYCLIC DIRECT SIMPLE SHEAR (CDSS) TESTING CONDUCTED AT THE UNIVERSITY OF BRITISH COLUMBIA (UBC) IN 2014.

- G/Gmax Area 2 (Fine Tailings)
- G/Gmax Area 3 (Coarse Tailings)
- G/Gmax - SAND, Lower Bound (Seed & Idriss 1970)
- Damping Ratio Area 2 (Fine Tailings)
- Damping Ratio Area 3 (Coarse Tailings)
- Damping - SAND, Lower Bound (Seed & Idriss 1970)

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	06JUL'16	ISSUED WITH LETTER	SB	DAY

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

SHEAR MODULUS REDUCTION AND DAMPING RATIOS TAILINGS

Knight Piésold CONSULTING

P/A NO. VA101-126/16

REF. NO. VA17-01098

FIGURE A.2

REV 0

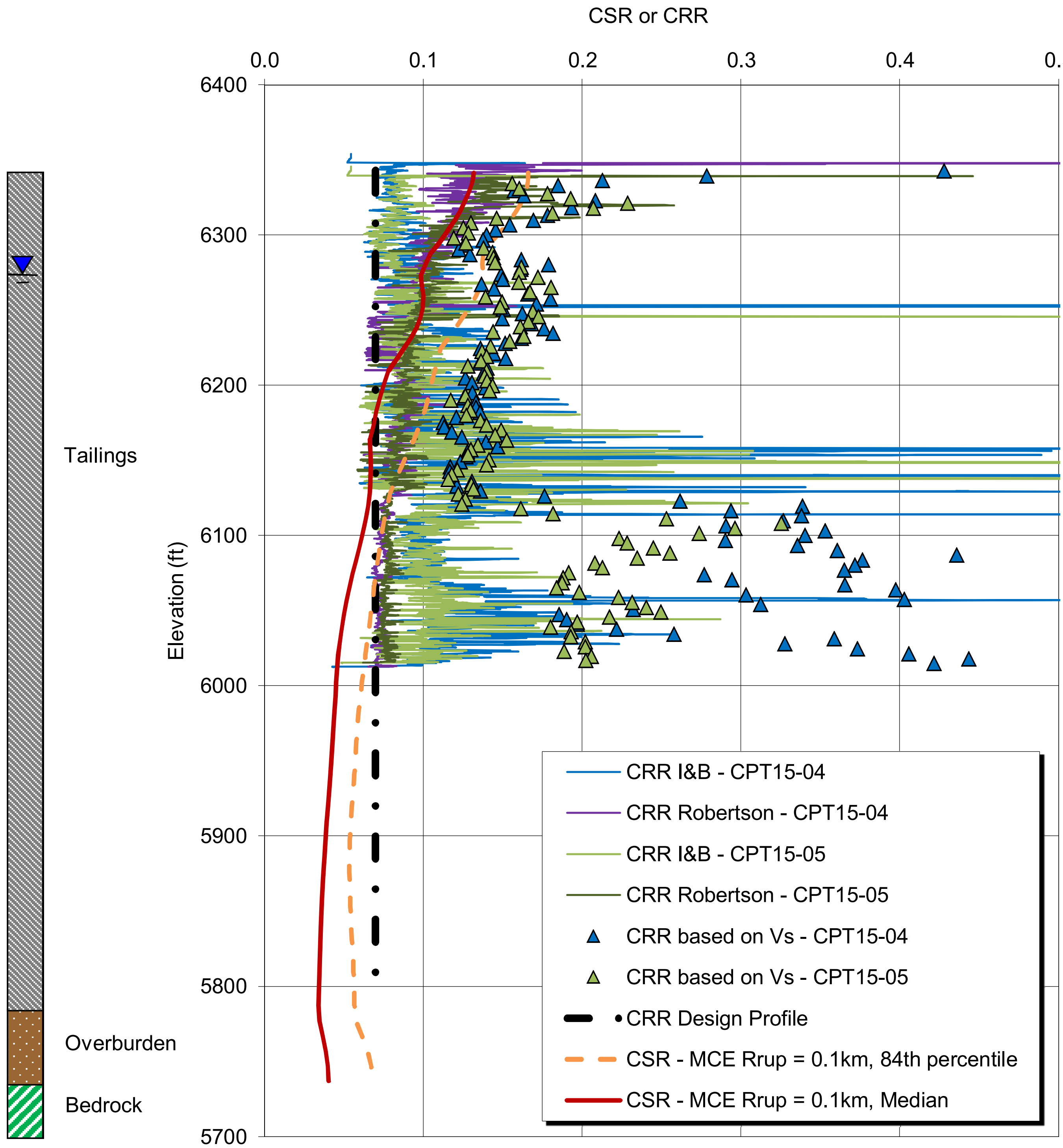
Knight Piésold CONSULTING

APPENDIX B

LIQUEFACTION ASSESSMENT FIGURES

(Pages B-1 to B-12)

VA17-01098
August 14, 2017



NOTES:
1. WATER LEVEL BASED ON PORE PRESSURE DISSIPATION TESTS.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

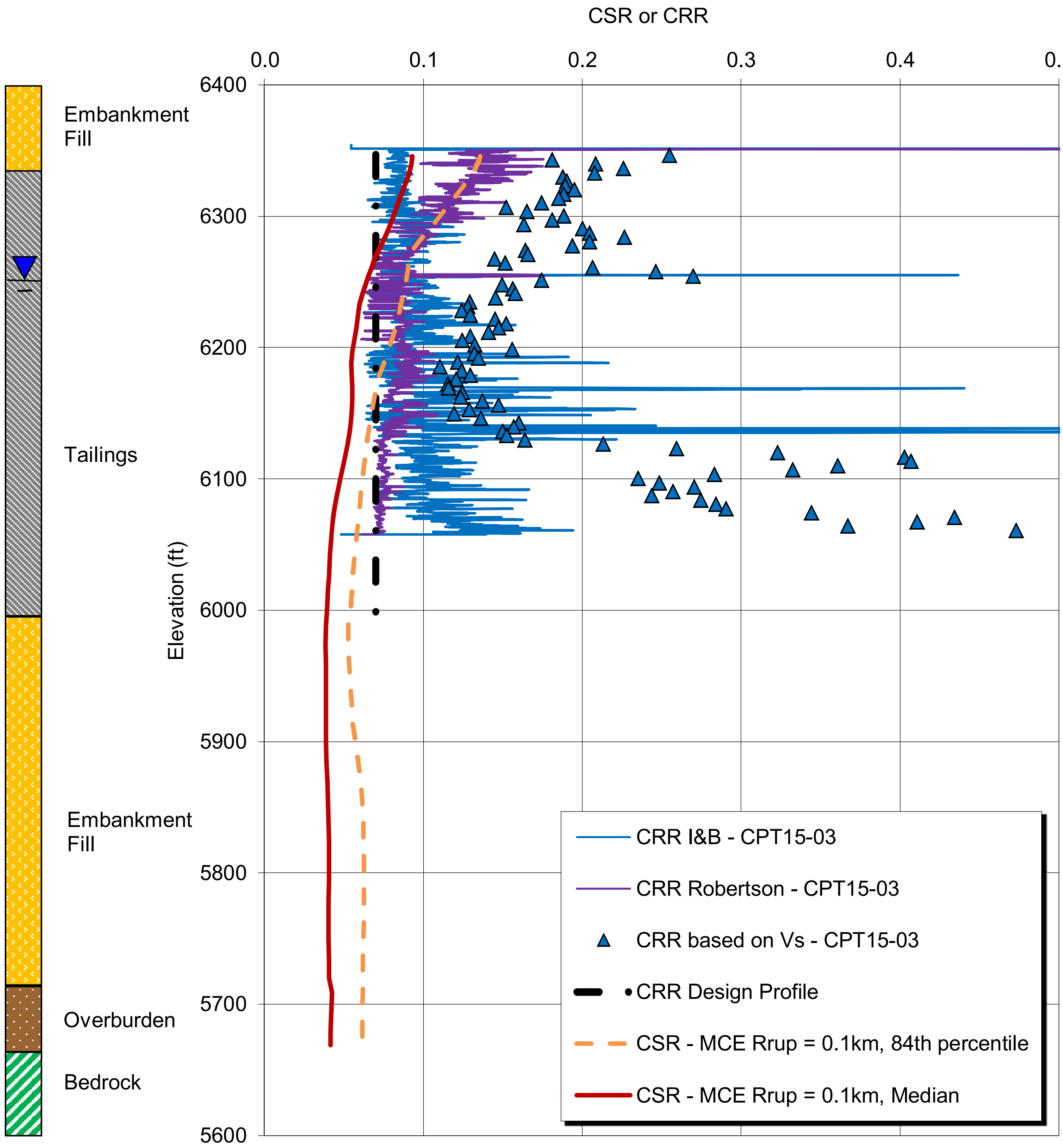
LIQUEFACTION ASSESSMENT
CURRENT CONDITIONS
SECTION 8+00W COLUMN 1

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.1 REV 0

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	08AUG'17	ISSUED WITH LETTER	AS2	DDF



NOTES:
1. WATER LEVEL BASED ON PORE PRESSURE DISSIPATION TESTS.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
CURRENT CONDITIONS
SECTION 8+00W COLUMN 2

Knight Piésold CONSULTING

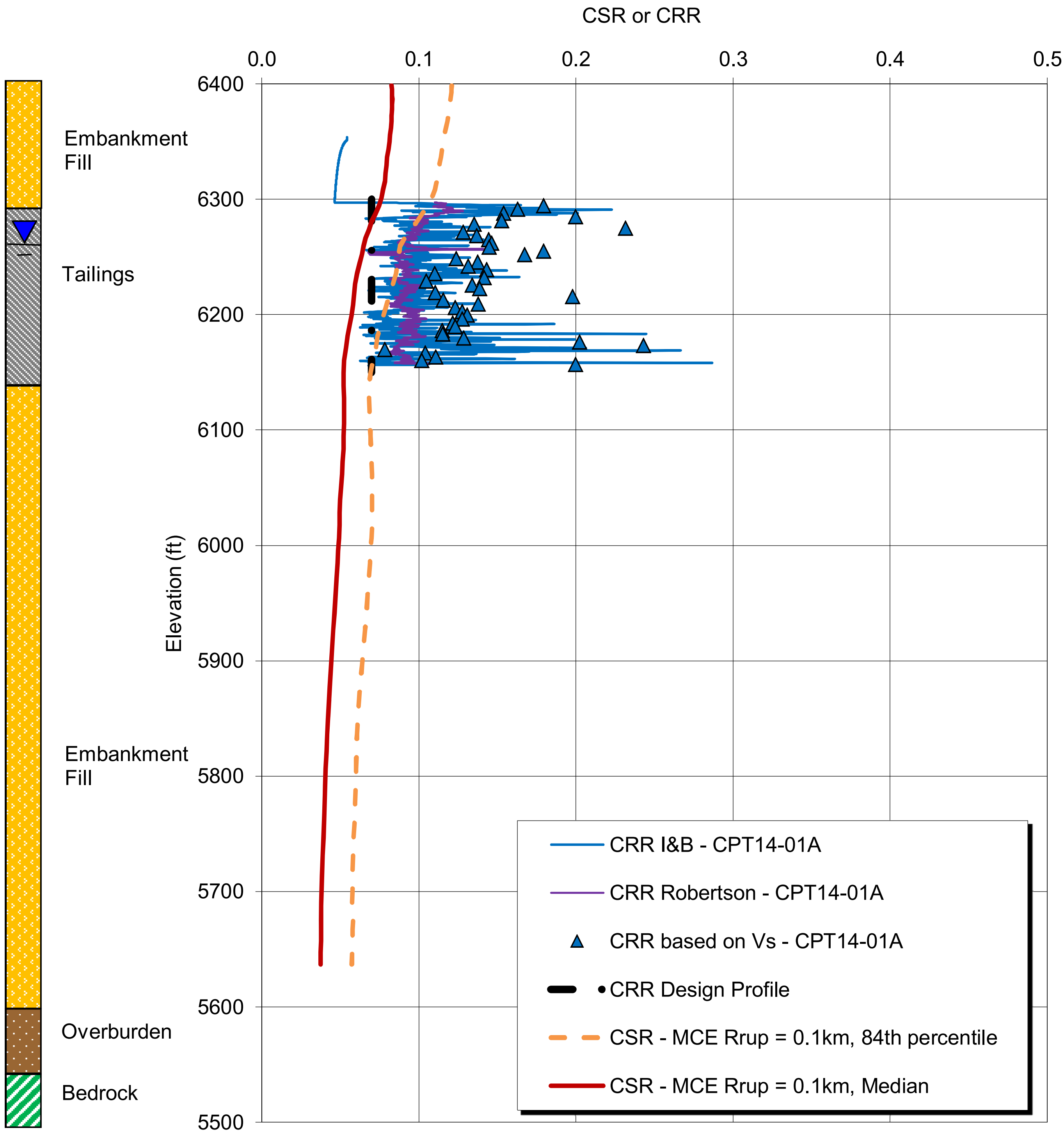
P/A NO. VA101-126/16

REF. NO. VA17-01098

FIGURE B.2

REV 0

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	08AUG'17	ISSUED WITH LETTER	AS2	DDF



NOTES:
1. WATER LEVEL BASED ON PORE PRESSURE DISSIPATION TESTS.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	08AUG'17	ISSUED WITH LETTER	AS2	DDF

MONTANA RESOURCES, LLP

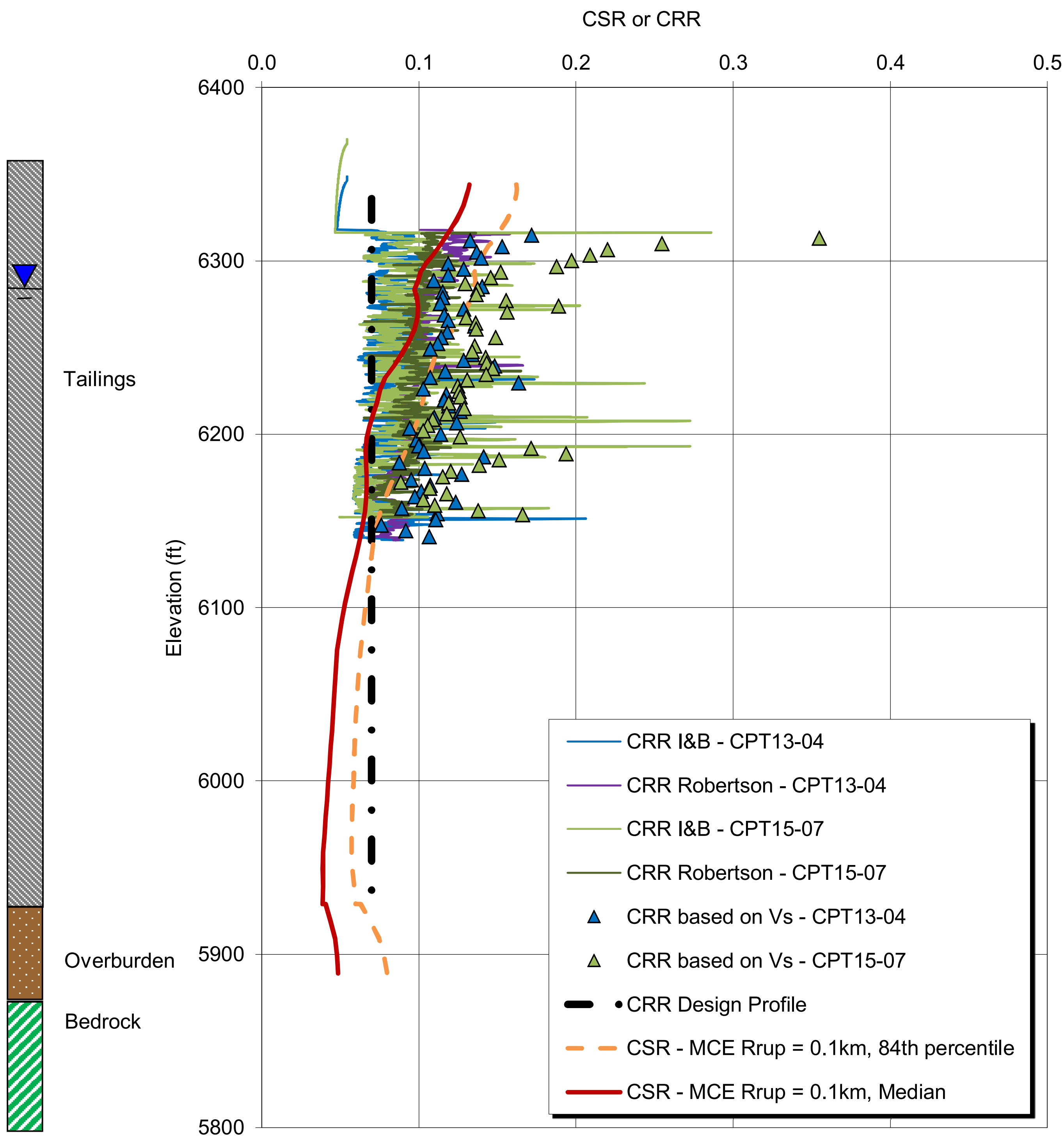
YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
CURRENT CONDITIONS
SECTION 8+00W COLUMN 3

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.3 REV 0



NOTES:
1. WATER LEVEL BASED ON PORE PRESSURE DISSIPATION TESTS.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

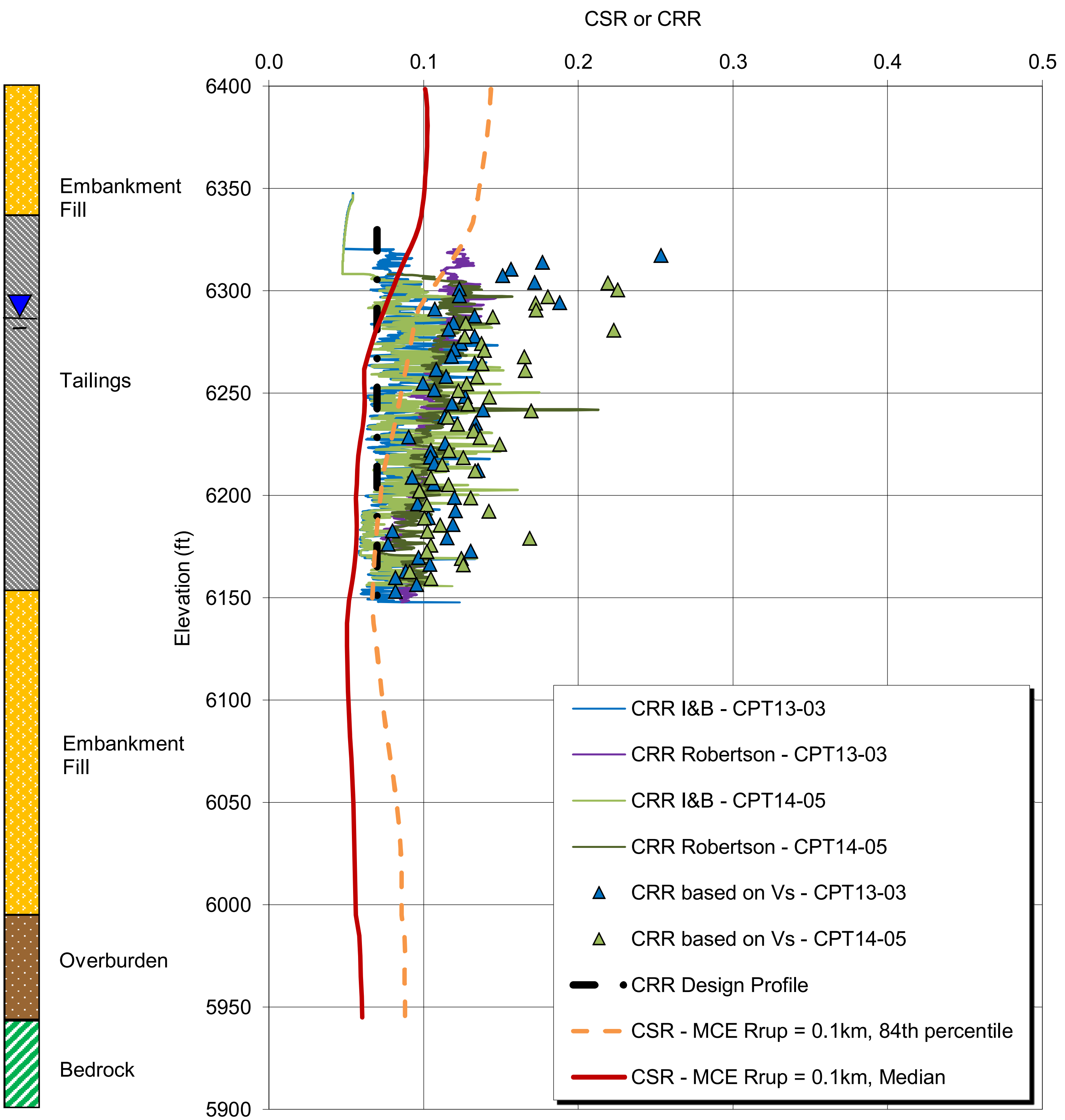
LIQUEFACTION ASSESSMENT
CURRENT CONDITIONS
SECTION 38+00NW COLUMN 1

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.4 REV 0

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	08AUG'17	ISSUED WITH LETTER	AS2	DDF



NOTES:
1. WATER LEVEL BASED ON PORE PRESSURE DISSIPATION TESTS.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

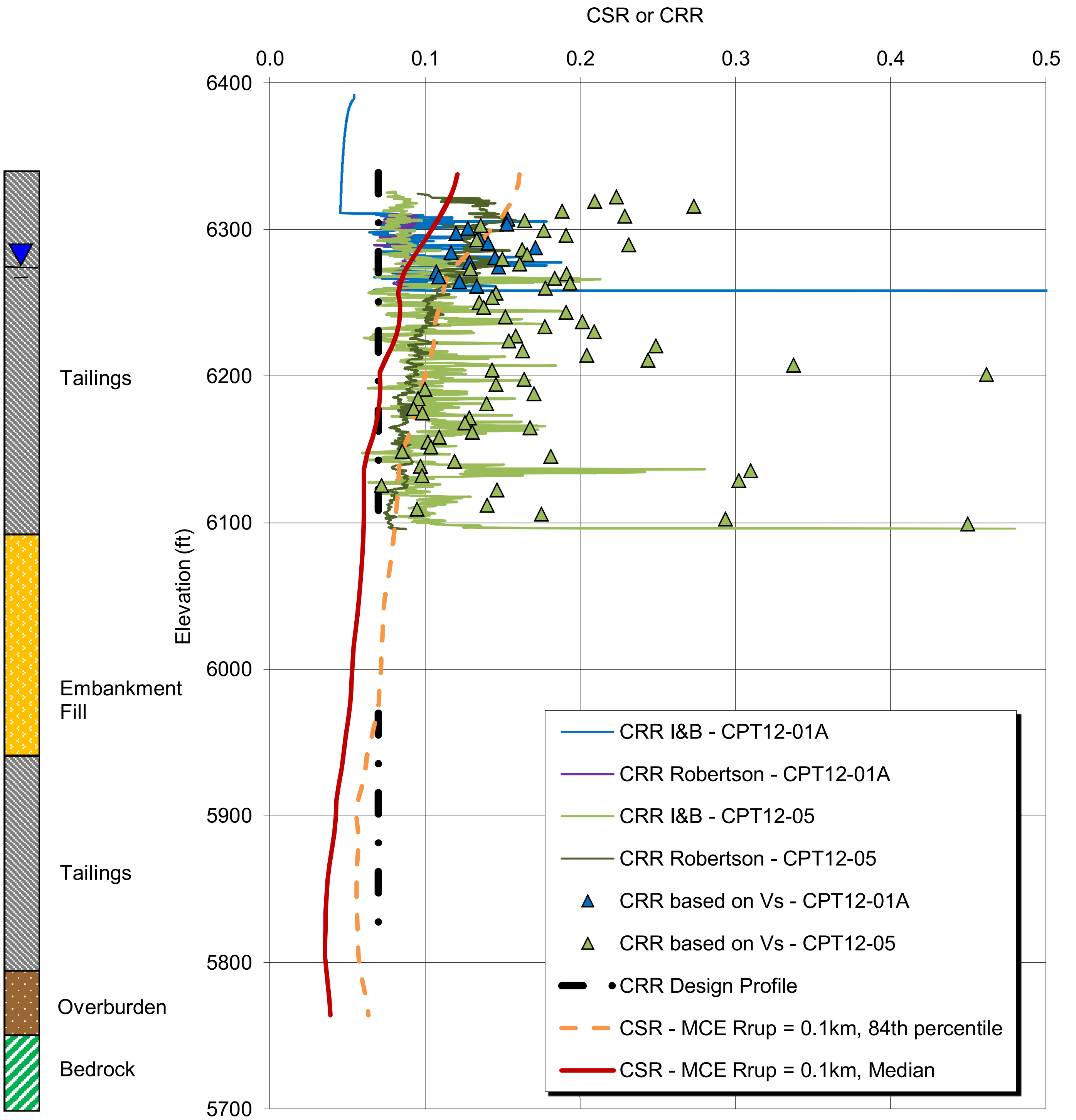
LIQUEFACTION ASSESSMENT
CURRENT CONDITIONS
SECTION 38+00NW COLUMN 2

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.5 REV 0

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	08AUG'17	ISSUED WITH LETTER	AS2	DDF



NOTES:
1. WATER LEVEL BASED ON PORE PRESSURE DISSIPATION TESTS.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.

0	08AUG'17	ISSUED WITH LETTER	AS2	DDF
REV	DATE	DESCRIPTION	PREP'D	RVW'D

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
CURRENT CONDITIONS
SECTION 18+00N COLUMN 1

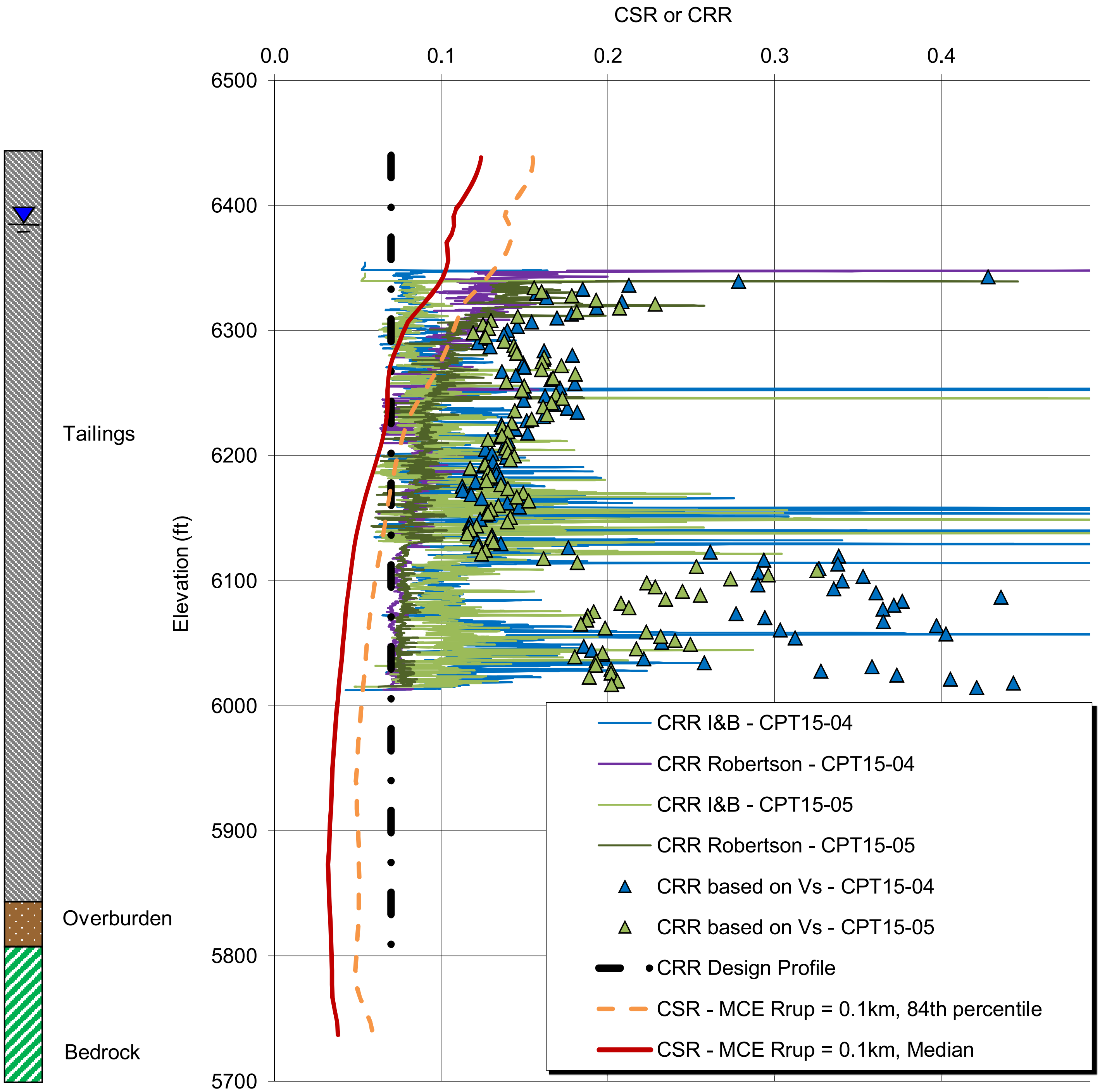
Knight Piésold CONSULTING

P/A NO. VA101-126/16

REF. NO. VA17-01098

FIGURE B.6

REV 0



NOTES:

1. WATER LEVEL ASSUMED TO BE 100ft ABOVE CURRENT ELEVATION..
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.
3. CSR INCORPORATED SURCHARGE LOADING OF TAILINGS AND/OR ROCKFILL.
4. CRR ASSUMED THE SAME AS CURRENT CONDITIONS.

0	14AUG'17	ISSUED WITH LETTER	AS2	DDF
REV	DATE	DESCRIPTION	PREP'D	RVW'D

MONTANA RESOURCES, LLP

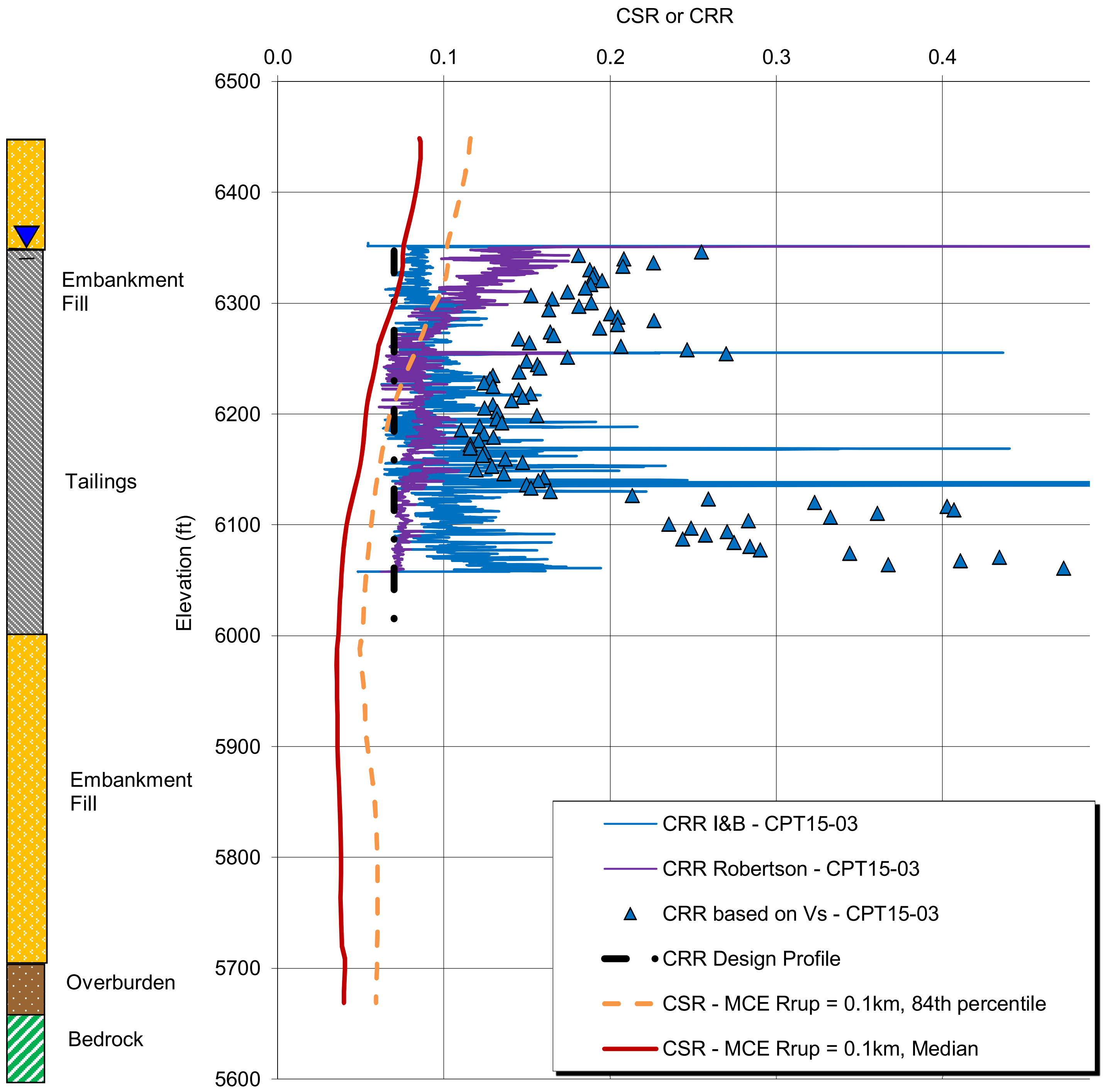
YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
FUTURE CONDITIONS - EL. 6,450 FT
SECTION 8+00W COLUMN 1

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.7 REV 0



NOTES:
1. WATER LEVEL ASSUMED AT BASE OF EMBANKMENT FILL.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.
3. CSR INCORPORATED SURCHARGE LOADING OF TAILINGS AND/OR ROCKFILL.
4. CRR ASSUMED THE SAME AS CURRENT CONDITIONS.

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	14AUG'17	ISSUED WITH LETTER	AS2	DDF

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
FUTURE CONDITIONS - EL. 6,450 FT
SECTION 8+00W COLUMN 2

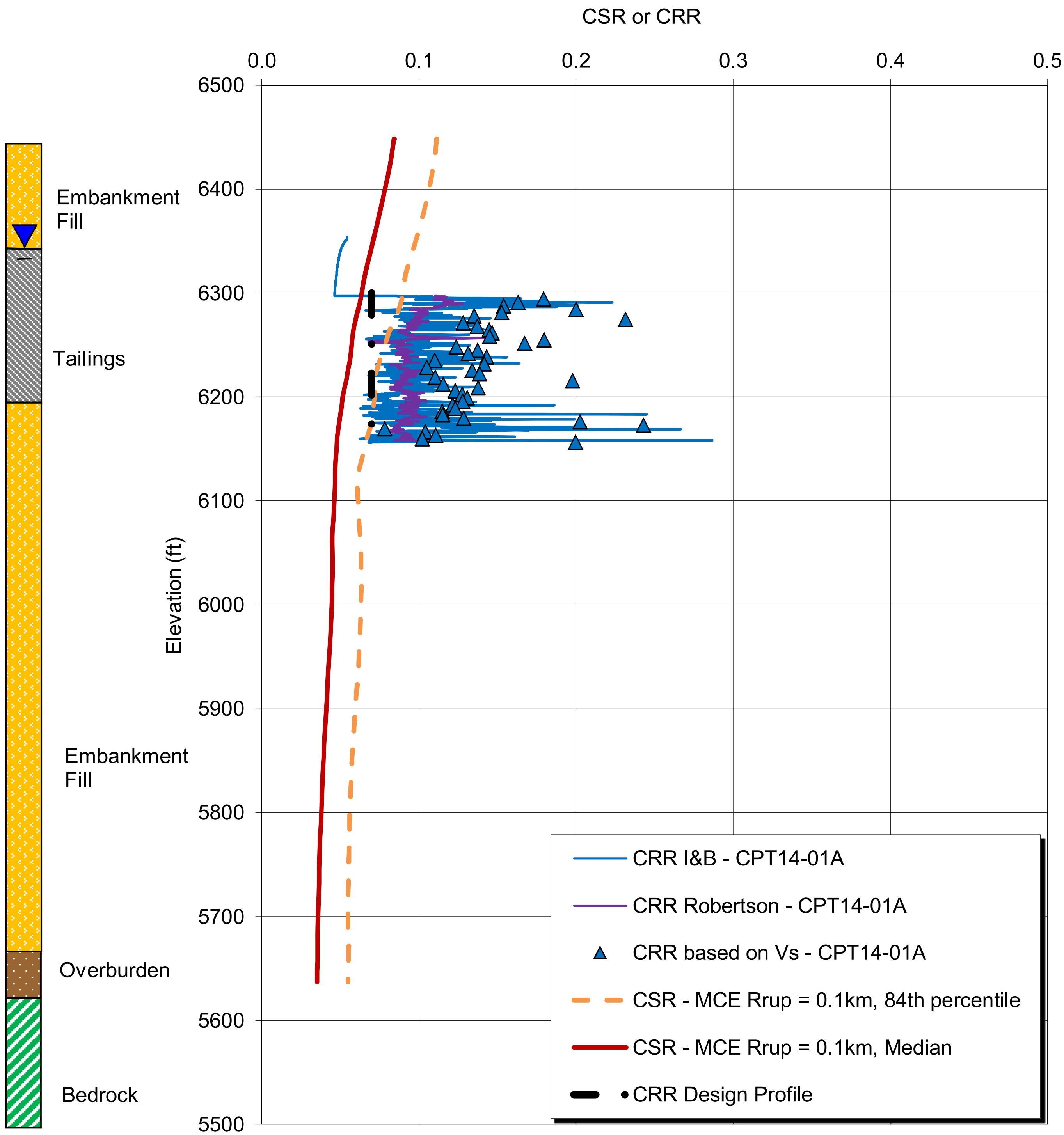
Knight Piésold CONSULTING

P/A NO. VA101-126/16

REF. NO. VA17-01098

FIGURE B.8

REV 0



NOTES:
1. WATER LEVEL ASSUMED AT BASE OF EMBANKMENT FILL.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.
3. CSR INCORPORATED SURCHARGE LOADING OF TAILINGS AND/OR ROCKFILL.
4. CRR ASSUMED THE SAME AS CURRENT CONDITIONS.

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

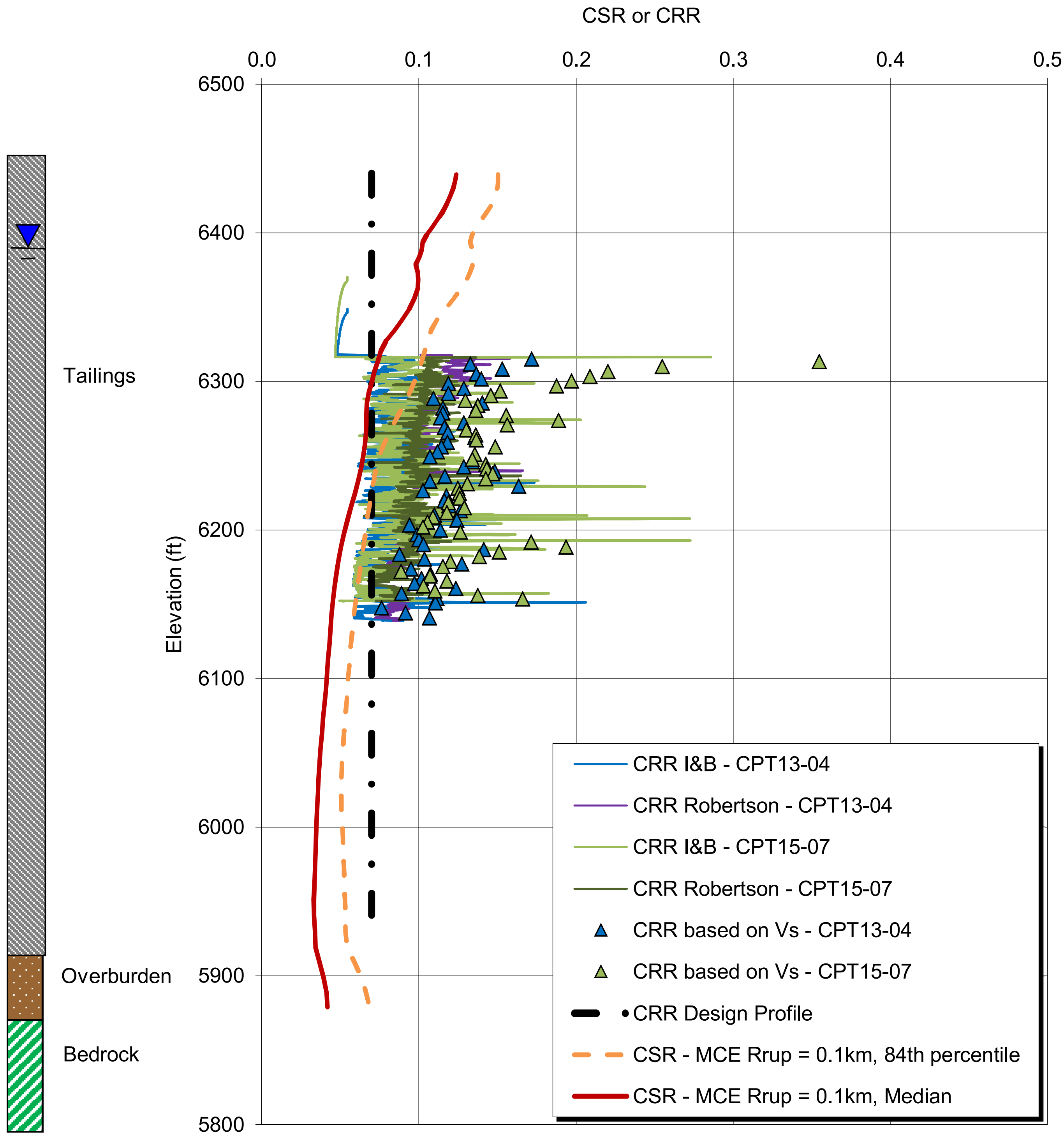
LIQUEFACTION ASSESSMENT
FUTURE CONDITIONS - EL. 6,450 FT
SECTION 8+00W COLUMN 3

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.9 REV 0

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	14AUG'17	ISSUED WITH LETTER	AS2	DDF



NOTES:
1. WATER LEVEL ASSUMED TO BE 100ft ABOVE CURRENT ELEVATION.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.
3. CSR INCORPORATED SURCHARGE LOADING OF TAILINGS AND/OR ROCKFILL.
4. CRR ASSUMED THE SAME AS CURRENT CONDITIONS.

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	14AUG'17	ISSUED WITH LETTER	AS2	DDF

MONTANA RESOURCES, LLP

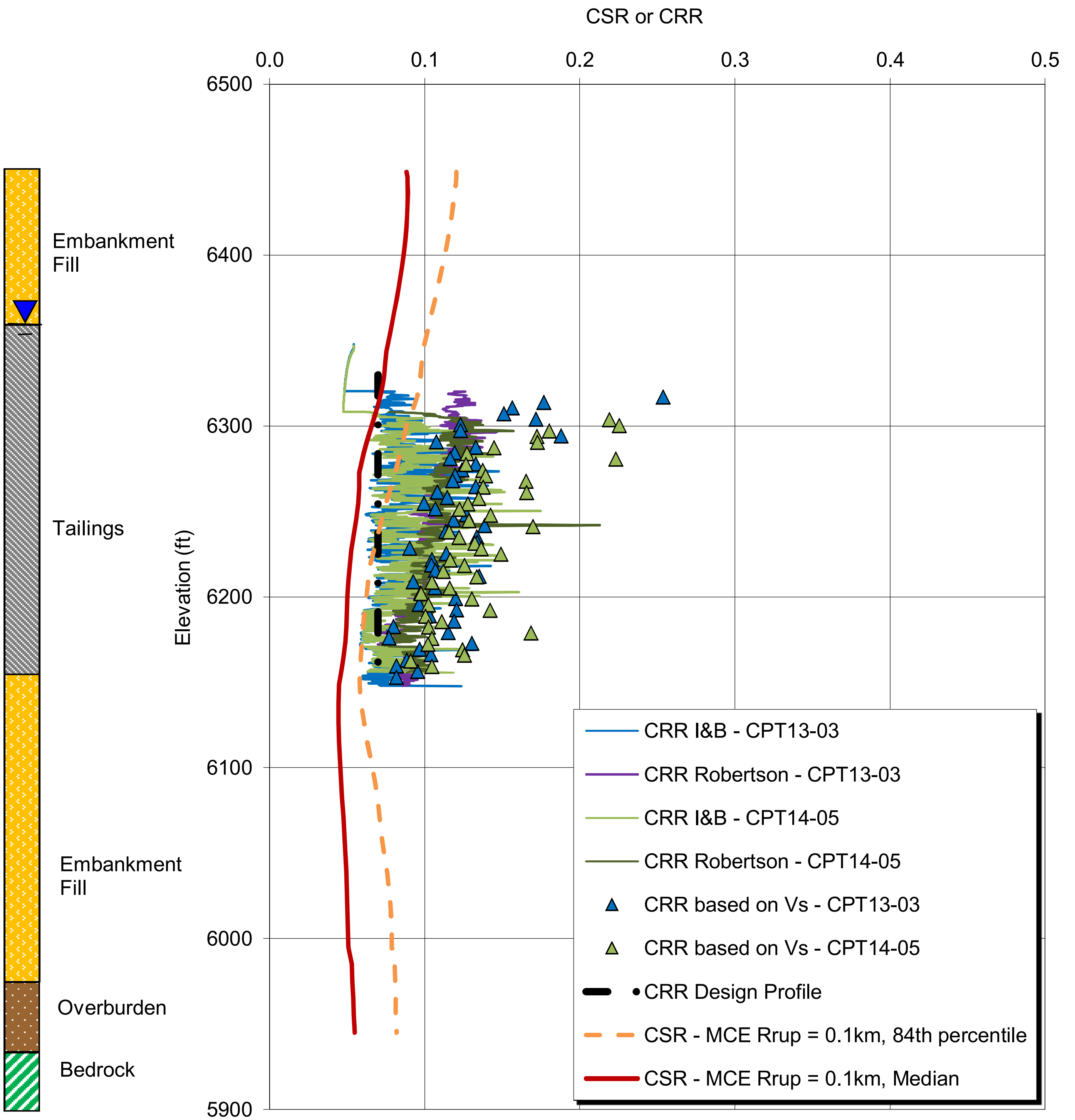
YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
FUTURE CONDITIONS - EL. 6,450 FT
SECTION 38+00NW COLUMN 1

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.10 REV 0



NOTES:
1. WATER LEVEL ASSUMED AT BASE OF EMBANKMENT FILL.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.
3. CSR INCORPORATED SURCHARGE LOADING OF TAILINGS AND/OR ROCKFILL.
4. CRR ASSUMED THE SAME AS CURRENT CONDITIONS.

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

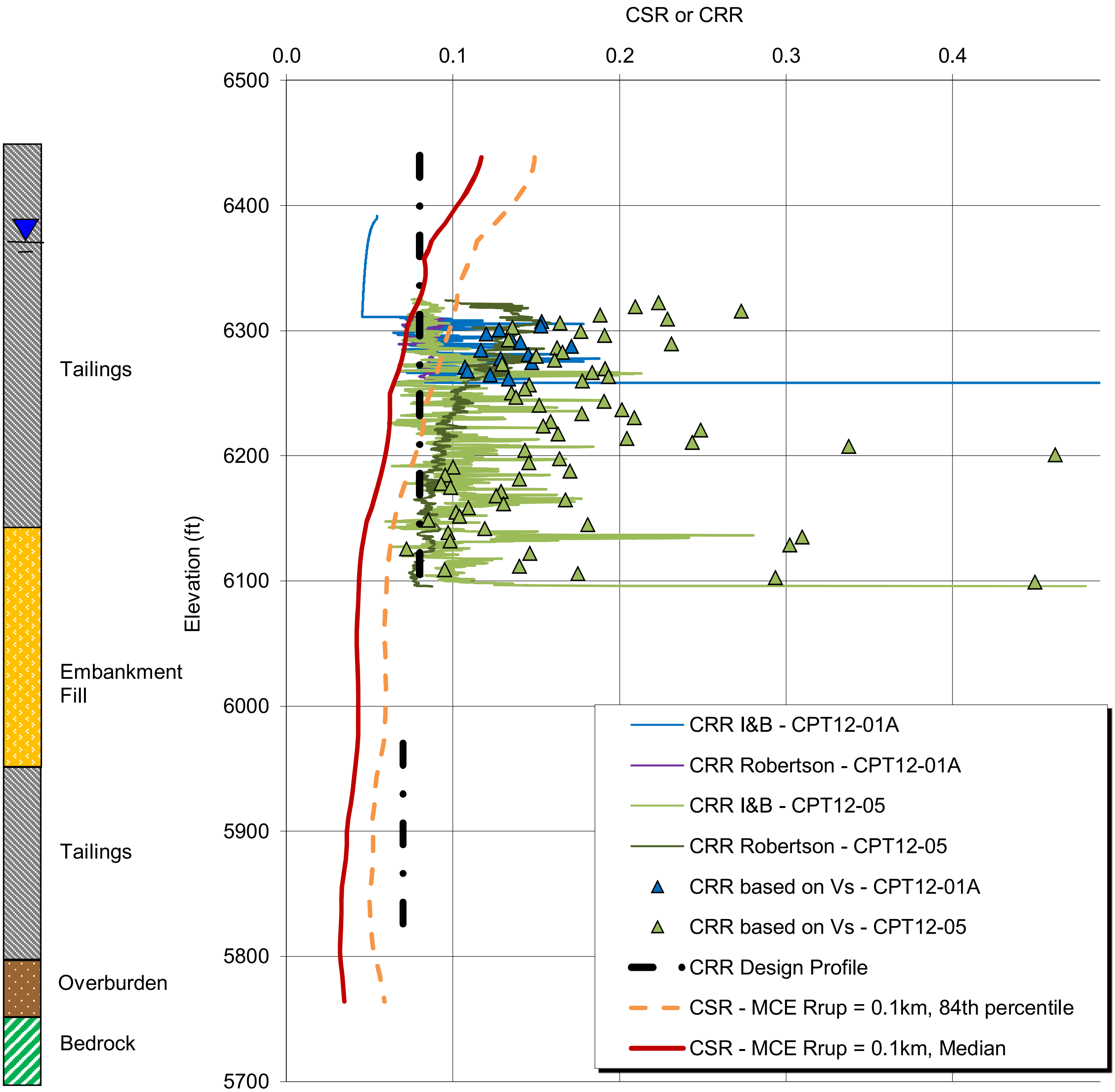
LIQUEFACTION ASSESSMENT
FUTURE CONDITIONS - EL. 6,450 FT
SECTION 38+00NW COLUMN 2

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/16	VA17-01098

FIGURE B.11 REV 0

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	14AUG'17	ISSUED WITH LETTER	AS2	DDF



NOTES:
1. WATER LEVEL ASSUMED TO BE 100ft ABOVE CURRENT ELEVATION.
2. PORE PRESSURES MODELLED AS 70% OF HYDROSTATIC PRESSURE.
3. CSR INCORPORATED SURCHARGE LOADING OF TAILINGS AND/OR ROCKFILL.
4. CRR ASSUMED THE SAME AS CURRENT CONDITIONS.

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	14AUG'17	ISSUED WITH LETTER	AS2	DDF

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

LIQUEFACTION ASSESSMENT
FUTURE CONDITIONS - EL. 6,450 FT
SECTION 18+00N COLUMN 1

Knight Piésold CONSULTING

P/A NO. VA101-126/16

REF. NO. VA17-01098

FIGURE B.12

REV 0

APPENDIX B

LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSES

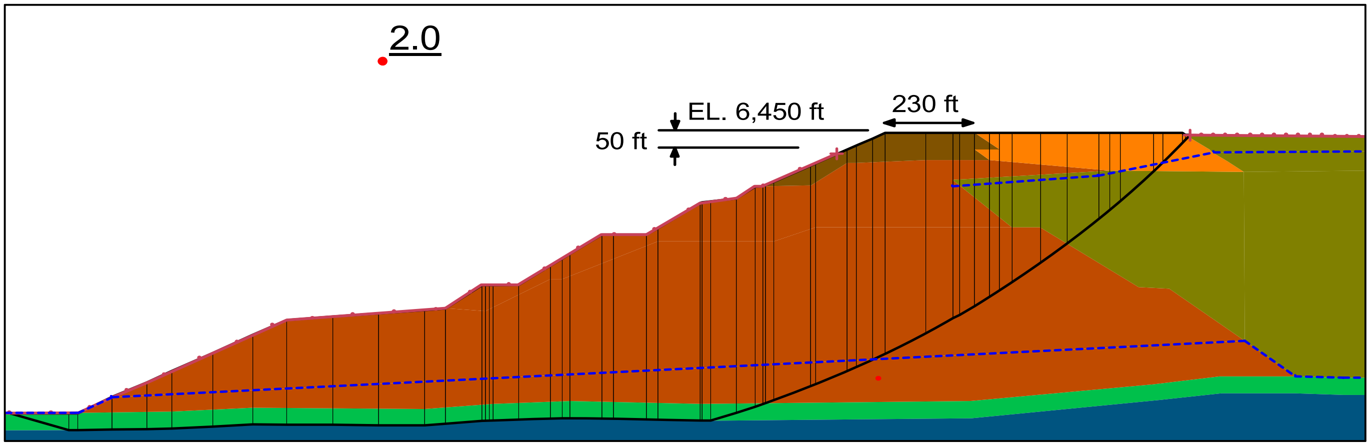
Appendix B1 Normal Operating Conditions
Appendix B2 Post-Earthquake Conditions
Appendix B3 Undrained Response Sensitivity Analysis
Appendix B4 Staged Sensitivity Analysis

APPENDIX B1

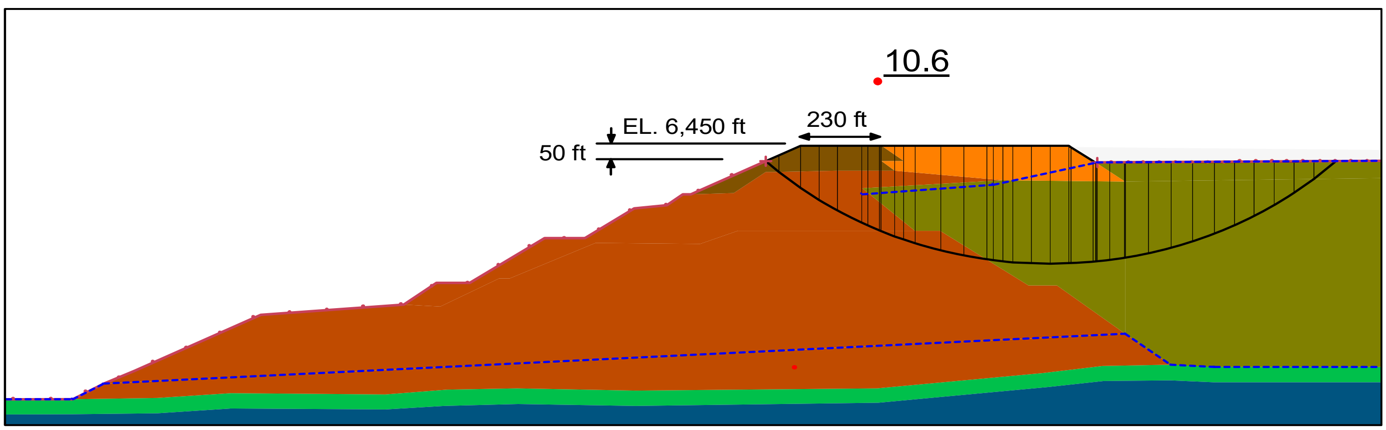
NORMAL OPERATING CONDITIONS

(Pages B1-1 to B1-4)

DOWNSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



UPSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

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YANKEE DOODLE TAILINGS IMPOUNDMENT

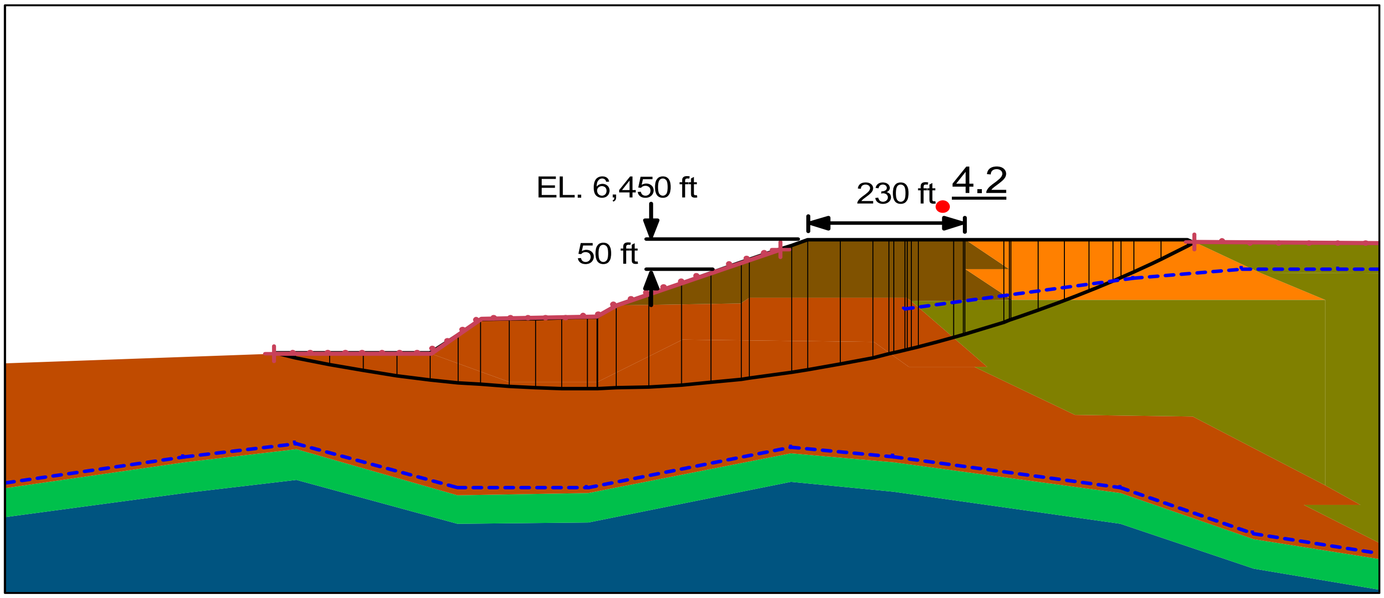
NORMAL OPERATING CONDITIONS
EAST-WEST EMBANKMENT
SECTION 08+00W

Knight Piésold CONSULTING

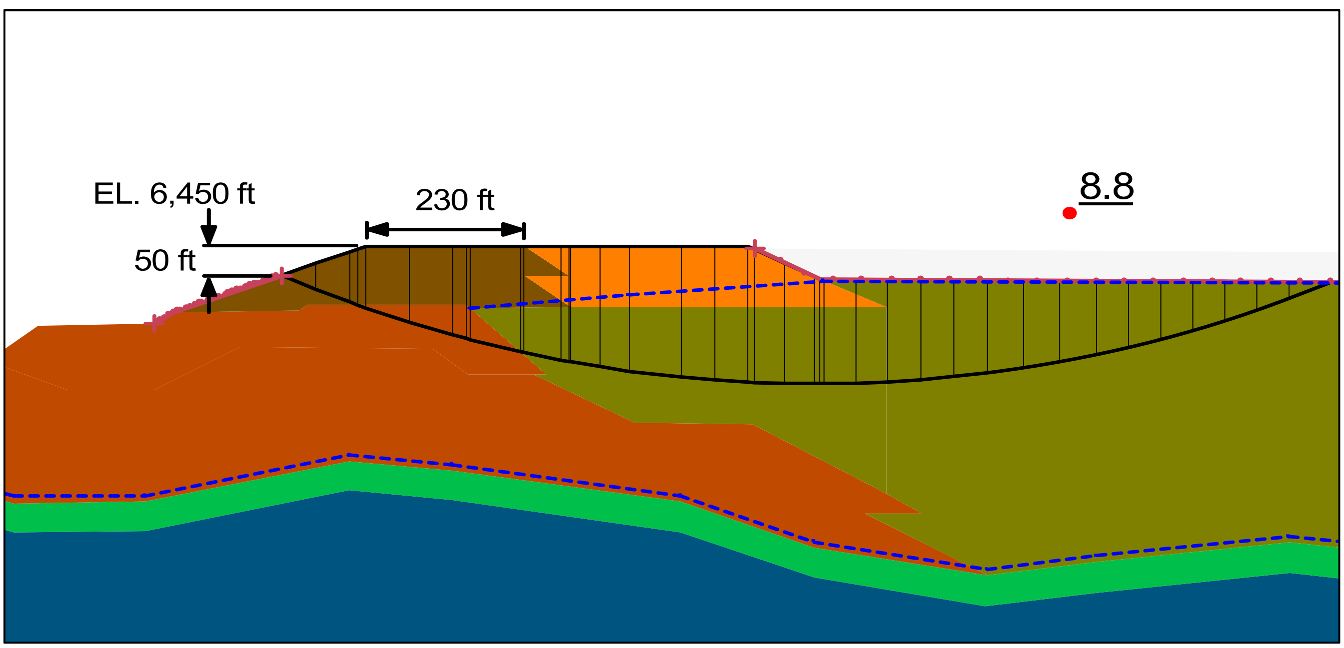
P/A NO.	REF. NO.
VA101-126/12	2

FIGURE B1.1 REV 2

DOWNSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



UPSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

NORMAL OPERATING CONDITIONS
EAST-WEST EMBANKMENT
SECTION 38+00NW

Knight Piésold CONSULTING

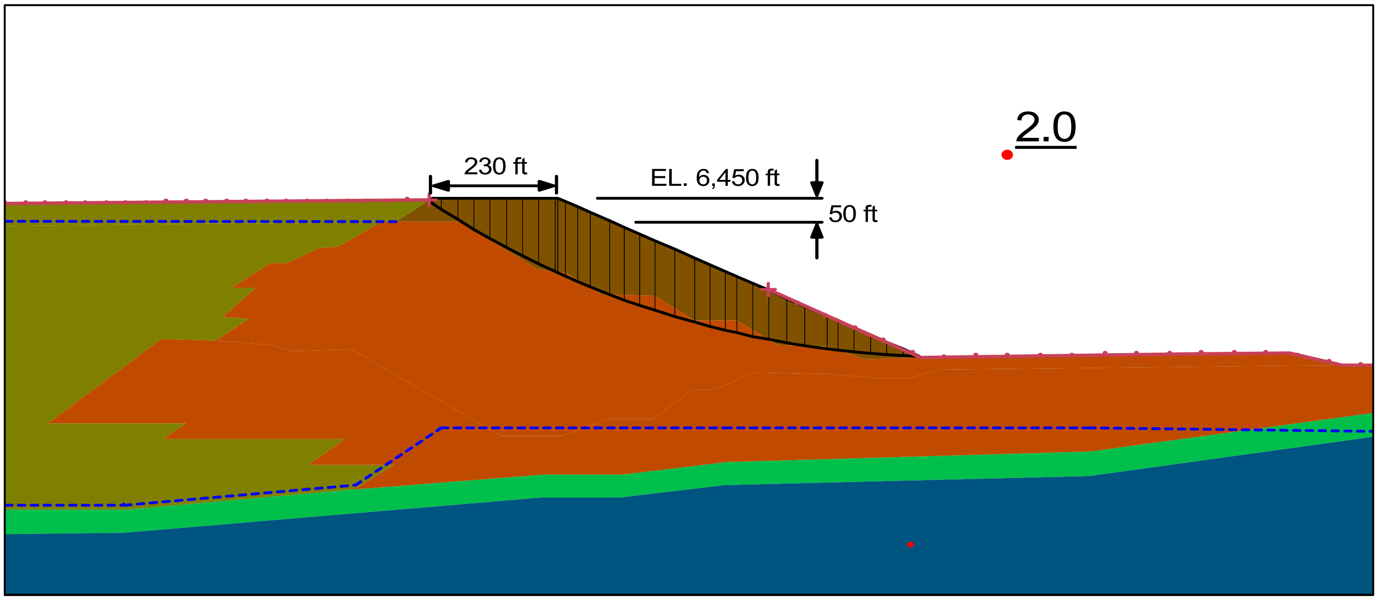
P/A NO. VA101-126/12

REF. NO. 2

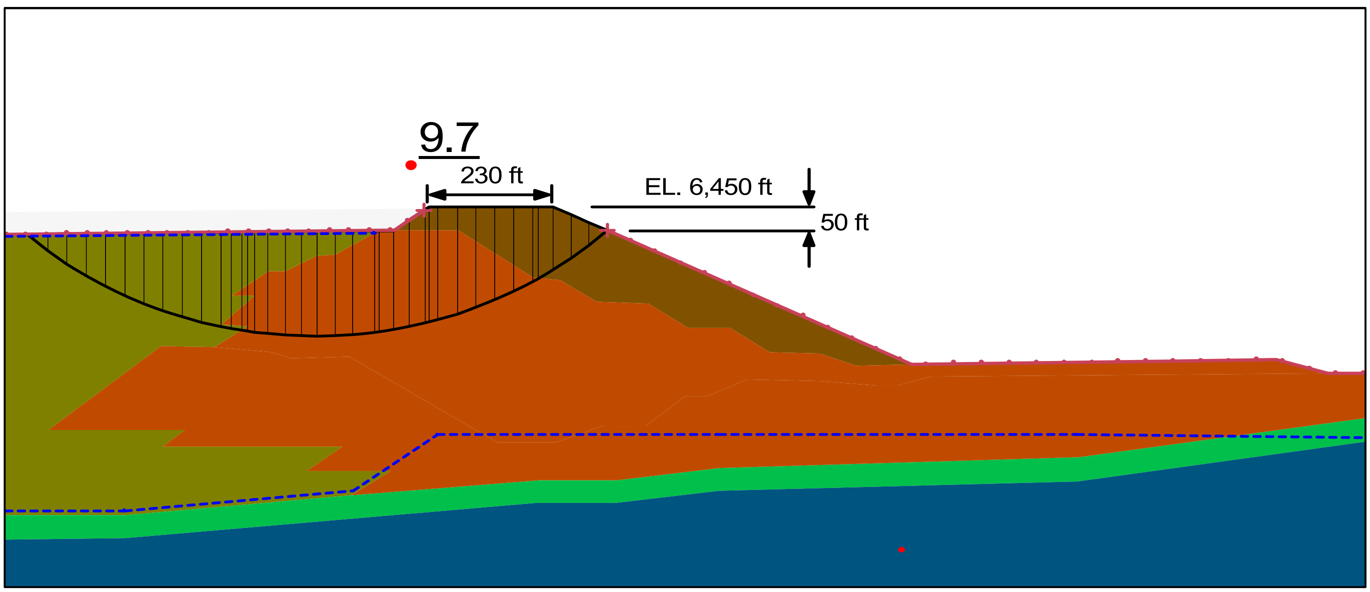
FIGURE B1.2

REV 2

DOWNSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



UPSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

2	10AUG'17	ISSUED WITH REPORT	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	RVW'D

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

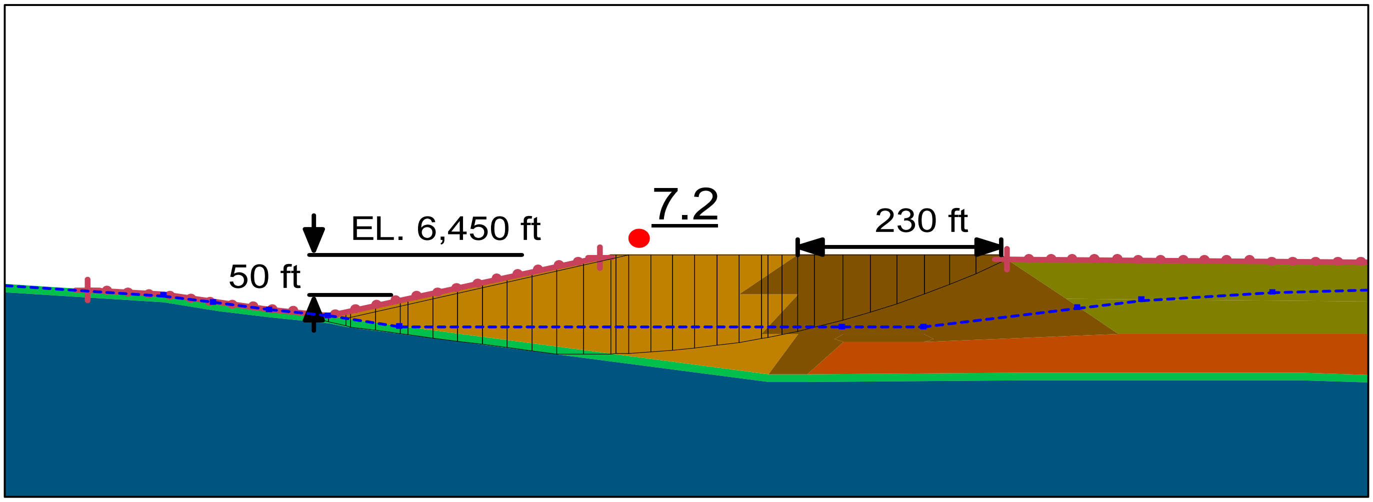
NORMAL OPERATING CONDITIONS
NORTH-SOUTH EMBANKMENT
SECTION 18+00N

Knight Piésold CONSULTING

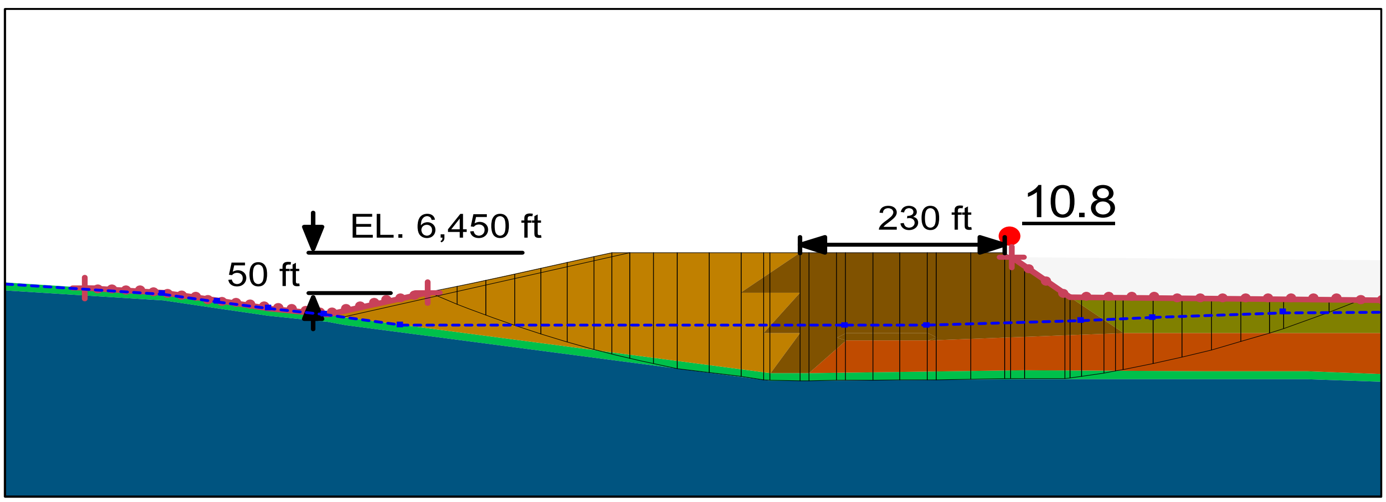
P/A NO. VA101-126/12 | REF. NO. 2

FIGURE B1.3 | REV 2

DOWNSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



UPSTREAM SLIP SURFACE, BASE CASE MATERIAL PROPERTIES



LEGEND:

- ZONE U ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ZONE D1 ROCKFILL - DRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

NORMAL OPERATING CONDITIONS
WEST EMBANKMENT
SECTION 108+40W

Knight Piésold CONSULTING

P/A NO. VA101-126/12

REF. NO. 2

FIGURE B1.4

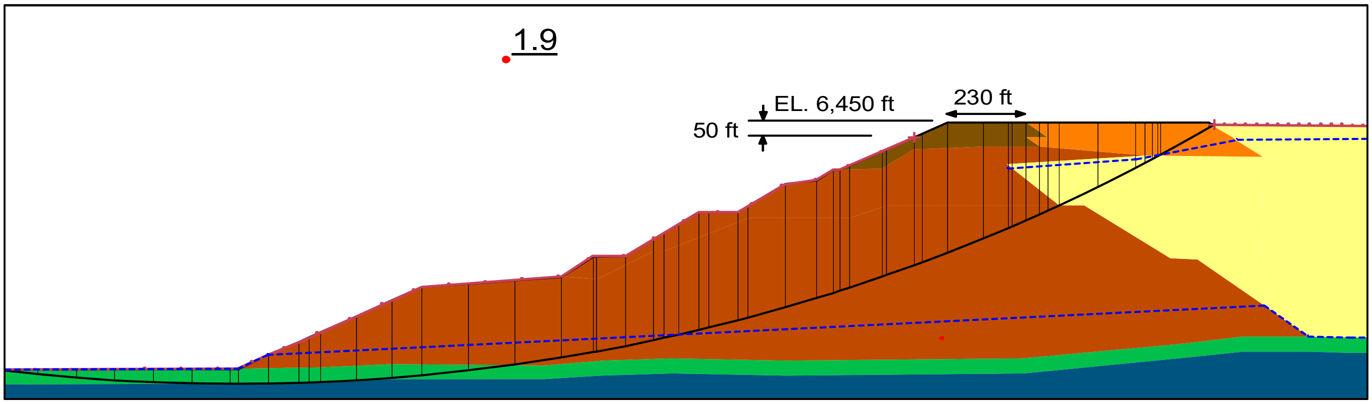
REV 2

APPENDIX B2

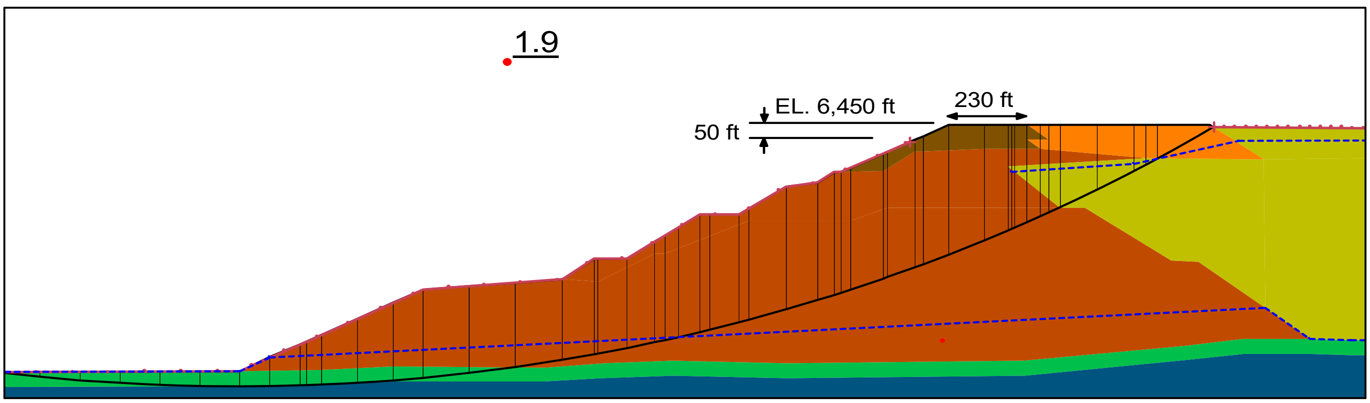
POST-EARTHQUAKE CONDITIONS

(Pages B2-1 to B2-8)

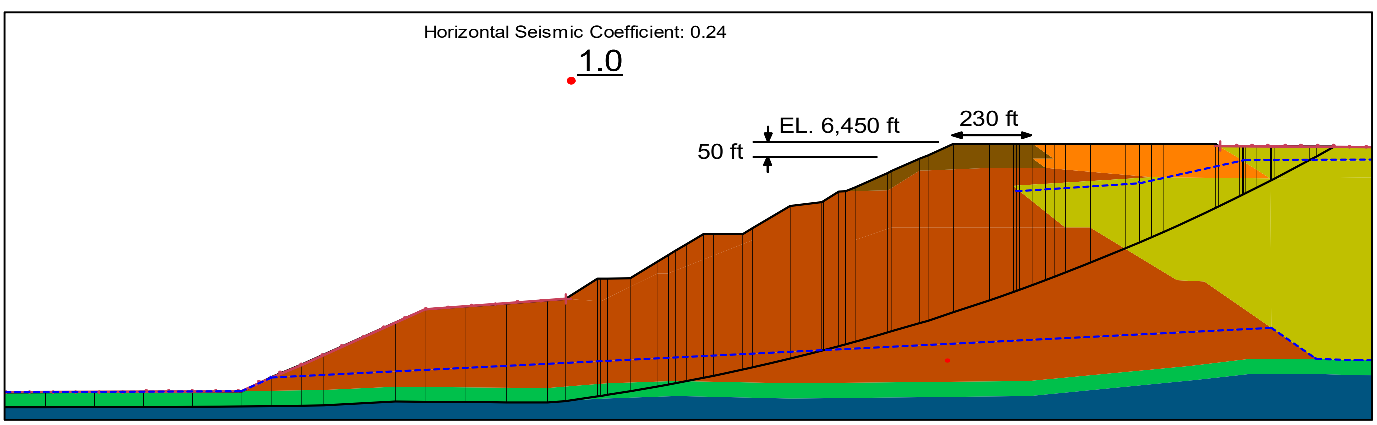
DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



DOWNSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

POST-EARTHQUAKE CONDITIONS
EAST-WEST EMBANKMENT
SECTION 08+00W

Knight Piésold CONSULTING

P/A NO. VA101-126/12

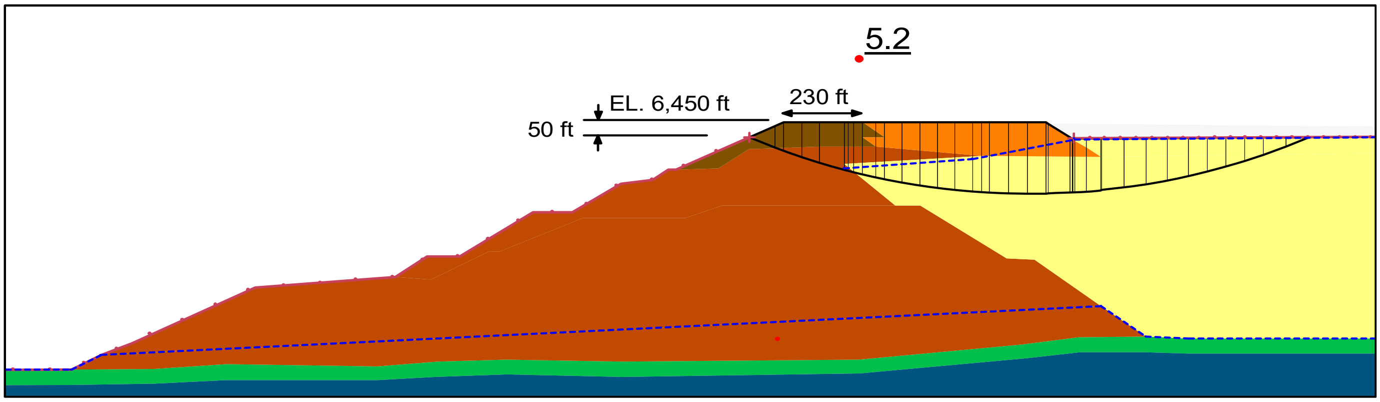
REF. NO. 2

FIGURE B2.1

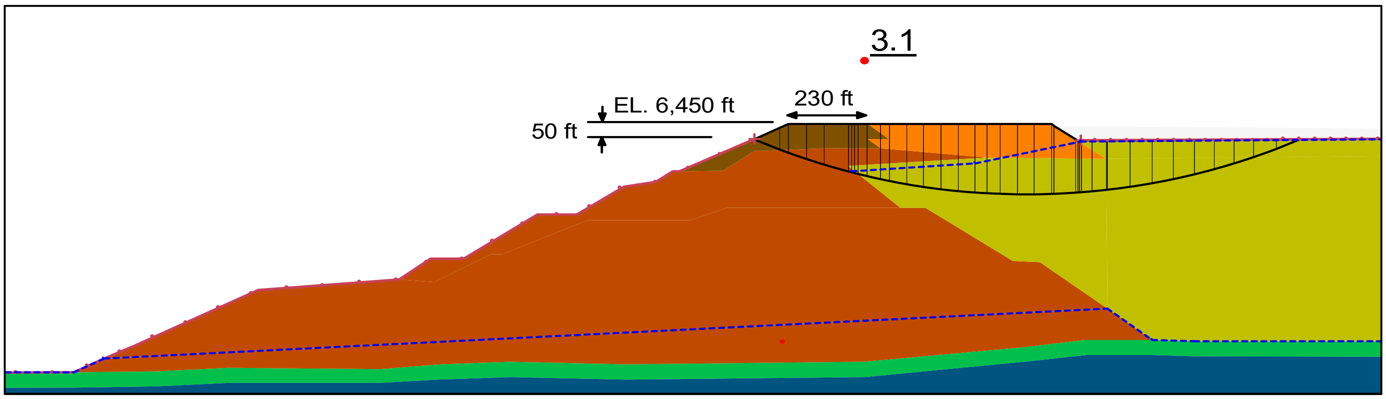
REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

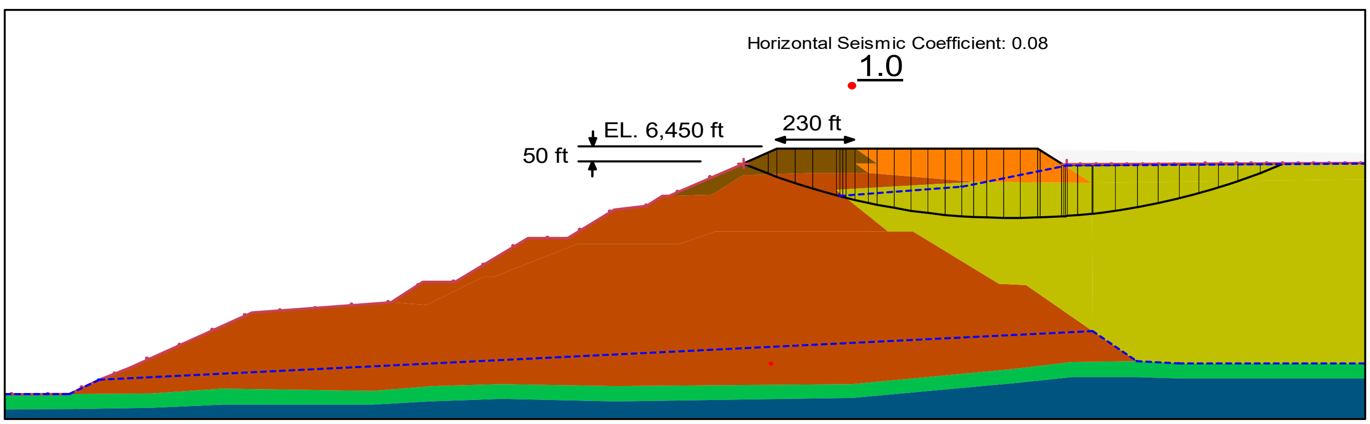
UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



UPSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

POST-EARTHQUAKE CONDITIONS
EAST-WEST EMBANKMENT
SECTION 08+00W

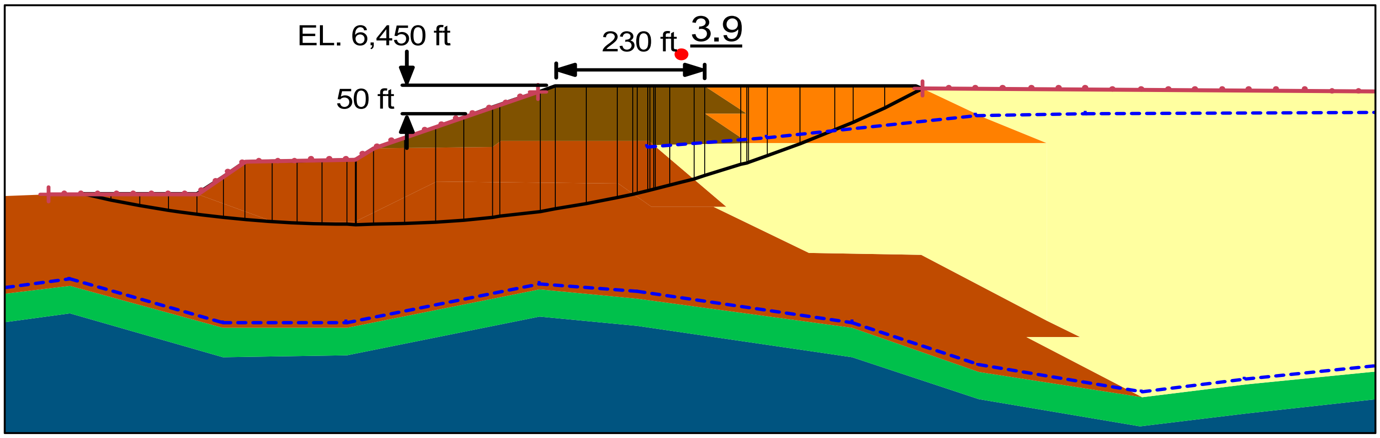
Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/12	2

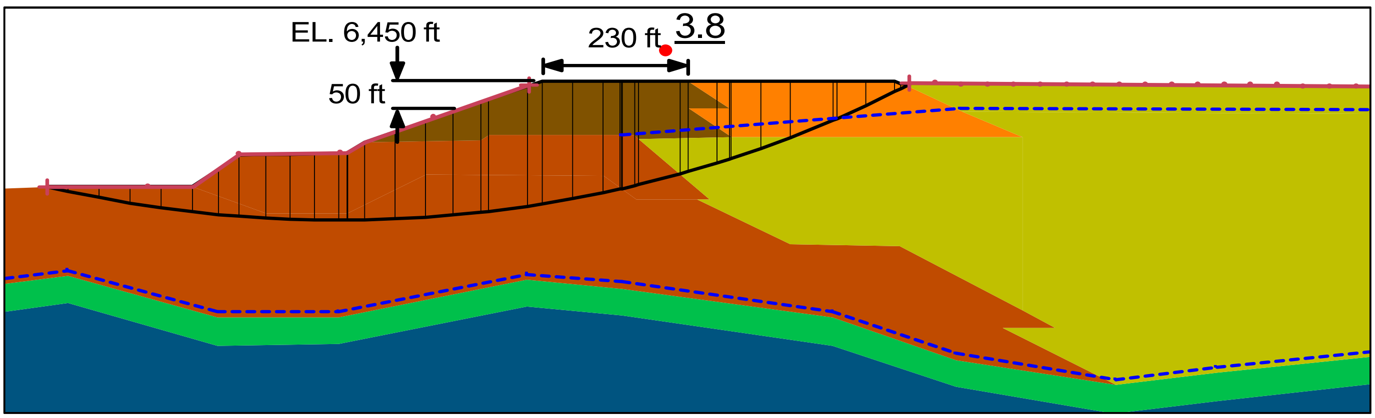
FIGURE B2.2 REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

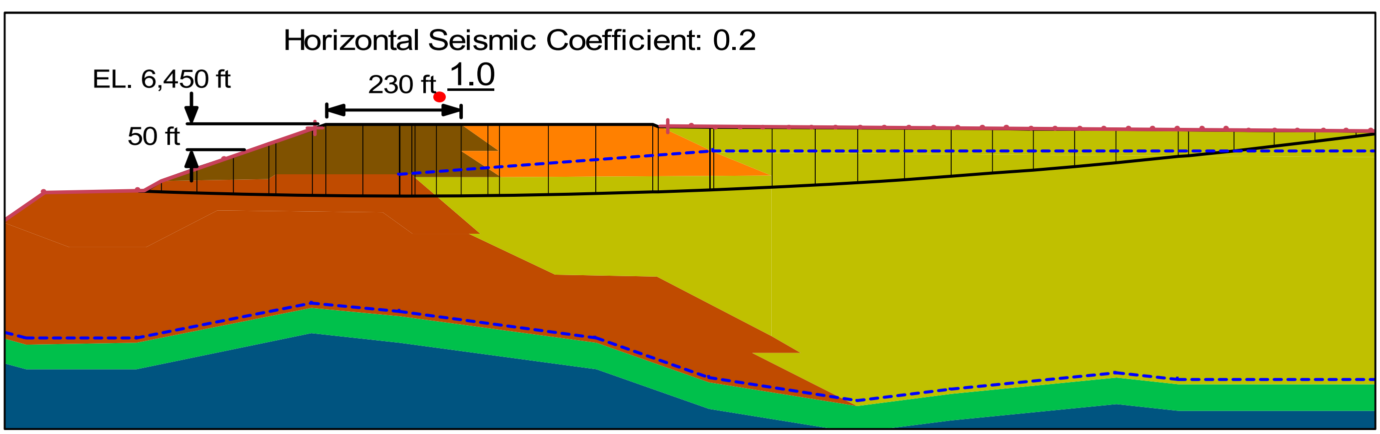
DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



DOWNSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

POST-EARTHQUAKE CONDITIONS
EAST-WEST EMBANKMENT
SECTION 38+00NW

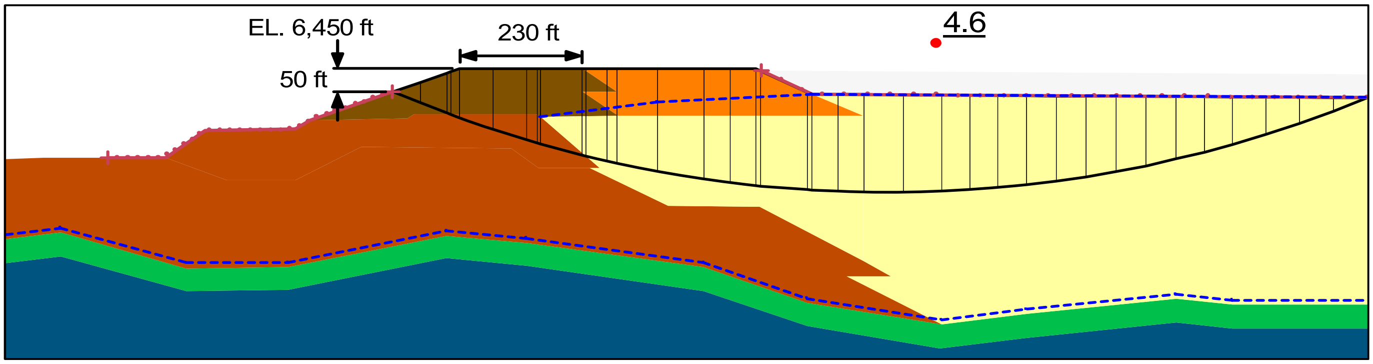
Knight Piésold CONSULTING

P/A NO. VA101-126/12 | REF. NO. 2

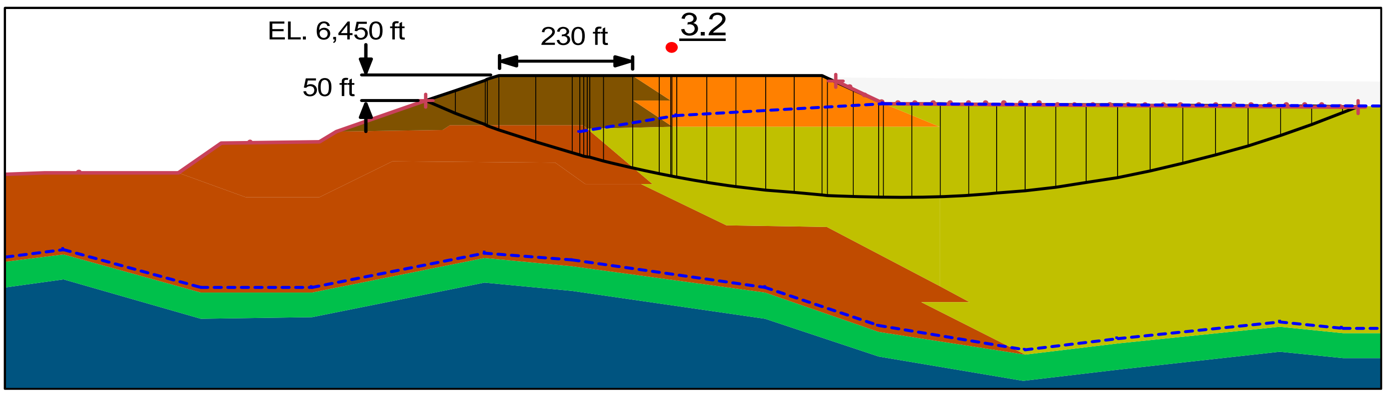
FIGURE B2.3 REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

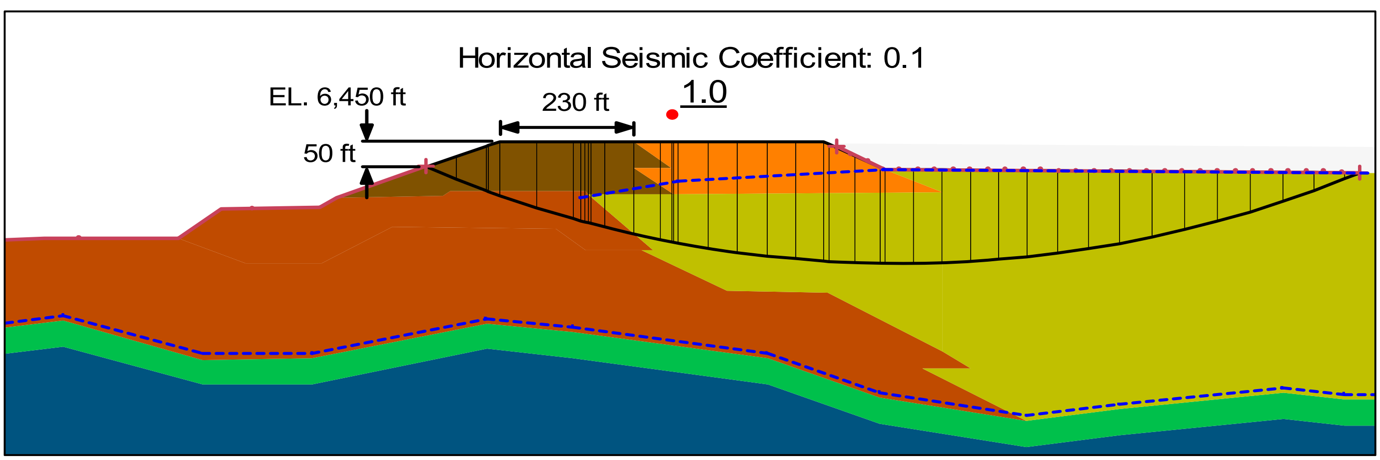
UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



UPSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

POST-EARTHQUAKE CONDITIONS
EAST-WEST EMBANKMENT
SECTION 38+00NW

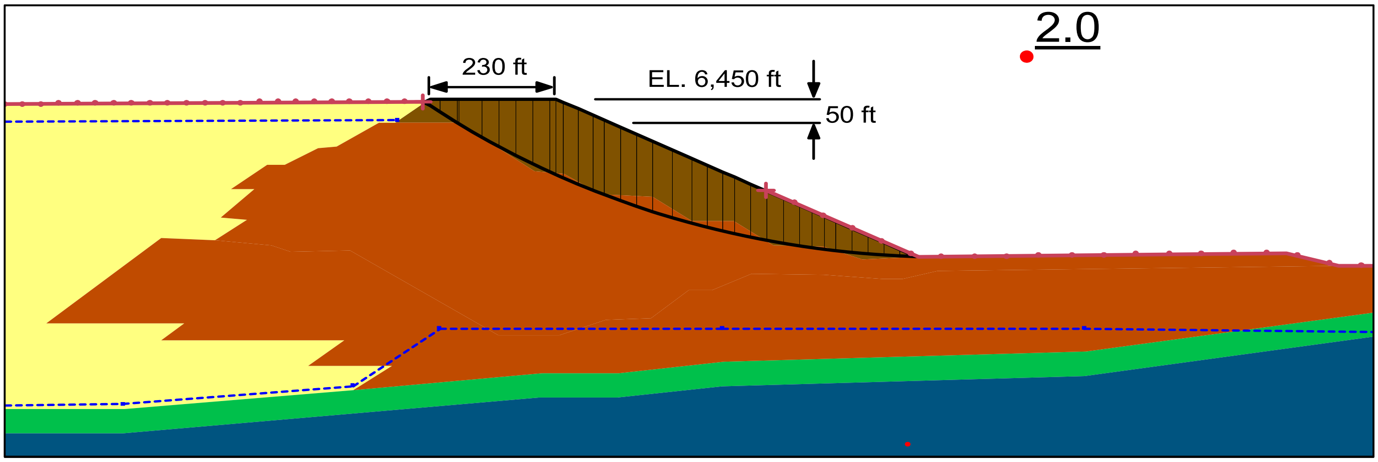
Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/12	2

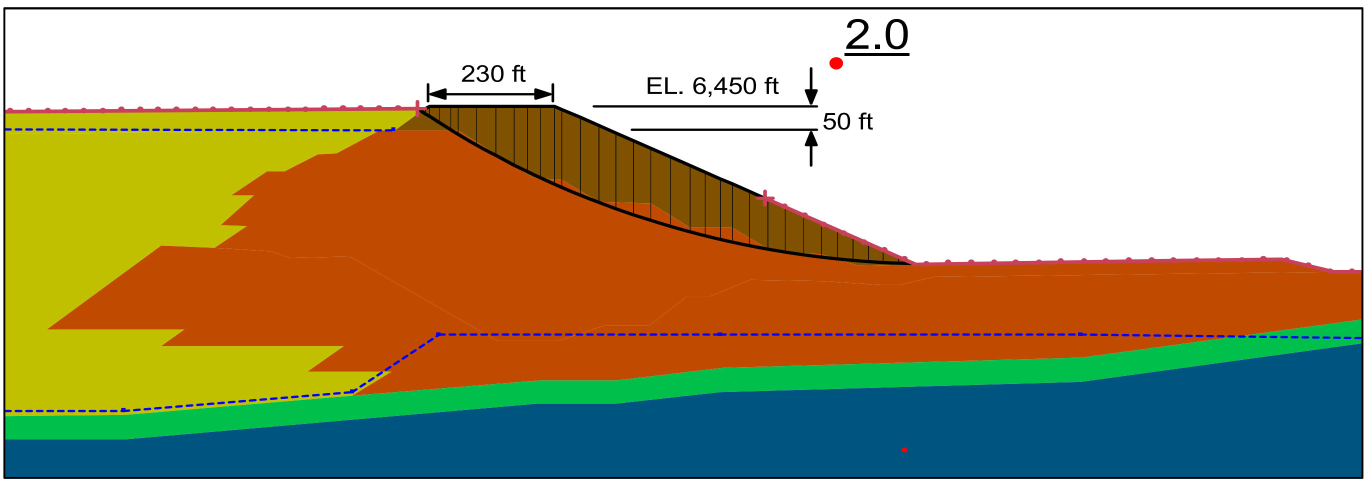
FIGURE B2.4 REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

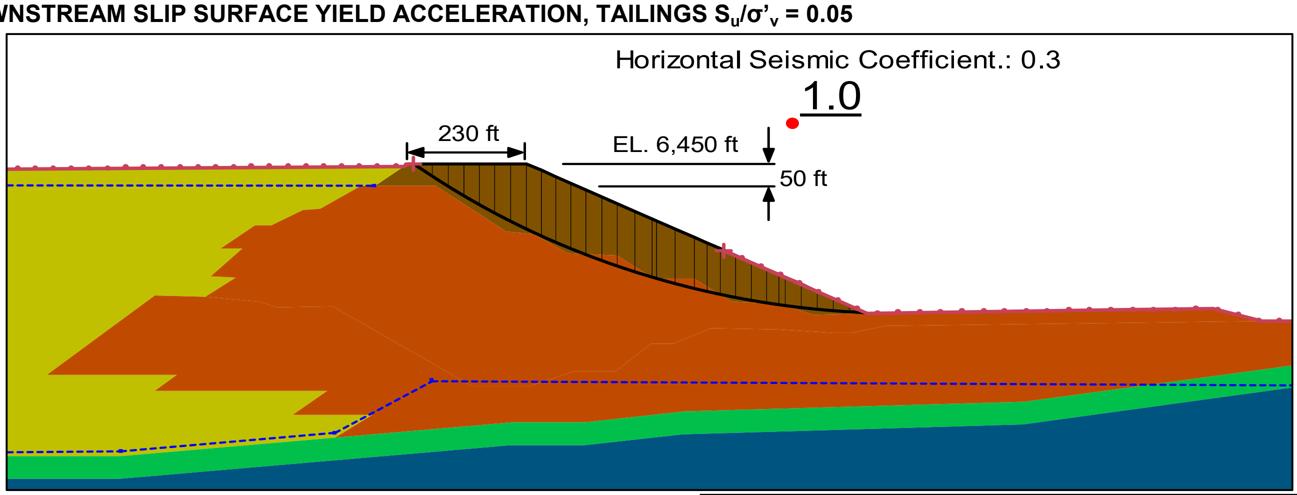
DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



DOWNSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

POST-EARTHQUAKE CONDITIONS
NORTH-SOUTH EMBANKMENT
SECTION 18+00N

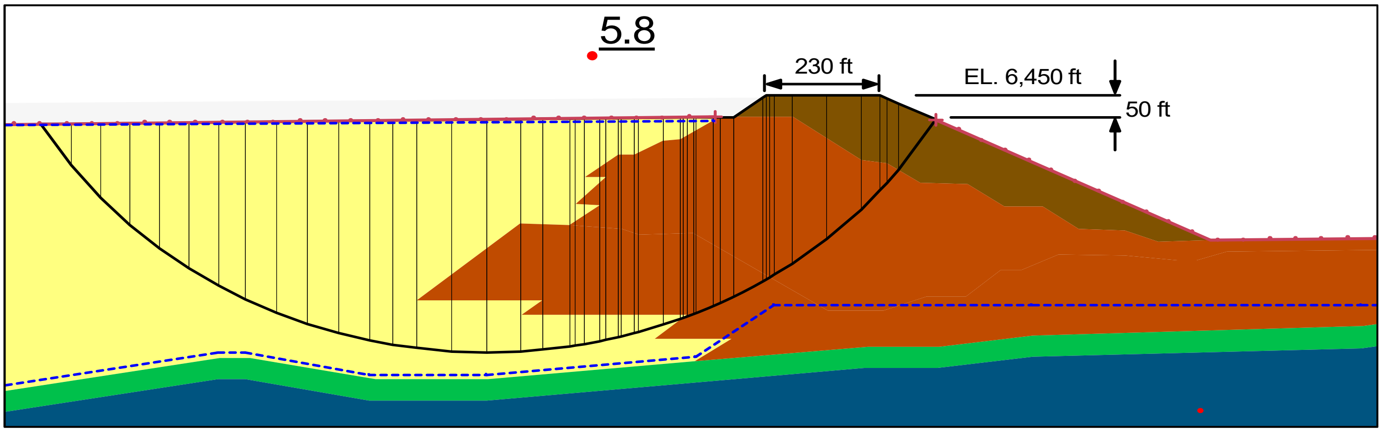
Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/12	2

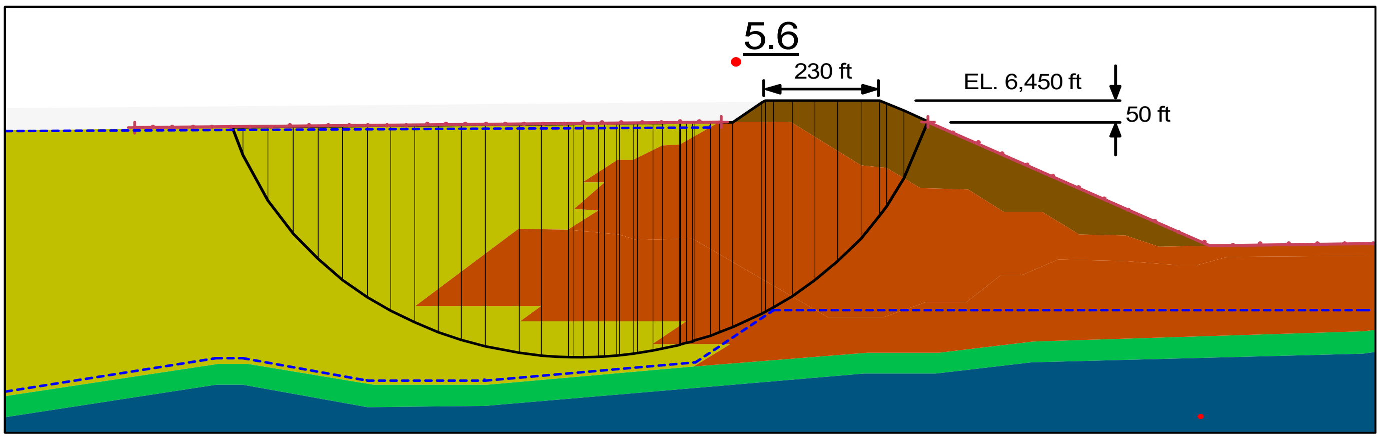
FIGURE B2.5 REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

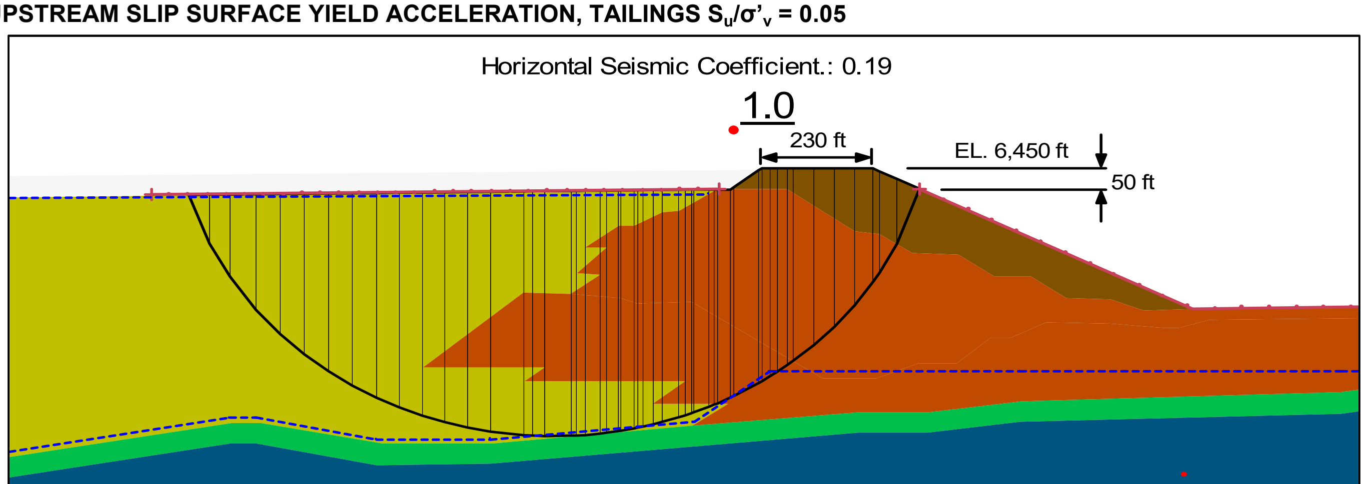
UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



UPSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

POST-EARTHQUAKE CONDITIONS
NORTH-SOUTH EMBANKMENT
SECTION 18+00N

Knight Piésold CONSULTING

P/A NO. VA101-126/12

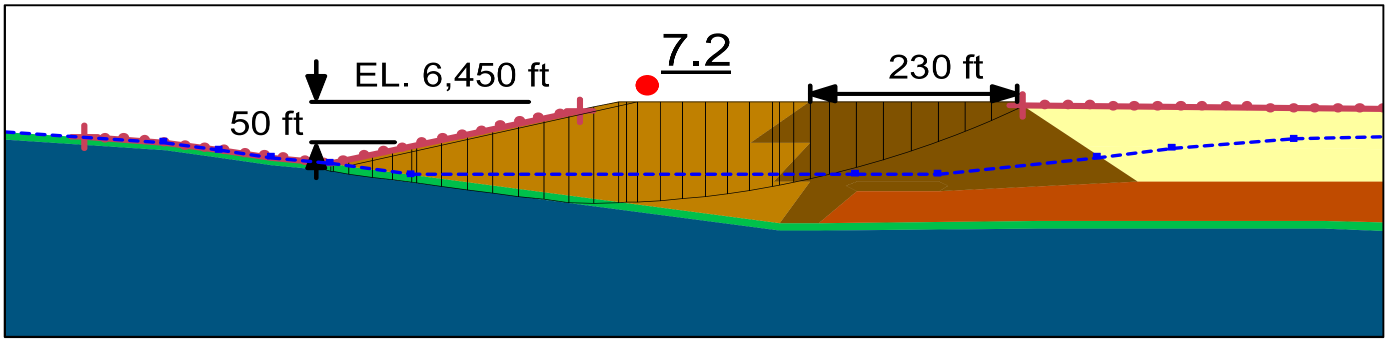
REF. NO. 2

FIGURE B2.6

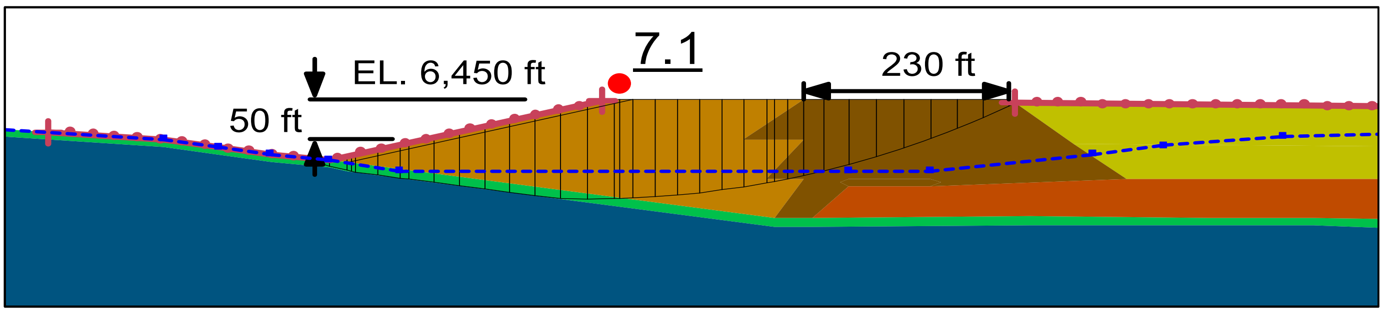
REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

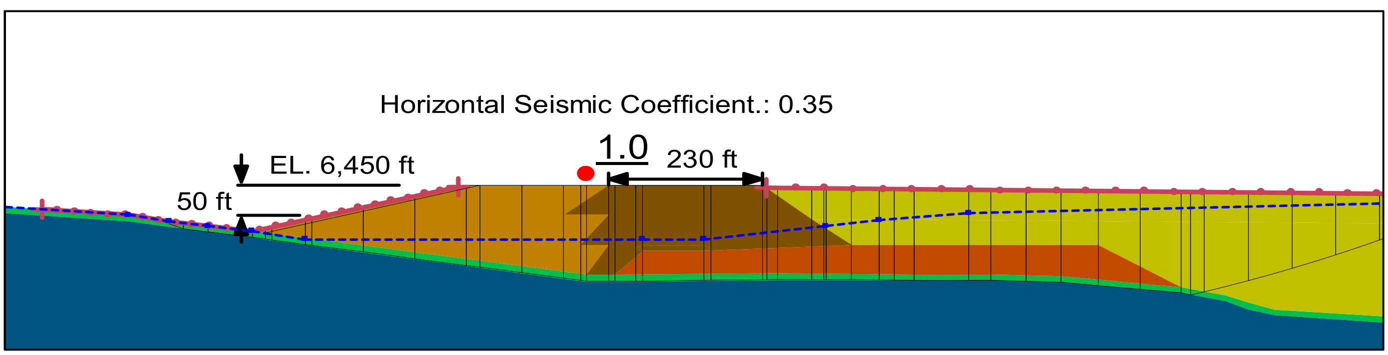
DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



DOWNSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



DOWNSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- ZONE U ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ZONE D1 ROCKFILL - DRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

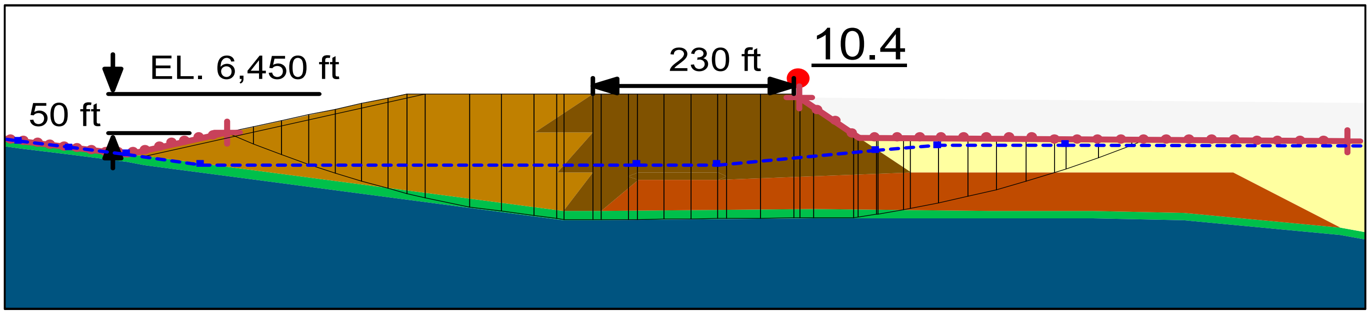
POST-EARTHQUAKE CONDITIONS
WEST EMBANKMENT
SECTION 108+40W

Knight Piésold CONSULTING

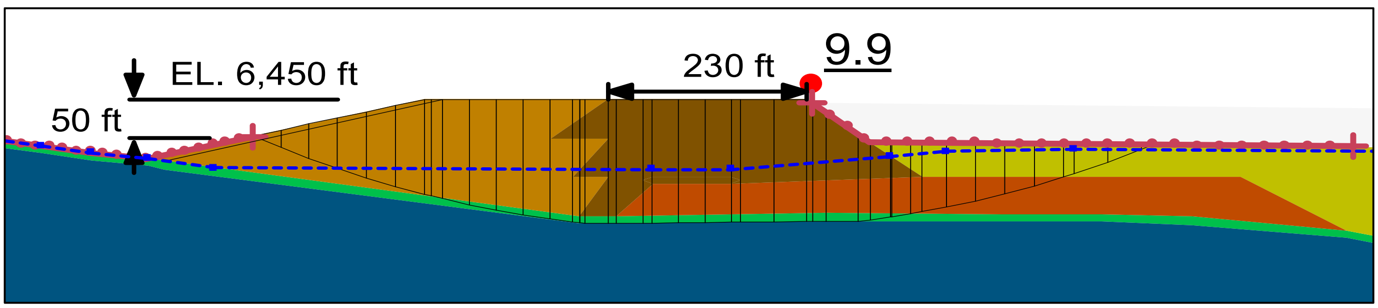
REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

P/A NO.	REF. NO.	FIGURE	REV
VA101-126/12	2	FIGURE B2.7	2

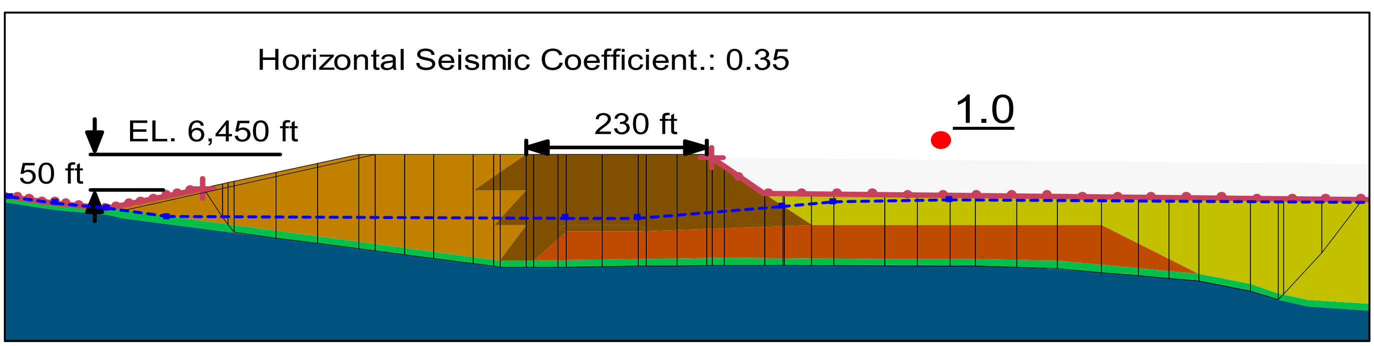
UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.20



UPSTREAM SLIP SURFACE, TAILINGS S_u/σ'_v = 0.05



UPSTREAM SLIP SURFACE YIELD ACCELERATION, TAILINGS S_u/σ'_v = 0.05



LEGEND:

- ZONE U ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ZONE D1 ROCKFILL - DRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

POST-EARTHQUAKE CONDITIONS
WEST EMBANKMENT
SECTION 108+40W

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/12	2

FIGURE B2.8 REV 2

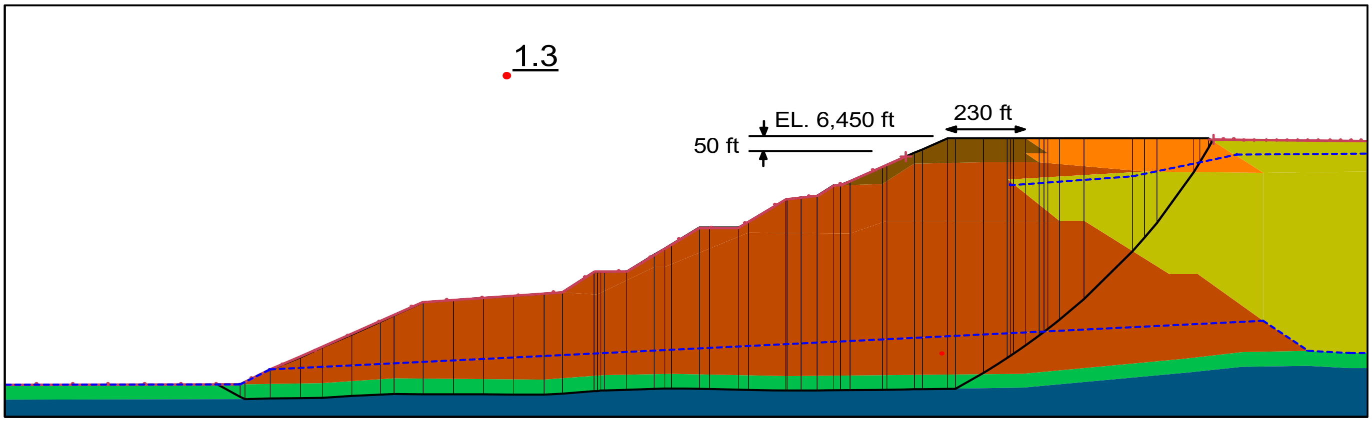
REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

APPENDIX B3

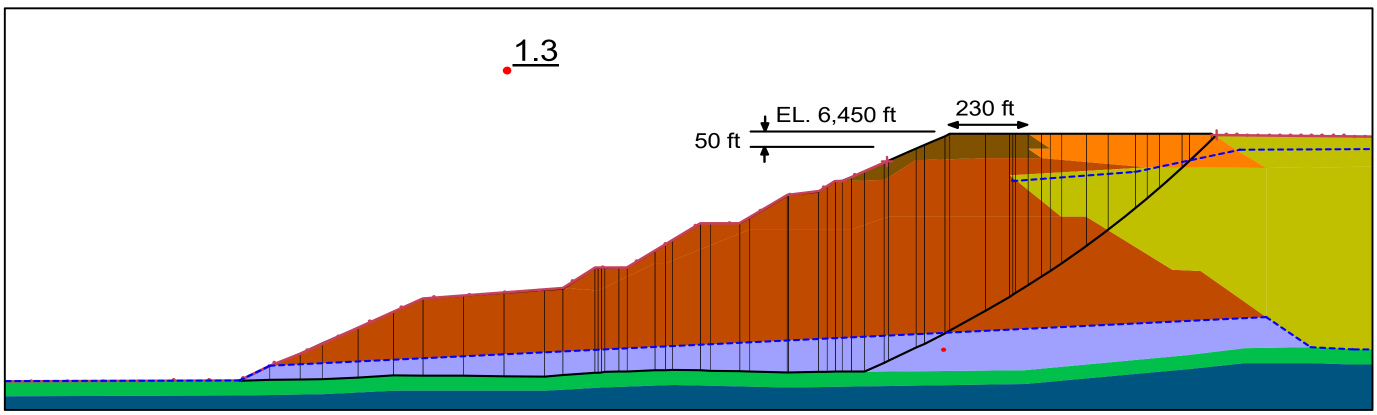
UNDRAINED RESPONSE SENSITIVITY ANALYSIS

(Pages B3-1 to B3-3)

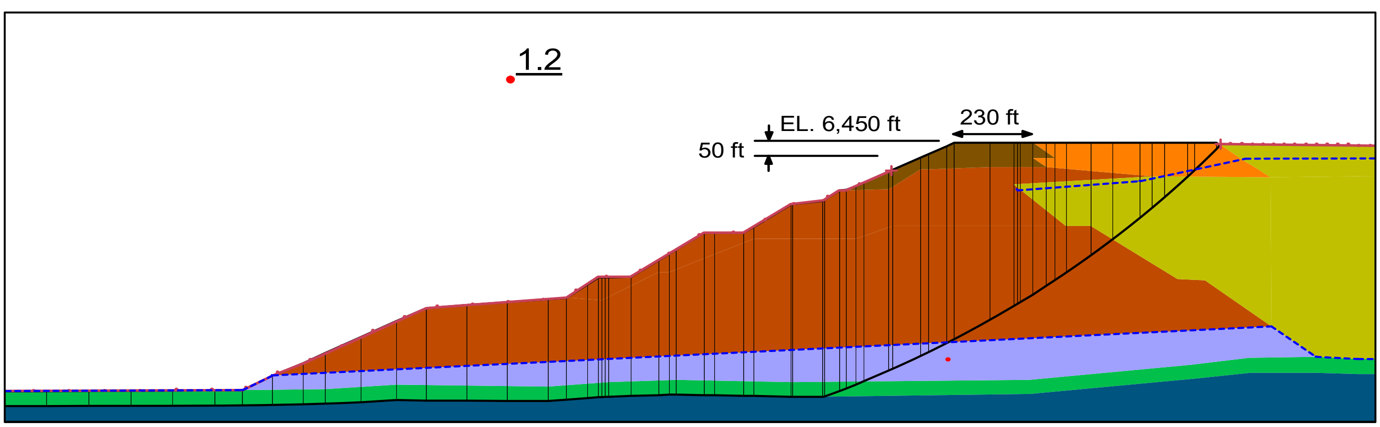
OVERBURDEN S_u/σ'_v = 0.23 (DSS)



SATURATED ROCKFILL S_u/σ'_v = 0.27 (DSS)



OVERBURDEN S_u/σ'_v = 0.23 (DSS), SATURATED ROCKFILL S_u/σ'_v = 0.37 (TX)



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

UNDRAINED RESPONSE SENSITIVITY ANALYSES
EAST-WEST EMBANKMENT
SECTION 08+00W

Knight Piésold CONSULTING

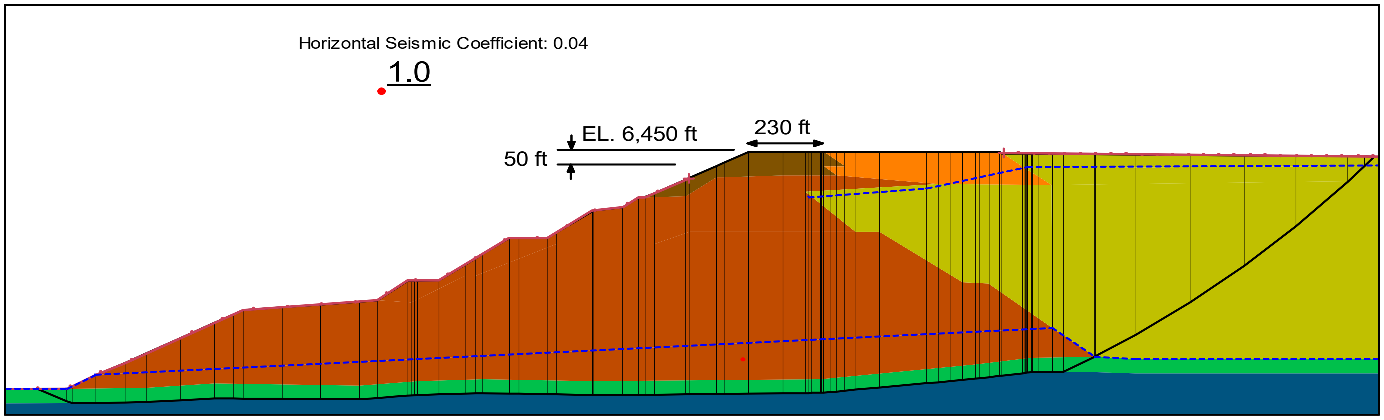
P/A NO. VA101-126/12

REF. NO. 2

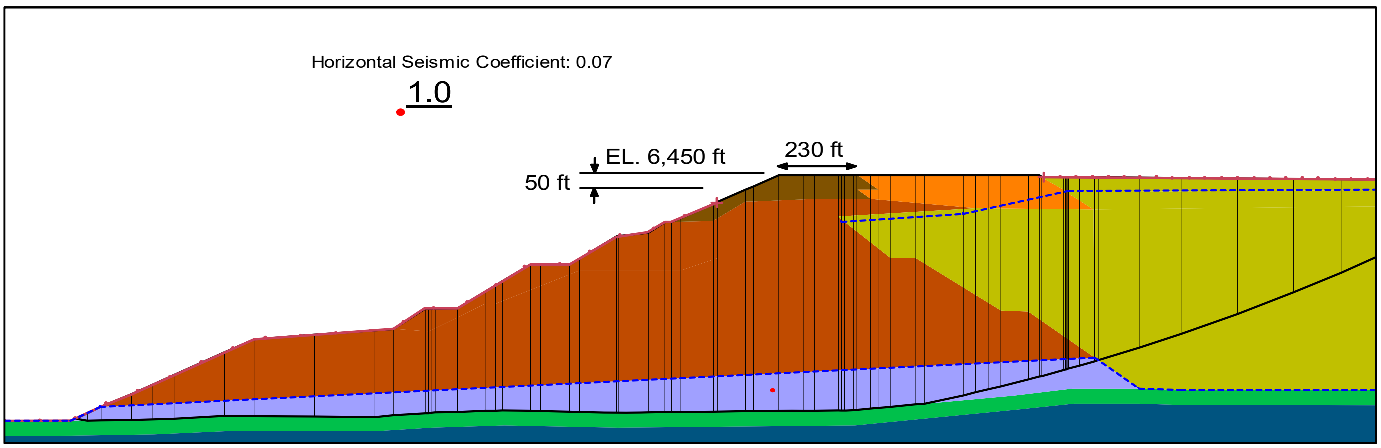
FIGURE B3.1

REV 2

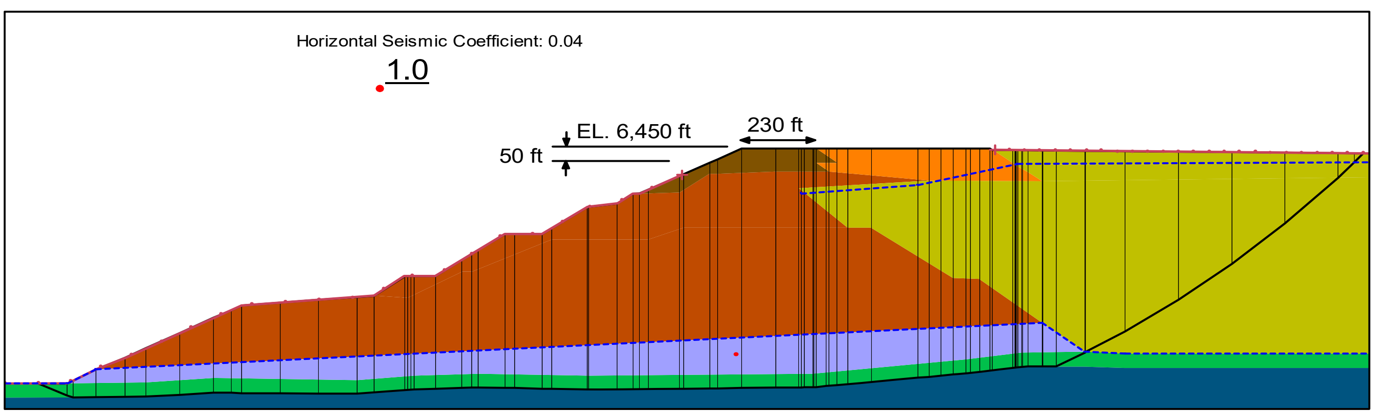
OVERBURDEN S_u/σ'_v = 0.23 (DSS)



SATURATED ROCKFILL S_u/σ'_v = 0.27 (DSS)



OVERBURDEN S_u/σ'_v = 0.23 (DSS), SATURATED ROCKFILL S_u/σ'_v = 0.37 (TX)



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

UNDRAINED RESPONSE SENSITIVITY ANALYSES
EAST-WEST EMBANKMENT
SECTION 08+00W

Knight Piésold CONSULTING

P/A NO. VA101-126/12

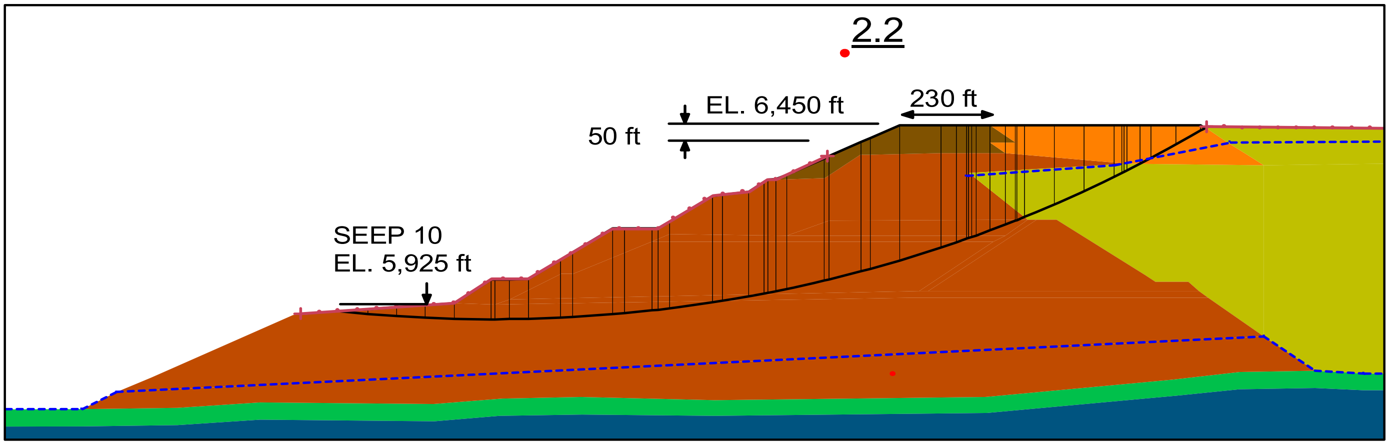
REF. NO. 2

FIGURE B3.2

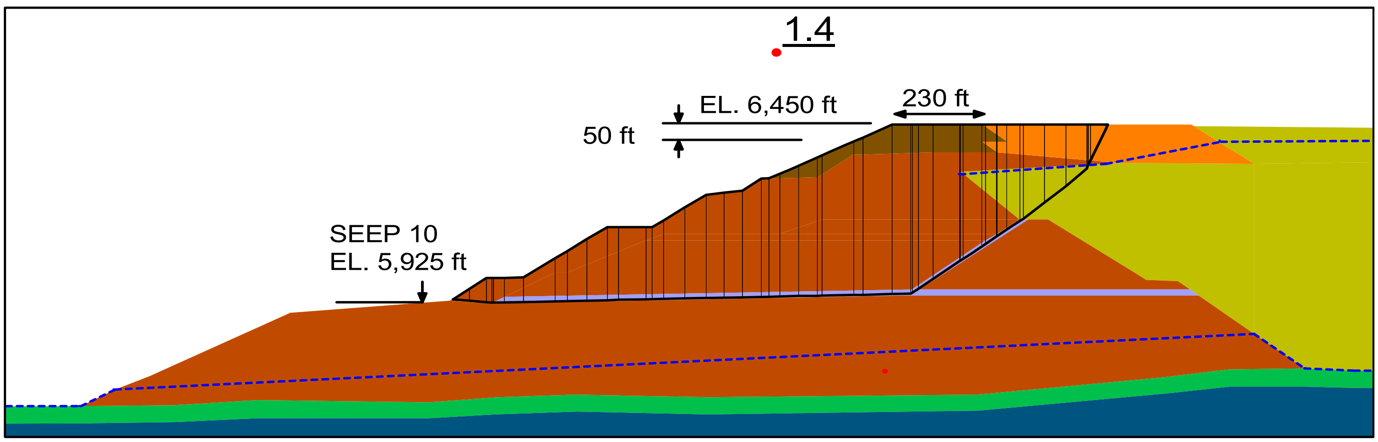
REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

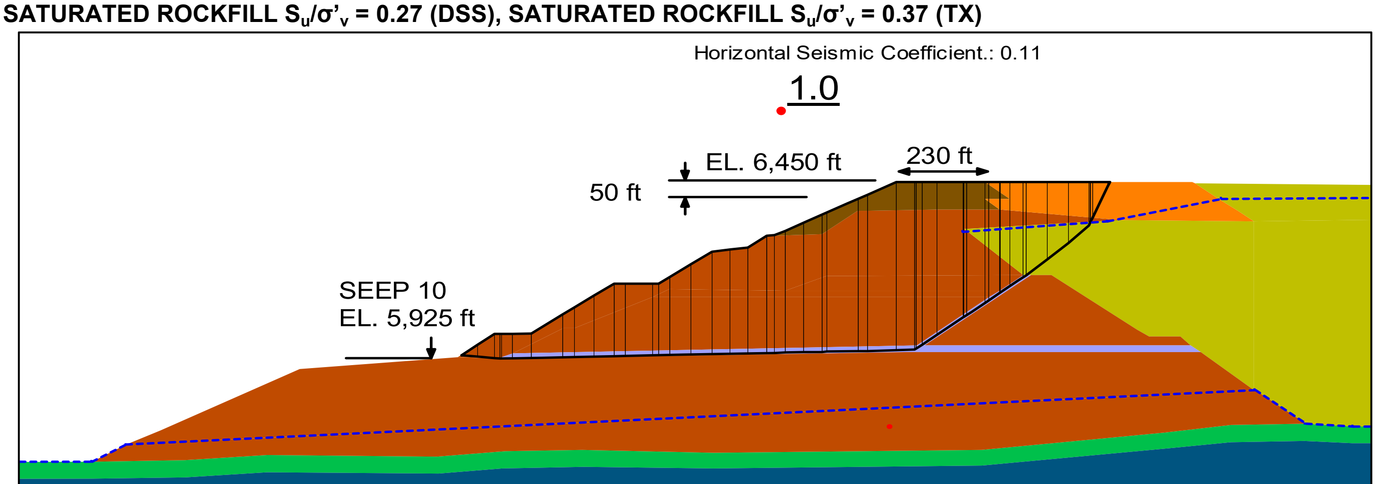
STATIC, POST-EARTHQUAKE CONDITIONS



SATURATED ROCKFILL S_u/σ'_v = 0.27 (DSS), SATURATED ROCKFILL S_u/σ'_v = 0.37 (TX)



SATURATED ROCKFILL S_u/σ'_v = 0.27 (DSS), SATURATED ROCKFILL S_u/σ'_v = 0.37 (TX)



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- DEGRADED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

UNDRAINED RESPONSE SENSITIVITY ANALYSES
EAST-WEST EMBANKMENT
SECTION 08+00W AT EL. 5,925 FT

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-126/12	2

FIGURE B3.3 REV 2

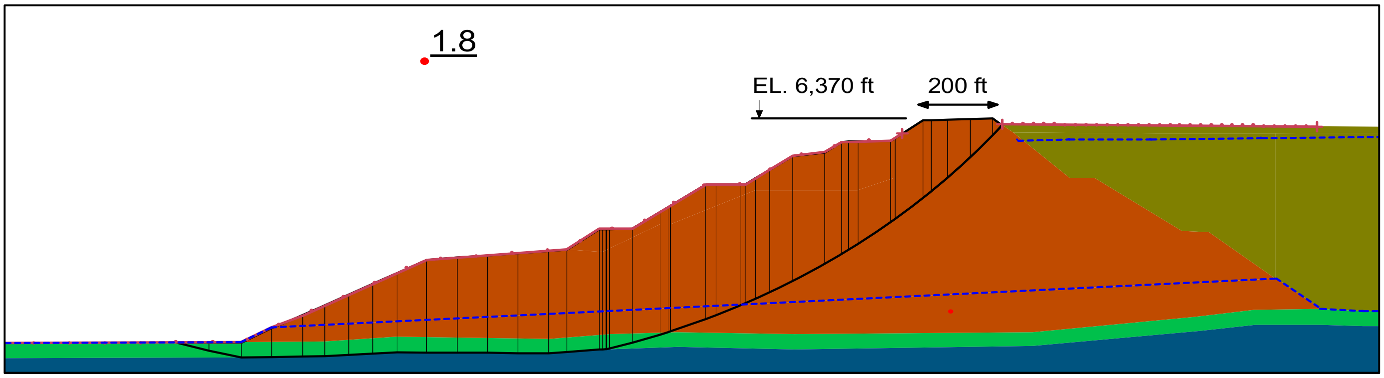
REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

APPENDIX B4

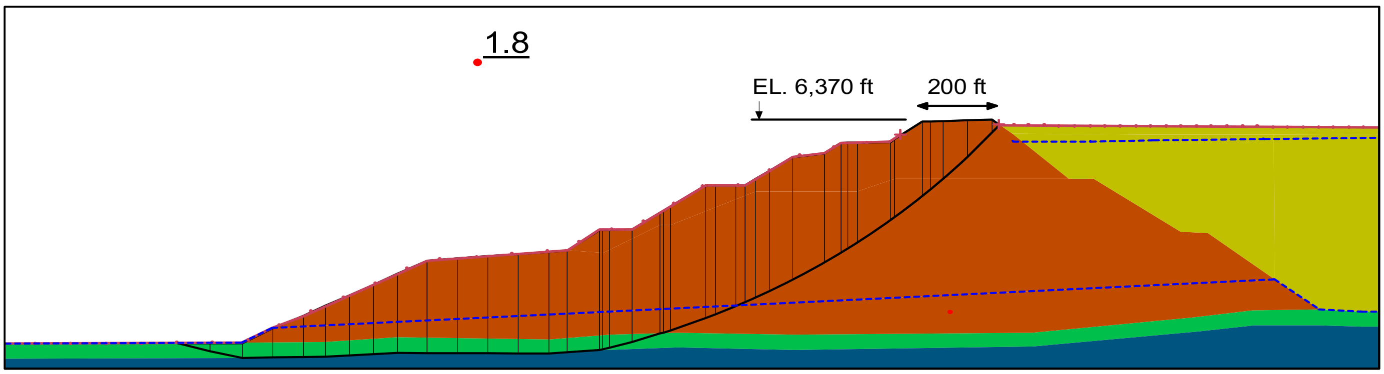
STAGED SENSITIVITY ANALYSIS

(Pages B4-1 to B4-3)

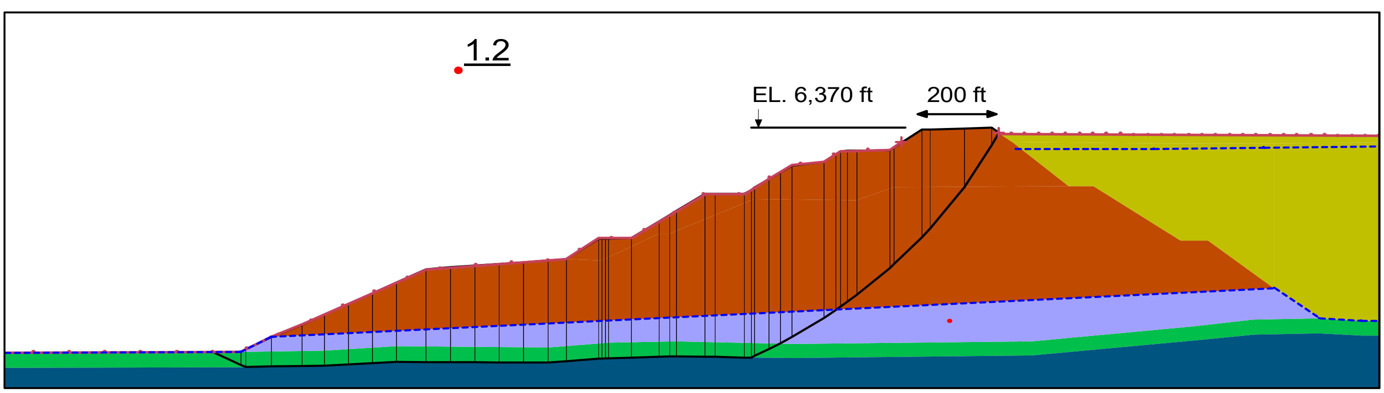
STATIC, NORMAL OPERATING CONDITIONS



STATIC, POST-EARTHQUAKE CONDITIONS



EXTREME SENSITIVITY ANALYSIS USING UNDRAINED RESPONSE
OVERBURDEN S_u/σ'_v = 0.23 (DSS), SATURATED ROCKFILL S_u/σ'_v = 0.37 (TX)



LEGEND:
- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- SATURATED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

EAST-WEST EMBANKMENT (2014)
SECTION 08+00W

Knight Piésold CONSULTING

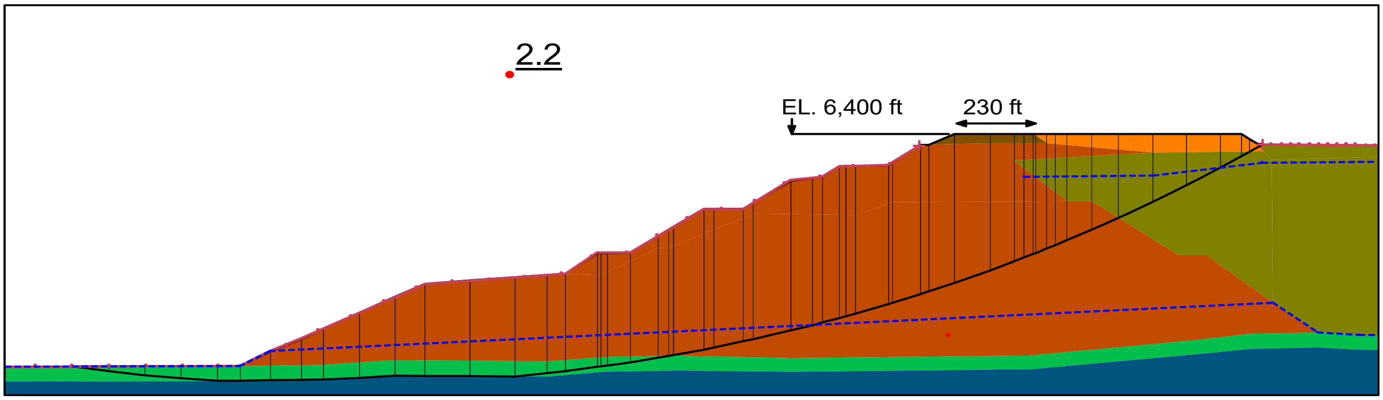
P/A NO. VA101-126/12

REF. NO. 2

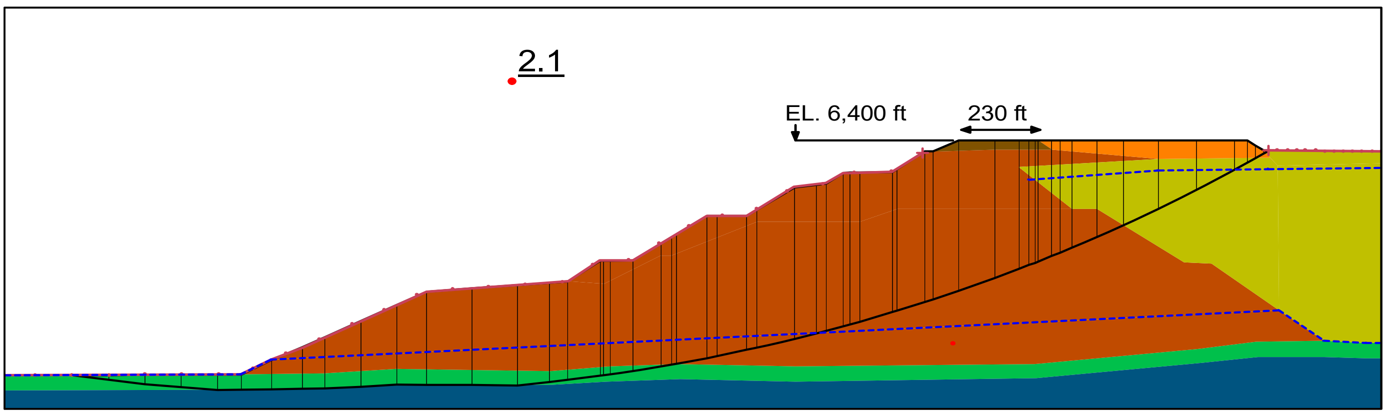
FIGURE B4.1

REV 2

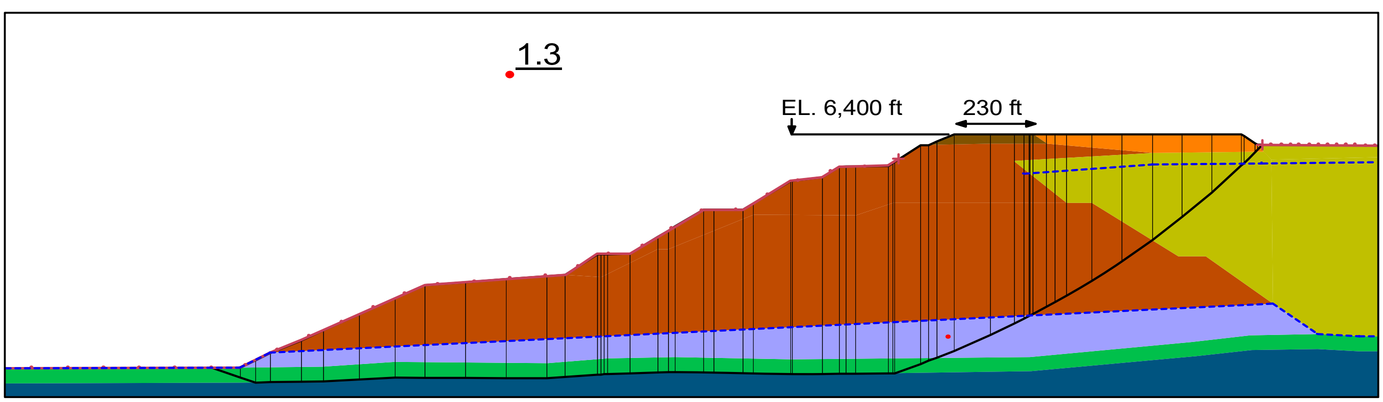
STATIC, NORMAL OPERATING CONDITIONS



STATIC, POST-EARTHQUAKE CONDITIONS



EXTREME SENSITIVITY ANALYSIS USING UNDRAINED RESPONSE
OVERBURDEN S_u/σ'_v = 0.23 (DSS), SATURATED ROCKFILL S_u/σ'_v = 0.37 (TX)



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- SATURATED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

EAST-WEST EMBANKMENT (2017)
SECTION 08+00W

Knight Piésold CONSULTING

P/A NO. VA101-126/12

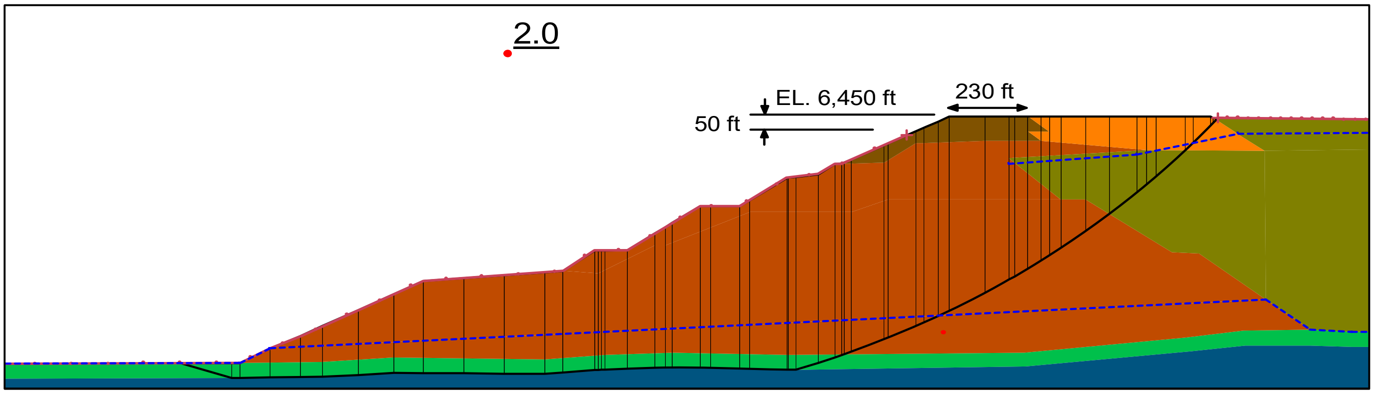
REF. NO. 2

FIGURE B4.2

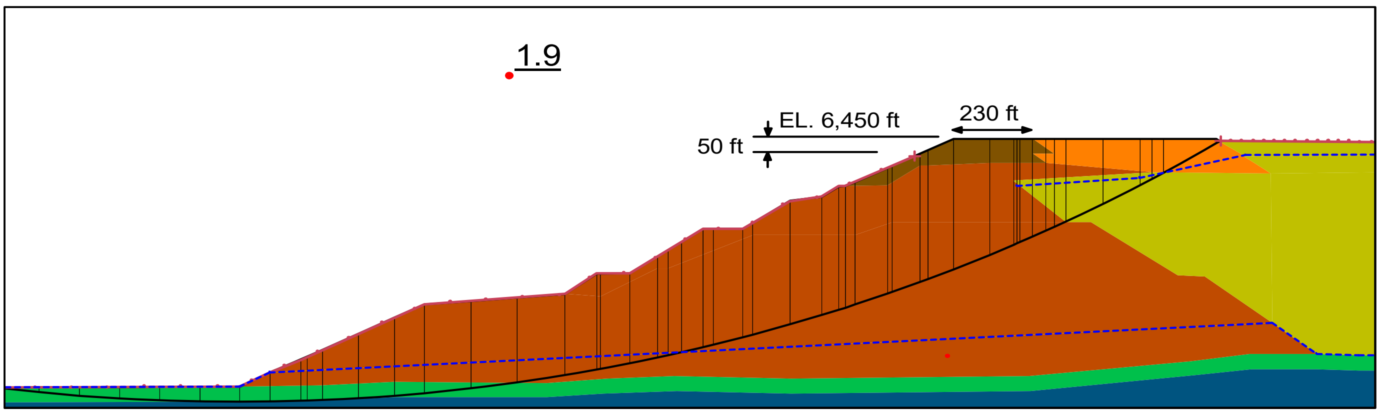
REV 2

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

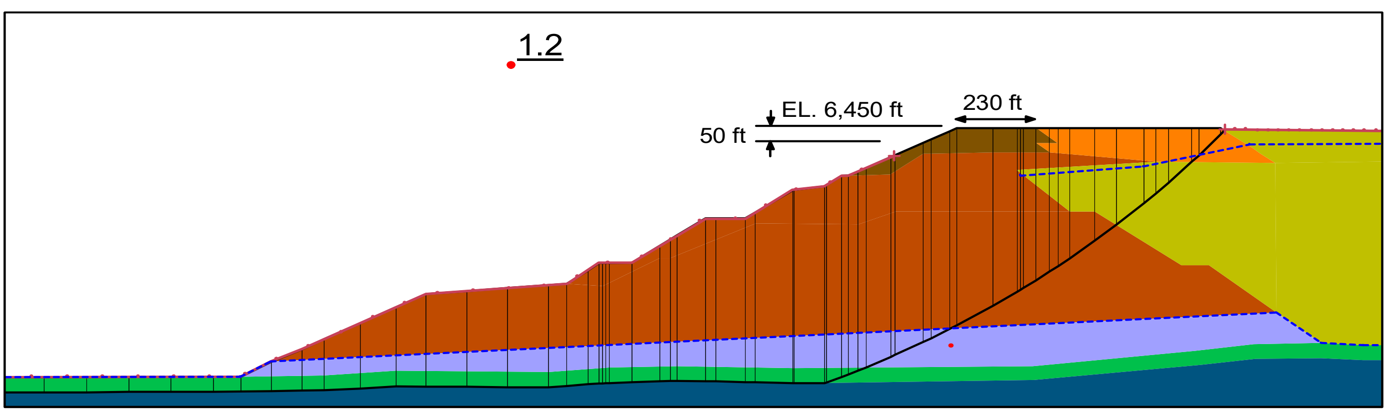
STATIC, NORMAL OPERATING CONDITIONS



STATIC, POST-EARTHQUAKE CONDITIONS



EXTREME SENSITIVITY ANALYSIS USING UNDRAINED RESPONSE
OVERBURDEN S_u/σ'_v = 0.23 (DSS), SATURATED ROCKFILL S_u/σ'_v = 0.37 (TX)



LEGEND:

- STRUCTURAL ROCKFILL - DRAINED
- HISTORIC ROCKFILL - DRAINED
- ROCKFILL SURCHARGE - DRAINED
- SATURATED ROCKFILL - UNDRAINED
- BEDROCK
- TAILINGS - DRAINED
- TAILINGS - UNDRAINED
- TAILINGS - LIQUEFIED
- OVERBURDEN

REV	DATE	DESCRIPTION	PREP'D	RVW'D
2	10AUG'17	ISSUED WITH REPORT	DDF	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

EAST-WEST EMBANKMENT (2031)
SECTION 08+00W

Knight Piésold CONSULTING

P/A NO. VA101-126/12

REF. NO. 2

FIGURE B4.3

REV 2