

MONTANA RESOURCES, LLP YANKEE DOODLE TAILINGS IMPOUNDMENT



DAM BREACH RISK ASSESSMENT

PREPARED FOR:

Montana Resources, LLP
600 Shields Avenue
Butte, Montana
USA, 59701

PREPARED BY:

Knight Piésold Ltd.
Suite 1400 – 750 West Pender Street
Vancouver, BC V6C 2T8 Canada
p. +1.604.685.0543 • f. +1.604.685.0147

VA101-126/12-3
Rev 3
March 13, 2018

Knight Piésold
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www.knightpiesold.com

**MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT**

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VA101-126/12-3**

Rev	Description	Date
0	Issued in Final	January 26, 2017
1	Issued with Marked Revisions	February 27, 2017
2	Issued with Marked Revisions for 6450 Crest	August 23, 2017
3	Issued with Revised Appendix A	March 12, 2018

Knight Piésold Ltd.

Suite 1400

750 West Pender Street

Vancouver, British Columbia Canada V6C 2T8

Telephone: (604) 685-0543

Facsimile: (604) 685-0147

www.knightpiesold.com

Knight Piésold
CONSULTING

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. This report summarizes the risk assessment of the YDTI at its proposed design crest elevation. Specifically, this assessment presents an examination of foundation and embankment instability, overtopping, and internal erosion and piping. The assessment considers loading during maximum normal operating conditions, loading from seismic events, flood events, and malfunctions of the reclaim water and tailings distribution systems.

▲ R2

The likelihood of embankment failure and uncontrolled loss of tailings due to foundation and slope instability under static conditions is very low. Overtopping of the embankment is only a credible failure mode for severe flood events and earthquake-induced deformation. The risk of flood-induced overtopping is very low, and is managed by maintaining the prescribed design freeboard through continued embankment construction up to the final design elevation. A closure spillway will prevent overtopping in the long-term after operations cease. The risk of earthquake-induced deformation leading to overtopping is very low. The seismic loading analysis considered both the operating conditions and long-term conditions following closure. The robustness of the free draining embankment, design freeboard, and extensive drained tailings beaches are sufficient to manage this risk. The pond will reduce in size following closure because pond evaporation exceeds precipitation at the site. Pore pressures will reduce over time and the tailings surface will be covered further limiting the potential for overtopping following an earthquake.

Internal erosion and piping of the embankment under normal operating conditions is not a credible failure mode. The tailings beaches work in conjunction with the free draining embankments to limit pore pressures at the interface between the tailings and embankment materials, and eliminate any substantial phreatic surface from developing in the embankment. Piping cannot develop without a continuous source of water eroding material along a seepage flow path. The risk of internal erosion and piping will increase if the supernatant pond or tailings stream is allowed to approach one of the embankments due to improper beach development or natural flooding.

The potential for internal erosion and piping initiated by natural flooding carries the greatest uncertainty for the YDTI. The flood events considered in this risk assessment are rare and therefore the likelihood of the flooded condition actually developing is very low. However, an analysis of internal erosion and piping potential under flooded conditions is difficult due to the variability in embankment fill consistency combined with the uncertainty of timing for development of such a condition. This uncertainty highlights the importance of water management and tailings beach development to manage risk. The alluvium facing, wide crest width, and the well graded particle size distribution of most of the embankment fill would provide some protection against internal erosion in many areas of the embankment. However, the ponding of water adjacent to the embankment could provide a pathway to a very large source of water that has the potential to cause concentrated leakage through gap-graded rockfill zones or coarse boulder layers within the embankment. These zones are known to exist and have been subjected to leakage caused by the tailings stream in the past.

The YDTI operates by keeping the pond separated from the embankments and by constructing free draining embankments. This concept, which has been successful over many decades to date, can be applied to the future development of the facility to mitigate the potential for internal erosion and piping under flooded conditions. Reducing the normal operating pond volume or improving the uniformity of tailings beach development will increase the storm storage that can be contained on the tailings beach without reaching the embankment. This will decrease the potential for internal erosion and piping under flooded conditions for the YDTI and will further enhance the safety of the facility under normal operating conditions.

There is an opportunity to utilize the observational method while considering the development of pore pressures within the tailings beach and embankment. This opportunity is particularly relevant now due to the change in approach of tailings beach development, and the importance of the tailings beaches to limit pore pressure development. Tailings beach development and long-term prediction of the normal operating pond volumes are estimated using models that are primarily based on past performance of the facility. There is some uncertainty in any model, and the predictions must be verified by observations. A plan for utilizing the observational method for managing residual risk is described. The plan includes foreseeable deviations from expected conditions, observational monitoring and analytical actions, and potential alternative solutions.

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 existing practice of placing higher strength rockfill materials in the higher sections of the embankment and weaker materials in non-structural areas should be continued. Monitoring of overall slope angles and pore pressures in the embankments will be required to demonstrate adequate performance of the facility.

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APPENDICES

Appendix A Tailings Flowability Assessment ▲ R3

ABBREVIATIONS

Montana Department of Environmental Quality	DEQ
Consolidated Undrained	CU
Direct Simple Shear	DSS
East-West	E-W
Elevation	EL
Emergency Action Plan	EAP
Factor of Safety	FS
Failure Modes Analysis	FMA
Horseshoe Bend	HsB
International Engineering Company Inc.	IECO
KirK Engineering and Natural Resources Inc.	KirK
Knight Piésold Ltd.	KP
Maximum Credible Earthquake	MCE
Million Gallons per Day	MGPD
Montana Code Annotated	MCA
Montana Resources LLP	MR
North-South	N-S
Probable Maximum Flood	PMF
Probable Maximum Precipitation	PMP
Quantitative Performance Parameters	QPPs
Tailings Operations, Maintenance and Surveillance	TOMS
Triaxial	TX
West	W
Yankee Doodle Tailings Impoundment	YDTI

1 – INTRODUCTION

1.1 GENERAL

Montana Resources, LLP (MR) operates an open pit copper and molybdenum mine located within the northeastern part of Butte, Montana. The operation includes a mill throughput of roughly 50,000 short tons per day and a small-scale dump leaching operation.

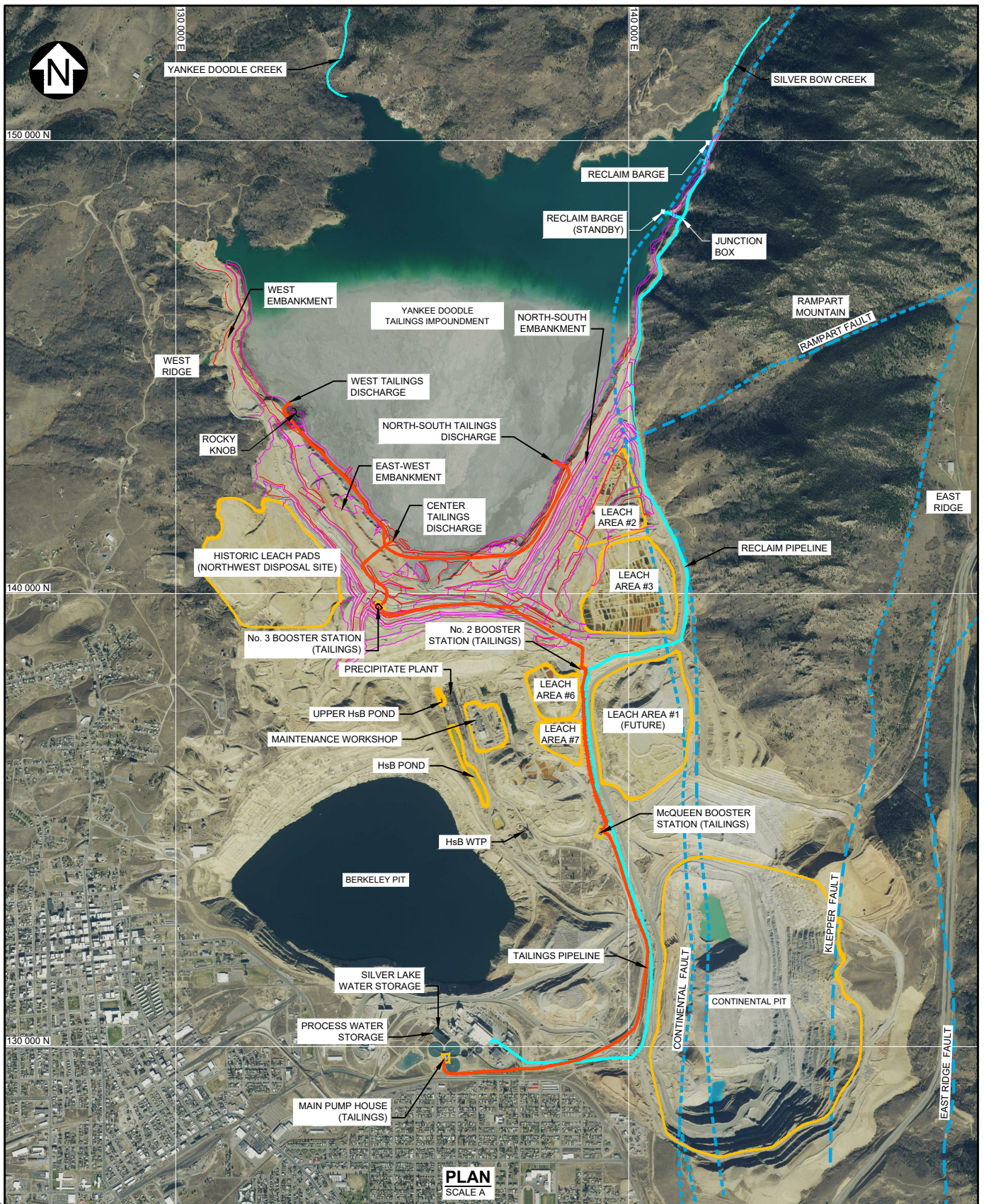
The Yankee Doodle Tailings Impoundment (YDTI) is the tailings storage facility for the mine. The YDTI was originally constructed in 1963 using rockfill obtained from Berkeley Pit stripping operations and has been continuously expanded to elevation (EL.) 6,400 ft using rockfill from the Berkeley Pit (until 1982) and from the Continental Pit (beginning in 1986). The YDTI comprises a valley-fill style impoundment created by a continuous rockfill embankment as shown on Figure 1.1. The embankment is divided into three rockfill embankments according to the general geometry of each limb of the continuous embankment for descriptive purposes. These embankments are the:

- North-South Embankment - The North-South Embankment forms the eastern to southeastern limb of the YDTI and runs approximately north to south in orientation. The North-South Embankment abuts onto the base of Rampart Mountain, forming the eastern limit of the MR mine site.
- East-West Embankment - The East-West Embankment forms the southwestern limb of the YDTI and runs approximately east to west in orientation. The East-West Embankment is constructed upstream of Horseshoe Bend and the Berkeley Pit.
- West Embankment - 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.

Historically the YDTI has been constructed by progressively placing rockfill to form free-draining rockfill embankments. The rockfill comprises pit-run material end-dumped in 30 to 100 ft lifts with the mine haul fleet. Ripping of the completed lift surfaces has been commonly completed to enhance vertical infiltration. The embankment design 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. A conceptual layout for the future facility is shown on Figure 1.2. The filling of the YDTI will be monitored throughout operations and construction sequencing will be evaluated periodically to confirm agreement with the design.

▲ R2



NOTES:

- COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
- IMAGERY FROM 2015 AIR PHOTO PROVIDED BY MONTANA RESOURCES.
- 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	15AUG'17	ISSUED WITH REPORT	DDF	MJC	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

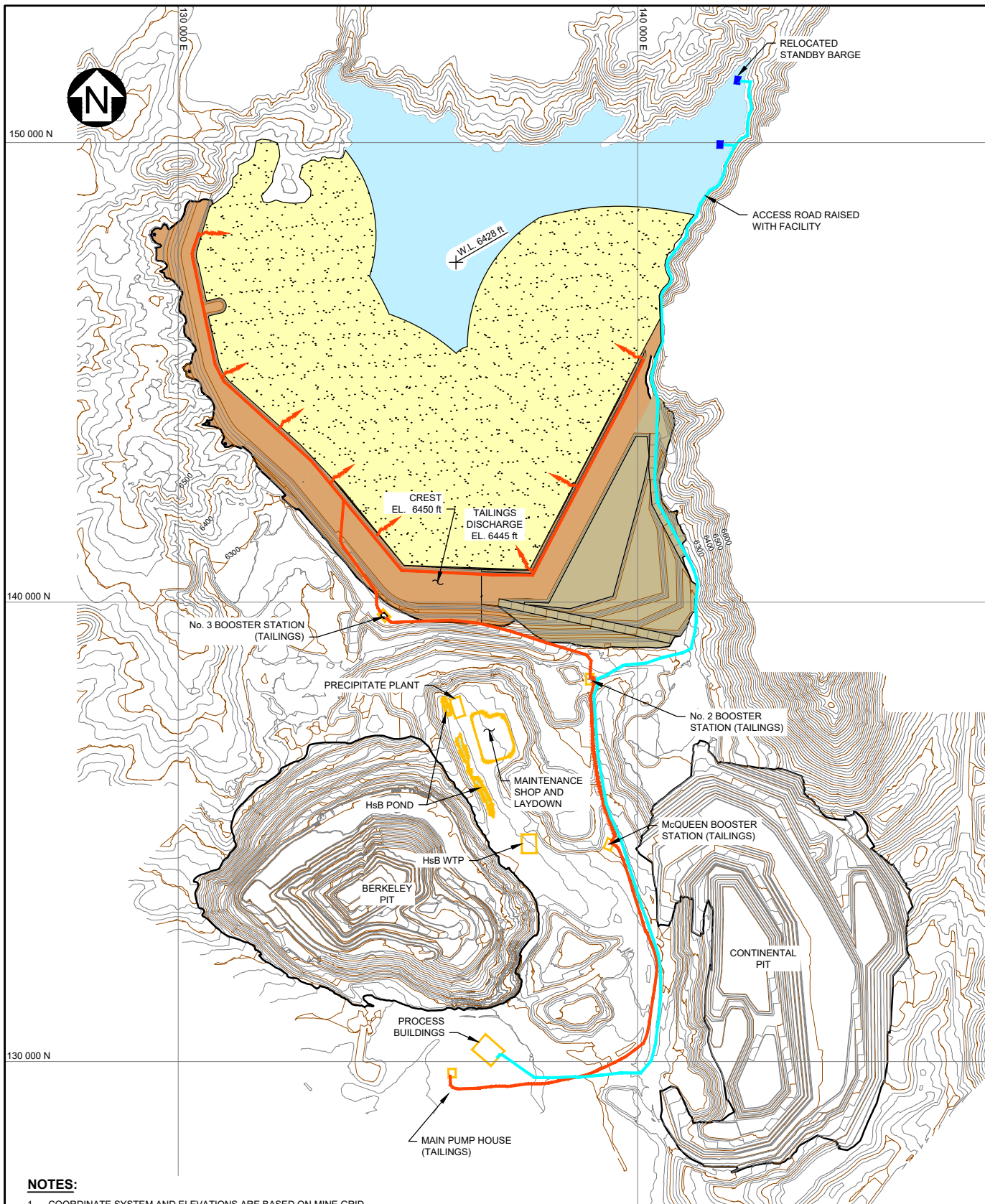
SITE LAYOUT (MARCH 2017)

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FIGURE 1.1

REV 1

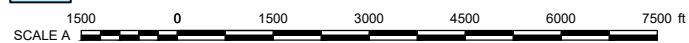


NOTES:

- 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



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YANKEE DOODLE TAILINGS IMPOUNDMENT

FUTURE FACILITY LAYOUT

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FIGURE 1.2

REV
1

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
1	15AUG'17	ISSUED WITH REPORT	DDF	KJM/MJC	KJB

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.

▲ R2

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.

1.2 PURPOSE AND SCOPE

▲ R2

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. The amendment will provide for approximately 12 years of additional mine life.

This report, prepared by Knight Piésold Ltd. (KP), summarizes the risk assessment of the YDTI at its proposed design crest elevation. Specifically, this assessment presents an examination of foundation and embankment instability, overtopping, and internal erosion and piping. The assessment considers loading during maximum normal operating conditions, loading from seismic events, flood events, and malfunctions of the reclaim water and tailings distribution systems.

The purpose of this study is to identify and characterize the risks associated with a potential dam breach of the YDTI. A dam breach is characterized by the sudden rapid and uncontrolled release of water or fluidized tailings solids from the impoundment. It is recognized that there are other types of failure, (e.g. uncontrolled contaminated seepage); however, these types of failure are not assessed in this study.

The risk analysis has also been used to define preliminary Quantitative Performance Parameters (QPPs) that will be used to monitor the YDTI to identify circumstances that could initiate emergency conditions. QPPs are measurements that can be easily made without complex calculation or interpretation, and are a good reference to quickly assess the performance of the YDTI.

1.3 LEGISLATED REQUIREMENTS

Montana Code Annotated (MCA) 82-4-376 describes the design document requirements for an operator proposing to expand an existing tailings storage facility and is the governing legislation for preparation of the expansion design (MCA, 2015). The requirements include:

“a dam breach analysis, a failure modes and effects analysis or other appropriate detailed risk assessment, and an observation method plan addressing residual risk,”

And:

“a list of quantitative performance parameters (QPPs) for construction, operation, and closure of the tailings storage facility. The QPPs may be expressed as minimums or maximums for the embankment crest width, embankment slopes, beach width, operating pool volume, phreatic surface elevation in the embankment and foundation, pore pressures, or other parameters appropriate for the facility and location;”

This report fulfills the above statutory requirements for the design document.

1.4 PREVIOUS RISK ANALYSIS STUDIES

KP completed a report titled “Failure Mode Analysis Information Summary” in early February 2013 (KP, 2013). The purpose of this report was to review the design and operation of the YDTI and to assist in the compilation of relevant information to support the simplified Failure Mode Analysis (FMA) workshop conducted by MR.

A summary report of the FMA workshop conducted by MR was compiled by KirK Engineering and Natural Resources Inc. (KirK) in late February 2013 (Kirk, 2013). The FMA workshop assessed potential modes of failure for the YDTI. Participants in the FMA workshop included representatives from MR, Silver Bow City County government, The Montana Department of Environment Quality (DEQ), the Montana Department of Natural Resources and Conservation, the Montana Bureau of Mines and Geology, and the U.S. Environmental Protection Agency.

2 – LIKELIHOOD AND CONSEQUENCE RATING CRITERIA

2.1 GENERAL

Risk is represented by the product of the likelihood of an event occurring and the consequences of that event:

$$\text{Risk} = \text{Likelihood} \times \text{Consequence}$$

In general terms, risk is higher when the likelihood and consequence of failure is higher, and risk is lower when the likelihood and consequence is lower. Likelihood can be further resolved into the probability of a certain event or loading condition occurring (e.g. the return period of an earthquake) and the probability of a failure occurring coincident with that event (e.g. how likely is it that the earthquake will cause deformation that constitutes failure). Therefore, likelihood can be further described as follows:

$$\text{Likelihood} = \text{Probability of Loading Conditions} \times \text{Probability of Coincident Failure}$$

The probability of an event occurring is generally easier to define quantitatively, whereas the probability of failure due to an event (the imposed loading conditions of the event) is considered through deterministic safety analyses.

2.2 LIKELIHOOD

2.2.1 Probability of Loading Conditions

The probability of the occurrence of a loading condition is directly associated with the risk of an event occurring. A more frequent or likely event or loading condition may have an increased need to reduce the probability of failure associated with the event if the potential consequences are severe and cannot be altered.

Normal operating conditions are considered certain, and are expected to occur once or more on an annual basis. Rainfall and seismic events can be classified based on the probabilistic return period associated with an event under consideration. For instance, a 1 in 1,000 year return period wet month is expected to occur once every 1,000 years. Such an event is unlikely to occur, but has a small probability of occurrence in each and every year. Larger rainfall events, like the Probable Maximum Precipitation (PMP), are even less likely and are considered to be very rare, maximum credible events.

The loading conditions imposed by an event considered in this assessment are defined based on the probability levels presented in Table 2.1. The category corresponding to the probability of the loading condition has been qualitatively defined.

Table 2.1 Probability of Loading Conditions

Category	Probability of Loading Condition
Very Rare	One event per 10,000 years or deterministic based maximum credible
Unlikely	One event per 1,000 years
Possible	One event per 100 years
Likely	One event per 10 years
Certain	One or more events per year

2.2.2 Probability of Coincident Failure

The definitions of four categories of failure probability are presented in Table 2.2 below. Each category is defined with a category value and a general description of the conditions for the value assigned. These categories provide a connection to the deterministic safety analyses used to demonstrate adequate dam safety under the loading conditions considered. The deterministic analyses will be referenced and summarized in subsequent sections supporting the analysis of each failure mode.

If a deterministic analysis cannot be completed to demonstrate adequate performance for a particular event, then a conservative assumption that failure probability is “Moderate” for that event can be made to simplify the risk analysis for dam safety decision making purposes.

Table 2.2 Probability of Coincident Failure

Category	Description
Not Credible	Failure mode not credible for loading condition and initiating events under consideration
Very Low	Robust analysis demonstrates stable condition, exceeds FS requirements, with appropriately conservative assumptions and with consideration of sensitivity analyses
Low	Analysis demonstrates marginal performance in considered loading condition, meets FS requirements, however analysis technique not as robust or sensitivity range does not capture full range of conditions
Moderate	Does not meet minimum requirements under loading conditions or failure mode cannot be analyzed in a practical manner

2.3 CONSEQUENCES

Four categories of consequence are defined in Table 2.3. The consequence of failure varies from Minor to Catastrophic and qualitatively describes the severity of the potential damage if failure were to occur. A consequence definition will not be evaluated if the probability of coincident failure is Not Credible.

Table 2.3 Consequence Definitions

Category	Description
Minor	Minor deformation, local area of impact
Moderate	Serious deformation, but no uncontrolled release of containment
Major	Uncontrolled release, contained within project area
Catastrophic	Uncontrolled release, off-site impact

Minor and Moderate consequence categories are generally consistent with Level 1 and Level 2 unusual occurrences, respectively, as defined in the Tailings Operations Maintenance and Surveillance (TOMS) Manual (MR, 2016). These two levels of consequence do not lead to uncontrolled release of impounded materials, and are acceptable if the level of deformation is expected for the loading condition under consideration.

A minor consequence is considered to be a deformation that is aesthetic and easily repairable. An example of a consequence with a minor severity is a ravelling or erosion of a local bench slope, or localized cracking on the YDTI embankment crest or slopes. Typically, this sort of deformation would require increased daily surveillance to monitor displacement until the problem is understood and minor repairs are completed. A moderate consequence is defined as a deformation or erosion impacting the crest width, or crest cracking that is progressively increasing provided there is no uncontrolled release of impounded materials. These are conditions that represent a potential emergency, if sustained or allowed to progress, but no emergency situation is imminent. A field investigation to identify the cause of the deformation will be required, and corrective repairs will be performed to return the facility to operating condition.

Major and Catastrophic categories are both consistent with Level 3 emergency conditions in the TOMS Manual, defined as an actual or imminent failure of containment. A consequence severity threshold adopted for this analysis defines the mine site boundary as a key spatial limitation to the consequence definitions. A Major consequence is defined as a breach outflow that is contained within the project area. A breach outflow that is not contained within the project area and has an off-site impact is defined as a Catastrophic consequence. A conservative assumption that failure consequences are potentially catastrophic can be made to simplify the risk analysis for dam safety decision making purposes.

Major and Catastrophic consequences are unacceptable. Either consequence requires a very low probability of failure or a very rare probability of the event occurring to manage risk. In both cases, QPPs must be established and incorporated into updates to the TOMS Manual.

3 – FOUNDATION AND SLOPE INSTABILITY

3.1 GENERAL

This section of the report summarizes the risk ratings for foundation and slope instability. The assessment considers normal operating conditions without any malfunctions. The assessment then uses an informal, deductive fault tree analysis structure to evaluate the causal events (i.e. malfunctions) necessary to induce failure for each failure mode. This is done by manipulating factors affecting the deterministic safety analyses to determine threshold levels for key factors that can be used as QPPs. Additional details supporting the risk ratings below in Table 3.1 are provided in the sections that follow.

Table 3.1 Risk Ratings for Foundation and Slope Instability

Failure Mode	Likelihood		Consequences
	Probability of Loading Conditions	Probability of Failure	
Foundation and Slope Instability ^{1,2}	Likely	Very Low	Moderate
Earthquake Events (or other loss of material strength)	Very Rare	Very Low	Moderate to Major
Flood Events	Very Rare	Very Low	Moderate to Catastrophic

▲ R2

▲ R2

NOTES:

1. THE LOADING CONDITION IS DEFINED AS NORMAL OPERATING CONDITIONS WITHOUT ANY MALFUNCTIONS.
2. FOUNDATION AND SLOPE INSTABILITY CONSIDERS THE MAXIMUM DRIVING FORCE AND THE MINIMUM RESISTING FORCE FOR STATIC STABILITY, WHICH IS PRESENT ONLY PERIODICALLY EITHER FOLLOWING A LIFT OF THE EMBANKMENT FOR UPSTREAM STABILITY OR AT THE ULTIMATE FILLING LEVEL FOR EACH LIFT (DOWNSTREAM STABILITY). THE LOADING CONDITION WILL OCCUR ROUGHLY EVERY TEN YEARS DURING OPERATIONS, AND THEREFORE THE PROBABILITY OF THE LOADING CONDITION IS 'LIKELY'.

3.2 NORMAL OPERATING CONDITIONS

3.2.1 Description of Failure Mode

▲ R2

Static instability through the foundation and embankment slope considers analyses of the potential for both upstream and downstream slope failure independently. 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 operations just prior to closure with the tailings surface adjacent to the embankment at 5 ft (minimum freeboard) below the crest.

The upstream instability mode considers a combination of the maximum driving force against the minimum resisting force, which is represented by an instantaneous raise of the final 50 ft embankment lift above the top of the tailings. The upstream instability is akin to bearing capacity failure, requiring displacement of embankment rockfill into the extensive drained tailings beach. 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 since the supernatant pond is far from the embankments.

Both instability modes simulate a potential for loss of containment and uncontrolled release of the impounded material. Slope failure of this kind may not actually produce a loss of containment; however, it was conservatively assumed to constitute a potential failure mode.

3.2.2 Method of Analysis and Summary of Results

Static slope stability was assessed by determining the Factor of Safety (FS) for the selected representative sections of the East-West (E-W), North-South (N-S) and West Embankments (W). A detailed description of the inputs and results is presented in the Stability Assessment Report (KP, 2018c). The layout of the sections selected for the slope stability analyses are shown on Figure 3.1. Typical embankment sections for the East-West, North-South and West Embankments are shown on Figure 3.2.

The minimum required factor of safety for static maximum normal operating conditions is 1.5. The resulting FS exceeds the minimum required for all upstream and downstream slip surfaces. The FS for the critical slip surface is shown in Table 3.2. The governing scenario for all sections analysed was a downstream slip intersecting overburden in the foundation.

▲ R2

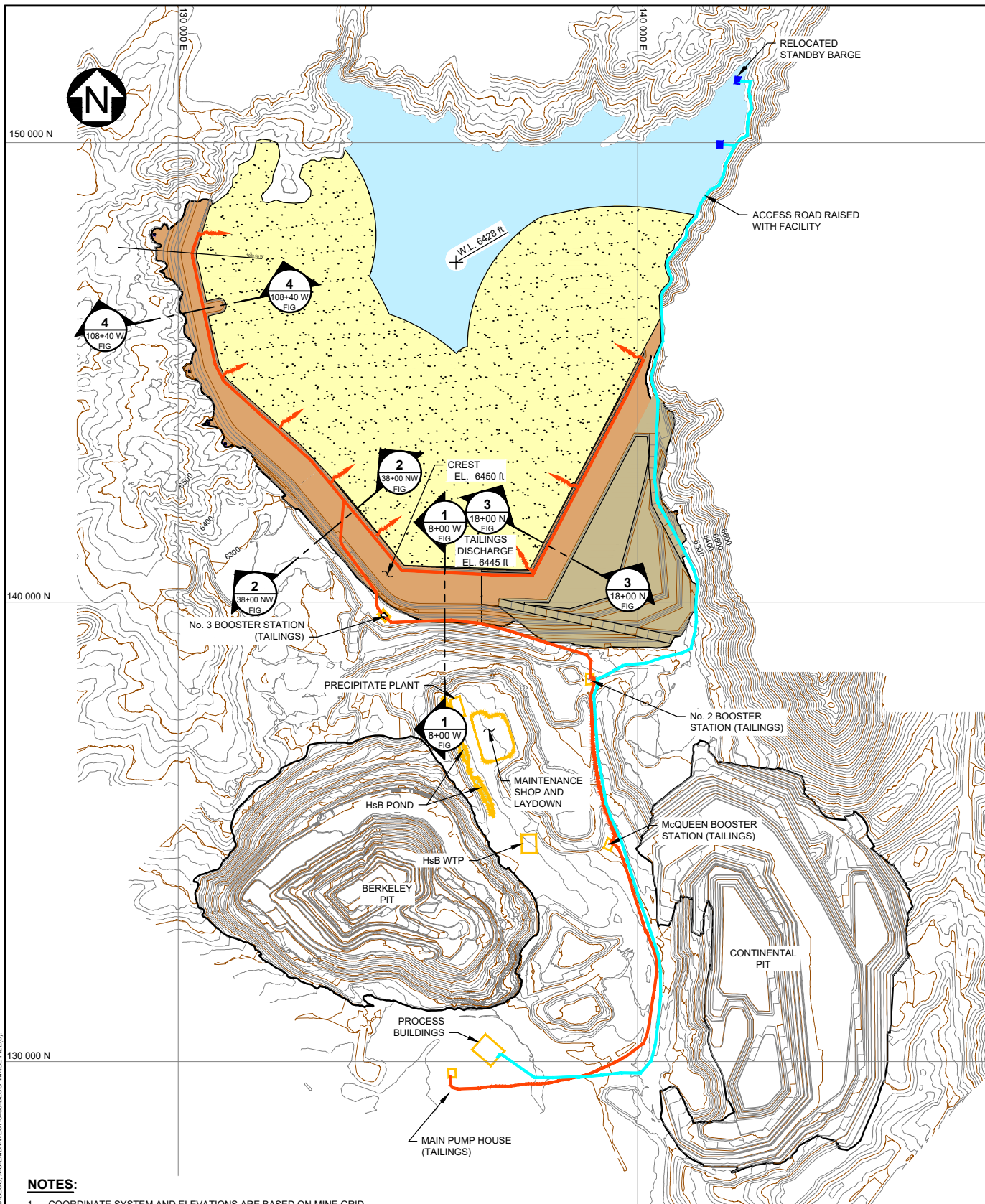
Table 3.2 Static Slope Stability Factors of Safety

Embankment Section	Minimum Required FS	Downstream FS ¹	Upstream FS ¹
08+00 W (E-W)	1.5	2.0	> 5
38+00 NW (E-W)	1.5	4.2	> 5
18+00 N (N-S)	1.5	2.0	> 5
108+40 (W)	1.5	7.2	> 5

▲ R2

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.
2. FACTORS OF SAFETY BASED ON VALUES FOR BASE CASE MATERIAL PROPERTIES IN TABLE 5.2 OF THE STABILITY ASSESSMENT REPORT (KP, 2018c).
3. '(E-W)' STANDS FOR EAST-WEST EMBANKMENT, '(N-S)' STANDS FOR NORTH-SOUTH EMBANKMENT, '(W)' STANDS FOR WEST EMBANKMENT.



NOTES:

- 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

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
LAYOUT OF SLOPE STABILITY SECTIONS
EMBANKMENT EL. 6450 (FULL)

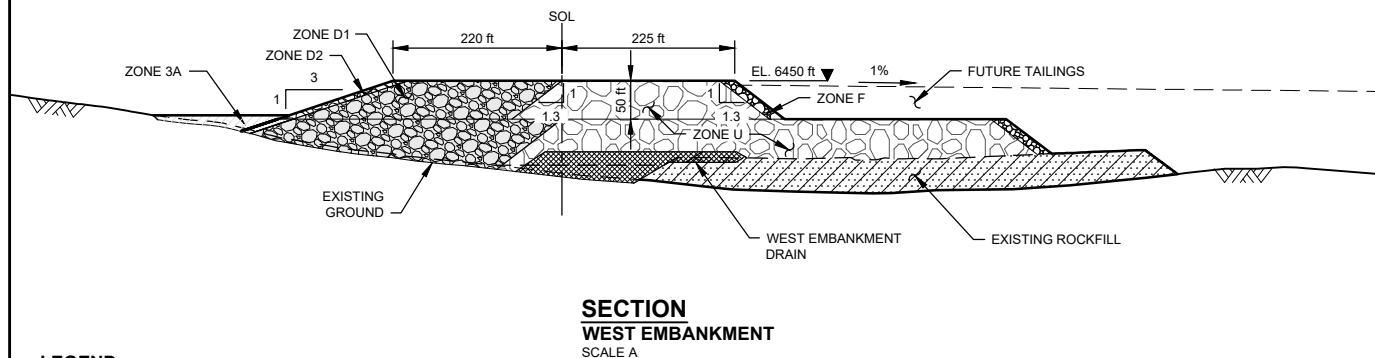
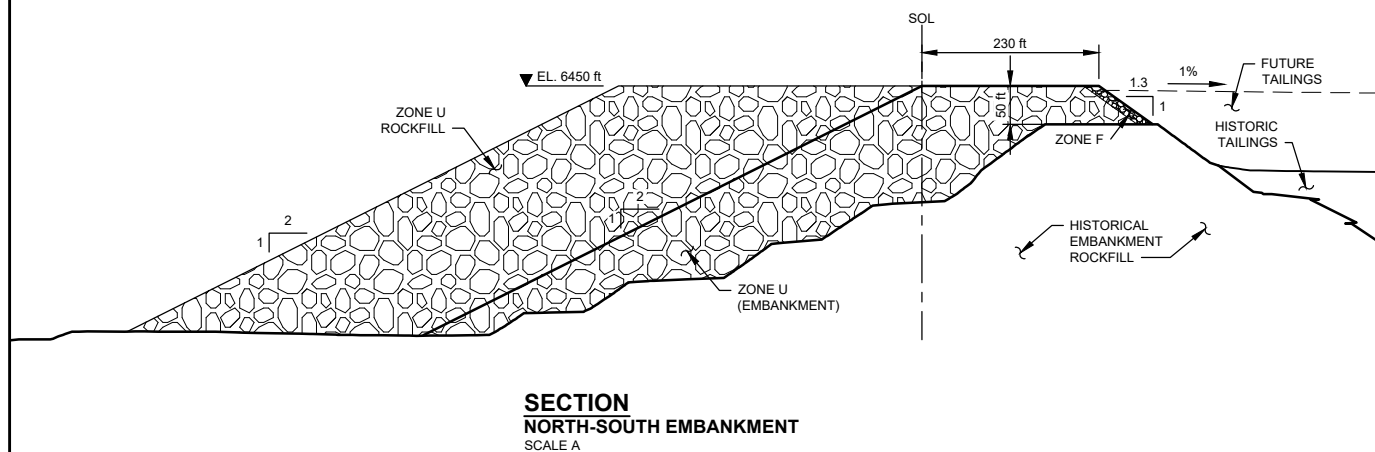
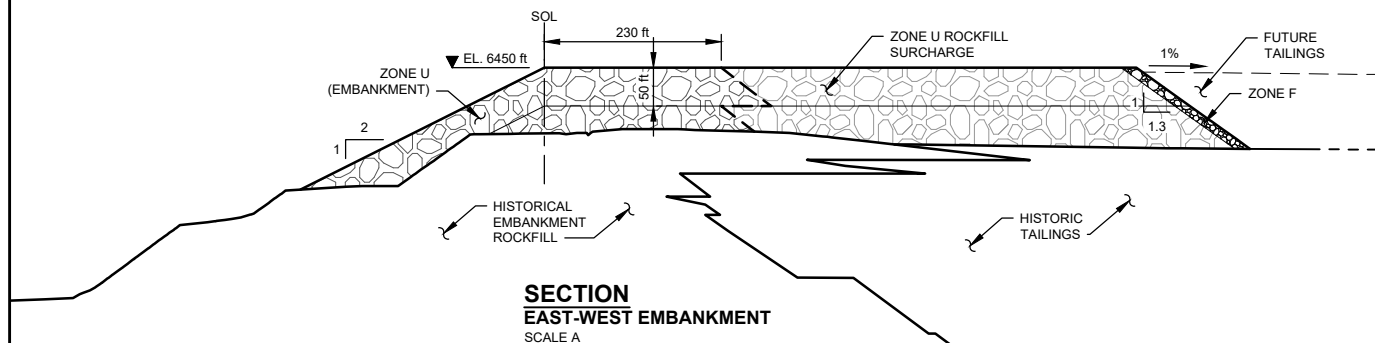
Knight Piésold
CONSULTING

P/A NO. VA101-126/12	REF NO. 3	REV 1
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FIGURE 3.1

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	15AUG'17	ISSUED WITH REPORT	DDF	MJC	KJB

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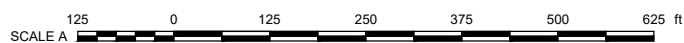


LEGEND:

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

NOTES:

- COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON MINE GRID.



MONTANA RESOURCES, LLP			
YANKEE DOODLE TAILINGS IMPOUNDMENT			
TYPICAL EMBANKMENT SECTIONS			
Knight Piesold CONSULTING		P/A NO. VA101-126/12	REF NO. 3
FIGURE 3.2			REV 1

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
1	15AUG'17	ISSUED WITH REPORT	DDF	MJC	KJB

3.2.3 Risk Rating

▲ R2

The slope stability analysis completed for static conditions considers a conservative scenario at ultimate filling of the impoundment. This is a condition that would develop only once during the operational life of the facility. The probability of this condition occurring meets the criteria for 'Likely' in Table 2.1. The predicted factors of safety of the slope stability analyses are based on the following:

- 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. The representation approximates the pore pressure conditions at the tailings discharge point, and is conservatively extrapolated through the remainder of the impoundment. The embankment fill and tailings materials below the piezometric lines are modelled as saturated with hydrostatic pore pressure development, although field data indicates that pore pressure development is less than hydrostatic.
- Material strength properties used to determine the factors of safety presented above are more conservative than past studies performed for the YDTI.
- The material properties adopted for rockfill are based on a series of relationships developed by Thomas Leps (Leps, 1970). The Leps non-linear shear strength functions recognize that rockfill (and sand) can maintain a higher effective angle of friction at lower confining pressures due to the dilatant behaviour of the material under shearing, while a lower strength may occur at higher stresses due to particle crushing and reduced dilation of the material. The stability analyses conservatively assume that the existing rockfill material degrades over time to the shear strength function of Leps Angular Sand, and that newer rockfill has the shear strength function of the Leps Lower Boundary.
- The base case frictional strength of 32 degrees for the tailings represents the 30th-percentile of a substantial data set estimated from the cone penetration test soundings between 2012 and 2015. Consolidated undrained (CU) triaxial testing has demonstrated higher effective friction angles for the tailings material.
- The silty-sand overburden comprised of quaternary alluvium and completely weathered bedrock is represented by a base case effective friction angle of 27 degrees, which represents a lower bound strength for this material.
- The impenetrable bedrock function is applied to bedrock in the stability assessment, which has the effect of limiting critical slip surfaces to the embankment and overburden.

The factors of safety under these conservative conditions exceed the requirements. The YDTI embankments are stable with a FS of 2.0 or greater. The probability of failure coincident with these loading conditions meets the criteria for 'Very Low' in Table 2.2.

The slip surfaces and conditions assessed simulate loss of containment and uncontrolled release of impounded material, however field data suggests that the tailings adjacent to the embankment have little flow potential since the top 50 ft or more are not saturated and the supernatant pond is far from the embankments. An assessment was performed to determine the potential for tailings to flow in the event of a hypothetical and sudden loss of containment due to static deformation (Appendix A). The assessment determined that the tailings below the phreatic surface are sufficiently dense to prevent flow, in the event they become unconfined, without a source of water to initiate erosion. The consequences of slope instability under normal operating conditions are assessed as 'Moderate'

based criteria in Table 2.3. The location of the supernatant pond and level of saturation in the tailings will have the greatest impact on the potential consequences of failure, and maintaining the pond remote from the embankments can effectively mitigate this very low risk.

In summary, the risk ratings for foundation and slope instability for normal operating conditions are as follows:

- Probability of Loading Condition: **Likely.**
- Probability of Coincident Failure: **Very Low.**
- Consequences of Failure: **Moderate.**

3.3 FACTORS INFLUENCING THE DETERMINISTIC ANALYSES

The FS against foundation or slope instability is dependent on a number of factors. Figure 3.3 shows a summary of the factors affecting static slope stability.

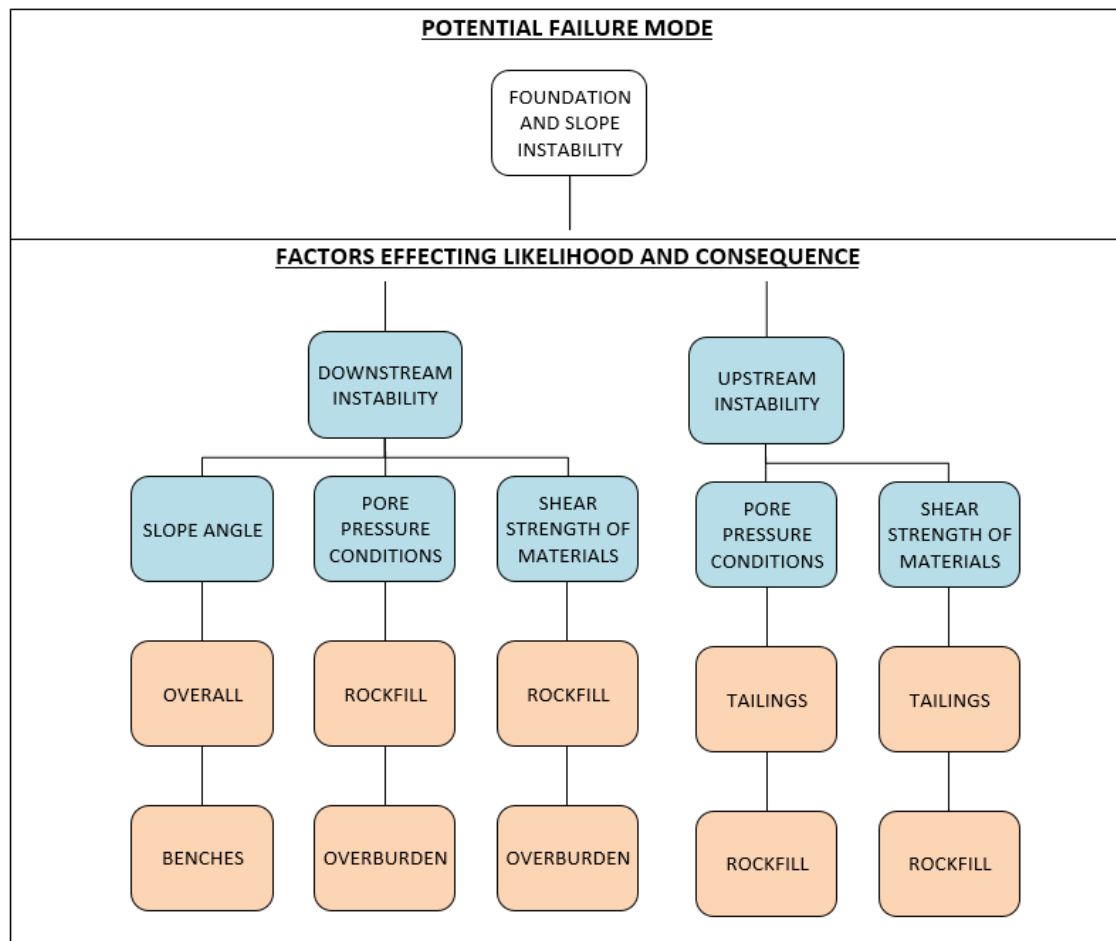


Figure 3.3 Factors Affecting Foundation and Slope Stability

Slope stability of the YDTI embankments is affected by:

- The downstream slope angle of the embankment
- Pore pressure conditions in the tailings, embankment, and foundation, and
- The shear strength of tailings, embankment rockfill and underlying overburden foundation.

Inter-bench slopes have been and continue to be developed at angle of repose with benches between successive lifts generating overall slope angles of 2 to 1 (horizontal to vertical) or flatter. Some local slope ravelling of inter-bench slopes is expected with material accumulating on the bench below. This type of small scale instability does not constitute a dam safety risk for the impoundment, and is acceptable as long as it does not present a hazard to worker safety.

MR prepared an inventory of embankment inter-bench slopes based on 1994 aerial mapping (MR, 1999). The mapping indicated that the average slope between catch benches on the downstream face of the embankment was 36 degrees for a distance of roughly 80 ft. The data indicated that high slopes are usually steeper (36 to 38 degrees) at the top with a break in slope near the toe at about 29 degrees. The mapping is consistent with visual observations of current conditions. The sensitivity of factors of safety against shallow downstream slope instability of the embankment (potentially impacting multiple benches, but not impacting the full crest width) can be evaluated in a rudimentary manner by applying slope stability theory for infinite slopes. The factors of safety for relevant overall slope angles are provided in Table 3.3. The risk of moderate consequence slope instability is managed by maintaining overall downstream slope angles of 2 to 1 (horizontal to vertical) or flatter.

Table 3.3 Sensitivity of FS to Overall Slope Angle

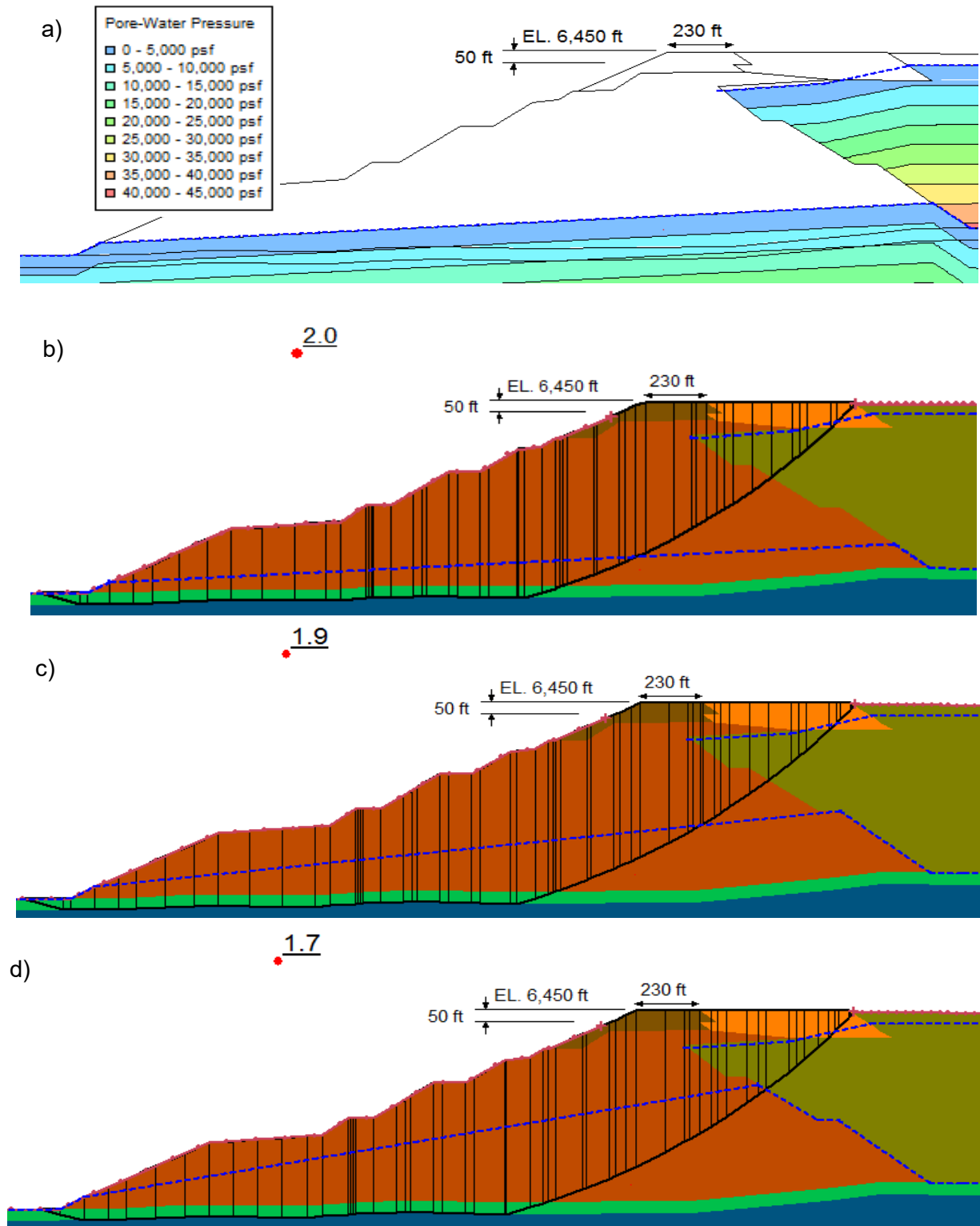
Overall Slope Angle ^{1, 2}	Rockfill Internal Angle of Friction	Factor of Safety ³
2H:1V (26°)	36°	1.5
2.25H:1V (24°)	36°	>1.6
2.5H:1V (22°)	36°	1.8
3H:1V (18°)	36°	>2.2

NOTES:

1. OVERALL SLOPE ANGLES ARE PRESENTED IN TERMS OF RATIO OF HORIZONTAL (H) TO VERTICAL (V) DISTANCE, AND IN DEGREES.
2. OVERALL SLOPE ANGLES ARE CONSISTENT WITH SLOPE ANGLES FOR THE EXPANSION OF THE NORTH-SOUTH, EAST-WEST, AND WEST EMBANKMENTS.
3. FACTOR OF SAFETY (FS) IS RELEVANT FOR AN INFINITE SLOPE OF DRAINED, COHESIONLESS SOIL.

The piezometric surfaces used in the stability assessment are simplified and conservative representations of the pore pressure conditions identified during the 2015 site investigation programs. The modelled piezometric lines conservatively apply hydrostatic conditions to the materials below the line. The piezometric surface approximates the pore pressure conditions at a tailings discharge point, and is conservatively extrapolated through the remainder of the impoundment for the static stability analyses. ▲ R2

The sensitivity of the slope stability FS to pore pressure conditions was evaluated by manipulating the pore pressure conditions in the lower bench of the embankment and the foundation for Section 8+00W of the East-West Embankment. The piezometric surface was increased to mimic a static rise in pore pressures. The analysis was performed by incrementally increasing the piezometric line in the embankment on the upstream side as indicated on Figure 3.4. These analyses were used to evaluate the effect of pore pressure conditions on global stability of the embankment. The calculated FS are included in Table 3.4. ▲ R2

**NOTES:**

- a) PIEZOMETRIC CONDITIONS APPLIED IN SLOPE/W BASE CASE.
 b) FACTOR OF SAFETY (FS) CALCULATED FOR SLOPE/W BASE CASE.
 c) SENSITIVITY OF FS TO 150 FT STATIC PORE PRESSURE RISE.
 d) SENSITIVITY OF FS TO 300 FT STATIC PORE PRESSURE RISE.

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

SLOPE STABILITY SENSITIVITY TO STATIC RISE IN PORE PRESSURE***Knight Piésold***
CONSULTINGP/A NO.
VA101-126/12REF. NO.
3**FIGURE 3.4**REV
1

1	22AUG'17	ISSUED WITH REPORT	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	RVW'D

Table 3.4 Sensitivity of Static FS to Pore Pressure Conditions

▲ R2

Section (Embankment Limb) ³	Piezometric Elevation on Upstream Side	Minimum Required Factor of Safety	Factor of Safety ¹
8+00W (E-W) ²	5,836 ft	1.5	2.0
8+00W (E-W)	5,995 ft	1.5	1.9
8+00W (E-W)	6,145 ft	1.5	1.7

NOTES:

1. THE FACTORS OF SAFETY FOR THE SENSITIVITY ANALYSES CONSIDER SLOPE INSTABILITY THROUGH THE EMBANKMENT CREST AND A SLIP SURFACE THAT INTERSECTS THE TAILINGS BEACH.
2. BASE CASE CONDITIONS AND FACTOR OF SAFETY FROM TABLE 3.2.
3. E-W STANDS FOR EAST-WEST EMBANKMENT.

The results indicate that the overall stability of the embankment under static conditions is insensitive to moderate increases in the pore pressure conditions in the embankment. This sensitivity case is reflective of unrealistic pore pressure increases in the embankment due to severe flooding and impeded drainage within the embankment. The consequence classification was altered to include the potential for 'Moderate to Catastrophic' consequences in the case of a severe flooded condition positioning the supernatant pond near to the embankment crest. The likelihood of the flooded condition developing is very rare, but the increased potential consequences of failure highlight the importance of water management and tailings beach development to limit risk. These analyses have not considered rockfill strength reduction for hypothetical undrained loading conditions within the embankment, which will be discussed in the post-earthquake analysis. The risk ratings for static instability caused by increased pore pressures following severe flooding are as follows:

▲ R2

- Probability of Loading Condition: **Very Rare**
- Probability of Coincident Failure: **Very Low**
- Consequences of Failure: **Moderate to Catastrophic**

The increase in pore pressure conditions that is required to reduce the FS to degree presented in Table 3.4 would be preceded by a very pronounced trend in elevated pore pressure conditions measured within the embankment. Developing extensive and relatively uniform tailings beaches around the facility is the best mitigation to limit pore pressure rises in the embankment under flooded conditions. Reducing the operating pond volume in conjunction with extensive beach development, as long as the pond volume is appropriate to meet water clarity objectives, will also reduce the likelihood and consequences of this failure mode without adversely affecting mine operations. The residual risk can be managed by monitoring pore pressures with established QPPs for specific instrumentation. The specified instrumentation and QPPs are defined in Section 3.4, and will be incorporated into the TOMS Manual (MR, 2016).

▲ R2

The 2015 SI classified the variable rockfill encountered as highly altered and weathered gravels, cobbles and boulders within a silty sand or sandy silt matrix. Particle strength of clasts ranged from hard competent rockfill to highly altered and friable. Observations made during drilling did not support a differentiation of shear strength parameters by historic pit source (KP, 2017a). As a result, the weakest Leps (1970) relation of Angular Sand was assigned to all historic rockfill material in recognition of the potential for site wide variability and long-term stability in closure. Use of this

function may be somewhat conservative where historically the most durable rockfill materials were placed. New rockfill material from the Continental Pit was assigned the slightly stronger Leps (1970) Lower Boundary relation for the new rockfill units.

An additional sensitivity analysis was completed to evaluate the sensitivity of the estimated FS for Section 8+00W of the East-West Embankment assuming the new rockfill material from the Continental Pit was assigned the weaker Leps (1970) Angular Sand relationship. The resulting factors of safety are shown in Table 3.5. The results indicate that the overall stability of the embankment is insensitive to the minor differences between the Leps Angular Sand and Lower Boundary relationships.

Table 3.5 Sensitivity of Static FS to Rockfill Strength

▲ R2

Section (Embankment Limb) ³	Slip Surface Location	Original Factor of Safety ¹	Modified Factor of Safety ²
8+00W (E-W)	Downstream	2.0	2.0
8+00W (E-W)	Upstream	> 5	> 5

NOTES:

1. ORIGINAL FACTORS OF SAFETY FROM TABLE 3.2.
2. MODIFIED FACTORS OF SAFETY CALCULATED ASSUMING NEW ROCKFILL MATERIAL WAS ASSIGNED LEPS ANGULAR SAND RELATIONSHIP INSTEAD OF LEPS LOWER BOUNDARY FUNCTION USED IN THE ORIGINAL ANALYSIS.
3. E-W STANDS FOR EAST-WEST EMBANKMENT.

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 for that particular sensitivity analysis.

▲ R2

An assessment was completed to determine the liquefaction potential of the tailings resulting from seismic loading from the design earthquake (KP, 2018c). The assessment indicated that 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 84th-percentile MCE while the tailings remain saturated.

▲ R2

An analysis of slope stability under static post-earthquake conditions was completed by conservatively assigning undrained and liquefied material strength properties to the entire tailings mass (KP, 2018c). The tailings were modelled with two undrained strength ratios to investigate the sensitivity of the FS to a variation in tailings material strength. Tailings were 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 tailings strength has very little impact on the predicted factor of safety for downstream slip surfaces. The calculated FS for a liquefied undrained strength ratio of 0.05 are included in Table 3.6. 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.

▲ R2

Table 3.6 Static Post-Earthquake Slope Stability Factors of Safety

▲ R2

Embankment Section	Minimum Required FS	Downstream FS ¹	Upstream FS ¹
08+00 W (E-W)	1.2	1.9	3.1
38+00 NW (E-W)	1.2	3.8	3.2
18+00 N (N-S)	1.2	2.0	> 5
108+40 (W)	1.2	7.1	> 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.
2. FACTORS OF SAFETY BASED ON STATIC POST-EARTHQUAKE CONDITIONS IN TABLE 5.3 OF THE STABILITY ASSESSMENT REPORT (KP, 2018c).
3. '(E-W)' STANDS FOR EAST-WEST EMBANKMENT, '(N-S)' STANDS FOR NORTH-SOUTH EMBANKMENT, '(W)' STANDS FOR WEST EMBANKMENT.

A sensitivity analysis was completed to determine the effect of decreasing the shear strength of the overburden on the global static stability of the embankment. A check was completed to determine the friction angle required in the overburden to reduce the FS to 1.5 for normal operating conditions and 1.2 for post-earthquake conditions. The results are shown in Table 3.7. The sensitivity analysis indicates that under base case conditions the FS will exceed the minimum required FS of 1.5 unless the effective friction angle of the overburden is reduced below 16 degrees. The lower bound case that considers post-liquefaction tailings residual strength in conjunction with reduced overburden strength would reduce the FS to less than 1.2 assuming an effective friction angle of 12 degrees for overburden.

▲ R2

Table 3.7 Sensitivity of Static FS to Overburden Effective Strength

▲ R2

Section (Embankment Limb) ⁴	Slip Surface Location	Original Effective Friction Angle (°)	Original Factor of Safety ^{1, 2}	Modified Effective Friction Angle (°)	Modified Factor of Safety ³
8+00W (E-W)	Downstream	27	2.0 ¹	16	1.5 ³
8+00W (E-W)	Downstream	27	1.9 ²	12	1.2 ³

NOTES:

1. ORIGINAL FS FROM TABLE 3.2, BASE CASE MATERIAL PROPERTIES APPLY TO ALL MATERIALS.
2. ORIGINAL FS FROM TABLE 3.6, POST-EARTHQUAKE MATERIAL PROPERTIES APPLY TO ALL MATERIALS.
3. MODIFIED FS CALCULATED ASSUMING OVERBURDEN MATERIAL WAS ASSIGNED A LOWER STRENGTH TO PRODUCE A FS < 1.5 FOR NORMAL OPERATING CONDITIONS AND A FS < 1.2 FOR POST-EARTHQUAKE CONDITIONS.
4. E-W STANDS FOR EAST-WEST EMBANKMENT.

The reduction in overburden strength required to reduce the FS to less than 1.5 is informative, but unlikely. Triaxial testing by Golder Associates in 1980 indicated that the angle of internal friction for alluvium recovered in the drilling ranged from approximately 25 to 35 degrees, which fits well with more recent testing completed by KP and typical values expected for this type of material. Direct shear tests were performed on samples of natural soils by Dames and Moore in the early 1960s prior to any substantial loading that these soils have undergone over the past 50 years. The results showed a wide range of variability, but the material was generally determined to be non-cohesive with angles of internal friction in the range of 16 to 42 degrees (KP, 2017a). This sensitivity analysis demonstrates an acceptable FS for significantly lower material strength properties, to allow for geotechnical uncertainties in the foundations.

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. 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.

▲ R2

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. Sensitivity analyses were performed and 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 (KP, 2018c).

▲ R2

The sensitivity analyses considered 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 3.8.

▲ R2

Table 3.8 Factors of Safety for Undrained Strength Sensitivity Analyses

▲ R2

Embankment Section	Material Properties ^{1,2}	Downstream FS ³
08+00 W	Overburden $S_u/\sigma'_v = 0.23$ (DSS)	1.3
	Saturated Rockfill $S_u/\sigma'_v = 0.27$ (DSS)	1.3
	Overburden $S_u/\sigma'_v = 0.23$ (DSS) & Saturated Rockfill $S_u/\sigma'_v = 0.37$ (TX)	1.2
08+00 W (Hypothetical Slip Surface at EL. 5,925 ft near Seep 10)	Perched Saturated Rockfill $S_u/\sigma'_v = 0.27$ (DSS) & $S_u/\sigma'_v = 0.37$ (TX)	1.4

NOTES:

1. MATERIAL PROPERTIES FOR TAILINGS ARE CONSISTENT WITH THE LIQUIFIED UNDRAINED STENGTH RATIO ($S_u/\sigma'_v = 0.05$) 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. FACTORS OF SAFETY BASED ON VALUES FROM TABLE 5.5 OF THE STABILITY ASSESSMENT REPORT (KP, 2018c).

These sensitivity analyses allow for geotechnical factors affecting stability to be indirectly considered. These factors include locally weaker foundation conditions, undrained pore pressure responses or other factors such as pre-shearing along discontinuous silty or clayey alluvium lenses. However, there is no evidence to suggest these potentially adverse conditions are present at the site, and thus these sensitivity analyses are only intended to illustrate the inherent contingency allowances that are present at the TSF.

An assessment was prepared to determine the potential for tailings to flow in the event of a hypothetical and sudden loss of containment due to earthquake deformation (Appendix A). The assessment determined that the tailings below the phreatic surface may have sufficient moisture content to flow in a viscous manner during seismic loading without confinement. Viscous flow is likely to stop without active shaking (i.e. at the end of the earthquake). The tailings are sufficiently dense to prevent flow under static conditions in the event they become unconfined without a source of water to initiate erosion. The consequences of slope instability under earthquake conditions are assessed as 'Moderate to Major' based criteria in Table 2.3. The estimated deformation and crest settlement would be classified as 'Moderate' consequences. The upper classification limit of 'Major' is consistent with the potential consequences of a viscous mudflow at the East-West Embankment based on the assessment described in Appendix A. The risk ratings for static instability caused by tailings and foundation material strength reduction due to seismic loading are as follows:

▲ R2

- Probability of Loading Condition: **Very Rare**
- Probability of Coincident Failure: **Very Low**
- Consequences of Failure: **Moderate to Major**

▲ R2

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 existing practice of placing higher strength rockfill materials in the higher sections of the embankment and weaker materials in non-structural areas should be continued. Monitoring of overall slope angles and pore pressures in the embankments will be required to demonstrate adequate performance of the facility.

▲ R2

3.4 QUANTITATIVE PERFORMANCE PARAMETERS

▲ R2

Foundation and slope stability of the embankment can be managed and monitored using the preliminary QPPs defined below in Table 3.9. The risk of moderate consequence slope instability is managed by maintaining overall downstream slope angles of 2 to 1 (horizontal to vertical) or flatter. Monitoring of embankment pore pressures is to be used to determine the pore pressures within the embankment are similar to those defined in the analysis. The QPP pore pressure values are consistent with the piezometric conditions applied to the analysis of slope stability under static normal operating conditions. The recommended values for these QPPs are simply an indicator that pore pressure in the base of the embankment has increased greater than approximately 30 ft over current measured conditions. These levels would indicate that an assessment of the cause of the pore pressure rise is appropriate; however, no immediate slope stability concern is directly related to a pore pressure increase of this magnitude.

Table 3.9 Preliminary QPPs – Foundation and Slope Instability

Location	QPP	Value
YDTI Embankments	Downstream Overall Slope	No steeper than 2H:1V
	Minimum Crest Width	> 200 ft
	Water Levels: DH15-S5 VW1	< 5,800 ft
	Water Levels: DH15-S4 VW1	< 5,750 ft
	Water Level: DH15-S3	< 5,700 ft
	Water Level: MW94-11	< 5,700 ft

▲ R2

4 – OVERTOPPING

4.1 GENERAL

This section of the report summarizes the risk ratings for overtopping of the embankments. The assessment considers normal operating conditions, pipeline malfunctions, earthquake-induced deformation, and flooding. The assessment describes key design criteria that are essential to manage the risk of overtopping, which can be used as QPPs for future monitoring. Additional details supporting the risk ratings below in Table 4.1 are provided in the sections that follow.

Table 4.1 Risk Ratings for Overtopping

▲ R2

Loading Condition	Likelihood		Consequences ⁴
	Probability of Loading Conditions	Probability of Failure	
Normal Operating Conditions ¹	Certain ²	Not Credible	-
Pipeline Rupture	Likely	Not Credible	-
Earthquake Events	Very Rare ³	Very Low	Moderate to Major
Flood Events	Unlikely ³	Very Low	Catastrophic
	Very Rare ³	Very Low	Catastrophic

NOTES:

1. THE LOADING CONDITION IS DEFINED AS NORMAL OPERATING CONDITIONS WITHOUT ANY MALFUNCTIONS.
2. OVERTOPPING CONSIDERS THE AERIAL EXTENTS AND ELEVATION OF A NORMAL DESIGN SUPERNATANT POND VOLUME OF 25,000 ACRE-FT.
3. RATINGS FOR THE PROBABILITY OF LOADING CONDITIONS FOR EARTHQUAKE AND FLOOD EVENTS ARE BASED ON THE RETURN PERIOD OF THE PARTICULAR EVENT UNDER CONSIDERATION.
4. A CONSEQUENCE RATING IS NOT PROVIDED IF FAILURE IS NOT CREDIBLE FOR THE LOADING CONDITION UNDER CONSIDERATION.

4.2 NORMAL OPERATING CONDITIONS

4.2.1 Description of Failure Mode

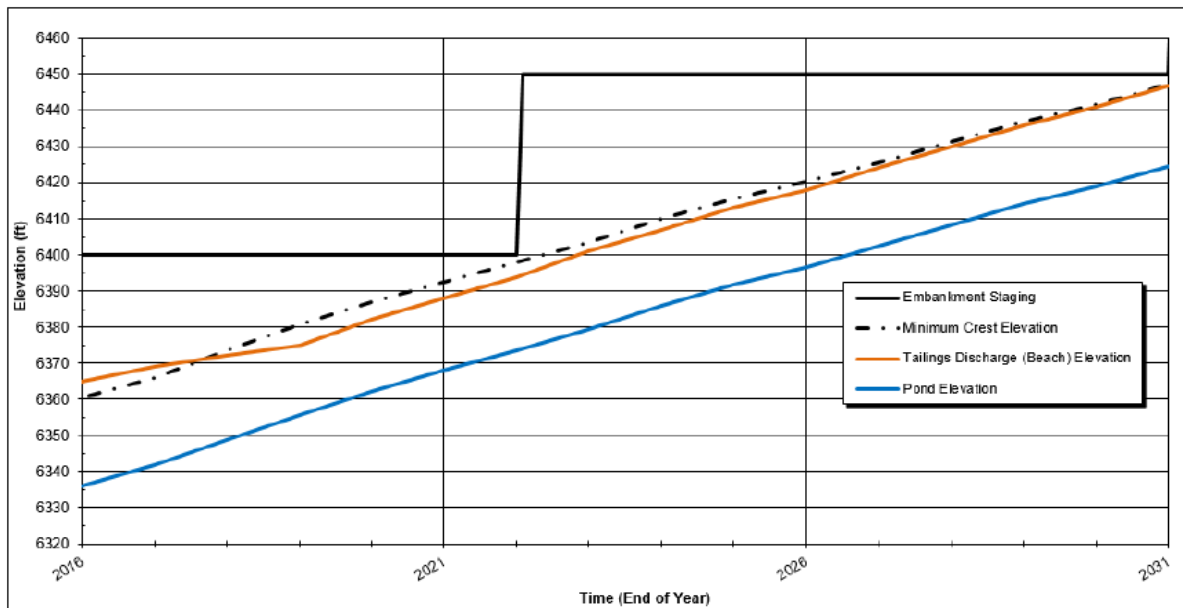
Overtopping considers the potential for supernatant pond to rise higher than the embankment crest leading to water or fluid tailings discharging over the embankment. A release of water over the top of the embankment has the potential to cause erosion of the embankment and breaching of the impoundment. The risk of overtopping is managed primarily through on-going embankment construction, the prescription of design freeboard, and appropriate water management practices to achieve a neutral water balance in the facility.

4.2.2 Method of Analysis and Summary of Results

The design of the YDTI, including timing for development of the staged lifts of the embankments was based on the filling schedule and the design freeboard requirements for the facility, which are described in the Design Basis Report (KP, 2017c). The filling curve for the facility and embankment

lift schedule is shown on Figure 4.1. The embankment lifting schedule is shown for simplicity as an instantaneous lift completed by 2022. The embankment crest elevation is maintained a minimum of 5 ft higher than the tailings discharge elevation. Embankment construction is completed in 50 ft high staged lifts, and therefore the total actual freeboard will tend to be larger than the design freeboard until just before operations cease. The estimated tailings discharge elevation and associated pond elevation for each year between 2016 and 2031 is shown on Figure 4.1. The rate of rise of the tailings will be approximately 6 ft per year, which is consistent with historical experience. The difference between the tailings discharge and supernatant pond elevation will typically be between 15 ft and 20 ft with a tailings beach developed at a slope less than approximately 1%.

▲ R2



▲ R2

NOTES:

- FIGURE ABOVE WAS REPRODUCED FROM FIGURE 4.3 OF THE DESIGN BASIS REPORT (KP, 2017c).

Figure 4.1 YDTI Filling Curve and Embankment Lift Schedule

4.2.3 Risk Rating

Normal operating conditions consider the maximum normal supernatant pond volume and extents, and excludes flood-induced loading, which will be evaluated in a subsequent section. The probability of the loading condition associated with the maximum normal pond volume is considered to be 'Certain' as defined in Table 2.1.

The design of the embankment under normal operating conditions provides an adequate design pond allowance and additional freeboard to eliminate the risk of overtopping. The probability of failure due to overtopping of the embankment under normal operating conditions meets the criteria for 'Not Credible' in Table 2.2.

In summary, the risk ratings for overtopping under normal operating conditions are as follows:

- Probability of Loading Condition: **Certain**
- Probability of Coincident Failure: **Not Credible**

4.3 FACTORS INFLUENCING THE ANALYSES

Overtopping under normal operating conditions has the potential to occur as a result of a prolonged malfunction of operational management plans for the impoundment. The potential malfunctions could include either mismanagement of freeboard through improper embankment construction and impoundment storage practices, or a rupture of the tailings distribution or reclaim water pipelines.

A major earthquake could induce deformation of an embankment, which could have the potential to lead to a loss of freeboard and overtopping.

Natural flooding could lead to a rise in the supernatant pond elevation and a corresponding loss of freeboard, and if freeboard is not sufficient to contain the flood it could result in overtopping of the embankment.

Embankment deformation under static conditions is another potential cause of overtopping if the embankment deforms in a manner that positions the embankment crest below the supernatant pond. Deformation of the embankment under static conditions due to foundation and slope instability was described in the previous section and is not repeated below.

The factors influencing the management of freeboard include:

- Embankment construction delays due to rockfill or equipment availability resulting in decreased freeboard
- Increased ore processing and tailings accumulation resulting in decreased available storage capacity, and
- Increased free water storage in the supernatant pond resulting in decreased available storage capacity.

These residual risks are managed through monitoring during operations to confirm that actual conditions are consistent with estimates made during the design process. Long range mine planning considers the need for rockfill for embankment construction during changes to the mine plan and will report on actual construction progress at the frequencies detailed in the Construction Management Plan (KP, 2017d).

Collection of the following monitoring data is necessary to evaluate the filling rate of the facility and should be reviewed at least annually by the EOR:

- Silver Lake make-up water volume (daily)
- Supernatant pond elevation (weekly)
- Rockfill placed in the embankment (monthly)
- Tailings discharge elevation (twice annually)
- Supernatant pond bathymetry (annually), and
- Mill throughput (annually).

The rupture of a tailings or reclaim pipeline could have the potential to cause embankment erosion due to the release of the pumped fluid or slurry, and could induce deformation of the embankment. The consequences of pipeline rupture are dependent on the location and size of the rupture, and the amount of time before repair or shut down of the damaged pipeline. The location and level of erosion at the time of discovery along with the pipeline flow rate will influence the potential consequences. The robustness of the embankment, substantial freeboard described previously, and the extensive drained tailings beaches make it very unlikely that damage to the embankment from a pipeline

rupture would cause embankment failure and uncontrolled release of contents of the YDTI. It is expected that some minor aesthetic damage would occur.

Pipeline pressure ratings are selected to withstand normal operating conditions and occasional surge conditions developed based on the design criteria of the system. Pipelines must be monitored to confirm they are operated within the design criteria by collecting regular flow meter readings to confirm design flow rate is not being exceeded and regular pressure gauge readings to confirm pipeline pressure rating is not exceeded. The risk ratings for pipeline rupture induced embankment deformation are as follows:

- Probability of Loading Condition: **Likely**
- Probability of Coincident Failure: **Not Credible**

Earthquake-induced deformations and natural flooding also have the potential to cause overtopping of the embankments. The evaluations of both loading conditions considered the maximum filling of the impoundment just prior to closure with the tailings surface adjacent to the embankment at 5 ft (minimum freeboard) below the crest. The facility only reaches this condition at the end of operations for a short period, and at other times there will be more freeboard to manage storm inflows or allow greater earthquake deformations without an impact on the safety of the facility.

The earthquake-induced overtopping failure mode was assessed by estimating the maximum earthquake-induced deformations along the critical slip surface and settlement of the crest at each embankment section (KP, 2018c). The yield acceleration required to reduce the FS to unity was determined for each section using post-earthquake material properties and maximum normal operating piezometric conditions. 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 seismic loading considered both the operating median Maximum Credible Earthquake (MCE) and post-closure 84th-percentile MCE. The MCE is a 'Very Rare' maximum credible event and deformations of the embankment would be expected during such a large seismic event given the proximity of the seismic sources.

▲ R2

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.

▲ R2

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. 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

▲ R2

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.

The deformation analysis examines an extreme scenario that includes the combined displacements relating to the following assumptions:

▲ R2

- 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.
- 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 (Swaigood, 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 Travarasrou, 2007).

The total tolerable displacement was reduced by the estimated potential fault displacement and crest settlement to estimate a tolerable displacement along the critical slip surface for each section. Displacements along the critical upstream and downstream slip surfaces were then estimated to check that predicted displacements were within the tolerable limits. The results of the earthquake-induced deformation estimates are summarized as follows:

▲ R2

- 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. 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.

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 probability of overtopping due to earthquake-induced deformation of the embankment meets the criteria for 'Very Low' in Table 2.2. In summary, the risk ratings for earthquake-induced embankment deformation are as follows:

- Probability of Loading Condition: **Very Rare**
- Probability of Coincident Failure: **Very Low**
- Consequences of Failure: **Moderate to Major**

Natural flooding can lead to a rise in the supernatant pond elevation and a corresponding loss of freeboard. The risk of flood-induced overtopping is managed by the prescription of design freeboard as described previously.

Analysis of the impact of flood events on the YDTI was undertaken using Muck 3D computer software to demonstrate the change in water level and pond extents compared to normal operating conditions. Muck 3D was used to model tailings beach development, as well as the location, extent and volume of the supernatant pond. Modelling was completed for two flood events, each considering a nominal operating pond volume of 25,000 acre-ft for normal operating conditions at the outset of flooding. The derivation of the storm events (KP, 2015a) considered in this analysis is included in Appendix B of the Design Basis Report (KP, 2017c). The two flood events considered were the Probable Maximum Flood (PMF), which is the design basis flood for the facility, and a lesser flood with a higher likelihood of occurrence. A return period of 1 in 1,000 years was selected for the latter event for the purpose of this risk assessment. The storm runoff volumes potentially generated by these two events are as follows:

- 1 in 1,000 Year 30 Day Rainfall – 6,500 acre-ft, and
- PMF Event – 20,000 acre-ft.

The resulting pond volume, changes to pond elevation and remaining available freeboard after flooding for each case are shown in Table 4.2. The anticipated extents of flooding are shown on Figure 4.2.

Table 4.2 Summary of the Effects of Natural Flooding

▲ R2

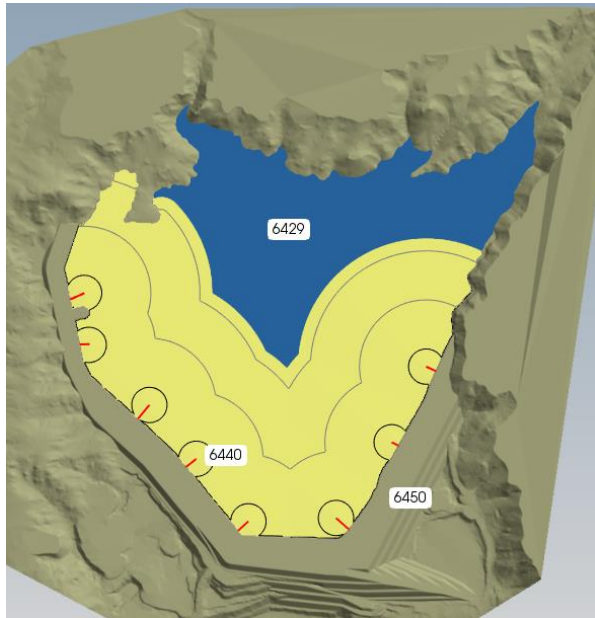
Loading Condition	Flood Effects on Supernatant Pond		Available Freeboard below Embankment Crest (ft)
	Total Stored Water Volume (acre-ft)	Increase in Pond Elevation (ft)	
Normal Operating Pond	25,000 ¹	Not Applicable	21 ft
1 in 1,000 Year 30 Day Rainfall	31,500 ²	7 ft	14 ft
PMF Event	45,000 ²	13 ft	6 ft

NOTES:

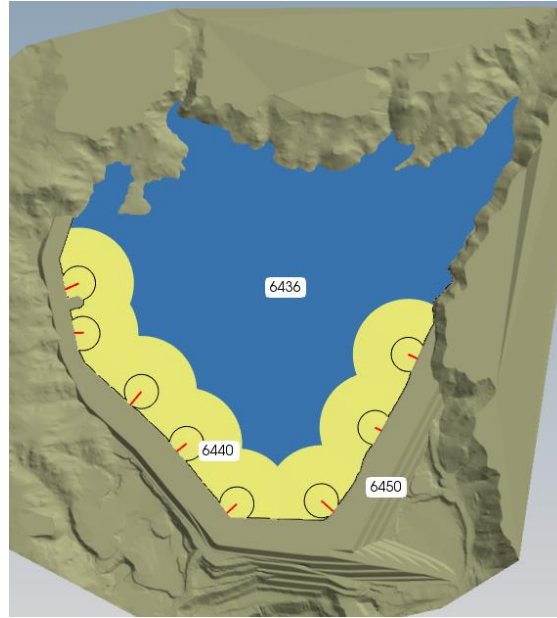
1. THE NORMAL OPERATING POND VOLUME FOR FUTURE OPERATIONS USED FOR THIS ANALYSIS WAS BASED ON THE NOMINAL POND ALLOWANCE OF 25,000 ACRE-FT AS DESCRIBED IN SECTION 4.3 OF THE DESIGN BASIS REPORT (KP, 2017c).
2. THE TOTAL VOLUME OF STORED WATER IS BASED A NOMINAL POND ALLOWANCE OF 25,000 ACRE-FT PLUS THE ESTIMATED STORM RUNOFF FROM THE FLOOD EVENT UNDER CONSIDERATION.

The results of the flood modelling indicate that under these flood conditions the pond will rise above normal operating level and remain below the embankment crest. Therefore, failure due to overtopping is 'Very Low'. In summary, the risk ratings for flood-induced overtopping of the embankments are as follows:

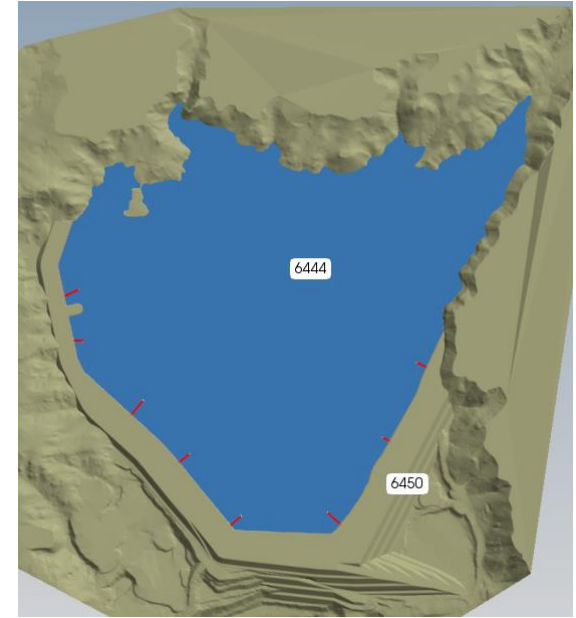
- Probability of Loading Condition: **Very Rare** for the PMF, and **Unlikely** for the 1 in 1,000 Year 30 Day Rainfall
- Probability of Coincident Failure: **Very Low**
- Consequences of Failure: **Catastrophic**



Normal Operating Conditions
Pond Volume - 25,000 acre-ft
Beach Length⁴ ~ 3,500 ft



1 in 1,000 Year 30-day Rainfall Event
Total Volume - 31,500 acre-ft
Beach Length⁴ ~ 1,500 ft



PMF Event
Total Volume - 45,000 acre-ft
Pond Against Embankment

LEGEND:

OPERATIONAL TAILINGS BEACH - ABOVE WATER

POND

NOTES:

1. ALL OUTPUTS ARE BASED ON MUCK 3D MODELLING OUTPUTS WITH 6450 ft CREST ELEVATION.
2. MODEL BASE USES JUNE 2016 TOPOGRAPHIC AND BATHYMETRIC SURVEY OF THE TAILINGS IMPOUNDMENT.
3. ALL CASES SHOW END OF FILLING TAILINGS DISCHARGE OF ELEVATION OF 6445 ft.
4. BEACH LENGTH IS APPROXIMATE MINIMUM LENGTH FROM ANY POINT ALONG THE EMBANKMENT

1	15AUG'17	ISSUED WITH REPORT	JRG	DDF
REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
DAM BREACH RISK ASSESSMENT MODELLING OF 25,000 ACRE-FT POND AND EXTENTS DUE TO FLOODING		
<i>Knight Piésold</i> CONSULTING	P/A NO. VA101-126/12	REF. NO. 3
	FIGURE 4.2	REV 1

4.4 QUANTITATIVE PERFORMANCE PARAMETERS

The risk of overtopping of the embankment is managed by following the embankment construction schedule established in the design. The suitability of the design to meet storage requirements in the future is evaluated using monitoring data periodically to confirm the performance of embankment construction and impoundment filling. The current acceptability of the facility to manage the risk of overtopping can be evaluated quickly using the preliminary QPPs defined in Table 4.3. The QPPs are designed to protect against overtopping during a severe flood event and embankment deformation during a severe earthquake event, and are therefore conservative for normal operating conditions.

Table 4.3 Preliminary QPPs – Overtopping

▲ R2

Location	QPP	Value
YDTI Embankment	Pond Elevation (below tailings discharge elevation) ¹	≥ 15 ft
	Minimum Freeboard (above tailings discharge elevation)	≥ 5 ft

NOTES:

1. THE TAILINGS DISCHARGE ELEVATION IS TYPICALLY PROGRESSIVELY RAISED AS TAILINGS ACCUMULATE. THIS DIFFERENCE IN ELEVATION RELATES THE ANTICIPATED ELEVATION DIFFERENCE BETWEEN THE SUPERNATANT POND AND TAILINGS DISCHARGE POINTS WITH EXTENSIVE TAILINGS BEACHES DEVELOPED IN BETWEEN, WHICH ALLOWS FOR STORAGE OF SPRING RUNOFF AND FLOOD INFLOWS ON THE TAILINGS BEACH WITHOUT APPROACHING THE EMBANKMENTS.

5 – INTERNAL EROSION AND PIPING

5.1 GENERAL

This section of the report summarizes the risk ratings for internal erosion and piping within the embankments. The assessment considers normal operating conditions, tailings beach development malfunctions, earthquake-induced cracking, and potential risks associated with flooding. The assessment describes key design criteria that are essential to manage the risk of internal erosion and piping, and describes opportunities to further limit the potential for this failure mode to develop. Additional details supporting the risk ratings below in Table 5.1 are provided in the sections that follow.

▲ R2

Table 5.1 Risk Ratings for Internal Erosion and Piping

Loading Condition	Likelihood		Consequences ⁴
	Probability of Loading Conditions	Probability of Failure	
Normal Operating Conditions ^{1, 2}	Certain	Not Credible	-
Tailings Stream Leakage	Likely	Moderate	Minor to Moderate
Earthquake Events	Very Rare	Very Low	Minor to Moderate
Flood Events ³	Unlikely	Low	Catastrophic
	Very Rare	Moderate	Catastrophic

▲ R2

NOTES:

1. THE LOADING CONDITION IS DEFINED AS NORMAL OPERATING CONDITIONS WITHOUT ANY MALFUNCTIONS.
2. INTERNAL EROSION AND PIPING CONSIDERS THE SUPERNATANT POND VOLUME AND EXTENTS FOR A NOMINAL OPERATING POND ALLOWANCE OF 25,000 ACRE-FT AS DESCRIBED IN SECTION 4.3 OF THE DESIGN BASIS REPORT (KP, 2017c).
3. FLOOD EVENT PROBABILITY OF LOADING CONDITION IS BASED ON THE RETURN PERIOD OF THE PARTICULAR EVENT UNDER CONSIDERATION.
4. A CONSEQUENCE RATING IS NOT PROVIDED IF FAILURE IS NOT CREDIBLE FOR THE LOADING CONDITION UNDER CONSIDERATION.

5.2 NORMAL OPERATING CONDITIONS

5.2.1 Description of Failure Mode

Internal erosion and piping has the potential to occur through an embankment or through the foundation of an embankment. Internal erosion typically develops progressively and is caused by seepage, starting from either the downstream or upstream side of the embankment or foundation. Piping can occur as the erosion develops to create a pathway for seepage. Fill materials can be carried with the water as the seepage path develops, which has the potential to gradually increase the erosion area. Potential breach mechanisms associated with internal erosion include gross

enlargement of the pipe hole, unravelling of the toe of the embankment slope, crest settlement leading to overtopping, and instability of the downstream slope.

The assessment of the potential for internal erosion and piping for the YDTI is complicated by the heterogeneity of the impoundment. Four conditions must typically exist for internal erosion and piping to occur (Fell et al., 2005):

1. There must be a seepage flow path and a source of water.
2. There must be erodible material within the flow path and this material must be carried by the seepage flow.
3. There must be an unprotected exit (open and unfiltered) from which the eroded material may escape.
4. The material directly above the material being eroded must be able to form and support the “roof” of the pipe.

Piping typically initiates by one of three processes: backward erosion, suffusion, or a concentrated leak. Backward erosion refers to initiation of erosion at the exit point of seepage and retrogressive erosion towards the source of water resulting in a continuous passage for seepage to travel along. Suffusion involves the washing out of fines from internally unstable soils, such as gap graded coarse sands and gravel. A concentrated leak involves the formation of a crack or passage directly from the source of water to an exit point and erosion initiates along the walls of the passage.

5.2.2 Method of Analysis and Summary of Results

The primary source of water for seepage is the supernatant pond, which is located at the north end of the YDTI. The elevation of the phreatic surface within the tailings beach generally decreases towards the embankment, and is typically highest where the tailings stream is actively recharging the water table due to vertical infiltration. The water table in the tailings adjacent to the embankment was measured at approximately 70 ft below the top of the tailings beach. Flow within the tailings beach is downwards and towards the embankment. Hydraulic conditions are generally less than hydrostatic. The pore pressures measured in the center of the embankment show saturated conditions at higher elevations than the downstream toe, but with a strong downward gradient. The presence of isolated perched wet zones in the embankment is more prominent with increasing proximity to the tailings impoundment. The measured data shows a low phreatic surface (about 30 ft above original ground) and hydrostatic conditions or small vertical gradients near the downstream toe of the embankment.

There are two potential unprotected exits from which eroded material could escape. The first is beyond the downstream slope of the embankment in natural soils and the second is within gap-graded zones of the rockfill that could form due to segregation during embankment construction. The process of backward erosion is relevant to the foundation and the embankment where a contrast in permeability exists and the exit point from lower permeability to higher permeability is unfiltered or inadequately filtered.

Initiation of backward erosion in the foundation would require static liquefaction (manifested as boiling or blowout) due to increased pore pressures beyond the downstream toe of the embankment. An upward hydraulic gradient of 0.3 ft/ft has been measured beyond the downstream toe of the embankment in a vibrating wire piezometer (VWP) installation (DH15-S1 VW2 and VW3) indicating, as expected, that the area downstream of the embankment is a seepage discharge zone. A critical gradient of 1 ft/ft is indicative of the potential for static liquefaction if no confining pressure is present.

The uppermost VWP is at a depth of 22 ft. The hydraulic gradient estimated at this point does not necessarily reflect the gradient at the seepage exit point, but it is the best available monitoring point for this condition and will be used to develop a QPP for head differential between the VWPs. The seepage path from natural soils downstream of the embankment to the tailings impoundment is in excess of 3,000 ft with an additional 5,000 ft or more distance to the supernatant pond. Even if static liquefaction (boiling) were to occur locally downstream of the drained embankment, it would not indicate imminent failure due to piping. A QPP for on-going monitoring of pore pressure conditions at the downstream toe of the embankment is discussed below.

Backward erosion in an embankment has the potential to initiate at the downstream side of a core zone in a zoned embankment as a result of the pore pressures exerted on the core and the resulting hydraulic gradient across the core zone. The YDTI embankments do not include a core zone and engineered filters, and the embankment behaves as a large drain that limits the development of saturated zones with elevated pore pressures. The contrast in permeability for the YDTI embankments occurs at the interface between the tailings and the rockfill embankment. This interface is typically covered with a thick separation zone of dumped earthfill (alluvium from pit stripping) to limit the potential for tailings migration into the rockfill. The direction of flow within the tailings is downward and towards the embankment. Localized and minor suffusion could occur separately or coincident with backward erosion at this interface, but only if pore pressures increased substantially and void space was present in the rockfill to allow erosion to occur.

There is no evidence presently of any seepage pathways from the supernatant pond mobilizing erodible material at the downstream exit point of the seepage. Seepage is monitored primarily at Horseshoe Bend (HsB) and the Number 10 Seep. The TOMS Manual (MR, 2016) contains response procedures for new seeps or increases in turbidity of the seepage water.

Concentrated leaking is only relevant to the embankments of the YDTI, and has the potential to occur in the gap-graded zones of the rockfill if a source of water is allowed to pond adjacent to the embankment. The supernatant pond is managed in a position that is remote from the embankments and separated by wide tailings beaches, which are essential to the management of risk associated with internal erosion and piping. Concentrated leaking of the embankment is not possible with the large separation from the pond and the embankment acting as a drain.

5.2.3 Risk Rating

Normal operating conditions consider the maximum normal supernatant pond volume and extents, excluding flood-induced loading or tailings beach mismanagement. The probability of the loading condition associated with the maximum normal pond volume is considered to be 'Certain' as defined in Table 2.1.

The primary source of water for seepage is the supernatant pond, which is located in the north end of the YDTI and is separated from the embankment by large tailings beaches. The pore pressures measured in the tailings adjacent to the embankment show downward gradients, and the embankment behaves as a large drain for the pore water in the tailings mass. The pore pressures in the embankment exhibit strong downward gradients with a low phreatic surface along the base of the embankment and as observed near the embankment toe. The probability of failure due to internal erosion and piping of the embankment under normal operating conditions meets the criteria for 'Not

Credible' in Table 2.2. In summary, the risk ratings for internal erosion and piping under normal operating conditions are as follows:

- Probability of Loading Condition: **Certain**
- Probability of Coincident Failure: **Not Credible**

The risk of internal erosion and piping will increase if the supernatant pond or tailings stream is allowed to approach one of the embankments.

5.3 FACTORS INFLUENCING THE ANALYSIS

Internal erosion and piping under normal operating conditions would require a malfunction of operational management plans for the impoundment. The potential malfunctions that could increase the risk of internal erosion and piping include improper beach development, excessive water storage, and concentrated leaking associated with the tailings stream flowing along the intersection of the tailings beach with the upstream face of the embankment. Seismic loading can induce longitudinal and transverse cracking, which could initiate internal erosion if connected to a source of water. Natural flooding will also increase the potential for internal erosion and will be discussed in this section.

▲ R2

Extensive tailings beaches are present that separate the supernatant pond from the embankments. The tailings beaches limit the potential for internal erosion and piping by controlling the source of water and seepage flow path. The beaches work in conjunction with the free draining embankments to limit pore pressures at the interface between the tailings and embankment materials, and eliminate any extensive zones of saturation from developing within the embankment rockfill. Piping cannot develop without a continuous source of water eroding material along a seepage flow path.

Five concentrated leakage events associated with the tailings stream flowing along or ponding against the upstream face of the embankment occurred between November 2015 and April 2016. These events were reported in the 2015 and 2016 Annual Inspection Reports (KP, 2015b and KP, 2017b). The filling level of the YDTI in these areas during this period coincides with the elevation of a segregated coarse boulder layer, which is created when placing rockfill in 50 ft lifts. The leaking events were observed by routine visual inspections and action was taken to eliminate leaking within 24 hours. This was achieved by placing more alluvium on the upstream side of the embankment and by adjusting the tailings discharge pipes to move the active tailings flow away from the embankment. Tailings leakage quickly ceased as the tailings flow in the impoundment moved away from the embankment. The damage caused by of the tailings leak events consisted of minor aesthetic erosion at the bench slope below the leak elevation caused by the exiting flows and deposition of tailings along embankment bench below. Alluvium facing was thickened along the North-South Embankment as a mitigation measure during and subsequent to these events.

The consequences of concentrated leakage caused by the tailings stream has been 'Minor' in the past, but conceivably could become 'Moderate' if left unchecked for a longer period of time. The events experienced over the past year coincided with the tailings filling level approaching the base of a 50 ft lift of the embankment. Although a series of leakage events were concentrated over the past year, the historical rate of occurrence is reported to be much less often in personal correspondence with MR. The tailings facility fills at a rate of roughly 6 ft per year. The base of the next embankment lift will be encountered in approximately 8 to 9 years at the North-South and East-West Embankments. Therefore, a probability of the loading condition occurring has been assessed as

'Likely' for the risk assessment. The probability of coincident failure has been assessed as 'Moderate' because minor failures have occurred before and the failure mode cannot be analyzed in practical manner.

In summary, the risk ratings for concentrated leakage associated with the tailings stream are as follows:

- Probability of Loading Condition: **Likely**
- Probability of Coincident Failure: **Moderate**
- Consequences of Failure: **Minor to Moderate**

The tailings leakage events have occurred because the tailings stream flows along the interface between the tailings beach and the embankment. The leakage occurs when the tailing stream becomes ponded in a localized low area adjacent to a coarse layer. This provides a source of water and seepage flow path, and an unprotected exit from which material can escape. The consequences have the potential to become worse than previously experienced if erodible material from the embankment is carried with the flow and if the material supports enlargement of the passage.

The long-term development of tailings beaches will be achieved using a discharge configuration that is progressively expanded from a single discharge point to multiple discharge points. Initially, additional discharge locations will be constructed along the West Embankment to allow tailings to fill in the low areas (below the current supernatant pond) in the northwest end of the impoundment, and create a tailings beach adjacent to the West Embankment. Development of the tailings beach along the West Embankment will push the pond towards the North-South Embankment. Tailings will also be discharged from the North-South Embankment to prevent the supernatant pond from approaching the embankment. It is essential to discharge from both sides of the facility concurrently to develop the tailings beaches appropriately.

Tailings beaches must be present at all times adjacent to the embankments to limit the potential for internal erosion. The tailings discharge should be directed towards the center of the facility. Tailings flow along the embankment interface should be limited as much as practical and monitored regularly when it does occur. These actions will reduce the risk of concentrated leakage caused by the tailings stream.

There is the potential for a secondary failure mode from seismic loading even if overtopping or slope instability does not occur. Earthquake deformation can cause longitudinal and transverse cracking, which can lead to an initiation of internal erosion if a source of water is present. Cracking typically occurs in the crest and upper portions of the embankment as a result of strong seismic loading. Longitudinal cracking would likely accompany any crest settlement or slip surface movement resulting from seismic loading. Visible longitudinal cracks typically occur when earth and rockfill embankments are subjected to strong earthquake ground motions approximately Magnitude 6.5 or greater, and a PGA greater than around 0.3 g. Embankments that experience relative crest settlement in excess of 0.2% are highly likely to experience transverse cracking (Fell et al., 2005). Transverse cracking would provide a potential pathway for internal erosion to develop if connected to a source of water that could further erode the pathway.

▲ R2

Seismic loading for the YDTI considers the operating median MCE and post-closure 84th-percentile MCE with a PGA of 0.45 g and 0.84 g, respectively, and a Magnitude of 6.5 (KP, 2017c). The seismic loading is from 'Very Rare' maximum credible events. The relative crest settlement was

▲ R2

estimated to be 0.2% for the median MCE and 2.2% for the 84th-percentile MCE to determine the estimated crest settlement in Section 4.3 following the methodology described in Swaisgood, 2014. Comparing the seismic loading at the YDTI and estimated crest settlement with a damage classification system for embankment cracking due to seismic loading (Fell et al., 2005; Fell et al., 2008) allows an empirical estimate of the probability of transverse cracking and maximum estimated crack width. The seismic loading and estimated relative crest settlement indicate that the damage classification based on this system could be 'minor to moderate' for the median MCE and 'major to severe' for the 84th-percentile MCE. The damage classification system is shown on Table 5.2.

Table 5.2 Damage Classification System for Embankments Under Earthquake Loading

Damage Class		Maximum Longitudinal Crack Width (mm)	Maximum Relative Crest Settlement (%)	Probability of Transverse Cracking	Maximum Likely Transverse Crack Width at the Crest (mm)
Number	Description				
0	None or Slight	< 10	< 0.03	0.001 to 0.01	5 to 20
1	Minor	10 to 30	0.03 to 0.2	0.01 to 0.05	20 to 50
2	Moderate	30 to 80	0.2 to 0.5	0.05 to 0.10	50 to 75
3	Major	80 to 150	0.5 to 1.5	0.2 to 0.25	100 to 125
4	Severe	150 to 500	1.5 to 5	0.5 to 0.6	150 to 175

▲ R2

NOTES:

1. DAMAGE CLASSIFICATION SYSTEM MODIFIED FROM TABLE 12.4 OF FELL ET AL., 2005 TO INCLUDE ADDITIONAL COLUMNS FROM FELL ET AL., 2008.

The estimated maximum transverse crack width ranges from 20 to 75 mm for operating conditions and up to 175 mm for closure conditions. The key protection against internal erosion due to transverse cracking is the 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. The likelihood of failure from internal erosion due to transverse cracking of the embankment initiated by seismic loading is assessed as 'Very Low' due to the presence of the large drained tailings beaches. The tailings may have sufficient moisture content to flow in a viscous manner during active seismic loading without confinement (Appendix A). If a transverse crack were to open up during active shaking, viscous tailings may flow like mud into the crack and would be expected to cease flowing without active shaking. The damage would be largely aesthetic and viscous tailings flow would be confined to the area of cracking within the embankment, if it occurred at all. The potential consequences are classified as 'Minor' to 'Moderate'.

▲ R2

In summary, the risk ratings for internal erosion resulting from embankment cracking due to seismic loading are as follows:

- Probability of Loading Condition: **Very Rare**
- Probability of Coincident Failure: **Very Low**
- Consequences of Failure: **Minor to Moderate**

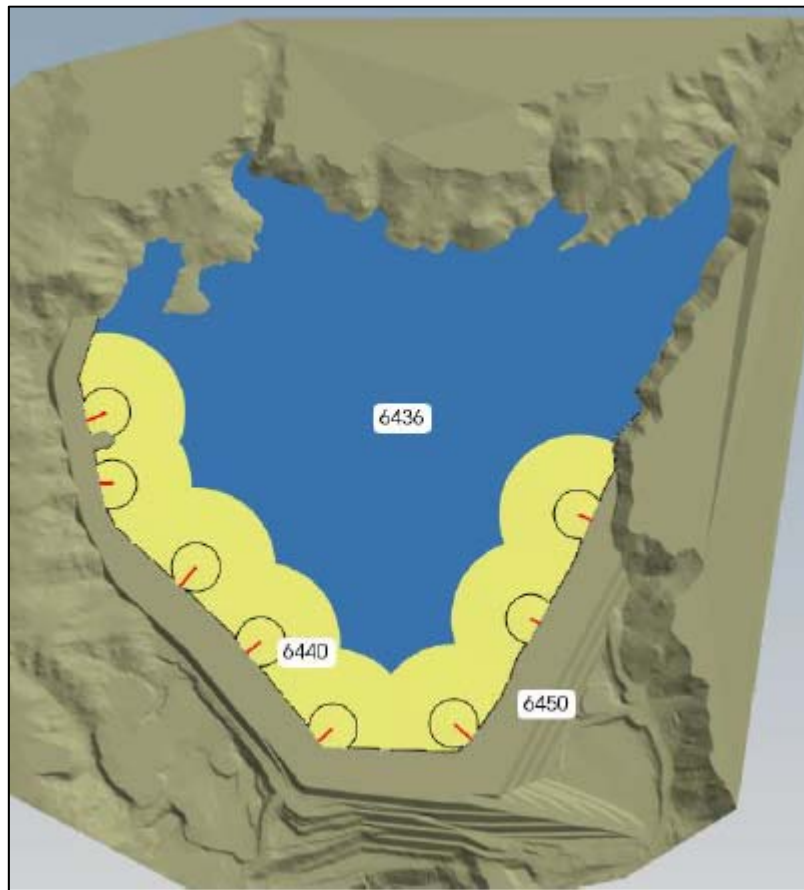
▲ R2

Proper tailings beach development will allow the supernatant pond to continue to be managed in a position that is remote from the embankments. The volume of water contained in the supernatant pond impacts the length of subaerial tailings beach. The pond surface area was increased between 2013 and 2015 to promote inundation and saturation of the tailings beach surface to manage wind-blown tailings generation in line with objectives of the DEQ Montana Air Quality Permit. However, to achieve the geotechnical objectives for beach development, enhancing embankment stability and limiting the potential for internal erosion, the practice of inundation of tailings beaches with water to manage wind-blown dusting will be phased out. The potential for tailings dusting will be managed through use of the multiple discharge points or by other means to wet the beach by recycling water within the mine area during critical periods.

The analysis of the potential for overtopping caused by flood events was described in Section 4.3. This section builds on the previous discussion and describes the potential impact of flooding on the risk of internal erosion and piping. Modelling of two potential flood events was completed to demonstrate the change in water level and pond extents compared to normal operating conditions. Each scenario considered a pond volume of 25,000 acre-ft for normal operating conditions at the outset of flooding. The storm runoff volumes added to the normal operating pond volume were 6,500 acre-ft for a wet month with a return period of 1 in 1,000 years and 20,000 acre-ft for the PMF. The pond extents for these flooded conditions were shown on Figure 4.2, and are reproduced as Figures 5.1 and 5.2 below.

The 1 in 1,000 year return period wet month with a storm runoff of 6,500 acre-ft would increase the pond level by approximately 7 ft and would be expected to flood a substantial portion of the lower elevation subaerial beaches as shown on Figure 5.1. A flood event of this magnitude meets the criteria for 'Unlikely' on Table 2.1. The supernatant pond in this scenario would remain separated from the embankments, which limits the potential for development of concentrated leakage. Seepage flows within the embankment could be expected to increase with corresponding increases in pore pressures in areas of the embankment that do not drain as effectively. There is some uncertainty in key starting conditions, particularly how the beaches will develop and what the operating volume of the supernatant pond would be at the outset of flooding. It is unknown whether flooding of this magnitude would even occur at all. The analysis meets the criteria for 'Low' in Table 2.2 since the analysis technique is not robust and there is some uncertainty of the sensitivity of key inputs to the analysis. The risk rating for internal erosion and piping caused by a flood with a return period of 1 in 1,000 years are as follows:

- Probability of Loading Condition: **Unlikely**
- Probability of Coincident Failure: **Low**
- Consequences of Failure: **Minor to Catastrophic**



▲ R2

Figure 5.1 Modelled Pond Extents for 1 in 1,000 Year Event

The PMF event with a storm runoff of 20,000 acre-ft would increase the pond level by approximately 15 ft and would be expected to flood all or most of the subaerial tailings beaches as shown on Figure 5.2. A flood event of this magnitude meets the criteria for 'Very Rare' in Table 2.1. This deterministic maximum credible flood event is in excess of three times the size of the storm runoff associated with the 1 in 1,000 year wet month discussed previously. The supernatant pond would not be isolated from the embankment under these conditions. The alluvium facing, wide crest width, and the well graded particle size distribution of most of the embankment fill would provide some protection against internal erosion in many areas of the embankment. However, the ponding of water adjacent to the embankment opens up a pathway to a very large source of water that has the potential to cause concentrated leakage through gap-graded rockfill zones or coarse boulder layers within the embankment. These zones are known to exist and have produced conduits for concentrated leaks caused by the active meandering tailings stream. An analysis of internal erosion and piping potential under these conditions cannot be analyzed in practical manner, and therefore meets the criteria for 'Moderate' in Table 2.2. The risk rating for internal erosion and piping cause by the PMF is as follows:

- Probability of Loading Condition: **Very Rare**
- Probability of Coincident Failure: **Moderate**
- Consequences of Failure: **Minor to Catastrophic**



▲ R2

Figure 5.2 Modelled Pond Extents for PMF Event

The flood events considered in this risk assessment are rare, but they highlight the importance of water management and tailings beach development to limit risk. The risk of internal erosion and piping of an embankment at the YDTI will be mitigated by keeping the pond separated from the embankments and by constructing free draining embankments. This concept, which has been successful for many decades, can be applied to the future development of the facility to mitigate the potential for internal erosion and piping under flooded conditions.

An analysis was undertaken to determine the maximum water storage volume that can be contained on the tailings beaches without water ponding against the embankment. A pond volume of 33,000 acre-ft can be contained on this beach arrangement without water reaching the embankment. Subtracting the storm runoff volume of the PMF of 20,000 acre-ft from this maximum water storage volume indicates that a normal operating pond volume of 13,000 acre-ft would limit the potential for water reaching the embankment under even the most severe flood event. Past operating experience tends to indicate that the minimum practical volume for the YDTI supernatant pond is approximately 15,000 acre-ft in order to promote adequate particle settling and maintain appropriate reclaim water supply and clarity to avoid compromising mill operations. The 1 in 1,000 year return period flood with a starting pond volume of 15,000 acre-ft would be less than the normal operating condition previously analyzed in this report as shown on Figure 5.3. There is a possibility of water reaching the embankment in the PMF as shown on Figure 5.3, but the likelihood is substantially reduced with a smaller normal operating pond. Reducing the normal operating pond volume, and developing

▲ R2

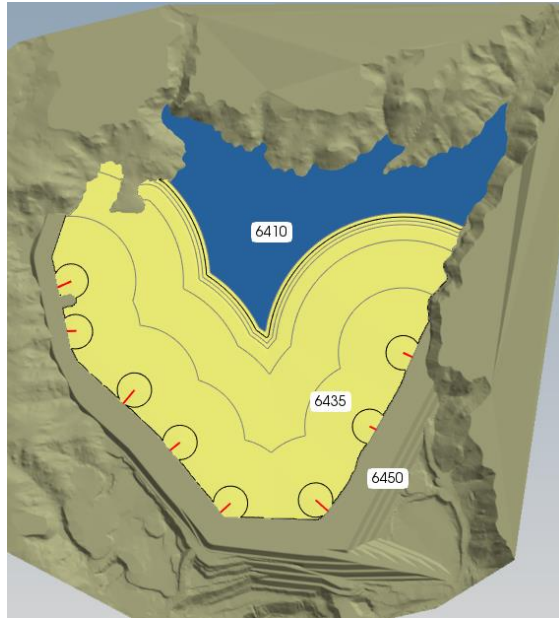
extensive and relatively uniform tailings beaches around the facility will decrease the potential for internal erosion and piping under flooded conditions for the YDTI. The revised risk ratings for flood event conditions with a maximum normal operating pond volume of 15,000 acre-ft are included in Table 5.3.

Table 5.3 Flood Event Risk Ratings with Reduced Normal Operating Pond Volume

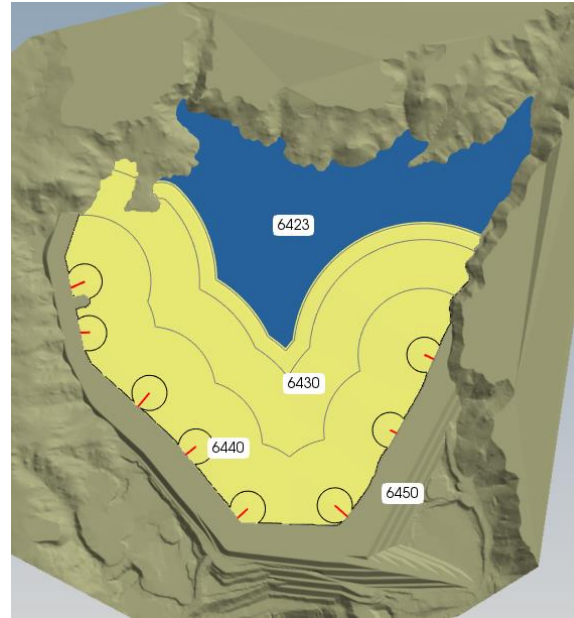
Loading Condition	Likelihood		Consequences ^{3, 4}
	Probability of Loading Conditions	Probability of Failure ³	
Flood Events	Unlikely ¹	Low	Catastrophic
Original Risk Ratings	Very Rare ²	Moderate	Catastrophic
Flood Events	Unlikely ¹	Not Credible	-
Revised Risk Ratings ³	Very Rare ²	Low	Catastrophic

NOTES:

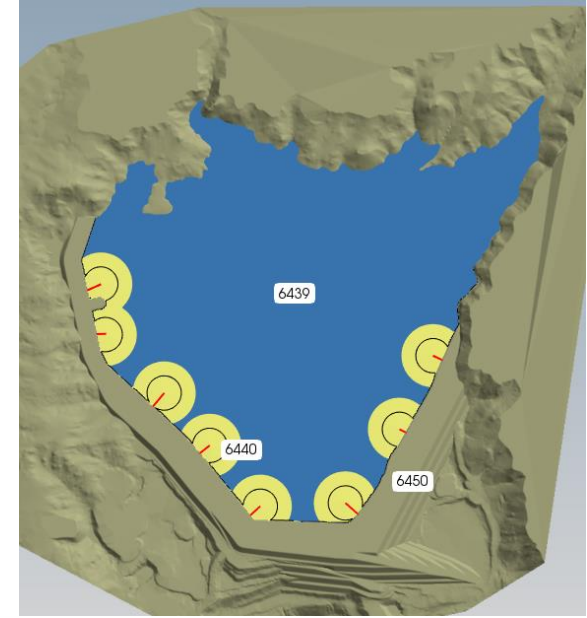
1. THE 1 IN 1,000 YEAR RETURN PERIOD FLOOD EVENT MEETS THE CRITERIA FOR 'UNLIKELY' IN TABLE 2.1.
2. THE PMF EVENT MEETS THE CRITERIA FOR 'VERY RARE' IN TABLE 2.1.
3. THE PROBABILITY OF FAILURE AND THE CONSEQUENCES OF FAILURE WERE REVISED ASSUMING THE NORMAL OPERATING POND VOLUME IS REDUCED OVER TIME TO 15,000 ACRE-FT OR LESS.
4. A CONSEQUENCE RATING IS NOT PROVIDED IF FAILURE IS NOT CREDIBLE FOR THE LOADING CONDITION UNDER CONSIDERATION.



Normal Operating Conditions
Pond Volume - 15,000 acre-ft
Beach Length⁴ ~ 3,900 ft



1 in 1,000 Year 30-day Rainfall Event
Total Volume - 21,500 acre-ft
Beach Length⁴ ~ 3,700 ft



PMF Event
Total Volume - 35,000 acre-ft
Localized Pond Against Embankment

LEGEND:

OPERATIONAL TAILINGS BEACH - ABOVE WATER

POND

NOTES:

1. ALL OUTPUTS ARE BASED ON MUCK 3D MODELLING OUTPUTS WITH 6450 ft CREST ELEVATION.
2. MODEL BASE USES JUNE 2016 TOPOGRAPHIC AND BATHYMETRIC SURVEY OF THE TAILINGS IMPOUNDMENT.
3. ALL CASES SHOW END OF FILLING TAILINGS DISCHARGE OF ELEVATION OF 6445 ft.
4. BEACH LENGTH IS APPROXIMATE MINIMUM LENGTH FROM ANY POINT ALONG THE EMBANKMENT

1	15AUG'17	ISSUED WITH REPORT	JRG	DDF
REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
DAM BREACH RISK ASSESSMENT MODELLING OF 15,000 ACRE-FT POND AND EXTENTS DUE TO FLOODING		
<i>Knight Piésold</i> CONSULTING	P/A NO. VA101-126/12	REF. NO. 3
	FIGURE 5.3	
		REV 1

Water balance modelling for the project has considered a target supernatant pond volume of 15,000 acre-ft to evaluate the potential to reduce water storage within the YDTI (KP, 2018a). The water available for reclaim is insufficient to support reclaim water demands for mill processing, and therefore make-up water is required from an outside source. The make-up water needs in each year varies with prevailing climate conditions, and has historically averaged approximately 2.8 million gallons per day (MGPD). Freshwater is required at the Concentrator to support processing, and typically between 1 and 3 MGPD is brought in from Silver Lake for this purpose.

The supernatant pond volume can be incrementally reduced to the target of 15,000 acre-ft during operations with a freshwater make-up rate maintained on an annual basis at an average below 2 MGPD. If this freshwater import rate does not support Concentrator operations, then alternative water reduction and/or water removal opportunities can be evaluated to produce similar reductions and the water management plan will be updated accordingly. Reducing the normal operating pond volume will progressively and incrementally lower the potential for internal erosion and piping under flooded conditions throughout operations. A fresh water import rate above 3 MGPD, without other water removal activities, would have the opposite effect on the potential for internal erosion and piping during flood events because this import rate is likely to increase the normal operating pond volume. A QPP related to this is described in Section 5.4 below, and a description of the observational method plan for managing residual risk is provided in Section 6.

The pond extents under flooded conditions during closure can be managed through construction of a closure spillway that passively limits the maximum pond elevation in the facility thereby eliminating the potential for piping in the long-term after closure. The design of the closure spillway to meet these objectives is described in the Reclamation Overview Report (KP, 2018b).

5.4 QUANTITATIVE PERFORMANCE PARAMETERS

The risk of internal erosion and piping is managed through freeboard criteria, tailings beach development criteria, and monitoring of pore pressure conditions. Extensive tailings beaches must be present to separate the supernatant pond from the embankments. The tailings beaches limit the potential for internal erosion and piping by controlling the source of water and seepage flow path. The beaches work in conjunction with the free draining embankments to limit pore pressures at the interface between the tailings and embankment materials, and eliminate any substantial phreatic surface from developing in the embankment. Piping cannot develop without a continuous source of water eroding material along a seepage flow path. The current acceptability of the YDTI to manage the risk associated with internal erosion and piping can be evaluated quickly using the preliminary QPPs included in Table 5.4.

Additionally, reducing the normal operating pond volume will decrease the potential for internal erosion and piping under flooded conditions for the YDTI and will further enhance the safety of the facility under normal operating conditions. A preliminary QPP for freshwater import is included in Table 5.4. This QPP is included to provide a quick understanding of the trajectory of the pond water balance. An average monthly freshwater import flowrate less than 2 MGPD indicates that the stored operating pond volume will decrease over time, which will further enhance the safety of the facility.

Table 5.4 Preliminary QPPs – Internal Erosion and Piping

Location	QPP	Value
YDTI Embankment	Differential Head: DH15-S1 VW2 and VW3	$\Delta H = VW2 - VW3 \leq 14 \text{ ft}$
	Differential Head: DH15-S2 VW2 and VW3	$\Delta H = VW2 - VW3 \leq 20 \text{ ft}$
YDTI Beach	Minimum Beach Length ²	No ponded water within 200 ft of the embankment
Freshwater import	Monthly average flowrate	< 2 MGD

▲ R2

NOTES:

1. THE TAILINGS DISCHARGE ELEVATION IS TYPICALLY PROGRESSIVELY RAISED AS TAILINGS ACCUMULATES. THIS DIFFERENCE IN ELEVATION RELATES THE ANTICIPATED ELEVATION DIFFERENCE BETWEEN THE SUPERNATANT POND AND TAILINGS DISCHARGE POINTS WITH EXTENSIVE TAILINGS BEACHES DEVELOPED IN BETWEEN, WHICH ALLOWS FOR STORAGE OF FRESHET AND FLOOD INFLOWS ON THE TAILINGS BEACH WITHOUT APPROACHING THE EMBANKMENTS.
2. THE MINIMUM BEACH LENGTH OF 200 FT ALLOWS TIME TO RESPOND AND MITIGATE WATER APPROACHING THE EMBANKMENT THROUGH MODIFACTION OF THE TAILINGS DISCHARGE POINTS OR PLACEMENT OF ADDITIONAL ALLUVIUM.

6 – OBSERVATIONAL METHOD PLAN FOR MANAGING RESIDUAL RISK

There were two methods for coping with uncertainties in applied soil mechanics prior to the formal recognition of the observational method. These two methods were to either adopt an excessive factor of safety or to make assumptions in accordance with general, average experience (Peck, R.B., 1969). An observational method of design and construction requires that gaps in information can be filled by observations, and foreseeable deviations from expected conditions have devised solutions that can be implemented. The failure modes analyzed in this risk assessment are fundamentally affected by the consistency and quality of the free-draining embankment fill, and the pore pressures within the tailings beach and embankment.

The embankments have been constructed almost continuously since the early in 1960s from rockfill that is highly variable in grain size distribution, geologic alteration, clast strength, and subsequent degradation since its initial placement. The material encountered in drilling investigations consisted of varying mixtures of gravel, cobbles and boulders within a silty clayey sand matrix. No distinguishing characteristics are apparent near the interface of the Berkeley and Continental rockfill zones in the embankment (KP, 2017a). The records documenting the historic embankment fill material consistency are sparse due in part to the state of practice at the time of construction and also to ownership changes in the 1980s. Available historic construction records have been compiled (KP, 2017a) and are indicative of what could generally be expected. The YDTI embankments at a crest elevation of 6,450 ft will range up to roughly 800 ft high in the maximum section and will be over 3 miles long. It is impractical to widely observe or characterize the existing embankment materials at this time, and therefore the observational method is inapplicable to this aspect of the design. Therefore, the design has been based on providing adequate factors of safety for the least-favorable conditions compatible with the available data.

▲ R2

The YDTI has been developed during the last few decades using a single tailings discharge point. An extensive drained tailings beach has developed, partly due to the shape of the valley and the position of the supernatant pond during that period. Property boundaries, topography, and groundwater conditions along the west side of the YDTI necessitated construction of the West Embankment. A single discharge point is no longer the most appropriate means of developing tailings beaches for the whole impoundment. During continued mining, tailings will be deposited at multiple locations to develop extensive drained tailings beaches adjacent to all the embankments. Development of the tailings beach along the West Embankment will push the pond towards the North-South Embankment. Tailings will also be discharged from the North-South Embankment to prevent the supernatant pond from contacting the embankment.

Internal erosion and piping of the embankment under normal operating conditions was determined to not be a credible failure mode because the supernatant pond is separated from the embankment by large tailings beaches. The beaches work in conjunction with the free draining embankments to limit pore pressures at the interface between the tailings and embankment materials, and eliminate any substantial saturated zones from developing in the embankment. The risk of internal erosion and piping will increase if the supernatant pond or tailings stream is allowed to approach one of the embankments due to improper beach development or natural flooding. The potential for internal erosion and piping initiated by natural flooding carries the greatest uncertainty for the YDTI. Reducing the normal operating pond volume or improving the uniformity of tailings beach

development will increase the storm storage that can be contained on the tailings beach without the pond perimeter reaching the embankment.

There is an opportunity to utilize the observational method while considering the development of pore pressures within the tailings beach and embankment. This opportunity is particularly relevant now due to the change in approach of tailings beach development, and the importance of the tailings beaches to limit pore pressure development. Tailings beach development and long-term prediction of the normal operating pond volumes are estimated using models that are primarily based on past performance of the facility. There is some uncertainty in any model, and the predictions must be verified by observations. The analysis of future conditions for the potential of internal erosion and piping uses the most-probable conditions rather than the most-unfavorable. The foreseeable deviations from expected conditions are included in Table 6.1. The planned observational monitoring and analytical actions associated with these deviations are included along with potential alternative solutions.

Table 6.1 Additional Observational Monitoring to Manage Residual Risk

Foreseeable Deviation	Planned Observational Monitoring or Analytical Action	Potential Alternative Solution
Freshwater import rate of 2 MGPD or less is not supportive of Concentrator operations.	Prepare water balance of freshwater process requirements, and highlight areas that can be optimized to reduce use.	Accept higher freshwater import rate, and determine alternative strategies to reduce volume of stored supernatant water.
Development of extensive tailings beaches adjacent to the entire embankment does not occur effectively as planned.	Observe beach development as per requirements of the TOMS Manual. Compare with modelled predictions.	Alter discharge locations to protect critical areas, increase number of discharge locations, decrease energy at outfall to improve beach development.
Pore pressure development increases in embankment fill.	Compare against QPP values and increase monitoring frequency.	Install additional pore pressure monitoring instruments to determine extents of pore pressure development. Evaluate need for vertical embankment drains.
Supernatant pond or tailings stream approaches within 200 ft of the embankment	Observe beach development, tailings stream, and supernatant pond position	Place additional alluvium on embankment face to provide additional protection. Adjust tailings discharge location to move tailings stream away or to develop beach.
The supernatant pond volume does not trend downwards towards the target of 15,000 acre-ft.	Monitor freshwater import rate or other alternative strategy for water reduction. Review annual pond bathymetry.	Improve beach uniformity by adding additional tailings discharge locations and rotating active discharge areas more frequently.
A closure spillway in the long-term after closure is eliminated from the closure plan.	Periodic review of the closure plan.	Adjust reclamation plan for the YDTI to store runoff from the PMF event without the pond reaching the embankment.

7 – SUMMARY OF RISK ANALYSIS

This assessment presented an examination of foundation and embankment instability, overtopping, and internal erosion and piping. The assessment considered loading during maximum normal operating conditions, and additional loading from seismic events, flood events, and malfunctions of the reclaim water and tailings distribution systems. The risk ratings for the failure modes and loading conditions analyzed are summarized, irrespective of embankment location, in Table 7.1. Descriptions of the qualitative likelihood and consequence ratings were provided in Section 2.

Table 7.1 Summary of Risk Ratings for All Failure Modes

▲ R2

Failure Mode	Loading Condition	Likelihood		Consequences
		Probability of Loading Conditions	Probability of Failure	
Foundation and Slope Instability	Normal Operating Conditions	Likely	Very Low	Moderate
	Earthquake Events	Very Rare	Very Low	Moderate to Major
	Flood Events	Very Rare	Very Low	Moderate to Catastrophic
Overtopping	Normal Operating Conditions	Certain	Not Credible	-
	Pipeline Rupture	Likely	Not Credible	-
	Earthquake Events	Very Rare	Very Low	Moderate to Major
	Flood Events	Unlikely	Very Low	Catastrophic
		Very Rare	Very Low	Catastrophic
Internal Erosion and Piping	Normal Operating Conditions	Certain	Not Credible	-
	Tailings Stream Leakage	Likely	Moderate	Minor to Moderate
	Earthquake Events	Very Rare	Very Low	Minor to Moderate
	Flood Events	Unlikely	Low to Very Low	Minor to Catastrophic
		Very Rare	Moderate to Low	Minor to Catastrophic

The likelihood of embankment failure and uncontrolled loss of tailings due to foundation and slope instability under static conditions is very low. The analyses considered conservative assumptions related to material properties and pore pressure conditions. The assessment also included relevant analyses to demonstrate the sensitivity of the predicted factors of safety to key factors affecting the slope stability of the embankment including changes to pore pressure conditions and loss of material strength.

Overtopping of the embankment is only a credible failure mode for severe flood events and earthquake-induced deformation. The risk of flood-induced overtopping is very low, and is managed by maintaining the prescribed design freeboard through continued embankment construction up to the final design elevation. The design freeboard is comprised of storm storage freeboard and additional minimum freeboard for wave run-up. The storm storage freeboard is based on the PMF, which is theoretically the largest flood resulting from a combination of the most severe meteorological and hydrologic conditions that could conceivably occur at the project site. The storm runoff volume for the PMF is over three times larger than a probabilistically determined 1 in 1,000 year return period wet month. Embankment construction will be completed in 50 ft high staged lifts, and therefore the total actual freeboard will tend to be larger than the design freeboard until just before operations cease. A closure spillway will prevent overtopping in the long-term after operations cease.

A large earthquake can induce deformations and settlement of the embankment crest, which could have the potential to lead to a loss of freeboard and overtopping. The risk of earthquake-induced deformation leading to overtopping is very low. The seismic loading considered both the operating conditions and long-term conditions following closure. 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 84th-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. The presence of the long tailings beach with a depth of 50 ft or more of unsaturated tailings that is not susceptible to liquefaction would prevent an uncontrolled release even if this large amount of embankment deformation occurred. The pond will reduce in size following closure due to climatic conditions, pore pressures will reduce over time and the tailings surface will be covered further limiting the potential for overtopping following an earthquake.

▲ R2

Internal erosion and piping of the embankment under normal operating conditions is not a credible failure mode. The primary source of water for seepage is the supernatant pond, which is located in the north end of the YDTI and is separated from the embankment by large tailings beaches. The tailings beaches limit the potential for internal erosion and piping by controlling the source of water and seepage flow path. The beaches work in conjunction with the free draining embankments to limit pore pressures at the interface between the tailings and embankment materials, and eliminate any substantial phreatic surface from developing in the embankment. Piping cannot develop without a continuous source of water eroding material along a seepage flow path. The risk of internal erosion and piping will increase if the supernatant pond or tailings stream is allowed to approach one of the embankments due to improper beach development or natural flooding.

The potential for internal erosion and piping initiated by natural flooding carries the greatest uncertainty for the YDTI. The flood events considered in this risk assessment are rare and therefore the likelihood of the flooded condition actually developing is very low. However, an analysis of internal erosion and piping potential under these conditions cannot be analyzed in a practical manner. This uncertainty highlights the importance of water management and tailings beach development to manage risk.

The 1 in 1,000 year return period wet month would be expected to flood a substantial portion of the lower elevation subaerial beaches. The supernatant pond in this scenario would remain separated from the embankments, which limits the potential for development of concentrated leakage. Seepage flows within the embankment could be expected to increase with corresponding increases in pore pressures in areas of the embankment that do not drain as effectively. The PMF event would be expected to flood all or most of the subaerial tailings beaches. The supernatant pond would not be isolated from the embankment under these conditions. The alluvium facing, wide crest width, and the well graded particle size distribution of most of the embankment fill would provide some protection against internal erosion in many areas of the embankment. However, the ponding of water adjacent to the embankment could provide a pathway to a very large source of water that has the potential to cause concentrated leakage through gap-graded rockfill zones or coarse boulder layers within the embankment. These zones are known to exist and have been subjected to concentrated leakage caused by the tailings stream in the past.

The YDTI operates by keeping the pond separated from the embankments and by constructing free draining embankments. This concept, which has been successful over many decades to date, can be applied to the future development of the facility to mitigate the potential for internal erosion and piping under flooded conditions. Reducing the normal operating pond volume or improving the uniformity of tailings beach development will increase the storm storage that can be contained on the tailings beach without reaching the embankment. This will decrease the potential for internal erosion and piping under flooded conditions for the YDTI and will further enhance the safety of the facility under normal operating conditions.

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9 – CERTIFICATION


This report was prepared and reviewed by the undersigned.

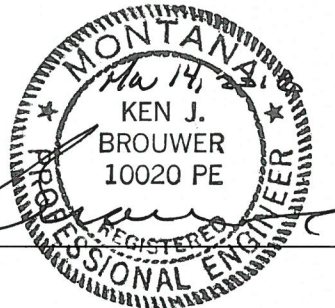
Prepared:


Jason Gillespie, P.Eng.
Project Engineer




Prepared:


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



Reviewed:


Ken Brouwer, P.E.
President

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APPENDIX A
TAILINGS FLOWABILITY ASSESSMENT
(Pages A-1 to A-32)

MEMORANDUM

To:	Mr. Ken Brouwer	Date:	February 8, 2018
Copy To:	Roanna Stewart	File No.:	VA101-00126/17-A.01
From:	Amy Adams	Cont. No.:	VA18-00103
Re:	Yankee Doodle Tailings Impoundment: Tailings Flowability Assessment		

1 – INTRODUCTION

Montana Resources, LLP (MR) 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 (EL.) of 6,450 ft. The amendment will provide for approximately 12 years of additional mine life. This memorandum summarizes the study of the potential flowability of the YDTI tailings in the event of a hypothetical and sudden loss of containment at the East-West Embankment due to static or earthquake-induced deformation. The study was performed by Knight Piésold Ltd. (KP) to provide input to a detailed dam breach risk assessment of the facility (KP, 2017a). The purpose of the study was to develop empirical information to assess the degree of mobility of the tailings through laboratory testing that can be considered along with tailings cone penetration testing (CPT) data and conventional liquefaction assessment to analyze the risks associated with the YDTI.

2 – BACKGROUND

The YDTI is a valley-fill style impoundment created by a continuous rockfill embankment that is referred to as three different embankment limbs (North-South, East-West, and West). The highest section of the embankment is located on the East-West limb of the embankment at the southern end of the impoundment. 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 rockfill has compressed over time under self-weight settlement and consolidation.

Tailings were historically discharged into the YDTI from a single point located on the southern end of the facility, such that extensive drained tailings beaches have developed adjacent to highest embankment section and the supernatant water pond is situated approximately 5,000 feet away as shown on Figure 1a. Changes to the tailings distribution system were made between 2016 and 2017, and eight discharge locations are now presently available to develop extensive drained tailings beaches adjacent to all three embankments. The tailings beaches will be capped and revegetated after closure, and the supernatant pond will shrink in size as illustrated in Figure 1b.

A stability assessment of the YDTI embankments was performed and considered static normal operating conditions, static post-earthquake conditions, earthquake-induced embankment deformation, and a series of sensitivity analyses analyzing hypothetical undrained loading conditions (KP, 2017b). The stability assessment concluded that continued filling of the YDTI can be completed while achieving factor of safety (FS) values that exceed legislated requirements, and are equivalent or improved as compared to 2014 conditions. 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 pressure in the tailings beach adjacent to the upstream face in the long-term. These key performance factors not only influence the FS values (i.e. the

likelihood of embankment deformation), but also the potential consequences of static or earthquake-induced deformation.

(a) During operations

(b) At closure

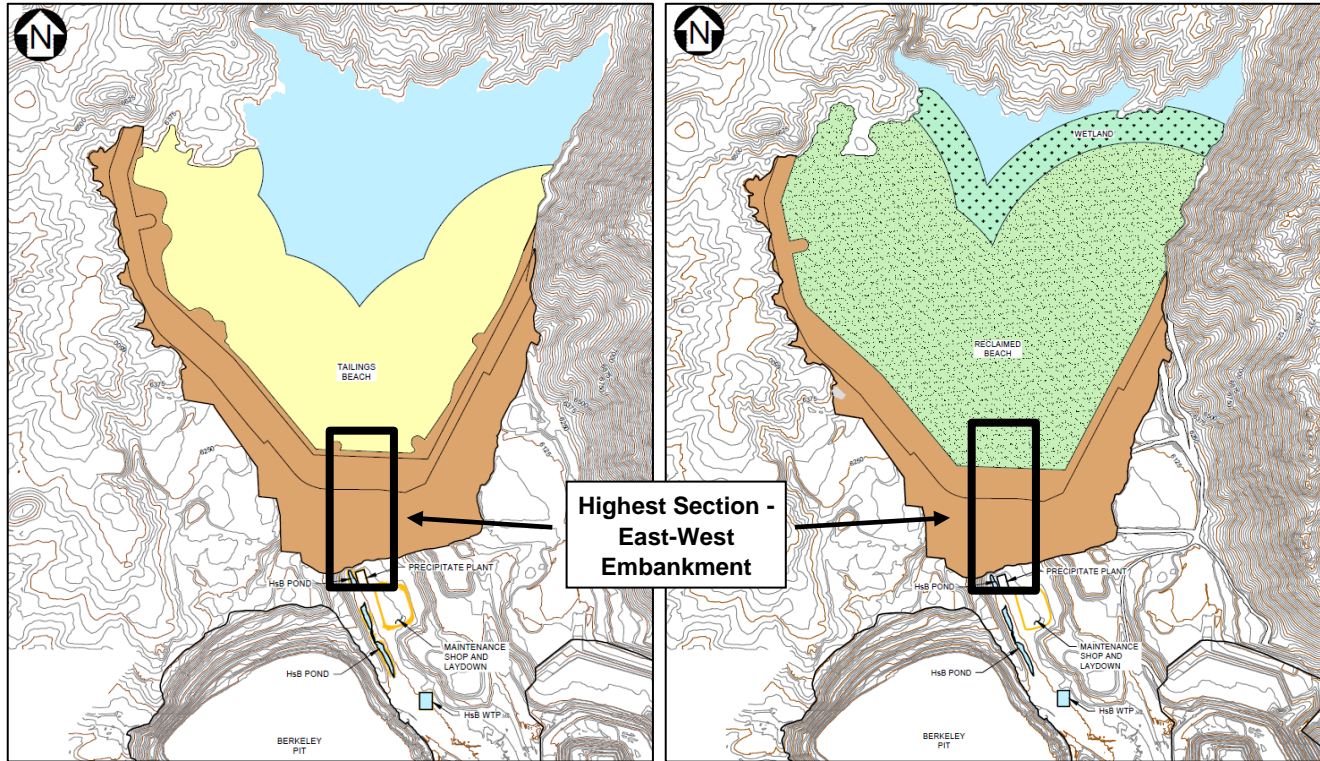


Figure 1 YDTI Arrangement During Operations and After Closure

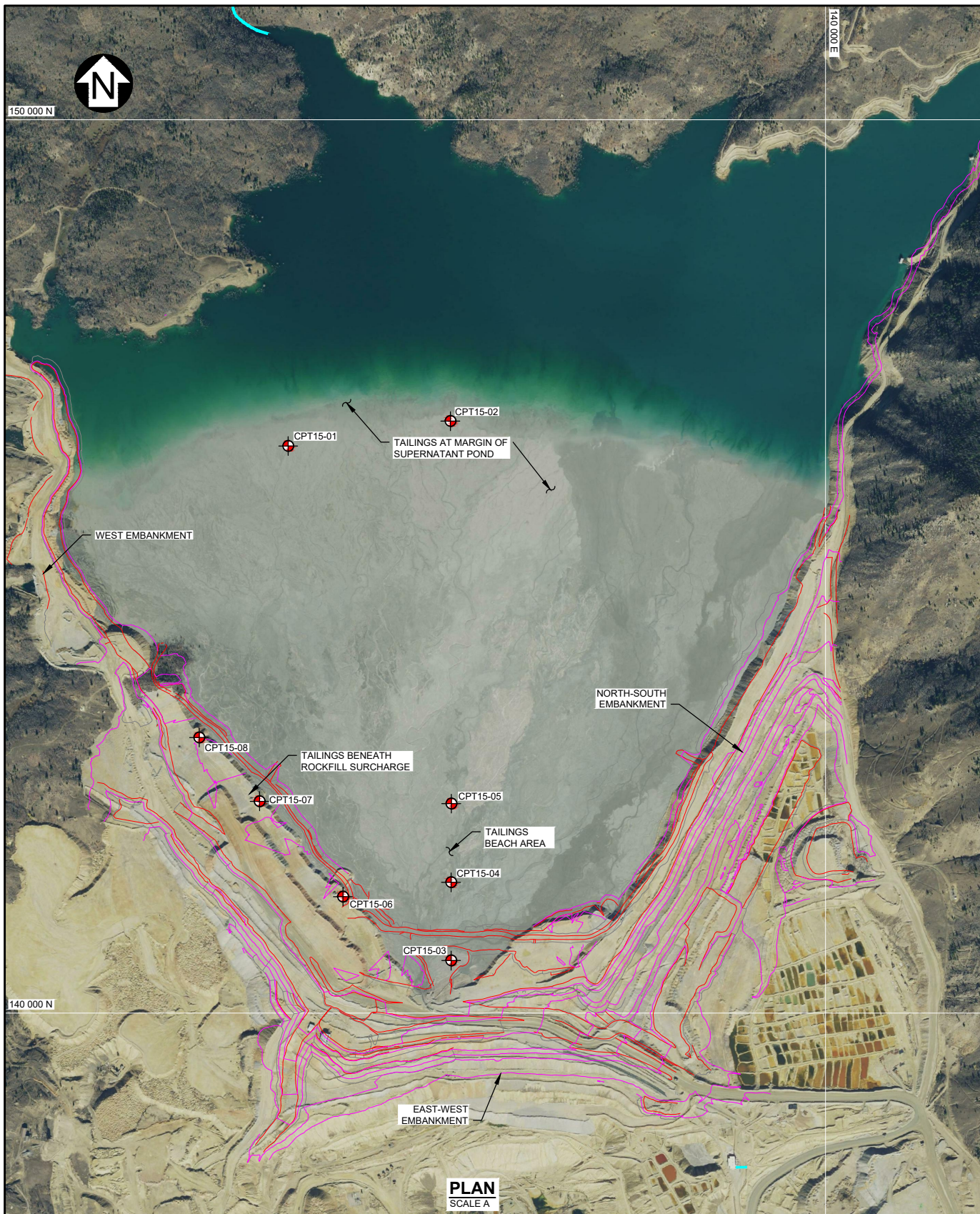
3 – TAILINGS CHARACTERIZATION

3.1 IN-SITU TAILINGS CHARACTERIZATION

Slurry tailings are generated in the milling process and generally consist of a predominately silty sand size fraction. The tailings solids are transported (pumped) as a slurry at about 33 to 37% gravimetric solids content (equivalent to 200% moisture content). The tailings slurry is deposited onto the subaerial tailings beaches where the solids settle and drain, while the transport fluid (water) flows to the supernatant pond for recovery and recycle to the milling process.

The YDTI embankments have been developed as free draining rockfill structures to allow for gravity drainage of the tailings beaches thereby reducing the tailings moisture content and saturation following deposition. The insitu density and moisture content in the tailings beaches adjacent to the East-West embankment have been evaluated from both field samples collected during the 2012 Site Investigation (SI) Program (KP, 2013) and extrapolated from CPT probings advanced during the 2015 SI Program (KP, 2016).

The location of the CPT probings completed during the 2015 SI Program are shown on Figure 2. The tailings dry density profiles from the CPT probings are shown on Figure 3. The profiles were produced using the data from the CPT probings (CPT15-03, CPT15-04, and CPT15-05) located in the vicinity of the maximum section of the East-West Embankment. The dry density profiles are based on measured tip resistance and sleeve friction (Robertson, P.K., and Cabal, K.L., 2010) and measured shear wave velocity (Mayne, P.W. 2014).



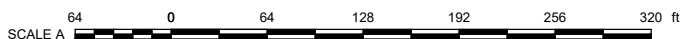
NOTES:

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

LEGEND:



DRILLHOLE AND CPTS - KNIGHT PIESOLD 2015



REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	19JAN'18	ISSUED WITH MEMO	DDF	PP	KJB

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

2015 CPT SOUNDINGS

Knight Piesold
CONSULTING

P/A NO. VA101-126/17	REF NO. VA18-00103
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FIGURE 2

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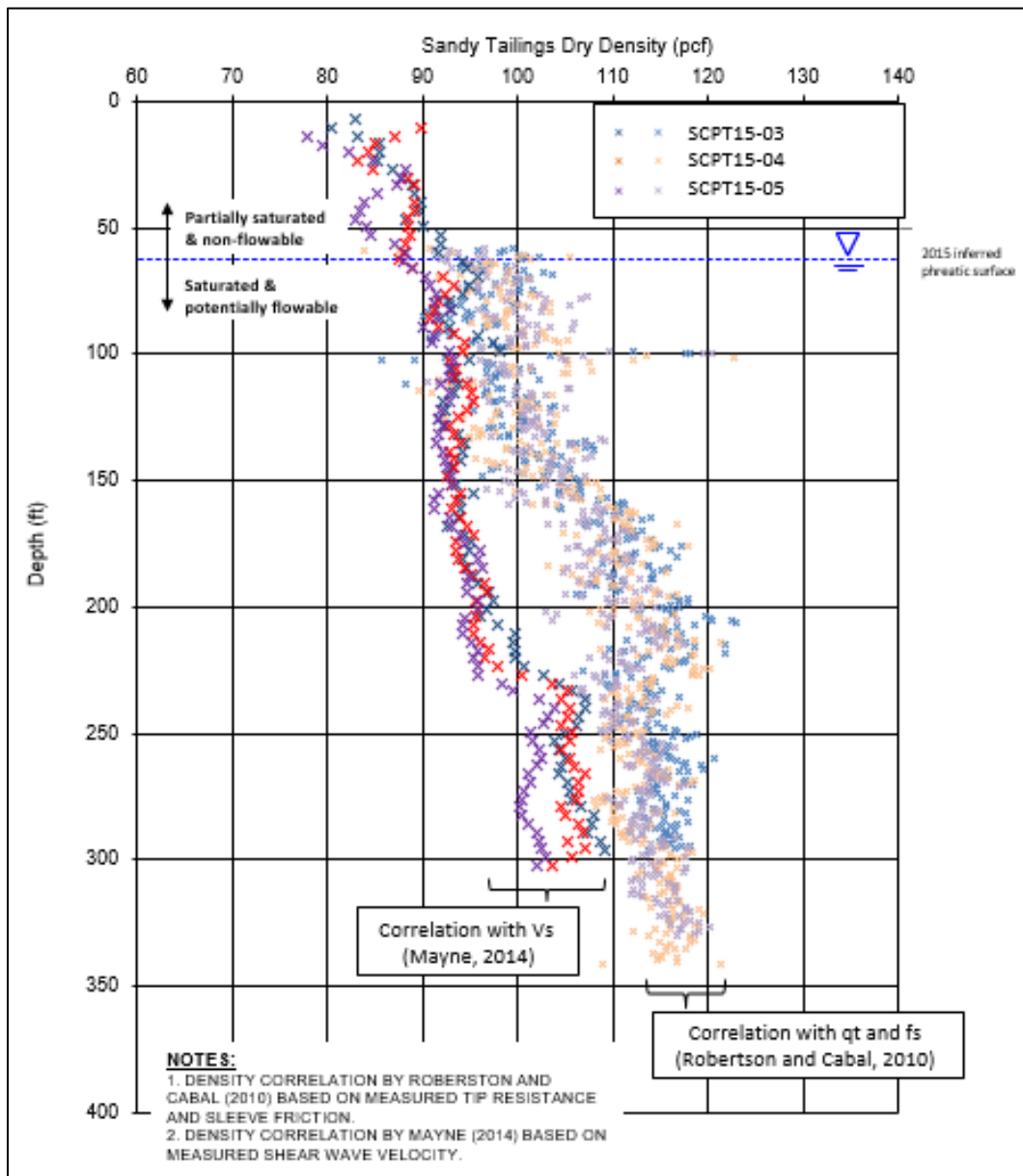


Figure 3 Silty Sand Tailings Dry Density vs. Depth

The phreatic surface in 2015 was determined to be approximately 60 ft deep (corresponding to EL. 6,290 ft above Anaconda datum). The tailings dry density progressively increases with depth as illustrated on Figure 3. The upper tailings with lower density typically drain downward and laterally into the embankment. 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. A saturated moisture content profile for the in-situ tailings was extrapolated from the dry density profile and is provided on Figure 4.

Laboratory measurements suggest that the moisture content of the upper 60 ft of tailings is variable, and that these tailings may be partially saturated. Small sections of the tailings beach immediately adjacent to the active tailings discharge points may also be temporarily saturated where localized recharge infiltration occurs. The tailings drain rapidly when the active discharge point is relocated.

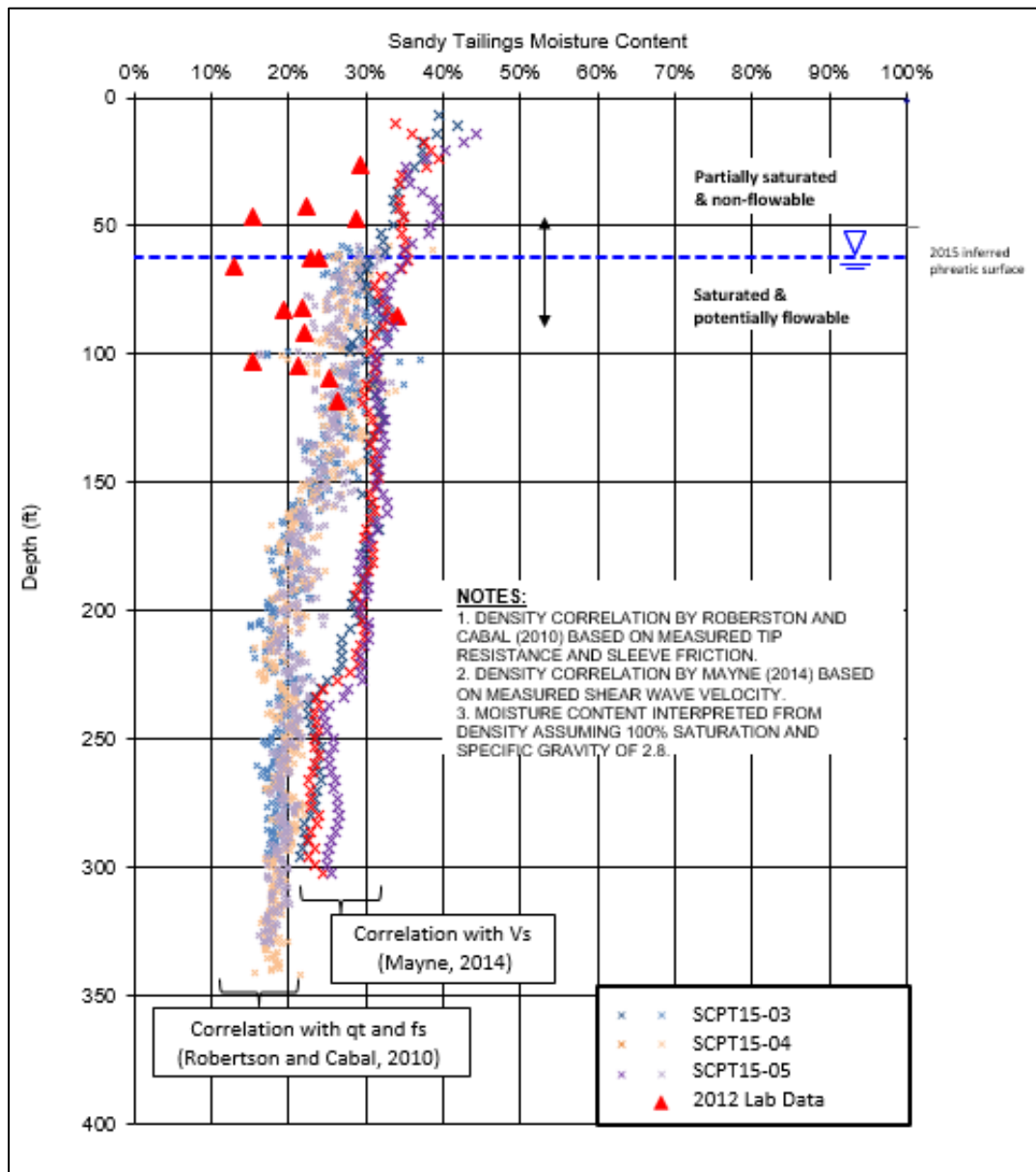
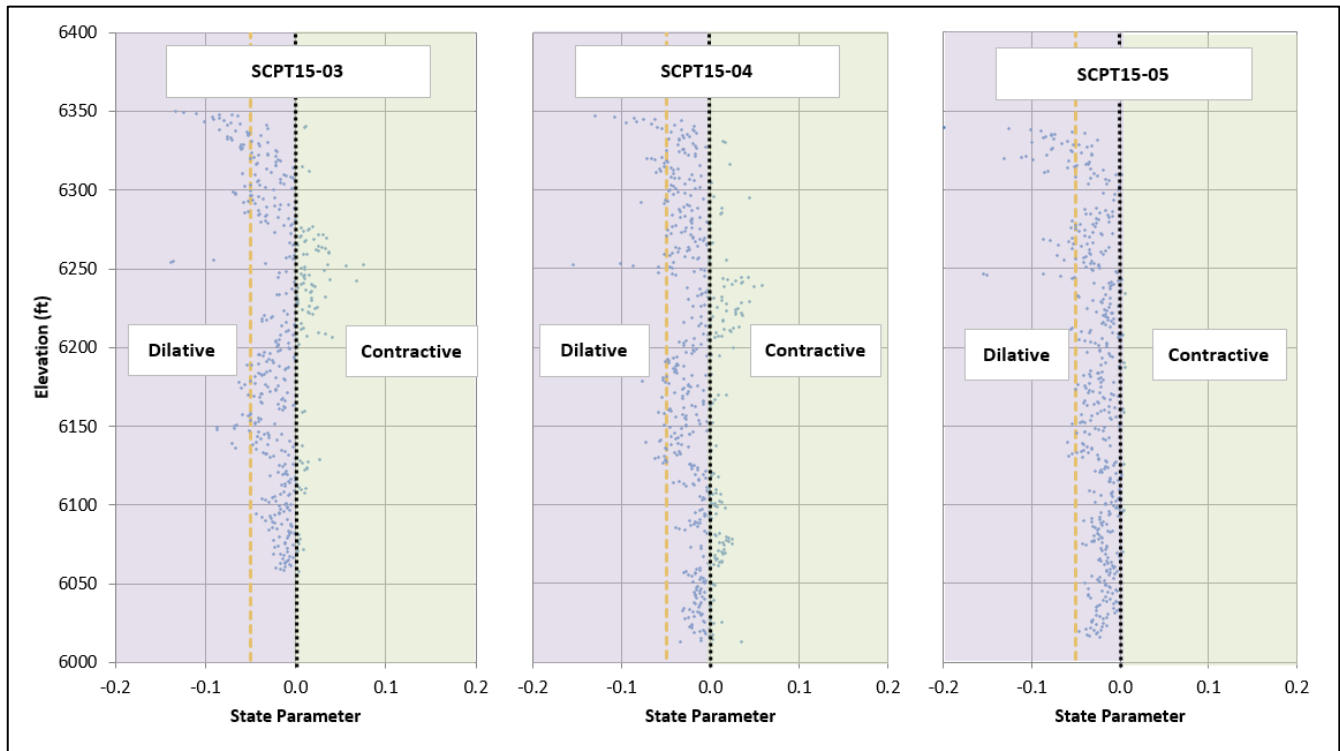


Figure 4 Silty Sand Tailings Saturated Moisture Content vs. Depth

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 (y) is the difference between the current void ratio of the tailings and the critical state void ratio. 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 profile plots for CPT15-03, CPT15-04, and CPT15-05 are shown on Figure 5.



NOTES:

1. The division between dilative and contractive behaviour is generally accepted to be -0.05, indicated by the yellow dashed line, although a state parameter of < 0 is indicative of potentially dilative material.

Figure 5 East-West Embankment Silty Sand Tailings State Parameter vs. Depth

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. The pore pressure difference profiles are shown on Figure 6.

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 under existing conditions.

Additional details from this analysis are provided in the Stability Assessment Report (KP, 2017b) and the Site Characterization Report (KP, 2017c).

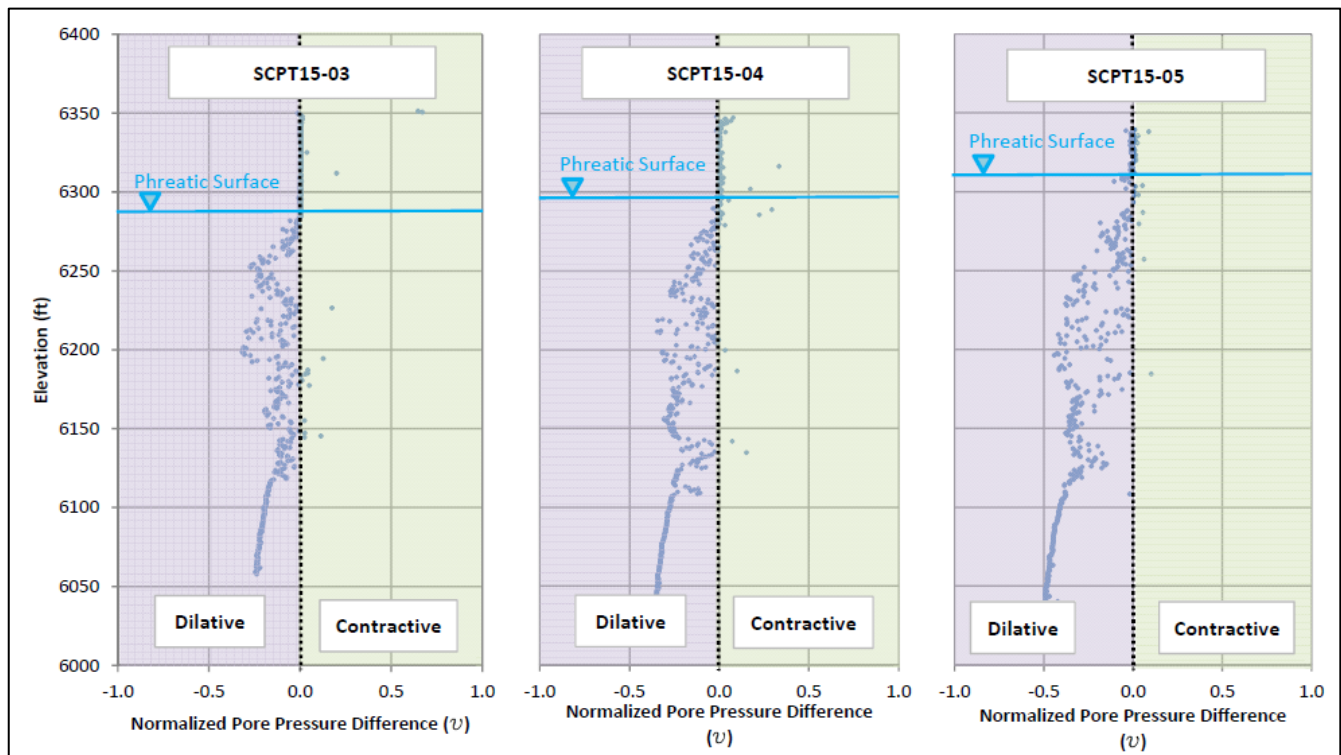


Figure 6 East-West Embankment Silty Sand Tailings Pore Pressure Differential vs. Depth

3.2 LABORATORY TAILINGS CHARACTERIZATION

3.2.1 Sampling Program and Test Specifications

Laboratory testing to characterize the potential mobility of the beach tailings adjacent to the East-West Embankment included the following:

- Index testing, including grain size, plasticity, and mineralogy, to evaluate general tailings characterization
- Rheological testing, including vane yield and Boger slump tests, to evaluate the flowability of the tailings at different water contents under static loading conditions, and
- Transportable Moisture Limit (TML) testing to evaluate the behaviour of unconfined tailings under dynamic loading conditions.

Eight (8) bucket samples of settled tailings solids were collected from various locations along the YDTI beach on April 13, 2017 by Montana Resources site staff. A ninth composite sample (KP17-09) was prepared using three Shelby tubes previously collected in 2015. This sample is representative of the 'slimes' sub-aqueous beach zone (tailings beach below water) located approximately 6,000 feet away from the maximum section of the East-West Embankment. The approximate locations of the tailings sample collection sites along the YDTI beach are shown on Figure 7. The samples were sent to the KP Geotechnical Laboratory in Denver, Colorado for soil index testing and then forwarded to Paterson and Cooke in Golden, Colorado for X-ray diffraction mineralogical analysis, and rheology and TML testing.

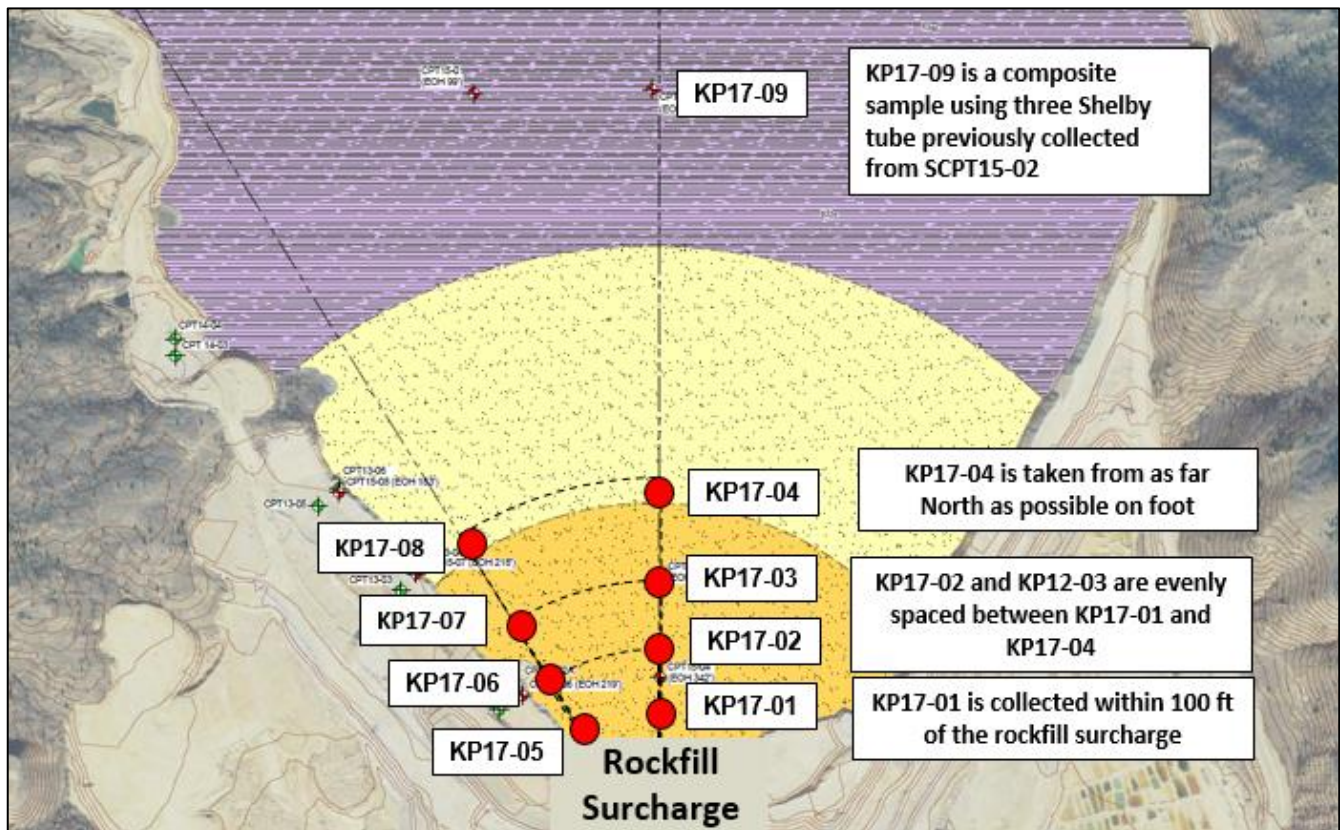


Figure 7 Approximate Tailings Sample Collection Locations

3.2.2 Tailings Index Testing

The results of the laboratory index testing are summarized in Table 1.

Table 1 Summary of Index Test Results

Sample No.	Particle Size Analysis (ASTM D6913)			Atterberg Limits (ASTM D4318)			Specific Gravity (ASTM D854)	Proctor Test (ASTM D698-12)	
	Sand (4.75 to 0.075 mm) (%)	Silt (0.075 to 0.002 mm) (%)	Clay (<0.002 mm) (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index (%)		Optimum Moisture Content (%)	Maximum Dry Density pcf
KP 17-01	84.7	13.7	1.6	N/P	21.0	N/P	2.78	17.5	102.4
KP 17-02	75.7	21.5	2.8	N/P	20.0	N/P	2.78	15.0	108.6
KP 17-03	76.3	21.4	2.3	N/P	22.0	N/P	2.78	16.5	104.8
KP 17-04	55.5	39.8	4.7	N/P	27.0	N/P	2.80	17.5	103.0
KP 17-05	89.5	9.1	1.4	N/P	21.0	N/P	2.77	17.4	102.2
KP 17-06	85.3	12.5	2.2	N/P	22.0	N/P	2.81	18.3	103.3
KP 17-07	87.9	10.1	2.0	N/P	20.0	N/P	2.77	15.4	103.3
KP 17-08	79.5	18.7	1.8	N/P	20.0	N/P	2.78	17.3	102.7
KP 17-09	28.0	61.2	10.8	24.0	31.0	7.0	2.77	16.8	106.6

The tailings are relatively homogenous non-plastic silty sand with trace clay. The specific gravity is 2.78, the liquid limit is 20%, the optimum moisture content is approximately 17%, and the standard proctor maximum dry density (SPMDD) is approximately 104 pcf. The tailings achieve the SPMDD between approximately 100 and 150 ft depth based on the CPT tip resistance and sleeve friction correlation shown on Figure 3.

The tailings show some increase in plasticity as the distance from the discharge point increases and the clay fraction increases. Sample KP17-09 has a liquid limit of 31% and a plasticity index of 7%. Mineralogical analyses were completed on sample KP17-02. The tailings is predominately comprised of quartz, K-feldspar, and plagioclase with only 5% clay minerals dominated by smectite.

3.2.3 Rheology testing

The yield stress is a measure of the activation energy required to cause a material to transition from semi-rigid behaviour to fluid-like behaviour. Water is a Newtonian fluid with zero yield stress, which allows it to flow under any level of applied stress until a flat surface is achieved. Materials with a non-zero yield stress are classified as Bingham plastics or pseudo-plastics. These materials experience viscous flow only when the applied stress exceeds the yield stress.

It is useful to visualize the yield stress in relation to the consistency and characteristics of other common fluid or near-fluid materials. For example, ketchup and mayonnaise are common household condiments that can be slowly poured, squeezed, or easily spread onto food. Grape jelly cannot be poured and must be spread to apply to food. Ketchup flows more easily when it is disturbed by shaking, but sets up rapidly once shaking has ended. Rheology for food processing is a mature science. Examples of the yield stress of these common materials are summarized in Table 2 below (Sun, A. and Gunasekaran, S. 2012, Cunningham, N. 2017). Comparing tailings consistency to common household items and foods has been a topic of recent research by other geotechnical practitioners as well (Mckenna, G. et al., 2016).

Table 2 Yield Stress as Indicator of Consistency of Flowability

Material	Yield Stress (Pa)
Ketchup	15
Mayonnaise	60 to 100
Grape Jelly (Refrigerator)	~ 500 Pa
Drywall Mud	~ 1000 Pa
Play Dough	~ 4500 Pa

The yield stress allows the consistency (and flowability) of tailings to be quantified independently of the solids content or composition. For saturated sandy tailings, the yield stress increases as the moisture content decreases. The potential for the tailings to flow is thus dependant on the moisture content and density of the tailings. Thickening and pumping of higher solids content (lower moisture content) tailings slurries at other mine operations where this alternative technology is used are at risk of thickeners and/or pipe works becoming plugged if the moisture content is reduced and the yield stress is increased above a pumpable threshold value. The same concept applies to the potential flowability of recently or historical deposited tailings within the YDTI.

The rheology of the saturated YDTI beach tailings was determined for four tailings samples after preparing the samples with different moisture contents. Two different methods were used to measure the yield stress of the samples as follows:

- **Vane yield test.** A vane is inserted into the specimen and rotated at a constant angular velocity. The peak torque corresponds to the yield stress.
- **Boger slump test:** A cylinder is filled with tailings and the cylinder is removed. The yield stress is calculated based on the measured slump of the unconfined tailings.

The results of the yield stress measurements are summarized in Table 3. The relationship between the measured yield stress and the moisture content is shown graphically in Figure 8. A key is provided showing the relationship between the tailings state (slurry, paste, or soil). A curve has been fit to the data based on the measured clay fraction of the samples. The yield stress of the tailings increases as the tailings moisture content decreases and the clay fraction increases.

Photographs taken during the Boger Slump test program are provided in Appendix A to illustrate the transition in unconfined behaviour from stable to fluid like flow under static stresses. A typical progression of unconfined behavior from Boger Slump testing of sample KP17-03 is shown on Figure 9. The tailings adjacent to the East-West Embankment begin to demonstrate highly viscous consistency deformation at moisture contents above approximately 35% when the yield stress is approximately 100 Pa (equivalent to mayonnaise). This yield stress is characteristic of the slurry to paste transition zone for tailings.

Less viscous fluid-like flow was observed when the moisture content was greater than approximately 40% corresponding to yield stresses less than 50 Pa. This yield stress is consistent with the characteristics for highly fluid and pumpable slurry tailings with a consistency that is roughly equivalent to ketchup.

The moisture content relating to a target yield stress increases with increasing clay fraction and thus distance from the point of deposition. The 'slimes' tailings sample KP17-09, which was collected from a barge mounted CPT rig, begins to exhibit highly viscous flow behaviour when the moisture content is higher than approximately 55%. Less viscous fluid flow behaviour was not observed until the moisture content reached approximately 70% for this sample.

Table 3 Summary of Rheological Test Results

Sample No.	Soil Type	Clay	Yield Stress Measurements		
			Moisture Content	Vane Yield	Boger Slump
		(%)	(%)	(Pa)	(Pa)
KP 17-01	Silty Sand	1.6%	25	1250	990
			28%	173	392
			31%	122	315
			32%	79	249
			35%	47	166
			44%	20	64
			53%	14	19
KP 17-02	Silty Sand	2.8%	20%	914	958
			25%	584	714
			27%	254	431
			28%	225	360
			31%	144	303
			34%	86	212
			36%	59	165
			42%	23	65
			47%	15	15
KP 17-04	Silty Sand	4.7%	26%	1110	930
			31%	239	327
			35%	118	222
			36%	116	193
			38%	50	92
			44%	24	40
			49%	20	22
KP 17-09	Silt with Sand (Slimes)	10.8%	34%	1890	428
			37%	1160	391
			45%	500	305
			53%	235	190
			60%	103	119
			69%	49	68
			86%	19	31

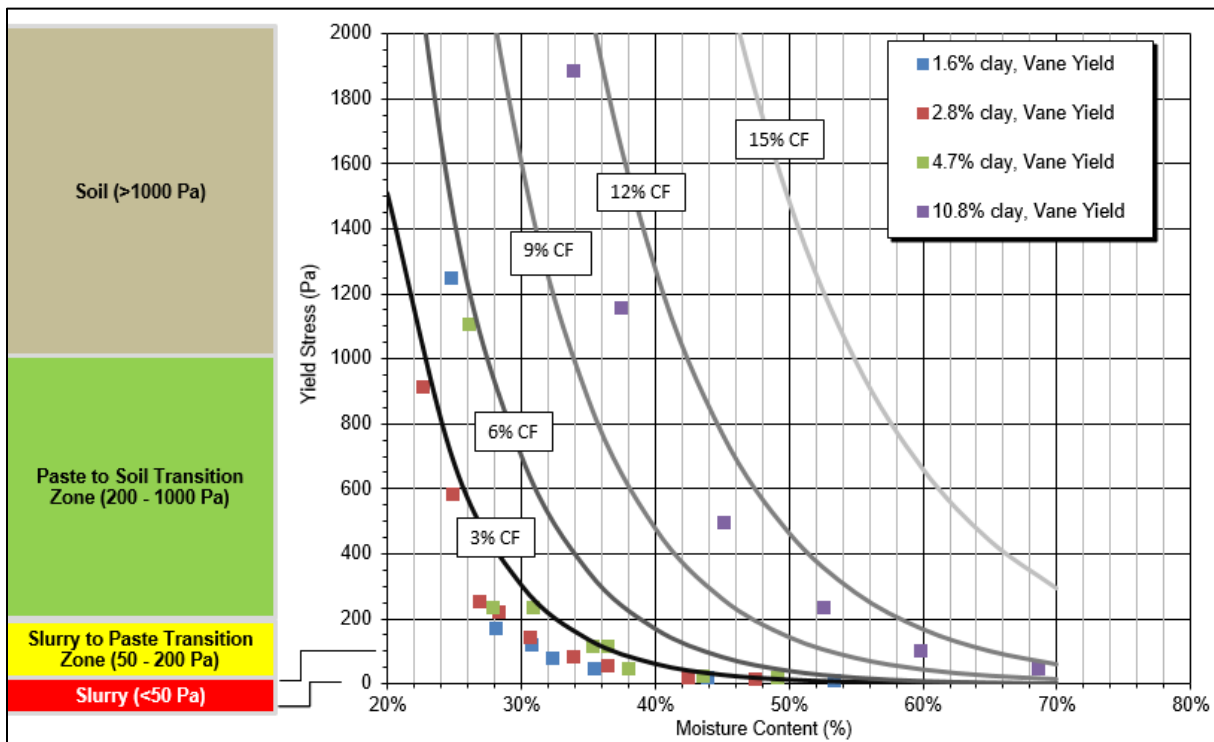


Figure 8 Tailings Rheology Model

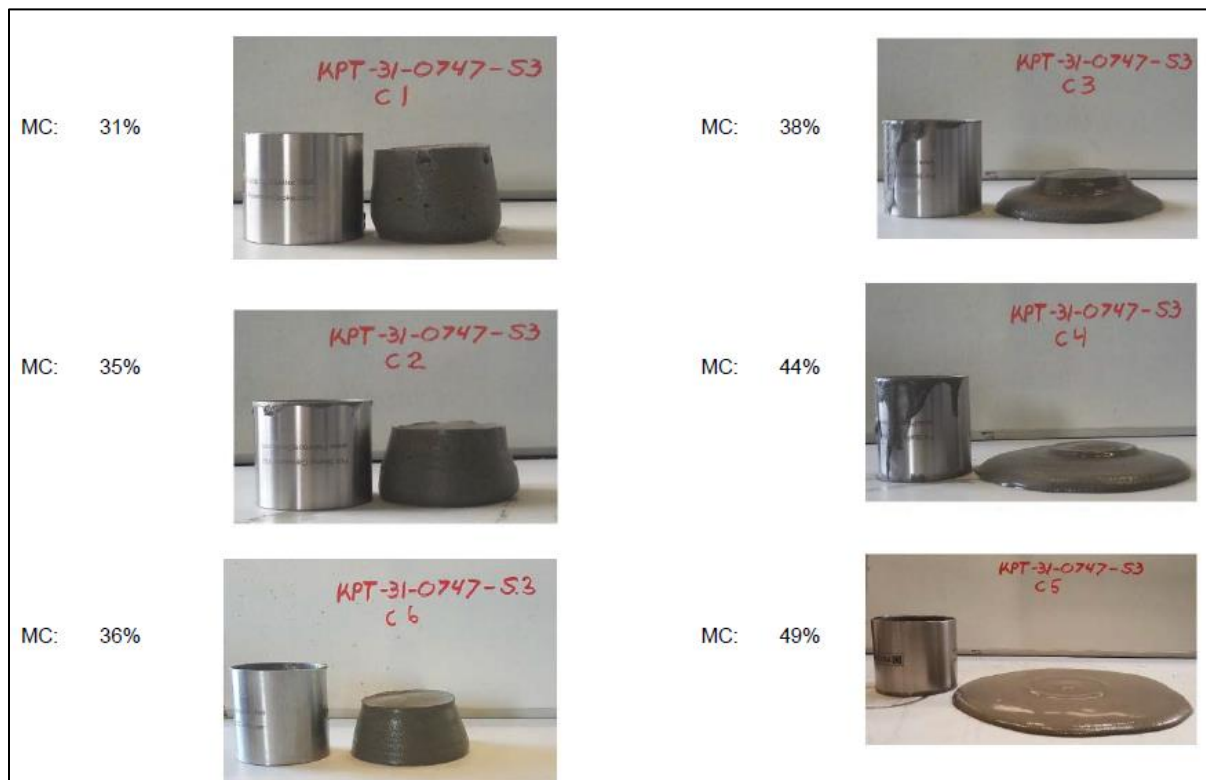


Figure 9 Boger Slump Yield Stress Test Results - KP17-03

3.2.4 Transportable Moisture Limit (TML)

The TML test was conducted on four of the tailings samples as an alternative method to determine the moisture content when unconfined tailings transition from a material with stable soil characteristics to a deformable viscous material under the constant input of dynamic stress. The TML test is commonly used to identify the maximum water content (by weight) of a liquefiable cargo that is to be transported on a bulk carrier such as a ship or on a conveyor belt system. The TML test is used to define the Flow Moisture Point (FMP), which is related to the moisture content where liquefaction or partial liquefaction of the sample could occur. The FMP is measured on the material compacted to a representative equivalent stress level (typically 1 m) under a constant dynamic input provided by a standard flow table (Figure 10). The TML is generally reported after adding a 10% factor of safety to the measured FMP. The TML test methodology is standardized under the International Maritime Solid Bulk Cargoes (IMSBC) code.



Figure 10 Flow table for TML testing (after Paterson and Cooke, 2017)

Typical photos of the TML tests for the tailings samples are shown on Figure 11 and Figure 12. The tests begin with a truncated circular pyramid of compacted tailings that is statically stable. The specimen diameter increases as the specimen and flow table shake. The diameter increase occurs from specimen fragmentation at low moisture contents (Figure 11) and due to liquefaction and viscous spreading at higher moisture contents (Figure 12).

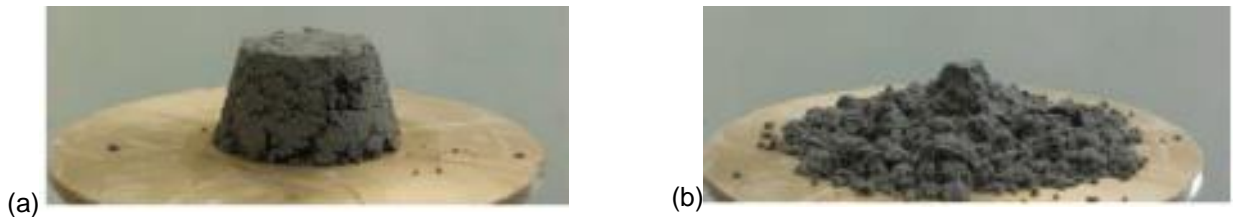


Figure 11 TML Test Specimen showing fragmentation (a) before test and (b) at completion

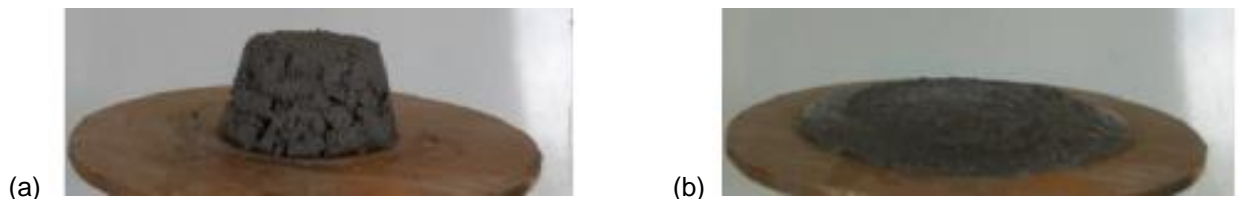


Figure 12 TML Test Specimen showing liquefaction (a) before test and (b) at completion

Photographs of the TML testwork for each sample are provided in Appendix B. The photographs indicate that isolated liquefaction begins at approximately 23% moisture content, with deformation characterized by slow, highly viscous flow deformation (similar to squeezed mayonnaise). Full liquefaction resulting in less viscous fluid-like flow of the whole specimen (similar to squeezed ketchup) did not occur until the moisture content was increased to

above approximately 25%. Flow does not continue upon removal of the dynamic stress. These critical moisture contents are higher for KP17-09 due to its higher liquid limit and clay fraction.

4 – TAILINGS FLOWABILITY POTENTIAL

4.1 COMPARISON OF CPT DATA AND LABORATORY DATA

The potential for the tailings to flow due to static and dynamic stresses can be evaluated based on the inferred in-situ saturated moisture content from the CPT soundings and the laboratory tailings characterization testing. The tailings flowability testwork was completed at low confining stresses equivalent to a maximum of 1 m of cover. The results are judged representative of the potential static and dynamic behaviour of the tailings in the unlikely scenario that the tailings become unconfined and need to be self-supporting (i.e. the vertical and horizontal confining stresses from overburden loading and/or embankment confinement were instantaneously removed). A comparison between the moisture content of the tailings and interpretation of the results of the rheology and TML testing is shown on Figure 13.

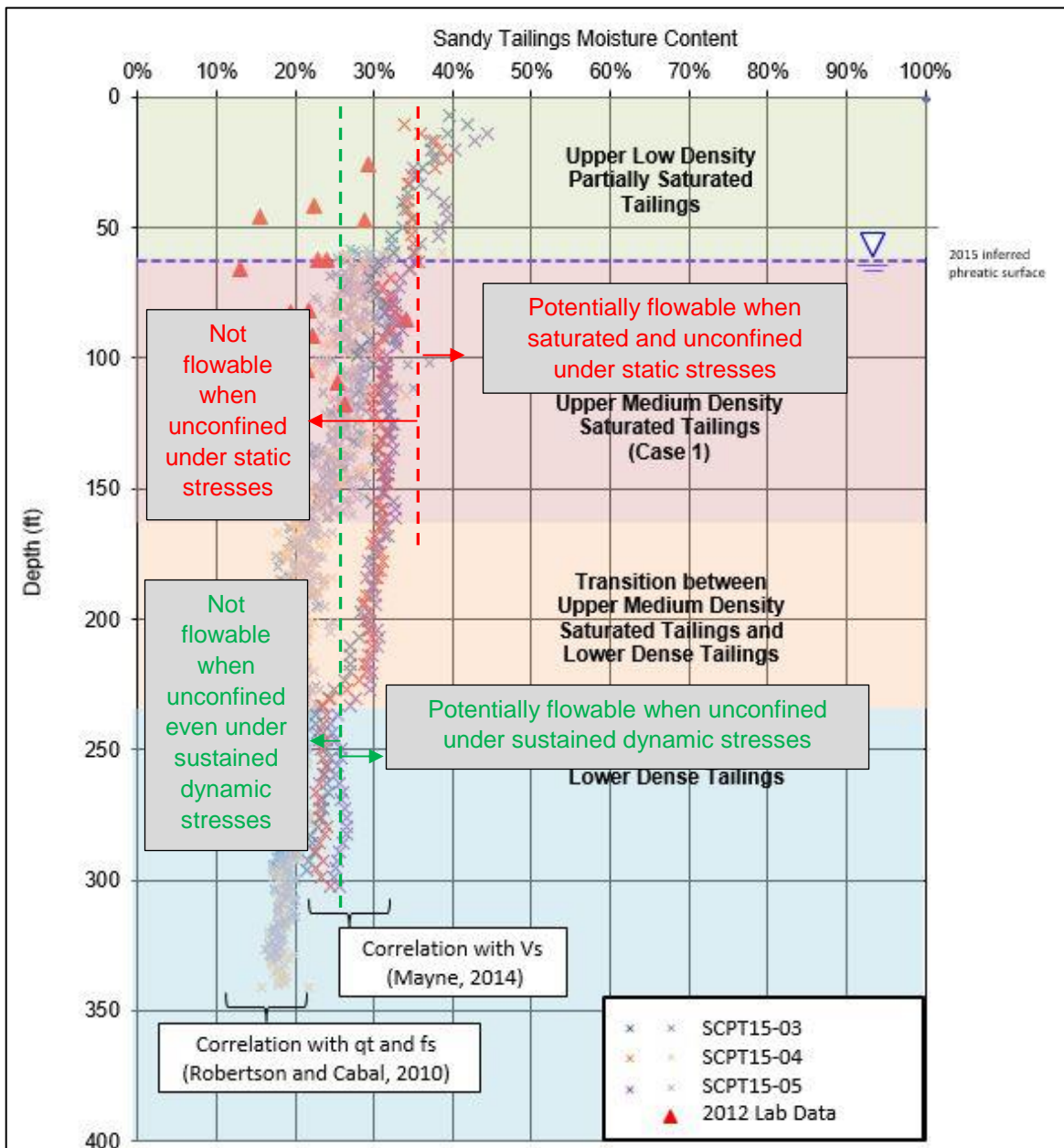


Figure 13 Silty Sand Tailings Flowability Potential

4.2 FLOW POTENTIAL UNDER STATIC CONDITIONS

The rheology testwork indicates that the silty-sand tailings do not have the potential to flow under static stresses at saturated moisture contents below approximately 35%, and fluid-like flow liquefaction would require a moisture content exceeding approximately 40%. A moisture content of 35% is indicated by the red dashed vertical line on Figure 13 for comparison with the inferred in-situ moisture content determine from the CPT soundings. The tailings below the inferred phreatic surface are sufficiently dense to prevent flow in the event they become unconfined without a source of surface water to initiate erosion. The existing beach tailings under static stresses are expected to behave like soil even if unconfined.

These findings are corroborated by the tailings state characterization, which indicated that although the tailings materials are on the boundary of potentially contractive or dilative based on state parameter, the excess pore

pressures required for contractive behavior are not present under existing conditions. Instability under static conditions leading to deformation of the East-West Embankment does not have the potential to initiate flow liquefaction because the large drained tailings beaches maintain the supernatant pond remote from the embankments, which reduces the pore pressure in the tailings beach adjacent to the upstream face.

4.3 FLOW POTENTIAL UNDER DYNAMIC CONDITIONS

The TML testwork indicates that under sustained dynamic loading, flow liquefaction has the potential to develop at moisture contents in excess of approximately 25%. Viscous flow initiated at these moisture contents stopped under laboratory test conditions when dynamic loading ceased. A moisture content of 25% is indicated by the green dashed vertical line on Figure 13 for comparison with the inferred saturated moisture content from the CPT soundings. The tailings in the beach area below the unsaturated zone may have sufficient moisture content to flow in a viscous manner during active excitation (i.e. during seismic loading) without confinement. Viscous flow is likely to stop without active shaking (i.e. at the end of the earthquake). The following tailings zones are inferred from the relationship between CPT based in-situ moisture content and the TML testwork:

- **Upper Low Density Partially Saturated Tailings (0 to 60 ft):** The surface tailings zone is unsaturated and the tailings do not contain sufficient moisture to develop fluid-like flow without the addition of large quantities of water from an external source. An external source of water is not realistically available as the supernatant pond is located about 5,000 feet north of the East-West Embankment.
- **Upper Medium Density Saturated Tailings (60 to 160 ft):** The middle tailings zone is mostly saturated and the tailings moisture contents varying from 25 to 35%. The correlation based on tip resistance and sleeve friction suggests that the tailings between 60 and 160 ft maximum depth have moisture contents higher than 25%. Flow liquefaction has the potential to develop in this zone during an active earthquake event without confinement. Embankment displacement would need to exceed 60 ft a vertical direction to expose the saturated tailings or a secondary failure mode would need to develop such as a vertical tear projected from the saturated tailings zone beyond the downstream slope of the embankment.
- **Transition between Upper Medium Density Saturated Tailings and Lower Dense Tailings (160 to 240 ft):** A 80 ft thick transition zone between the Medium and Lower Dense Tailings occurs between 160 ft and 240 ft. The correlation between the measured shear wave velocity and moisture contents suggests that 240 ft depth is a conservative lower boundary for upper medium density saturated tailings with moisture content above 25%.
- **Lower Density Tailings (below 240 ft):** The lowest tailings zone is sufficiently dense due to self-weight consolidation such that the in-situ moisture content would be insufficient for development of a tailings flow either under static conditions or during an earthquake event.

The findings above are similar to the findings of the conventional cyclic liquefaction assessment (KP, 2017b). The liquefaction assessment indicated that saturated tailings within the YDTI and without the rockfill surcharge are potentially liquefiable under the design earthquakes below the unsaturated zone. The top 50 to 80 ft of tailings adjacent to the embankment is unsaturated or partially saturated, dilative, and therefore not susceptible to liquefaction. The potentially liquefiable tailings extend from the bottom of unsaturated zone to approximately 200 to 270 ft depth in the central impoundment area. Tailings below the surcharged areas have very low liquefaction potential. 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 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 Stability Assessment Report (KP, 2017b) demonstrated that the YDTI embankments are stable and design freeboard tolerances are appropriate to prevent a loss of confinement. The tailings strength has very little impact on the predicted factor of safety for downstream slip surfaces. The YDTI embankments are stable with a factor of safety (FS) of 1.9 or greater under post-earthquake conditions. 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 legislative requirements for static post-earthquake loading conditions is $FS \geq 1.2$.

The earthquake deformation analysis indicates that the maximum estimated earthquake-induced deformation will be within design tolerances. 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. Earthquake-induced embankment displacement would need to exceed 60 ft in a vertical direction to expose the upper medium density saturated tailings, which is well above the estimated maximum displacement levels.

5 – HYPOTHETICAL CONSEQUENCES OF FAILURE

5.1 MECHANICS OF MUDFLOWS

A mudflow is a form of mass wasting that can result when saturated fine-grained sediments are mobilized and flow as a fluid down a sufficiently steep slope. Some mudflows are rather viscous and therefore slow, while others are very fluid and can flow rapidly. The rheological characteristics (viscosity) of a fine-grained mudflow depend on the grain size distribution, clay content, and moisture content of the saturated (or nearly saturated) soil mass. The velocity and extent of transport for a viscous mudflow will depend on the slope height and inclination (energy).

The 2015 Samarco tailings failure (Morgenstern et al., 2016) provides an example where loose saturated tailings liquefied and created a fluid mudflow that rapidly migrated downstream mixing with water in a down gradient reservoir, which overtopped and inundated a downstream village.

Liquefaction is a soil mechanics term implying a substantial loss of shear strength in response to an applied stress. Liquefaction typically occurs in saturated soils following rapid undrained or dynamic (i.e. earthquake) loading that cause pore pressures to increase and the effective stress to decrease. Unsaturated soils are typically not liquefiable. Liquefaction of a saturated soil does not necessarily imply the material characteristics will change to a highly mobile fluid state, but rather it implies that soil deformation can more readily occur under applied loading. Low viscosity fluid-like behaviour can occur when the moisture content is sufficiently high, whereas more viscous deformation will occur at lower moisture contents for a given soil type.

While liquefaction of saturated soil material is a prerequisite for flow, liquefaction alone does not imply that a highly fluid mudflow will occur. Both a loss of confinement and a sufficiently high moisture content are required before a mudflow can develop in a saturated soil, and a source of energy (e.g. a steep slope) must be present to maintain flow. The threshold moisture content required to initiate flow in a soil varies with the grain size characteristics of the material, the slope of the flow path, and the rate of flow as follows:

- **Grain Size:** Sandy material such as the beach tailings will typically require a lower moisture content to sustain fluid-flow characteristics as compared to finer grained materials with a higher clay content, which require more moisture to hydrate and coat the particles and sustain fluidity.
- **Flow Path:** Steep slopes allow for mud to flow at lower moisture contents as compared to gentler slopes.
- **Flow Rate:** Slow, highly viscous flow will occur at lower moisture contents and along shallower slopes, as compared to more rapid, high energy, fluid flow that may occur at higher moisture contents and down steeper slopes.

5.2 ESTIMATING THE GEOMETRY OF A HYPOTHETICAL MUDFLOW

The estimated in-situ density and the associated saturated moisture content of the tailings materials in the beaches adjacent to the East-West Embankment were evaluated in order to assess the extent of a hypothetical mudflow at this location. The Upper Medium Density Saturated Tailings zone may have a sufficient moisture content to support deformation as a slow and viscous flow until active earthquake movement ceases and a stable slope is achieved in the liquefied tailings material. The Upper Low Density Partially Saturated Tailings would be expected to deform if the underlying tailings were mobilized during seismic loading. An estimate of the stabilized slope for a hypothetical mudflow following liquefaction can be inferred geometry from the results of the Boger Slump, Yield Stress and TML tests.

The static angle of repose (i.e. Factor of Safety of 1.0) for tailings at 35% moisture content (corresponding to approximately 100 Pa yield stress) is approximately 45 degrees (1H:1V) or greater at low effective stresses as illustrated on Figure 14.

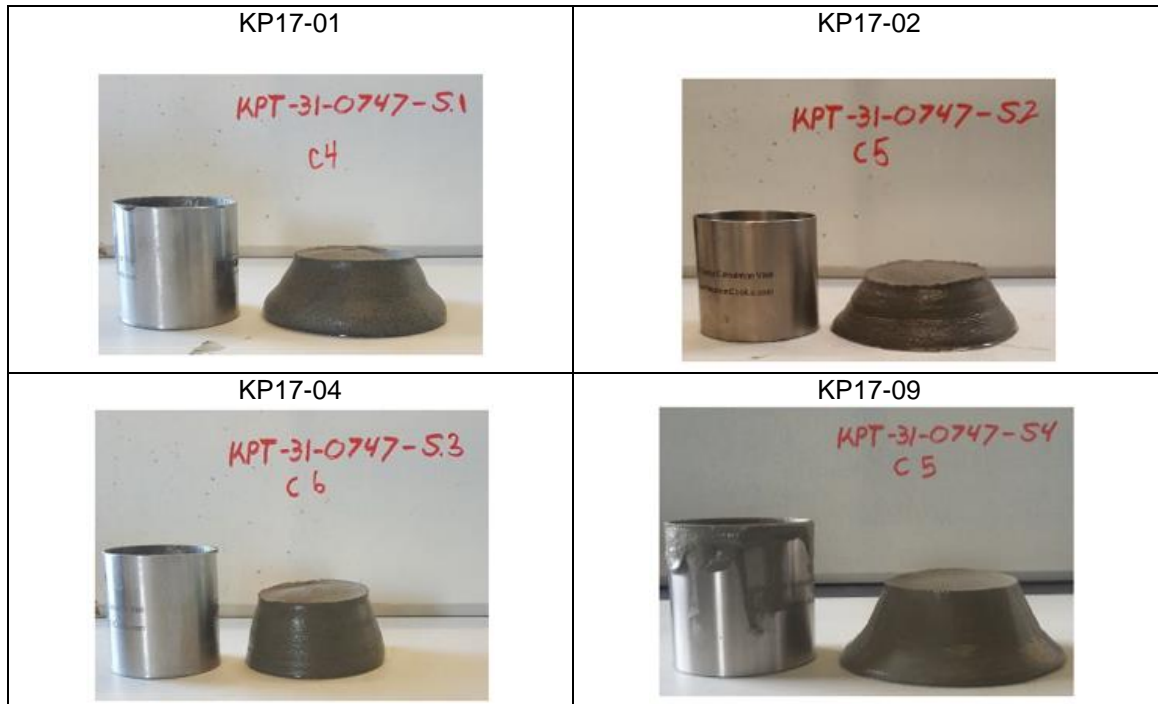


Figure 14 Static Angle of Repose for Silty Sand Tailings with 35% Moisture Content (100 Pa Yield Stress)

The dynamic angle of repose, which may occur if the tailings liquefy and are allowed to flow, varies from approximately 10 to 20% for these tailings at 25% moisture content as shown on Figure 15 and is expected to decrease as the moisture content of the tailings increases.

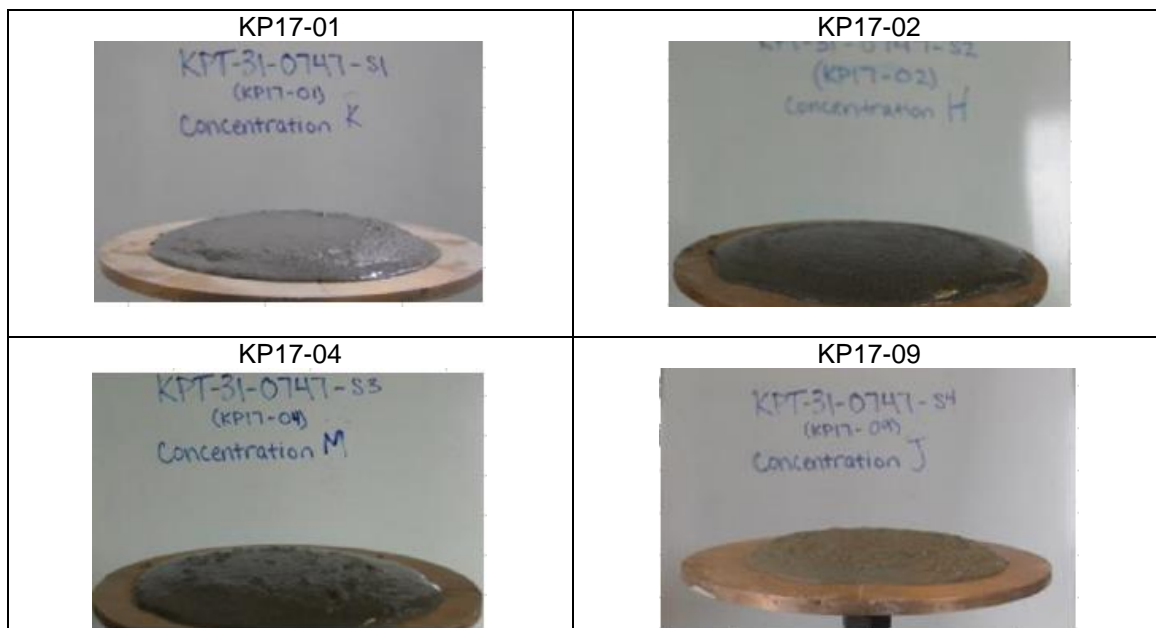


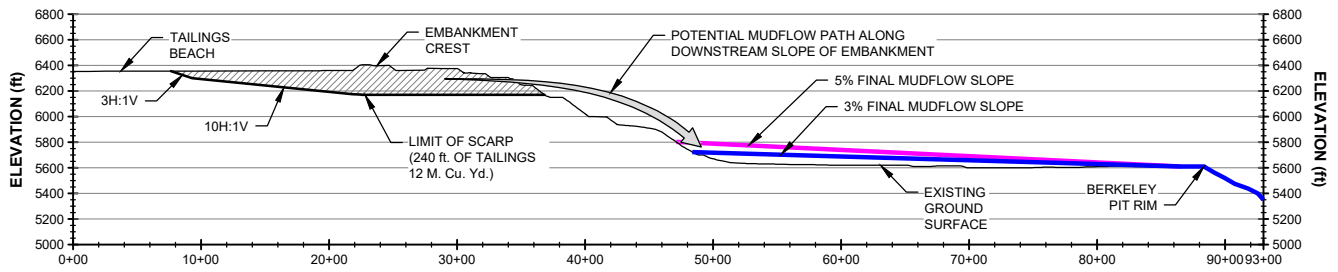
Figure 15 Dynamic Angle of Repose for Silty Sand Tailings with 25% Moisture Content

5.3 MUDFLOW MODEL

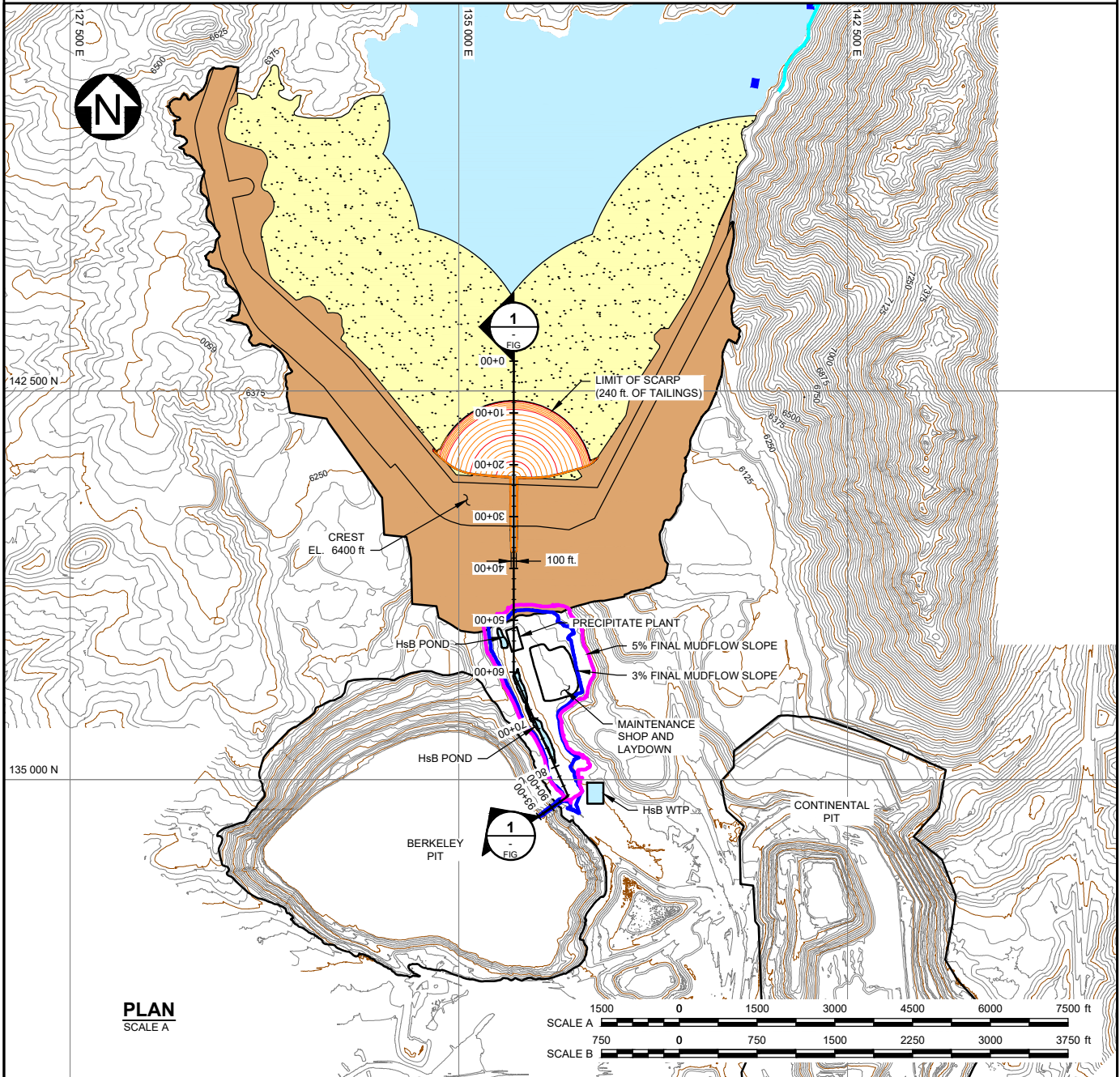
A conservative estimate of the potential consequences of a hypothetical mudflow from a breach of the East-West Embankment was developed considering the results of the tailings flowability assessment. The volume and extent of tailings that could potentially be mobilized after a hypothetical breach of the East-West Embankment were estimated based on the tailings static and dynamic angles of repose described above and available case histories, while considering potential flowability under sustained seismic loading of the Upper Medium Density Saturated Tailings zone to a depth of 240 ft. The consequence assessment specifically ignores the likelihood of occurrence, which is address in the Dam Breach Risk Assessment, and specifically assumes that a breach or vertical tear develops in the embankment exposing the full depth of tailings that are potentially flowable under dynamic conditions. The existing beach tailings under static stresses are expected to behave like soil and be non-flowable even if unconfined.

A 100 ft wide breach through the East-West Embankment was assumed along with final slope angles of approximately 33% (3H:1V) for the Upper Low Density Partially Saturated Tailings and 10% (10H:1V) for the Upper Medium Density Saturated Tailings. A gentler slope is a reasonable assumption for fully mobilized tailings at the base of the embankment. The final slope angle for transported mudflow tailings would be in the range of 3 to 5% based on case histories for mudflows generated for tailings dams, waste impoundments, and other earth structures (Lucia et al, 1981; Blight and Fourie, 2003).

The extents of this hypothetical mudflow would be limited to within the project area. The hypothetical mudflow would flow down gradient towards Berkeley Pit. The geometric conditions indicate that the mudflow would pass through the Horseshoe Bend (HsB) Area and bypass the HsB Water Treatment Plant (WTP). The mudflow would stop moving adjacent to the WTP, prior to the Berkeley Pit. The maximum potential limits are illustrated on Figure 16. The total volume of the mudflow would be in the order of up to 12 million cubic yards of tailings for a hypothetical 240 ft deep scarp. This hypothetical event would be preceded by an extreme earthquake ground motion directly in the vicinity of the project, which would initiate emergency response procedures even prior to the possible initiation of described potential consequences.



1
FIG
SECTION
PROFILE ALONG POTENTIAL MUDFLOW PATH
SCALE B



PLAN
SCALE A

LEGEND:

- TAILINGS BEACH
- EMBANKMENT FILL
- MINE WATER
- 5% FINAL MUDFLOW SLOPE
- 3% FINAL MUDFLOW SLOPE

NOTES:

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

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

**POTENTIAL CONSEQUENCES OF
HYPOTHETICAL MUDFLOW
PLAN AND SECTION**

Knight Piésold
CONSULTING

PIA NO. VA101-126/17 REF NO. VA18-00103

FIGURE 16

REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	19JAN'18	ISSUED WITH MEMO	ALA	SIR	DDF

6 – SUMMARY

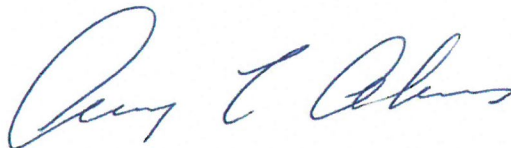
This memorandum summarized the study of the potential flowability of the YDTI tailings in the event of a hypothetical and sudden loss of containment at the East-West Embankment due to static or earthquake-induced deformation. The purpose of the study was to develop empirical information to assess the degree of mobility of the tailings through laboratory testing, which was considered along with tailings cone penetration testing (CPT) data and conventional cyclic liquefaction assessment to analyze the risks associated with the YDTI.

The tailings below the inferred phreatic surface are sufficiently dense to prevent flow in the event they become unconfined without a source of surface water to initiate erosion. The unsaturated or partially saturated tailings above the phreatic surface are not susceptible to flow. The existing beach tailings are expected to behave like soil under static stresses even if unconfined. These findings were corroborated by the tailings state characterization, which indicated that although the tailings materials are on the boundary of potentially contractive or dilative based on state parameter, the excess pore pressures required for contractive behavior are not present under existing conditions.

The tailings in the beach area below the unsaturated zone may have sufficient moisture content to flow in a viscous manner during active excitation (i.e. during seismic loading) without confinement. This zone of potentially flowable tailings is between approximately 60 ft and 240 ft depth below surface. The conventional cyclic liquefaction assessment indicated that potentially liquefiable tailings extend to approximately the same depth. Viscous flow is likely to stop without active shaking (i.e. at the end of the earthquake). Earthquake-induced embankment displacement would need to exceed 60 ft or more in a vertical direction to expose the upper medium density saturated tailings, which is well above the estimated maximum displacement levels. The tailings pore water will drain down and the thickness of the unsaturated zone will increase following closure, which progressively reduces the potential flowability of the tailings in the vicinity of the embankment. The beach tailings will eventually transform into a non-flowable mass following closure, which will preclude the development of any significant mudflow in the hypothetical event of a sudden breach at the East-West Embankment.

We trust this meets your needs at this time. Please contact the undersigned with any questions.

Prepared:



Amy Adams, Ph.D., P.Eng. – Project Engineer

Reviewed:



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

Approval that this document adheres to Knight Piésold Quality Systems:

DDF

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/ala

APPENDIX A

BOGER SLUMP YIELD STRESS TEST RESULTS

(Pages A-1 to A-4)

MC: 28%



MC: 35%



MC: 31%



MC: 44%



MC: 32%



MC: 53%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.

0	23AUG'17	ISSUED FOR INFORMATION	ALA	DDF
REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
BOGER SLUMP YIELD STRESS TEST RESULTS KP17-01		
Knight Piésold CONSULTING	P/ANO. VA101-126/17	REF. NO. VA18-00103
	FIGURE A-1	
		REV 0

MC: 27%



MC: 42%



MC: 31%



MC: 47%



MC: 36%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.

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REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
BOGER SLUMP YIELD STRESS TEST RESULTS KP17-02		
<i>Knight Piésold</i> CONSULTING	P/A NO. VA101-126/17	REF. NO. VA18-00103
	FIGURE A-2	
		REV 0

MC: 31%



MC: 38%



MC: 35%



MC: 44%



MC: 36%



MC: 49%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.

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REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
BOGER SLUMP YIELD STRESS TEST RESULTS KP17-03		
Knight Piésold CONSULTING	P/ANO. VA101-126/17	REF. NO. VA18-00103
	FIGURE A-3	
		REV 0

MC: 37%



MC: 60%



MC: 45%



MC: 68%



MC: 53%



MC: 86%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.

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REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
BOGER SLUMP YIELD STRESS TEST RESULTS KP17-04		
Knight Piésold CONSULTING	P/ANO. VA101-126/17	REF. NO. VA18-00103
	FIGURE A-4	
		REV 0

APPENDIX B

TRANSPORTABLE MOISTURE LIMIT TEST RESULTS

(Pages B-1 to B-4)

Before Test (typical)



MC: 20%

MC: 20%



MC: 21%

MC: 20%



MC: 22%

MC: 20%



MC: 22%



MC: 22%



MC: 23%



MC: 24%



MC: 25%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.

0	23AUG'17	ISSUED FOR INFORMATION	ALA	DDF
REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
TRANSPORTABLE MOISTURE LIMIT TEST RESULTS KP17-01		
Knight Piésold CONSULTING	P/ANO. VA101-126/17	REF. NO. VA18-00103
	FIGURE B-1	
		REV 0

Before Test (typical)



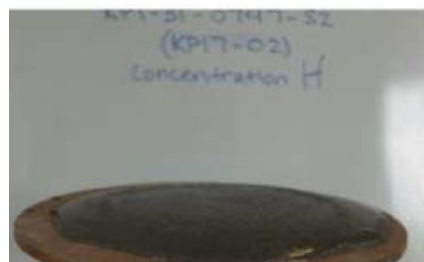
MC: 19%



MC: 21%



MC: 25%



MC: 12%



MC: 20%



MC: 22%



MC: 15%



MC: 20%



MC: 23%



MC: 18%



MC: 21%



MC: 24%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.

0	23AUG'17	ISSUED FOR INFORMATION	ALA	DDF
REV	DATE	DESCRIPTION	PREP'D	RW'D

MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
TRANSPORTABLE MOISTURE LIMIT TEST RESULTS KP17-02		
Knight Piésold CONSULTING	P/ANO. VA101-126/17	REF. NO. VA18-00103
	FIGURE B-2	
		REV 0

Before Test (typical)



MC: 22%

MC: 15%



MC: 24%

MC: 17%



MC: 20%

MC: 19%



MC: 22%



MC: 23%



MC: 23%



MC: 24%



MC: 25%



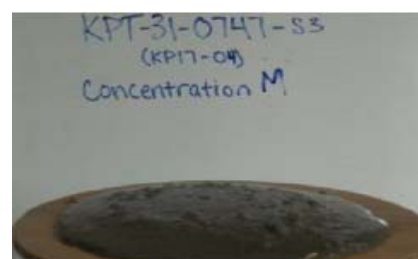
MC: 26%



MC: 26%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.



MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
TRANSPORTABLE MOISTURE LIMIT TEST RESULTS KP17-04		
Knight Piésold CONSULTING	P/ANO. VA101-126/17	REF. NO. VA18-00103
	FIGURE B-3	
		REV 0

0	23AUG'17	ISSUED FOR INFORMATION	ALA	DDF
REV	DATE	DESCRIPTION	PREP'D	RW'D

Before Test (typical)



MC: 24%



MC: 25%



MC: 27%



MC: 30%



MC: 32%



MC: 35%



MC: 39%

**NOTES:**

1. MC IS THE GRAVIMETRIC MOISTURE CONTENT BASED ON SOIL MECHANICS PRINCIPLES.

0	23AUG'17	ISSUED FOR INFORMATION	ALA	DDF
REV	DATE	DESCRIPTION	PREP'D	RWV'D

MONTANA RESOURCES, LLP.		
YANKEE DOODLE TAILINGS IMPOUNDMENT		
TRANSPORTABLE MOISTURE LIMIT TEST RESULTS KP17-09		
<i>Knight Piésold</i> CONSULTING	P/ANO. VA101-126/17	REF. NO. VA18-00103
	FIGURE B-4	
		REV 0