SITE CHARACTERIZATION REPORT

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# Montana Resources, LLP
## Yankee Doodle Tailings Impoundment
### Site Characterization Report

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EXECUTIVE SUMMARY

Montana Resources, LLP (MR) completed a phased geotechnical and hydrogeological site investigation program throughout 2015 and 2016 commensurate with the information needs to support the design and permitting of increased the storage capacity within the Yankee Doodle Tailings Impoundment (YDTI). The scope of the site investigation programs was adjusted and expanded as additional information related to the ground conditions became available. The findings of this site investigation work were integrated with information from previous site investigation work that was completed during various stages of design and construction of the YDTI between 1962 and the present.

There have been several investigations completed in the vicinity of the East-West and North-South Embankment areas spanning over five decades. The investigations were completed by several different engineering consultants in coordination with the mine operator of the time using a variety of methods. The recent drilling investigations collected continuous core samples of the embankment rockfill by sonic drilling through the existing embankment to natural ground beneath. The rockfill encountered was highly variable, and generally consisted of 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. The variability encountered during the recent investigations, and recognition of the potential for site wide variability and long-term degradation after mine closure suggests that it would be appropriate to adopt conservative shear strength parameters for the rockfill in the static stability analyses.

The elevation of the phreatic surface within the tailings beach gradually decreases southward from the supernatant pond towards the East-West Embankment. The direction of flow within the tailings beach is downward and towards the embankment. The phreatic surface within the embankment downstream of the crest sits deep within the embankment within the bottom 50 to 120 ft of rockfill, and is expected to vary depending on rockfill hydraulic conductivity and the original ground elevation along the base of the permeable embankment rockfill. Perched water conditions are expected within the embankment rockfill, particularly on historic road surfaces. Flow of seepage from the East-West Embankment and North-South Embankment is inferred to follow the historic drainages that pre-existed construction of the YDTI. The topography underlying the embankment suggests that seepage along the East-West and North-South Embankment alignments will flow towards the central embankment section following the historical surface topography and discharge from the embankment to Horseshoe Bend.

Piezometric elevations along the West Ridge typically range from 6,430 ft to 6,440 ft with elevations to the north and south of the ridge increasing to 6,460 ft and 6,480 ft. Groundwater flow in the West Ridge area is influenced by the presence of structural lineaments. A shallow piezometric low exists in the centre of the West Ridge and is structurally bounded to the north and south by east-west striking lineaments. The locally depressed groundwater elevations are interpreted to be a result of the structural boundaries impeding the flow of groundwater into the piezometric low from adjacent regions to the north and south. This effect is compounded by the relatively narrow surface topography of the West Ridge at this location, which is expected to limit meteoric recharge to the groundwater low and further reduce piezometric elevations within the low.
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MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDERMENT

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ABBREVIATIONS

ACC .......................................................... Anaconda Copper Company
ASTM .......................................................... American Society for Testing and Materials
BGS ............................................................ Below Ground Surface
BQM ............................................................ Butte Quartz Monzonite
Cc ............................................................. compression index
CPT ............................................................ Cone Penetration Testing
CD ............................................................. consolidated drained
CU ............................................................. consolidated undrained
DEQ .......................................................... Department of Environmental Quality
DH ............................................................. Drillhole
eVST .......................................................... Electronic Vane Shear testing
EL ............................................................. elevation
EOE ............................................................ Engineer of Record
ft ............................................................... Feet
Golder .......................................................... Golder Associates
GWIC ........................................................ Ground Water Information Center
HLA ........................................................... Harding Lawson Associates
HsB ............................................................ Horseshoe Bend
Hydrometrics .............................................. Hydrometrics, Inc.
IRP ........................................................... Independent Review Panel
IT ............................................................. Infiltration Tests
IECO ........................................................ International Engineering Company Inc.
KP ............................................................. Knight Piérdol Ltd.
Kes ........................................................... Hydraulic Conductivity
MCE .......................................................... Maximum Credible Earthquake
m/s ............................................................. Meters per second
MW .......................................................... Monitoring Well
MBMG ...................................................... Montana Bureau of Mines and Geology
MCA ........................................................ Montana Code Annotated
MPa ........................................................... Megapascal
MR ............................................................ Montana Resources, LLP
SBTn ........................................................ Normalized Soil Behavior
PSD .......................................................... Particle Size Distribution
BCPTu ........................................................ Penetration Testing
CPTu ........................................................ piezocone penetration testing
PPD .......................................................... Pore Pressure Dissipation
Pc ............................................................. pre-consolidation pressure
R7W .......................................................... Range 7 West
RMR .......................................................... Rock Mass Rating
RQD .......................................................... Rock Quality Designation
SOIL ......................................................... Setting Out Line
SM ........................................................... Silty Sand
SI ............................................................. Site Investigation
SPT ................................................................. Standard Penetration Test
TP ................................................................. Test Pit
TAC ................................................................. The Anaconda Company
t/m³ ................................................................. Tonnes per cubic meter
tsf ................................................................. Tonnes per square foot
T3N ................................................................. Township 3 North
T4N ................................................................. Township 4 North
Ueq ................................................................. equilibrium pore pressure
UCS ................................................................. Unconfined Compressive Strength
USACE ............................................................ US Army Corps of Engineers
VWP ................................................................. Vibrating Wire Piezometer
WED ................................................................. West Embankment Drain
YDTI ................................................................. Yankee Doodle Tailings Impoundment
1 – INTRODUCTION

1.1 INTRODUCTION

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 approximately 50,000 short tons per day and a small-scale leaching operation. The project is located in Butte, Silver Bow County, in Sections 5 and 6 Township 3 North (T3N), Range 7 West (R7W) and Sections 31 and 32 Township 4 North (T4N), Range 7 West (R7W) of the Montana Principal Meridian. The site is bounded by Interstate 15 and the Continental Divide on the east, Moulton Reservoir Road on the west, and Farrell Street, Continental Drive and Shields Avenue to the south.

1.2 YANKEE DOODLE TAILINGS IMPOUNDMENT (YDTI)

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.

The mine has the relevant operating permits for continued mining in the Continental Pit, rockfill disposal areas and ancillary facilities with the exception of the YDTI. An amendment to the operating permit for the YDTI is required to allow continued filling of the facility to an embankment crest of EL. 6,450 ft. A plan view showing the conceptual layout of the facility at this embankment crest elevation is included on Figure 1.2.
NOTES:
1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
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LEGEND:
- TAILINGS BEACH
- TAILINGS DEPOSITION
- EMBANKMENT FILL
- ROCK DISPOSAL SITE
- TAILINGS PIPELINE
- RECLAIM PIPELINE
- MINE WATER
- RECLAIM BARGE

SCALE: 1500 ft
0 1500 3000 4500 6000 7500 ft

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
FUTURE FACILITY LAYOUT
1.3 LEGISLATED REQUIREMENTS

Montana Code Annotated (MCA) is a codification and compilation of existing Montana state general and permanent law. MCA is arranged topically and is continuously rearranged to maintain an orderly and logical arrangement. 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 design (MCA, 2015). The requirements include:

“A site geotechnical investigation commensurate in detail and scope with the complexity of the site geology and proposed tailings storage facility design. The investigation must include a geological model of site conditions and a rationalization of the site investigation process”

This report fulfills the above requirements of the legislation and also provides available details of historic investigations dating back to the early 1960s.

1.4 INDEPENDENT REVIEW PANEL

An Independent Review Panel (IRP) for the YDTI has been selected. The IRP consists of three independent review engineers or specialists, as stipulated by Montana Code Annotated (MCA) Title 82 Chapter 4 Part 3 Section 76. The members of the MR IRP are as follows:

- Dr. Dirk Van Zyl
- Dr. Leslie Smith, and
- Mr. Jim Swaisgood.

1.5 ENGINEER OF RECORD

The requirement for an Engineer of Record (EOR) for the YDTI is described in Montana Code Annotated (MCA) Title 82 Chapter 4 Part 3 Section 75 (MCA 82-4-375). The EOR is required to be a Professional Engineer licensed in the State of Montana. The EOR for the YDTI is Mr. Ken Brouwer, P.E., of Knight Piésold Ltd.

The EOR is responsible for the following:

- Review the design and other documents pertaining to the tailings storage facility.
- Certify and seal designs or other documents pertaining to the tailings storage facility submitted to the Department of Environmental Quality (DEQ).
- Complete an annual inspection of the tailings storage facility.
- Notify the operator when credible evidence indicates the tailings storage facility is not performing as intended.
- Immediately notify the operator and the DEQ when credible evidence indicates that the tailings storage facility presents an imminent threat or a high potential for imminent threat to human health or the environment.

1.6 COORDINATE SYSTEM

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

1.7 PURPOSE AND SCOPE

This Site Characterization Report summarizes the relevant historical and recent geotechnical and hydrogeological investigations that support the design of the YDTI.

MR completed a phased geotechnical site investigation program throughout 2015 and 2016 commensurate with the information needs to support the design and permitting of the YDTI. The 2015 and 2016 programs were completed in the following phases:

- Phase 1: West Embankment test pit and test trench excavations
  - Phase 1A: Test pit excavation
  - Phase 1B: Test trench excavation
- Phase 2: West Embankment and West Ridge drilling and Vibrating Wire Piezometer (VWP) installation
  - Phase 2A: West Embankment drilling
  - Phase 2B: West Embankment infill drilling
  - Phase 2C: West Ridge drilling
- Phase 3: East-West Embankment sonic drilling and VWP installation
- Phase 4: Cone Penetration Testing (CPT) and VWP installation in the YDTI, and
- Phase 5: West Embankment and West Ridge infill drilling and VWP installation.

The findings of this site investigation work were integrated with information from previous site investigation work that was completed during various stages of design and construction of the YDTI between 1962 and the present. A complete summary of the investigations, including historic information and recent work to support permitting, is provided in Section 2.

The main objectives of this report are to:

- Describe the physiography, site geology, and known faults in the vicinity of the YDTI
- Describe the surficial material (alluvium) properties and distribution underlying the YDTI
- Describe the geotechnical and hydrogeological conditions along the West Ridge
- Describe the foundation conditions for the existing rockfill embankment
- Describe the geotechnical properties and hydrogeological conditions for the existing rockfill embankment
- Describe the geotechnical and hydrogeological conditions of the tailings contained in the YDTI, and
- Describe the instrumentation monitoring network.
2 – REVIEW OF SITE INVESTIGATION PROGRAMS

2.1 SUMMARY OF INVESTIGATIONS

2.1.1 Previous Investigations

Site investigation programs were conducted by several different engineering consultants in coordination with the mine operator of the time using a variety of methods. The work spans over five decades from 1962 to present, and typically coincided with certain periods of project development. It is also possible that other site investigation work was carried out by ACC before MR took ownership of the property in 1986; however, only the known, available, and useful investigative studies are described in this report. Some programs included a number of drillholes and laboratory test work while others were literature reviews of previous investigations. The site investigation programs are summarized by report in Table 2.1. The available logs for the historic drillholes, standpipe piezometers, monitoring wells, and test pits are compiled by report in Appendix A, and the locations are shown on Figure 2.1.

Dames and Moore (1962) conducted a feasibility study of the proposed tailings pond prior to construction which characterized the site with five drillholes (62 T-1 to 62 T-5), laboratory test work and alluvium mapping. Dames and Moore (1963) completed an additional drillhole (62-T6) and continued development of design and construction criteria of the tailings impoundment.

M.K. Botz (1969a) investigated the geology and hydrogeology of the Upper Silver Bow Creek drainage basin and produced a report, Bulletin 75, published by the Montana Bureau of Mines and Geology (MBMG). M.K. Botz (1969b) performed drilling investigations beneath the South and Main leach pads to determine the condition of the asphalt pad beneath the Main leach pad and the water transmitting ability of the alluvium beneath both leach areas. Botz and Knudson (1970) conducted an investigation of the hydrogeology of the Berkeley Pit area on behalf of ACC. The only information presently available from the latter report is a map of the surficial geology and bedrock contours in the Berkeley Pit area, which is included in Appendix C.

Golder Associates (Golder) conducted a site investigation including three drillholes (311-398, 297-390/B) on the alluvium deposits south of the Berkeley Pit and completed laboratory test work on select samples to determine geotechnical strength characteristics of the alluvial material for pit wall stability evaluations. The test work included particle size analysis, moisture content, in-situ density, and triaxial tests. The original purpose of the investigation was to provide shear strength characteristics for the alluvium for the evaluation of pit wall stability for a proposed expansion of the Berkeley Pit (Golder, 1980).

The next phase of investigation was completed in 1980 by International Engineering Company Inc. (IECO, 1981). The IECO study of geotechnical and hydrological conditions was initiated in response to an inspection of the YDTI by the US Army Corps of Engineers (USACE, 1980). Fourteen drillholes (DH-1 to DH-13, and DH-6A) were completed in the vicinity of the East-West and North-South Embankment areas with a focus on drilling along the embankment crest and the maximum embankment section at the time. The report by IECO included a description of regional and site geology, described regional seismicity and established seismic design criteria for the seismic stability analyses.
Goldberg (1990) completed an engineering literature review of the YDTI embankment and proposed a site investigation program to establish an instrument and monitoring network.

Harding Lawson Associates (HLA) completed two boreholes in 1992 (92-1 and 92-2B) using a mobile drill equipped with a casing advance system to compliment the studies performed by IECO and to further investigate the in-place density of the tailings at the upstream edge of the North-South Embankment (HLA, 1993). HLA also updated the seismic design criteria for the YDTI.

MR installed standpipe piezometers in 1993 and 1994 to define and monitor the pore pressure conditions within the tailings embankment subsequent to the HLA study. Piezometers were constructed at seventeen locations in the East-West Embankment and five locations in the tailings basin upstream of the North-South Embankment. The pore pressure monitoring network for the embankment was expanded by MR between 2005 and 2014 to replace damaged or abandoned wells. Eleven standpipe piezometers were installed in the vicinity of the North-South Embankment and three were installed in the East-West Embankment.

Hydrometrics, Inc. (Hydrometrics) performed a hydrologic evaluation of the West Ridge area of the YDTI to support the design. The work by Hydrometrics included evaluations of surface hydrology patterns and seep surveys in the vicinity of the West Ridge, physiography and geology of the ridge and up-slope drainage basins, and hydrogeology of the West Ridge area. The work also included review of information on existing residential wells from the MBMG Ground Water Information Center (GWIC) database. Eight monitoring wells were installed on the West Ridge in 2012 (MW12-11 to MW12-18), and a draft report (Hydrometrics, 2014) was prepared. Evaluation of the site hydrogeology continued in 2015 with the installation of an additional 13 monitoring wells (MW15-01 to MW15-13) and extension of the monitoring well network north and east of the YDTI supernatant pond. Three additional monitoring wells were completed in the West Ridge area in 2016 (MW16-01, MW16-02D, and MW16-02S) for a total of 24 monitoring wells completed at the site by Hydrometrics.

KP completed annual site investigation programs between 2012 and 2014 using the ODEX drilling technique with periodic SPT and CPT soundings (KP, 2013; KP, 2014; KP, 2016d). The CPT equipment and methods used during these programs allowed for measurement of penetration (tip) resistance, sleeve friction, and the dynamic pore pressure generated in the tailings. Pore pressure dissipation (PPD) testing was completed every 10 to 15 ft and seismic shear wave velocity measurements were completed every 3 ft. The primary focus of these programs was to collect field information to support geotechnical and hydrogeological characterization of the tailings mass adjacent to the embankment for use in updated liquefaction and stability assessments. The areas investigated were mainly along the East-West Embankment and along the North-South Embankment in the vicinity of historic drillholes by IECO and HLA. The site investigation work included six drillholes, 16 CPTs, and eleven test pits.
### Table 2.1 Site Investigation Summary

<table>
<thead>
<tr>
<th>Report Objectives</th>
<th>Site Investigations</th>
</tr>
</thead>
</table>
| **Dames and Moore (1962)** | • 5 drillholes (62-T1 to 62-T5)  
Characterized the North Embankment and Horseshoe Bend, described surface geology and foundation conditions prior to substantial YDTI development, evaluated the stability of embankments.  
Index and shear testing  
Permeability testing  
1 Test Pit |
| **Dames and Moore (1963)** | • 1 drillhole (62-T6)  
Index testing  
Summarized subsurface conditions of YDTI based on 1962 site investigation results, developed design and construction criteria for YDTI, and developed tailings deposition plan.  
Laboratory test work reportedly included shear strength and permeability testing of alluvium, but results unavailable. |
| **Botz (1969a, 1969b and 1970)** | • 9 drillholes (SD-1 to SD-3, MD-1 to MD-5, and MD-5A)  
Drilling investigation of asphalt pad and alluvium beneath leach pads downstream of the YDTI.  
Laboratory test work including consolidated drained triaxial tests on alluvium samples |
| **Golder Associates (1980)** | • 3 drillholes (311-398, 297-390, and 297-390B)  
Characterized the alluvium strength for East Berkeley Pit Expansion  
Laboratory test work including index, direct shear and triaxial shear testing  
Standpipe piezometer installations |
| **IECO (1981)** | • 14 drillholes (DH-1 to DH-13 and DH-6A)  
Evaluated site geology and seismicity, described foundation conditions and characteristics of existing embankment, and evaluated the stability of existing embankment for an expansion.  
Various sampling devices  
Index, direct shear and triaxial shear testing  
Standpipe piezometer installations |
| **Goldberg (1990)** | None  
Engineering literature review of the embankment |
| **Harding Lawson Associates (1993)** | • 2 drillholes (92-1 and 92-2B)  
Investigated in-place density of tailings along North-South Embankment and updated seismic design criteria |
| **MR Engineering Department (1993 to 1994)** | • 22 standpipe piezometers (93-1 to 93-5, 94-1 to 94-12, 94-B1 to 94-B5)  
Water level monitoring  
Investigated pore pressure conditions in the YDTI and East-West Embankment. |
| **MR Engineering Dept. (1999)** | • Summarize existing information  
Stability analysis modeling for static and pseudo-static conditions  
Tailings auger boring sampling  
Evaluation of YDTI embankment stability throughout design life, review of the foundation, embankment, and tailings engineering properties and seismic hazard analysis. |
| **Hydrometrics (2012)** | • 8 monitoring wells (MW12-11 to MW12-18)  
Groundwater sampling of monitoring wells  
Characterize West Ridge area hydrogeology, establish baseline hydrogeology and groundwater chemistry data and review of existing residential wells from MBMG and GWIC database. |
| **Knight Piésold (2012)** | • 4 drillholes with SPTs (DH12-01B, DH12-03, DH12-04, DH12-05A)  
4 CPTs (CPT12-01A, CPT12-03A, CPT12-04, CPT12-04A, CPT12-05)  
Index tests on tailings sand  
3 VWP installations (CPT12-03A, 04 and 04A)  
Investigated tailings material and hydrogeological conditions within YDTI. |
Report Objectives | Site Investigations
---|---
Knight Piésold (2013) Investigated tailings material and hydrogeological conditions within YDTI. | • 2 drillholes with SPTs (DH13-07 and DH13-09) • 6 CPTs (CPT13-01 to 06) • 3 settlement monitoring points on test pads • 6 VWP installations (CPT13-01 to 06) • Index tests on tailings sand

Knight Piésold (2014) Investigated deep tailings material of YDTI, tailings material north of ‘Rocky Knob’, collected tailings samples for shear testing, investigated the strength of embankment fill near Horseshoe Bend, and assessed hydrogeological conditions within YDTI. | • 5 CPTs (CPT14-01, CPT14-01A, CPT14-02, CPT14-04, CPT14-05) • 11 test pits at Horseshoe Bend (TP14-01 to TP14-11) • 3 VWP installations (CPT14-01A, 02, 04) • 5 index tests on tailings sand, 8 on rockfill, proctor and triaxial test on rockfill

Hydrometrics (2015) Investigated West Ridge area hydrogeology, established baseline hydrogeology and groundwater chemistry data and established inventory and quality of nearby residential wells. | • 13 monitoring wells (MW15-01 to MW15-13)

Knight Piésold (2015) Investigated West Ridge geotechnical and hydrogeological conditions, East-West Embankment and foundation conditions, tailings impoundment materials and installed VWP monitoring network. Identified presence of low piezometric conditions in a deep isolated fracture system. | • 14 drillholes (DH15-01 to DH15-14) • 6 sonic drillholes (DH15-S1 to DH15-S6) • 8 CPTs (CPT15-01 to CPT15-08) • 33 test pits (TP15-01 to 14, 18 to 31, 33, 34, and 38 to 40) • 11 trenches (T-1 to T-11) • 76 VWP installations • Laboratory test work

Knight Piésold (2016) Investigated West Ridge area hydrogeology targeting deep isolated fracture system identified by KP investigation in 2015. | • 5 drillholes (DH16-01 and DH16-05) • 25 VWP installations

Hydrometrics (2016) Investigated West Ridge area hydrogeology targeting deep isolated fracture system identified by KP investigation in 2015. | • 15 trenches T-12 to T-26 • 3 monitoring wells (MW16-01, MW16-02D and MW16-02S)

2.1.2 2015 Site Investigations

MR completed a 2015 phased site investigation program to evaluate geotechnical and hydrogeological conditions to support the design and permitting of the YDTI. KP provided geotechnical and hydrogeological field services including drilling supervision, logging, in-situ testing, and instrument installation. The complete 2015 geotechnical and hydrogeological site investigation program is presented in the following documents:

- Phase 1A – West Embankment test pit program to investigate foundation materials (test pits TP15-01 to 14, 18 to 31, 33, 34, and 38 to 40). KP memo “Phase 1A West Embankment Test Pit Program Summary”, Ref. No. VA15-03370 dated December 24, 2015.
- Phase 1A Addendum - West Embankment foundation infiltration tests (IT-1 to IT-6). KP memo “Addendum to Phase 1A West Embankment - Infiltrometer Testing”, Ref. No. VA16-00184 dated May 12, 2016.
- Phase 1B – West Embankment trenching program to investigate structural lineaments (trenches T-1 to T-11). KP memo “Phase 1B West Embankment Trench Program Summary”, Ref. No. VA15-03525 dated January 21, 2016.
• Phase 4 – Cone Penetration Testing (CPT) program to investigate the tailings material within the YDTI (drillholes CPT15-01 to CPT15-08). KP memo “Phase 4 Tailings Impoundment SCPT Program Summary”, Ref. No. VA16-00014 dated March 18, 2016.

Geotechnical site investigation programs carried out in 2015 collected significant amounts of data to characterize the geology, hydrogeology, and geotechnical conditions. Memos presenting the factual data are included in Appendix D. The logs from the drillholes and test pits have been compiled with the available historic logs in Appendix A.

2.1.3 2016 Site Investigations

Hydrometrics and KP conducted additional site investigation work in 2016 to evaluate the presence of a zone with low piezometric conditions in the West Ridge. The 2016 site investigation work is presented in the following documents:
• Phase 5 – Five hydrogeology drillholes (DH16-01 and DH16-05) and installation of 25 VWPs to investigate structural geology and hydrogeology in West Ridge. “Phase 5 West Embankment Hydrogeological Drilling Program Summary”, Ref. No. VA16-00856 dated November 14, 2016.
• Trench program completed by MR and Hydrometrics based on information from surface exposures created by topsoil removal, structural projections from trenches/drillholes, and geologic maps on file at MR. The trench program is reported in the following memos included in Appendix D:
  o Trenches T-12 to T-19 are reported in “Additional West Ridge Trenching; 2/24/16 to 3/29/16” dated April 6, 2016.
  o Trench T-20 is reported in “West Ridge Trenching; Trench T-20” dated July 22, 2016.
  o Trench T-21 is reported in “West Ridge Trenching; Trench T-21” dated September 21, 2016.
  o Trenches T-22 to T-26 are reported on in “West Ridge Trenches T-22 through T-26” dated January 11, 2017
• Three monitoring wells were installed by Hydrometrics in 2016 (MW16-01, MW16-02S and MW16-02D). Monitoring well MW16-02D was constructed as a 6-inch diameter well to facilitate hydraulic testing. A pumping test and a recharge test were conducted at the well. Well construction and testing details are presented in a standalone report by Hydrometrics entitled “Hydrogeologic Evaluation of the Yankee Doodle Tailings Impoundment West Ridge Area Silver Bow County” (Hydrometrics, 2017).
Memos presenting the factual data from the Phase 5 drilling program and trenching investigations are included in Appendix D. The logs from the drillholes and monitoring wells have been compiled with the available historic logs in Appendix A.

2.2 GEOLOGIC SETTING

An overview of the regional geologic setting of Silver Bow County was prepared by MBMG (MBMG, 2009). A summary of relevant information is provided in this section. The report by MBMG compiles details from the work of several other authors and provides the additional references to that work, which have not been reproduced in this summary. Silver Bow County lies within the Rocky Mountains of the Cordillera or “backbone” of the North American continent. Local geology is affected by tectonic zones and features including those described below and shown in Figure 2.2:

- The Basin and Range Province: a broad zone of crustal extension west of the Rocky Mountains that extends into southwestern Montana. The Montana extension direction is East-Northeast to West-Southwest. The Basin and Range began to form about 17 million years ago.
- The Great Falls Tectonic Zone: a broad northeast trending belt from Salmon, Idaho to Great Falls, Montana to Saskatchewan, Canada. A zone of distinctive igneous rocks and northeast trending faults. Some faults in this zone have moved as recently as the Quaternary Period.
- Helena Salient: a bulge of Front Range style folding and thrust faulting that protrudes eastward into the Rocky Mountain Foreland.
- Boulder batholith: a large Cretaceous granitic mass that dominates Silver Bow County.
- Lewis and Clark Line: a topographic and geologic lineament that extends over 500 miles from Washington through central Montana, and passes just north of Silver Bow County.
- Intermountain Seismic Belt: a zone of elevated seismicity that encompasses western Montana.

The geology of Silver Bow County is both diverse and distinctive. It is diverse in the variety of rock types and geologic structures, and distinctive in its location at the intersection of these numerous geologic provinces. The county is divided into five geologic domains based on rock type and age as follows:

- Precambrian metamorphic rocks
- Paleozoic and Mesozoic sedimentary rocks
- Boulder batholith
- Lowland Creek volcanic rocks, and
- Cenozoic valley sediments.
1. Figure reproduced from Figure 3.1 of the Montana Bureau of Mines and Geology Open-File Report 585: Geologic Map and Geohazard Assessment of Silver Bow County, Montana (MBMG, 2009).

**Figure 2.2**  Tectonic Zones of Western Montana
The Boulder batholith dominates Silver Bow County and is the relevant geologic domain for the YDTI area. It is composed of a number of plutons, the largest of which is Butte Granite, which is 74 to 76 million years old, and includes a number of smaller bodies. Intrusion of the multiple plutons that form the batholith was part of the tectonic event that built the Rocky Mountains and caused the low-grade metamorphism that produced quartzite, marble, and hornfels bodies where granitic magma heated sedimentary rocks.

Boulder batholith rocks are mechanically homogeneous where intact, but jointing has created dominant planes of weakness that control rock behaviour near the surface. The dominant joint sets are vertical and north-south trending, but other orientations exist.

2.3 PHYSIOGRAPHY

Upper Silver Bow Basin is a drainage basin that consists of a relatively flat alluvial valley surrounded by mountains. The basin is bounded on the east by a steep ridge, known locally as the East Ridge, which in several places exceeds 8,000 ft in altitude and is dissected by numerous small streams. The alluvial-filled central valley is approximately 3.5 miles wide and 7 miles long. The alluvium in the valley is derived from weathering and erosion of rocks from the adjacent mountains. Surface slopes on the alluvium steepen rapidly toward the surrounding mountains. Terraces within the valley alluvium may have resulted from increased stream competence during pluvial conditions, but other geologic factors, such as faulting, may have contributed to the present valley topography (Botz, 1969a).

The YDTI is located approximately two miles northeast of Butte, Montana and is immediately to the north of the Berkeley Pit. The YDTI lies in the northern end of the basin near the historical confluence of Yankee Doodle Creek and Silver Bow Creek. Yankee Doodle Creek drains the northwestern portion of the up-slope basin, and Silver Bow Creek drains the eastern portion. A smaller drainage, Dixie Creek, drains a small basin between the two. Vegetation cover in the drainage basins includes grasses, sagebrush, and forests of pine and aspen. The soil mantle is thin over the majority of the basin.

The YDTI is bordered on all sides except the south by mountainous terrain. The eastern slopes are the steep terrain of Rampart Mountain rising up to the Continental Divide. Rampart Mountain is the upthrown side of the Continental Fault, which traces along the eastern edge of the YDTI along the valley floor (IECO, 1980). The West Ridge slopes are a relatively low ridgeline of rolling hills with elevations of approximately 6,550 ft.

2.4 SITE FAULTS

Silver Bow County has many geologic faults, some of which date back more than 1.7 billion years. Many have not moved for hundreds of millions of years. A few have moved more recently and must be considered to have the potential for future movement. It is impossible to say with certainty exactly when or how large the latest fault movement was, but there is evidence that faulting did occur in geologically recent times. The faults that have demonstrably moved most recently trend northeast and displace Quaternary sediments. These faults and others with the same orientation overprint the Rocker Fault and possibly the Continental Fault, based on relationships observed outside the county in Elk Park. Northeast-trending faults in Silver Bow County belong to a broad northeast-southwest belt of faults associated with the Great Falls Tectonic Zone (MBMG, 2009).
The faults that created Rampart Mountain, Elk Park, and the East Ridge form a linked fault zone that generally strikes in a north-south direction (MBMG, 2009). The faults in this linked zone that are located in the vicinity of the YDTI are the Continental Fault, Klepper Fault and the East Ridge Fault as shown on Figure 2.3. All three are normal faults. The Continental Fault and Klepper Fault intersect the Continental Pit located south of the YDTI. The Continental Fault traces along the eastern edge of the YDTI below the North-South Embankment along the East Ridge beneath an apron of granitic alluvium. The Continental Fault is overprinted by the northeast-trending Rampart Fault in the vicinity of the North-South Embankment. The East Ridge Fault lies further up the slope and trends parallel to the East Ridge (MBMG, 2009).

Overprinting of the Continental and Rocker Faults by the northeast faults does not necessarily mean that the former are now inactive. The northeast faults are appropriately oriented to act as transfer faults that accommodate movement on offset segments of the larger Continental and Rocker Faults (MBMG, 2009).

The timing of movement on the Continental Fault is constrained only by the youngest dated geologic unit known to predate movement, which is the 58.8 million-year-old dike reported by Czehura (2006). The fault cuts the alluvium in the Continental Pit, and because no dikes are known to intrude the alluvium it is reasonable to conclude that the fault moved more recently than 58.8 million years ago, but how recently is unknown. The strongest evidence suggestive of movement in the past 1.8 million years is the overall character of the range front (MBMG, 2009).

The Rocker Fault is located to the southwest of the YDTI at a distance of over 8.5 km (Al Atik and Gregor, 2016). The evidence related to movement on the Rocker Fault during the Quaternary period is more convincing than the Continental Fault. Surface exposures suggest the Rocker Fault has had ground-rupturing movement within the past 1.8 million years. The timing of the last movement was constrained to somewhere between 130,000 years and 5 million years through a review of previous studies, and is based primarily on age dating of faulted and undisturbed sediments along the fault (MBMG, 2009).

The Continental and Rocker Faults were included in the seismic hazard source models for the design, primarily due to the proximity to the YDTI site and because they were included as active sources in past studies (Al Atik and Gregor, 2016).
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 25 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

LINEAMENT TRACE (HYDROMETRICS, 2014)
PROPERTY BOUNDARY

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
UPPER SILVER BOW CREEK BASIN
PHYSIOGRAPHY AND FAULT STRUCTURES

FIGURE 2.3
2.5 SEISMIC HAZARD

The design earthquake has been selected to meet the obligations as stipulated in MCA 82-4-376 (2), (m), (i) and (l). The legislation requires a probabilistic and deterministic seismic evaluation for the area and assessment of peak horizontal ground acceleration. The legislation requires either of the following for the expansion of an existing tailings storage facility:

- An analysis showing the proposed design meets the minimum design requirements for a new tailings storage facility, or
- An analysis showing the proposed design does not reduce the original design factors of safety and seismic event design criteria.

The requirement for a new tailings storage facility is for an analysis showing that the seismic response of the tailings storage facility does not result in the uncontrolled release of impounded materials when subject to the ground motion associated with the 1 in 10,000 year event, or the maximum credible earthquake (MCE), whichever is larger.

The seismic event design criteria for the YDTI have been updated periodically. The latest criteria preceding the recent seismic hazard assessment described below were developed by HLA (HLA, 1993). HLA prepared a deterministic estimate of the MCE for movement along the Continental Fault. The study defined the MCE as a Magnitude 6.5 event with a peak bedrock acceleration of 0.6 g.

MR chose to update the seismic event design criteria although it was not required by the legislation. An updated seismic hazard analysis was considered prudent at this time to demonstrate that the YDTI meets state-of-practice engineering design standards due to the close proximity of the Continental Fault and developments in seismic hazard assessment methods since 1993. A site specific probabilistic and deterministic seismic hazard analysis was conducted as part of the YDTI design. The report (Al Atik, L. and Gregor, N., 2016) is included as Appendix B. The study included derivation of the following seismic response spectra:

- Probabilistic spectra with return periods of 475, 1,000, 2,475, and 10,000 years, and
- Deterministic 50th (median) and 84th percentile response spectra for the MCE scenarios on the Continental fault with rupture distances of 1.2 and 0.1 km.

The resulting peak ground accelerations of the seismic hazard analyses are summarized in Table 2.2, and the horizontal design spectra for the YDTI are shown in Figure 2.4.

**Table 2.2 Summary of Probabilistic and Deterministic Seismic Hazard Analysis**

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Probabilistic UHS PSA (g)</th>
<th>Deterministic PSA (g) Rrup = 1.2 km</th>
<th>Deterministic PSA (g) Rrup = 0.1 km</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA</td>
<td>Median</td>
<td>84th Percentile</td>
</tr>
<tr>
<td>475</td>
<td>0.08</td>
<td>0.42</td>
<td>0.84</td>
</tr>
<tr>
<td>1,000</td>
<td>0.12</td>
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</tr>
<tr>
<td>2,475</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>10,000</td>
<td>0.37</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
1. Peak ground accelerations are for rock site conditions (\(V_{S30} = 760\) m/s).
2. Source: Table 6-2 of Al Atik, L. and Gregor, N., 2016 included as Appendix B.

Figure 2.4 shows the horizontal design spectra for the YDTI. The deterministically derived MCE spectra exceed those for the probabilistically derived 1 in 10,000 year event. The MCE was therefore
selected as the design earthquake. The MCE with a rupture distance of 0.1 km produces spectral accelerations that are greater than the MCE with a rupture distance of 1.2 km.

![Horizontal Spectra](image)

**NOTES:**
1. Source: Figure 6-2 of Al Atik, L. and Gregor, N., 2016 included as Appendix B.

**Figure 2.4 Probabilistic and Deterministic Horizontal Design Spectra**

The selection of the design earthquake is described in the Design Basis Report (KP, 2017a). The analysis demonstrating the seismic response of the facility, and describing the loading conditions and methods of analysis is provided in the Stability Assessment Report (KP, 2017b).

### 2.6 SITE GEOLOGY

Three main material types are present in the foundation of the YDTI study area:
- Competent Bedrock
- Weathered Bedrock, and
- Alluvium.
2.6.1 Bedrock

The basin and surrounding mountains lie entirely within the intrusive rock for the Boulder Batholith and have been subjected to intense faulting. The rock of the batholith is a composite body made up of numerous separate plutons. Butte Quartz Monzonite (BQM) is the dominant rock type, but the intrusive mass is cut by many alaskite and aplite bodies of various sizes, shapes, alterations and textures. In the upper areas of Silver Bow Creek Basin, the batholith intrudes Paleozoic sedimentary rock and Upper Cretaceous volcanic rock. The batholith is approximately 70 million years old and was emplaced subsequent to the formation of Rocky Mountains at the end of the Cretaceous Period (Botz, 1969a).

The bedrock consists of two distinct subunits or zones, a weathered or leached zone, and a competent zone. The weathered zone constitutes the uppermost portion of the bedrock unit immediately underlying the alluvium, where the alluvium is present, and often has an iron-stained appearance. The competent zone underlies the weathered zone and is usually grey in colour and is very hard. The weathered zone was exposed to natural chemical and physical processes which decomposed or weathered the upper portion. Eventually, the upper portion of the bedrock unit became a weathered zone tens of ft thick (the resulting alteration material of decomposed plagioclase and potassium feldspar) interspersed with gravel to cobble sized rock fragments of remnant, non-altered quartz monzonite (Canonie, 1994).

Mountains on the northern skyline of the tailings basin are capped with Tertiary volcanics that overlap and partly conceal the quartz monzonite basement. Intrusive rhyolite dikes are seen at the base of the East Ridge (MR, 1999). Aplite dikes have been mapped more recently by MR at the northern end of the YDTI as shown previously on Figure 2.3.

The valley slopes in the vicinity of the YDTI are generally formed by granitic bedrock covered by a layer of residual soils up to 10 ft thick. These residual soils are the foundation for the embankment in many places, and probably are a result of slope wash deposition or weathered-in-place granite (Dames and Moore, 1962). The residual soils are overlain in places by alluvium as described below.

2.6.2 Alluvium

The Tertiary to Recent age alluvium is derived from weathering and erosion of the igneous terrain. The alluvial deposits may include landslide debris, talus, and braided fan gravels transitional to lake beds extending into the central portion of the Butte-Silver Bow valley. The stratigraphy within the alluvium varies through the valley. Interlayered mixtures of sand, gravel, clayey sand and silt are common in localized braidplains where sections have been exposed by mining operations. Fan gravels, characterized by matrix supported gravels and cobbles, appear where steeper drainages outwash into the valley. Alluvial deposits generally consist of poorly sorted angular sands interbedded with clayey and silty sands (MR, 1999).

Recent stream deposits are present along the historic Silver Bow Creek channel and its tributaries below the present day YDTI. The developed channels were filled with dark brown well-sorted and moderately-loose sands and gravels with occasional isolated lenses of silt. The stream deposits are approximately 800 ft wide and up to 45 ft deep (Dames and Moore, 1963).

In the vicinity of the YDTI, a broad band of alluvial outwash material is located directly east of the historic Silver Bow Creek channel and forms a moderately-sloping plain adjacent to the East Ridge.
The outwash deposits increase in depth to more than 80 ft in the southern portion of the site. Material consisting of silt to cobble sized particles make up the upper soils. The deeper soils are interbedded with firm silts, sandy loams and sands (Dames and Moore, 1963).

These deposits extend south to the central portion of the Butte-Silver Bow valley, filling an asymmetrical graben formed by the Continental Fault at the base of the East Ridge. The thickness exceeds 1,000 ft immediately south of the mine complex (MR, 1999).

Three historic geologic maps representing the extent of the alluvium deposits in the Silver Bow Creek basin were reviewed as part of the study. The previous geologic maps showing the interpreted extents of the alluvial deposits are included in Appendix C. The following information was adopted from each of these maps:

- Dames and Moore (1963) mapped the alluvial deposits prior to the original tailings impoundment dyke construction; this map is centered on the confluence of Yankee Doodle Creek and Silver Bow Creek where the original embankment was located. This mapping separated the alluvial deposits into recent stream deposits located along the creeks and outwash deposits located close to East Ridge.
- Botz and Knudson (1970) mapped the northern tributaries of the Silver Bow Creek Basin after the tailings impoundment was constructed.
- MR mapped the West Ridge alluvial tributaries in 2014 and 2015.

KP combined these geologic maps into an alluvial map of the YDTI area that differentiates the recent stream deposits from the outwash deposits, and included historic and more recent drillholes that have reported intercepting the alluvial deposits. The amalgamated mapping is shown on Figure 2.5.

The alluvium mapping is overlaid on aerial imagery of the YDTI from 2015. Natural ground contours from available 1956 topography (predating YDTI development) are superimposed on the mapping and aerial imagery. The blue arrows indicate predevelopment surface drainage patterns interpreted from the 1956 topography, which are generally consistent with the stream deposit mapping. The combined mapping can be used to understand drainage patterns in the vicinity of the YDTI at the foundation level beneath the existing embankments.

A total of twenty-eight (28) drillholes and monitoring wells reported encountering the alluvial deposits. Fourteen of these drillholes reported intersecting alluvial deposits ranging between 4 and 44 ft thick before terminating in bedrock. The remainder of the drillholes did not report reaching bedrock, and three drillholes encountered in excess of 70 ft of alluvium material.
<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>EASTING (m)</th>
<th>NORTHING (m)</th>
<th>ELEVATION (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>YANKEE DOODLE CREEK</td>
<td>149,000 N</td>
<td>141,500 N</td>
<td>5900</td>
</tr>
<tr>
<td>DIXIE CREEK</td>
<td>141,500 N</td>
<td>139,000 N</td>
<td>5700</td>
</tr>
<tr>
<td>SILVER BOW CREEK</td>
<td>147,500 N</td>
<td>140,500 E</td>
<td>5800</td>
</tr>
</tbody>
</table>

**NOTES:**
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 25 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.
4. SUPERIMPOSED TOPOGRAPHY IS 1956 TOPOGRAPHY OF THE DRAINAGE BASIN PRIOR TO DEVELOPMENT OF THE FACILITY.

**SCALE:** A

**LEGEND:**
- 2015 SONIC DRILLHOLE
- OUTCROP DEPOSITS (HANES AND MOREAU, 1966)
- QUATERNARY ALLUVIUM (WETZEL, 1975)
- HISTORIC DRILLHOLE
- ALLUVIUM (MONTANA RESOURCES WEST RIDGE GEOLOGICAL MAPPING 2015)
- OUTWASH DEPOSITS (DAMES AND MOORE, 1962)
- YEARLY STREAM DEPOSITS (DAMES AND MOORE, 1962)

**MONTANA RESOURCES, LLP**

**YANKEE DOODLE TAILINGS IMPOUNDMENT**

**ALLUVIUM DISTRIBUTION AND THICKNESS**

**REV DATE**

**DRAWN**

**DESIGNED**

**REVIEWED**

**1/10/2017 8:12:16 AM**

**SAVED:** M:\1\01\00126\14\A\Acad\FIGS\B43,

**PRINTED:** 1/10/2017 8:12:16 AM,

**XREF FILE(S):** Topo 5 ft 2013; DISTURBANCE BOUNDARY; ALLUVIUM; DRILL HOLES; X-C-PROP-BNDY-2015; 1956 Contours - 20'; V-HYDRO-FLOW-PATH

**IMAGE FILE(S):** 01 2015_NAIP_Silver_Bow Scan for Josie

**FIGURE 2.5**
3 – WEST RIDGE SITE CONDITIONS

3.1 GENERAL

The West Ridge area is a relatively low ridgeline bordering the west side of the impoundment. Ground elevations along the West Ridge range from 6,470 to 6,550 ft. The east flank of the West Ridge is heavily forested and slopes steeply towards the impoundment. The west flank slopes more gently due west and is mixed forest and open range land. Moulton Road trends north-south along the ridge crest and a number of private residences are located along the west flank of the ridge.

A total of 19 monitoring wells, 21 drillholes, 33 test pits and 26 trenches have been advanced along the West Ridge over a series of hydrogeological and geotechnical site investigation programs conducted between 2012 and 2016. Hydrometrics (2014) initially investigated the hydrogeologic conditions of the West Ridge with the installation of eight monitoring wells (MW12-11 to MW12-18) in the area. One of the monitoring wells from this investigation, MW12-16, identified a local low groundwater level of approximately 6,378 ft elevation that initiated further investigation. Hydrometrics investigated the groundwater level low with the installation of an additional eight monitoring wells in the West Ridge in 2015 (MW15-01 to MW15-08). Monitoring well MW15-03 encountered a similar groundwater level low as reported at monitoring well MW12-16.

KP conducted a phased geotechnical and hydrogeological site investigation in 2015 to support engineering design of the West Embankment. Phases 1A, 1B, 2A, 2B, 2C and two drillholes from Phase 3 of this site investigation program focused on the West Ridge area as discussed below.

- Phase 1A test pit program included 33 test pits (TP15-01 to 14, 18 to 31, 33, 34, and 38 to 40) to investigate the surficial material and weathered bedrock horizon and identify areas of thicker surficial material in the foundation of the West Embankment. The study also included infiltration testing of overburden soil.
- Phase 1B trench program included ten trenches (T-1 to T-4 and T-6 to T-11) to investigate the surface expression of possible structural lineaments that could explain the groundwater low on the West Ridge.
- Phase 2A and 2B consisted of eleven drillholes along the proposed West Embankment centreline and footprint. Five Phase 2A drillholes (DH15-01 to DH15-05) were located along topographic lows on the embankment centerline. Phase 2B included four drillholes that were located along topographic highs on the embankment centerline (DH15-07, DH15-08, DH15-11 and DH15-13) and two drillholes located in the vicinity of proposed West Embankment Drain (DH15-09 and DH15-12).
- Phase 2C consisted of three deep angled drillholes (DH15-06, DH15-10, and DH15-14) located close to monitoring wells MW12-16 and MW15-03 to investigate the subsurface conditions around the groundwater low.
- Phase 3 included two sonic drillholes (DH15-S6 and DH15-S7) to investigate displaced or consolidated tailings beneath the recently placed rockfill located to the north of 'Rocky Knob'.

Installation of DH15-14 and subsequent monitoring of piezometric elevations therein identified an additional anomalously low piezometric elevation at depth within the West Ridge to the south of the previously identified groundwater low. KP and Hydrometrics conducted the following additional site investigation work in 2016 to further investigate this second piezometric low, termed the deep
isolated fracture system, and to target structural lineaments (shear zones) identified as potential boundaries to flow in the West Ridge:

- KP conducted Phase 5 of the site investigation program, consisting of one vertical and four angled drillholes (DH16-01 to DH16-05) located in the vicinity of DH15-14.
- Hydrometrics installed an additional three monitoring wells in 2016, two of which (MW16-01 and MW16-02D) were installed within the deep isolated fracture system. Monitoring well MW16-02D was installed as a 4-inch diameter well to facilitate hydraulic testing.
- Hydrometrics conducted hydraulic testing in MW16-02D to characterize the extent of the isolated fracture system. Testing included a 14-day pumping test with water level recovery monitored over four weeks and a 7-day recharge test. Response to the hydraulic testing was monitored using the network of monitoring wells and VWPs in drillholes in the West Ridge area.
- MR and Hydrometrics completed 15 additional trenches (T-12 to T-26) in 2016 to identify surface expressions of key structural lineaments (shear zones) that could explain the low piezometric elevations in MW12-16 and MW15-03 and in the deep isolated fracture system. The trenches were sited based on the projections of shear zones encountered in the 2015 and 2016 drillholes, and based on the west ridge structural geology model presented in Section 3.5.3.

Monitoring wells, drillholes, test pits, and trenches advanced in the West Ridge area are shown on Figure 3.1. Details related to the location, depth, and monitoring instrumentation of the West Ridge drillholes and monitoring wells are summarized in Table 3.1. Memos detailing the 2016 trenching program and findings are presented in Appendix D.
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

LEGEND:
- TRENCH

SCALE:

- 400 0 400 800 1200 1600 2000 ft

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST EMBANKMENT
DRILLHOLE AND TEST PIT LOCATIONS

FIGURE 3.1

REV
0

DATE
10JAN'17

DESCRIPTION
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KTD
RM
DOF

10-126/14
2

Figure 3.1

YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST EMBANKMENT
DRILLHOLE AND TEST PIT LOCATIONS

SCALE A

NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

LEGEND:
- TRENCH

SCALE:

- 400 0 400 800 1200 1600 2000 ft

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST EMBANKMENT
DRILLHOLE AND TEST PIT LOCATIONS

FIGURE 3.1

REV
0

DATE
10JAN'17

DESCRIPTION
Issued with Report

KTD
RM
DOF

10-126/14
2

Figure 3.1
# Table 3.1 West Embankment and West Ridge Drillhole Summary

<table>
<thead>
<tr>
<th>Monitoring Well</th>
<th>Northing (ft)</th>
<th>Easting (ft)</th>
<th>Elevation (ft)</th>
<th>Azimuth (°)</th>
<th>Dip (°)</th>
<th>Total Depth (ft)</th>
<th>Installations</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW12-11</td>
<td>144,632.8</td>
<td>129,595.7</td>
<td>6,521.4</td>
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<td>-</td>
<td>200</td>
<td>Monitoring well</td>
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<tr>
<td>MW12-12</td>
<td>146,996.7</td>
<td>129,116.2</td>
<td>6,475.9</td>
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<td>-</td>
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<td>Monitoring well</td>
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<tr>
<td>MW12-13</td>
<td>148,087.7</td>
<td>128,735.9</td>
<td>6,490.3</td>
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<td>-</td>
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<td>Monitoring well</td>
</tr>
<tr>
<td>MW12-14</td>
<td>146,991.1</td>
<td>129,103.9</td>
<td>6,476.5</td>
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<td>-</td>
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<td>MW12-15</td>
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<td>-</td>
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<td>-</td>
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<td>-</td>
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<td>-</td>
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<td>-</td>
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<td>-</td>
<td>-</td>
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<td>MW15-05</td>
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<td>-</td>
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<td>6,469.0</td>
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<td>-</td>
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<td>Monitoring well</td>
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<td>MW15-07</td>
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<td>-</td>
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<td>MW15-08</td>
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<td>6,504.1</td>
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<td>-</td>
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<td>6,501.5</td>
<td>-</td>
<td>-</td>
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<td>MW16-02D</td>
<td>145,508.0</td>
<td>129,730.0</td>
<td>6,499.4</td>
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<td>-</td>
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<td>131,599.2</td>
<td>143,505.4</td>
<td>6,408.5</td>
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<td>200</td>
<td>3 VWPs</td>
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<tr>
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<td>146,376.3</td>
<td>6,374.1</td>
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<td>-</td>
<td>300</td>
<td>3 VWPs</td>
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<tr>
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<td>147,448.6</td>
<td>6,328.3</td>
<td>-</td>
<td>-</td>
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<td>3 VWPs</td>
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<td>DH15-04</td>
<td>131,190.9</td>
<td>143,704.6</td>
<td>6,351.4</td>
<td>-</td>
<td>-</td>
<td>200</td>
<td>3 VWPs</td>
</tr>
<tr>
<td>DH15-05</td>
<td>130,540.0</td>
<td>145,047.8</td>
<td>6,338.6</td>
<td>-</td>
<td>-</td>
<td>200</td>
<td>3 VWPs</td>
</tr>
<tr>
<td>DH15-06</td>
<td>129,435.4</td>
<td>145,751.8</td>
<td>6,485.3</td>
<td>360</td>
<td>64</td>
<td>508</td>
<td>5 VWPs</td>
</tr>
<tr>
<td>DH15-07</td>
<td>130,556.2</td>
<td>145,499.7</td>
<td>6,393.5</td>
<td>-</td>
<td>-</td>
<td>140</td>
<td>3 VWPs</td>
</tr>
<tr>
<td>DH15-08</td>
<td>130,900.9</td>
<td>144,460.3</td>
<td>6,399.9</td>
<td>-</td>
<td>-</td>
<td>193</td>
<td>3 VWPs</td>
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<tr>
<td>DH15-09</td>
<td>130,400.7</td>
<td>147,474.3</td>
<td>6,347.4</td>
<td>-</td>
<td>-</td>
<td>235</td>
<td>3 VWPs</td>
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<tr>
<td>DH15-10</td>
<td>129,455.8</td>
<td>146,314.1</td>
<td>6,481.5</td>
<td>181</td>
<td>64</td>
<td>700</td>
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</tr>
<tr>
<td>DH15-11</td>
<td>130,208.8</td>
<td>147,971.5</td>
<td>6,370.5</td>
<td>-</td>
<td>-</td>
<td>200</td>
<td>3 VWPs</td>
</tr>
<tr>
<td>DH15-12</td>
<td>131,122.0</td>
<td>146,329.3</td>
<td>6,347.6</td>
<td>-</td>
<td>-</td>
<td>150</td>
<td>3 VWPs</td>
</tr>
<tr>
<td>DH15-13</td>
<td>130,272.0</td>
<td>146,801.5</td>
<td>6,431.1</td>
<td>-</td>
<td>-</td>
<td>180</td>
<td>3 VWPs</td>
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<tr>
<td>DH15-14</td>
<td>129,473.3</td>
<td>145,823.7</td>
<td>6,483.4</td>
<td>181</td>
<td>64</td>
<td>700</td>
<td>5 VWPs</td>
</tr>
<tr>
<td>DH15-15</td>
<td>144,623.9</td>
<td>131,667.2</td>
<td>6,353.6</td>
<td>-</td>
<td>-</td>
<td>86</td>
<td>-</td>
</tr>
<tr>
<td>DH15-16</td>
<td>144,122.2</td>
<td>131,297.4</td>
<td>6,347.8</td>
<td>-</td>
<td>-</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>DH16-01</td>
<td>130,544.4</td>
<td>145,498.8</td>
<td>6,393.6</td>
<td>-</td>
<td>90</td>
<td>402</td>
<td>4 VWPs</td>
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<td>DH16-02</td>
<td>129,550.9</td>
<td>145,500.1</td>
<td>6,502.3</td>
<td>181</td>
<td>73</td>
<td>602</td>
<td>3 VWPs</td>
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<tr>
<td>DH16-03</td>
<td>129,722.5</td>
<td>145,359.6</td>
<td>6,505.8</td>
<td>182</td>
<td>70</td>
<td>758</td>
<td>5 VWPs</td>
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<td>DH16-04</td>
<td>130,284.3</td>
<td>145,471.0</td>
<td>6,469.5</td>
<td>175</td>
<td>71</td>
<td>850</td>
<td>6 VWPs</td>
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<td>DH16-05</td>
<td>129,196.5</td>
<td>145,518.3</td>
<td>6,508.1</td>
<td>181</td>
<td>64</td>
<td>999</td>
<td>7 VWPs</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Coordinate system and elevations are based on the Anaconda Mine Grid.
2. Azimuth and dip of drillholes based on MR drillhole survey. Drillhole dip varied slightly throughout the drillhole. The reported value represents the average dip along the drillhole length. The drillhole is vertical if no value is provided.
3.2 MONITORING INSTRUMENTATION

Piezometric elevations are monitored in a network of drillholes and monitoring wells using a combination of VWPs, pressure transducers and manual water level measurements. The locations of the monitoring wells and drillholes in the West Ridge area are shown on Figure 3.1.

A total of 19 monitoring wells were installed along the West Ridge by Hydrometrics between 2012 and 2016. The monitoring wells range in depth from 102 to 558 ft below ground surface (bgs). Monitoring well depths and screened intervals are summarized in Table 3.2 along with the measured piezometric depths and elevations at the end of October 2016.

A total of 73 VWPs were installed in 19 drillholes advanced in the area of the West Embankment footprint and West Ridge. Three VWPs were installed in each Phase 2A and 2B drillhole, five VWPs were installed in each Phase 2C drillhole, and three to seven VWPs were installed in each Phase 5 drillhole. VWPs range in depth from 28 to 888 ft bgs. VWP installation details are summarized in Table 3.3 along with the measured piezometric depths and elevations at the end of October 2016.

### Table 3.2 West Ridge Monitoring Well Summary

<table>
<thead>
<tr>
<th>Monitoring Well</th>
<th>Total Drillhole Depth</th>
<th>MW Screen Interval</th>
<th>Piezometric Depth</th>
<th>Piezometric Elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW12-11</td>
<td>200</td>
<td>145 - 195</td>
<td>60</td>
<td>6,461</td>
</tr>
<tr>
<td>MW12-12</td>
<td>200</td>
<td>165 - 200</td>
<td>47</td>
<td>6,429</td>
</tr>
<tr>
<td>MW12-13</td>
<td>200</td>
<td>150 - 200</td>
<td>23</td>
<td>6,467</td>
</tr>
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<td>MW12-14</td>
<td>150</td>
<td>100 - 150</td>
<td>40</td>
<td>6,436</td>
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<tr>
<td>MW12-15</td>
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<td>150 - 200</td>
<td>36</td>
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<td>MW12-16</td>
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<td>141 - 191</td>
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<tr>
<td>MW12-17</td>
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<td>155 - 195</td>
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<td>6,435</td>
</tr>
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<td>MW12-18</td>
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<td>80 - 115</td>
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<td>MW15-01</td>
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<td>170 - 230</td>
<td>70</td>
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<td>MW15-02</td>
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<td>141 - 199</td>
<td>71</td>
<td>6,412</td>
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<td>MW15-03</td>
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<td>335 - 389</td>
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<td>6,384</td>
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<td>72</td>
<td>6,393</td>
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<td>72 - 102</td>
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<td>244 - 264</td>
<td>65</td>
<td>6,433</td>
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</table>

**NOTES:**

1. Piezometric depths represent water level measurements taken on October 26, 2016.
2. Piezometric elevations are calculated at the midpoint of the well screen.
3. Piezometric elevations are based on the Anaconda Mine Grid.
4. Piezometric elevations at MW16-01 and MW16-02D are influenced by a 14-day pumping test that was conducted from August 17 to 31, 2016 and a recharge test conducted from October 5 to 11, 2016.
### Table 3.3 West Ridge VWP Summary

<table>
<thead>
<tr>
<th>Drillhole ID</th>
<th>Downhole Depth (ft)</th>
<th>VWP ID Number</th>
<th>Sensor Depth (ft)</th>
<th>Sensor Elevation (ft bgs)</th>
<th>Piezometric Depth (ft)</th>
<th>Piezometric Elevation (ft)</th>
</tr>
</thead>
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<td>195</td>
<td>6,214</td>
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<td></td>
<td>VW2</td>
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<td>6,283</td>
<td>51</td>
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<td></td>
<td></td>
<td>VW3</td>
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<td>300</td>
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NOTES:
1. Piezometric data represent VWP readings taken on October 26, 2016 unless otherwise noted.
2. Piezometric data presented for DH15-02 and DH15-05 are VWP readings taken on October 7, 2016.
3. Piezometric data presented for DH15-07 are VWP readings taken on November 2, 2016.
4. Sensor and piezometric elevations are based on the Anaconda Mine Grid.
5. Piezometric elevations at DH15-14 VW1 and VW2 and DH15-01 are influenced by a 14-day pumping test conducted from August 17 to 31, 2016 and a recharge test conducted from October 5 to 11, 2016.
6. Piezometric elevations at sensors in DH15-03 are above ground surface. Drillhole DH15-03 is located within a surface depression at Drain Pod #1.
3.3 GEOLOGIC SECTIONS

The data collected during the site investigation work was used to prepare a series of geologic cross sections for the West Embankment and West Ridge area. The cross sections present the following information:

- Geology, including weathered zones, highly fractured zones, oxidation and structural features such as shear zones and dykes
- Results from hydraulic conductivity testing
- Location of instrumentation
- Piezometric levels, and
- Projections of inferred shear zones.

Sections presented in this section are organized according to their location relative to the West Embankment footprint and the West Ridge. Locations of sections beneath the West Embankment are shown on Figure 3.1 and West Embankment sections follow as Figures 3.2 to 3.4. Section locations were selected to coincide with key alignments of the West Embankment design. A long section trending generally north-south following the setting out line (SOL) for the West Embankment is presented as Figure 3.2. Four sections oriented relatively perpendicular to the SOL of the West Embankment are oriented along topographic lows along the alignment where the Extraction Basin and drain pods are located. These sections are included on Figure 3.3 and Figure 3.4.

Locations of sections through the West Ridge that support the investigation of the piezometric lows are shown on Figure 3.5. One section, presented on Figure 3.6, is oriented approximately east-west crossing the deep isolated fracture system. Four of the sections are oriented approximately north-south and parallel the angled drillholes of the Phase 2c and Phase 5 site investigation programs. A longitudinal section along the top of the West Ridge is included as Figure 3.8.
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS OTHERWISE NOTED.
3. GROUNDWATER LEVEL AT VWP SENSORS REPORTED ON OCTOBER 26, 2016, EXCEPT VWP SENSORS IN DH15-05.
4. GROUNDWATER LEVEL AT VWP SENSORS IN DH15-05 REPORTED ON OCTOBER 7, 2016.
5. GROUNDWATER LEVEL AT CPT14-04 REPORTED ON JULY 10, 2015.
6. GROUNDWATER LEVEL AT CPT14-04 REPORTED ON JULY 10, 2015.
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

LEGEND:
- Projected Shear Zone
- Mapped Shear Zone
- Inferred Structural Lineament
- Aplite Dyke

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
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LEGEND:
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SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

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SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

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SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

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SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

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- Aplite Dyke

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

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LEGEND:
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- Inferred Structural Lineament
- Aplite Dyke

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

NOTES:
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LEGEND:
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- Mapped Shear Zone
- Inferred Structural Lineament
- Aplite Dyke

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

NOTES:
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- Mapped Shear Zone
- Inferred Structural Lineament
- Aplite Dyke

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MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
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WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

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- Projected Shear Zone
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- Inferred Structural Lineament
- Aplite Dyke

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

LEGEND:
- Projected Shear Zone
- Mapped Shear Zone
- Inferred Structural Lineament
- Aplite Dyke

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

Figure 3.5

REV 0

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
WEST RIDGE SITE PLAN AND CROSS-SECTION LOCATIONS

NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

LEGEND:
- Projected Shear Zone
- Mapped Shear Zone
- Inferred Structural Lineament
- Aplite Dyke

SCALE A
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. DIMENSIONS AND ELEVATIONS ARE IN FEET UNLESS OTHERWISE NOTED.
3. GROUNDWATER LEVEL AT VWP SENSORS REPORTED ON OCTOBER 26, 2016, EXCEPT DH15-07.
4. GROUND WATER LEVEL AT VWP SENSORS IN DH15-07 REPORTED NOVEMBER 2, 2016.
6. PROJECTION OF THE DH16-03-3W SHEAR ZONE BASED ON CONTACTS ENCOUNTERED IN SURFACE TRENCH T-20 AND SUBSURFACE CONTACTS IN DH15-04 (INFERRED).
YANKEDoodle TAILINGS IMPOUNDMENT

SHEAR ZONE PROJECTION INFERRED FROM MINE GRID.

NOTES:
1. COORDINATES AND ELEVATIONS ARE IN FT UNLESS OTHERWISE NOTED.
2. MONITORING LEVELS AT VWP SENSORS AND MONITORING WELLS REPORTED ON OCTOBER 26, 2016.

LEGEND:
- SHEAR ZONE
- ALTERED ZONE
- SHEAR ZONE PROJECTION INFERRRED FROM TRENCH
- WIRE CEMENTED CEMENTED ZONE
- 1 5/8" HYDRAULIC CONDUCTIVITY TEST
- END OF SOIL AT EACH DRILLHOLE (ZONES VARY)
- SHEAR ZONE
- SHEAR ZONE
- SHEAR ZONE
- SHEAR ZONE
- MONTANA RESOURCES, LLP
- YANKEDoodle TAILINGS IMPOUNDMENT

SECTION 3

WEST RIDGE

VA0001-0014 1/2 SCALE 8 X 11

FIGURE 3.8
3.4 MATERIAL TYPES

Bedrock geology of the West Ridge area is dominated by BQM bedrock. Limited outcrops of volcanic rocks cover the BQM in the extreme northern portion of the study area, shown on Figure 2.3. The following material types were encountered in the West Ridge area in the drillhole and test pit programs:

- Fill material
  - Rockfill
  - Tailings
- Surficial Material
  - Alluvium
  - Colluvium
  - Completely Weathered Bedrock (residual soil)
- Bedrock
  - Weathered BQM Bedrock
  - Competent BQM Bedrock
- Structures (joint sets, shear zones, aplite dykes and quartz veins)

3.4.1 Fill Materials

3.4.1.1 Rockfill

Four drillholes (DH15-09, DH15-12, DH15-S6, and DH15-S7) were located on the rockfill access road that was constructed to salvage topsoil in the West Ridge area in January 2013. Drillholes DH15-09 and DH15-12 located in the vicinity of proposed West Embankment Drain (WED) encountered rockfill material 60 ft and 32 ft deep, respectively. The rockfill road at DH15-09 was constructed from variable sources of pit-run rockfill end dumped in roughly a 50 ft lift by the 240-ton mine haul trucks. Drilling between depths of 40 ft and 60 ft recovered BQM of varying geologic alteration interpreted to be boulder fragments.

Drillholes DH15-S6 and DH15-S7 were located in an area to investigate displaced and consolidated tailings adjacent to the WED alignment and encountered rockfill material 47 ft and 17 ft deep, respectively.

3.4.1.2 Tailings

Drillholes DH15-S6 and DH15-S7 were located at the southern edge of the West Embankment where rockfill placement has occurred over existing tailings. The intent of these drillholes was to investigate the extent to which the rockfill has displaced or consolidated the tailings slimes.

Tailings material was encountered in DH15-S6 at a depth of 47 to 61 ft. This material is characterized as medium grey, very fine grained clayey silty sand. Tailings were very soft and wet to moist with no plasticity. Tailings were not encountered in drillhole DH15-S7. Additional characterization of tailings within YDTI is provided in Section 5.

3.4.2 Surficial Materials

MR identified the presence of alluvium deposits in topographic low areas at the locations shown on Figure 2.5 with geologic mapping as described in Section 2.6.2. These deposits form six drainage
channels with ephemeral tributaries draining towards the basin that now contains the YDTI. The alluvium material distribution and consistency was investigated with a test pit program (KP, 2015). Topsoil removal was underway during the program and many test pits were located in areas where topsoil and portions of the surficial material had already been removed, or was later removed. The drainage channels contained loose to dense silty sand that varied from 1 ft to greater than 16 ft thick but was generally less than 10 ft thick. Surficial materials were generally found to be thicker near the current extent of the YDTI than high on the West Ridge.

The test pits and drillholes often encountered a relatively thin layer of loose to dense silty sand below the alluvium and overlying highly to moderately weathered bedrock characterized as completely weathered bedrock. Completely weathered bedrock is decomposed to soil with little original rock texture or fabric preserved. This material is referred to interchangeably in this report as completely weathered bedrock and residual soil. Completely weathered bedrock was discoloured throughout to an orange-brown and the grain size is related to the relic granitic texture of the source rock. The completely weathered bedrock was extremely weak and required no force to be broken with fingers. Complete bedrock weathering was thickest in the topographic lows and thinnest (or not present) on topographic highs, and was generally less than 10 ft thick. Alluvial material overlying completely weathered bedrock in a test pit completed on the West Ridge is shown in Photograph 3.1.

Photograph 3.1  Typical profile of Alluvium, Completely and Highly Weathered Bedrock
It was difficult to distinguish whether the overburden deposits were alluvium, colluvium, or completely weathered bedrock without signs of alluvial deposition, such as stratified layering. Stratified layering in the alluvial material was only found in three test pits (TP15-12, 14, and 29).

PSD testing was completed on samples of the alluvium/colluvium and completely weathered bedrock (KP, 2015). Particle Size Distribution (PSD) test results indicate a material with approximately 80% sand, 10% silt, and 10% clay. The USCS designation for these materials is Silty Sand (SM). Grain size distributions are similar for the alluvium/colluvium and completely weathered bedrock material types as shown on Figure 3.11.

![Gradation Summary of Surficial Materials](image)

**NOTES:**
1. Orange indicates silty sand from Alluvium/Colluvium samples.
2. Grey indicates Completely Weathered Bedrock samples.

**Figure 3.11  Gradation Summary of Surficial Materials**

Infiltration testing to investigate the field saturated hydraulic conductivity (Ks) of the surficial materials was completed using an infiltrometer (KP, 2016g). The estimated Ks ranged from 1x10⁻⁵ to 2x10⁻⁴ m/s (3 to 50 ft/day) for all six tests completed in alluvium/colluvium and completely weathered bedrock material. The estimated Ks for the one test completed in completely weathered bedrock was at the low end of the range of Ks results for the five tests completed in alluvium/colluvium material.

The alluvium/colluvium and completely weathered bedrock material have similar characteristics based on the field description, grain size analysis, and infiltration testing. Distinction of the overburden material is not considered to be important to the engineering design as a result of the similar geotechnical and hydrogeological properties.
3.4.3 Bedrock

3.4.3.1 Weathered Bedrock

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

The BQM in the west ridge area shows a typical meteoric weathering profile that grades from completely weathered (residual soil) to highly weathered to moderately weathered. The profile of weathering within the bedrock was distinguished during the KP site investigations based on characteristics of rock fracture spacing, intactness, and discolouration observed in the test pits and drill core. RMR classifications generally decrease as the degree of weathering increases from fresh, competent bedrock through moderately and highly weathered to completely weathered (residual soil). A similar trend is observed in RQD as rock becomes increasingly jointed at higher degrees of weathering. A typical weathering profile within a test pit completed on the West Ridge is shown in Photograph 3.1.

Highly weathered bedrock is the middle weathered horizon where the rock is discoloured predominantly along joint faces and more than half of the rock material is decomposed. Highly weathered bedrock is weak and requires moderate force to crumble. The thickness of the horizon varied in the test pits and drillholes from not present to 50 ft thick. RMR classifications for highly weathered bedrock range from POOR to GOOD with an average rating of FAIR. RQD ranges from 0 to 50 % with an average of 20 %.

Moderately weathered bedrock is the bottom weathered horizon where less than half the rock is decomposed or disintegrated and the original rock texture or fabric is preserved. Fresh rock fragments are typically present as blocks or boulders that fit together. The excavator bucket generally refused to advance on this horizon during the test pit program. Test pits located on the topographic high points encountered shallow moderately weathered bedrock and little weathering profile. Drillholes encountered a moderately weathered bedrock thickness that generally varied from 10 to 100 ft, and exceeded one hundred ft thick in several of the drillholes on the West Ridge. RMR classifications for moderately weathered bedrock range from POOR to VERY GOOD with an average rating of GOOD and an average RQD of approximately 33 %.

3.4.3.2 Competent Bedrock

Slightly weathered to fresh BQM bedrock is classified as competent bedrock with little to no weakness or weathering noted. Competent BQM is described as dark grey and medium to coarse grained. Slightly weathered to fresh bedrock was encountered in all but one (DH15-08) of drillholes in the West Ridge area at depths ranging from approximately 5 ft to 200 ft bgs. The RMR classification for the competent BQM is FAIR to VERY GOOD with an average rating of GOOD indicative of strong intact rock strength. The average RQD for competent BQM was approximately 50%.

Laboratory testing of competent BQM bedrock was completed as part of the Phase 2A site investigation program (KP, 2016b). A total of 15 samples were tested from 5 drillholes (DH15-01 through DH15-05) advanced in the West Embankment area.

- Wet density typically ranged from 2.6 to 2.7 tonnes per cubic meter (t/m³)
• Young’s Modulus for samples tested range from 210 Mpa to 70,400 Mpa with an average of 33,320 Mpa
• Unconfined Compressive Strength (UCS) ranged from 2 MPa to 154 Mpa, and
• Poisson’s Ratios range from 0.05 to 0.3 with an average ratio of 0.2 for the 15 samples.

Altered BQM bedrock was encountered in numerous drillholes in proximity to structural lineaments (shear zones). Shear zones are typically surrounded by an alteration halo (zone of altered or fractured/altered rock bounding the shear). The material within the altered zone is typically subjected to green argillic and/or white argillic alteration. Bedrock was classified as altered on the cross-sections and drillhole logs to indicate rock that has been weakened and altered due to chemical weathering. This alteration may be attributed to the alteration halo of a larger shear or many multiple shear zones that are each less than 5 ft thick that were not explicitly described as shear zones on the drillhole logs. RMR99 classifications for altered bedrock range from POOR to VERY GOOD with an average rating of FAIR. The RMR ratings for altered rock are similar to moderately weathered BQM bedrock. The average RQD for altered BQM was approximately 37%.

Highly fractured bedrock and highly fractured altered bedrock were encountered in several drillholes in the West Ridge. Highly fractured zones are present at depth in DH15-14 and MW16-01 in proximity to significant structural features (shear zones). These fractured zones may result from movement of an individual shear zone, the extent of the shear alteration halo (zone of altered or fractured/altered rock bounding a shear zone) or a high concentration of shear zones within a local area. RMR99 classifications for highly fractured bedrock had an average rating of FAIR.

3.4.4 Geologic Structures

Geologic structures encountered within the West Ridge included shear zones, aplite dikes, and quartz veins and quartz vein breccia/fragments. General characteristics of each structure type includes the following:
• Several east-west striking, steeply dipping shear zones cross-cut the West Ridge. Shear zones are typically characterized by highly weathered rock with clay gouge and associated friable zones, broken zones or rubble zones. Shear zones commonly included strong green argillic and white sericite alteration and an absence of iron oxide staining. The RMR99 classification in shear zones ranged from POOR to GOOD with an average classification of POOR. RQD values for shear zones range up to 47%, but are generally low with an average value of 7 %. The RQD values are indicative of the fractured, broken nature of the shear material.
• Aplite dikes are typically less than 2 inches in thickness. One or more dikes were encountered in DH15-06, DH15-09, DH16-02, DH16-03, DH16-04, and DH16-05.
• Quartz veins were commonly encountered within the subsurface geology and surface trenching, typically associated with shear zones. Quartz veins or breccia were typically surrounded by green argillic and white argillic alteration and clay gouge or friable bedrock. Quartz veining could be up to two inches thick and commonly encountered as breccia or fragments with a shear zone matrix. Drillhole DH15-07 was advanced along a sub-vertical quartz vein described as brittle and consisting of dissolution features such as vugs that were not interconnected.

Details of all structural features encountered during drilling and trenching are provided in the site investigation memos included in Appendix D. Numerous structural lineaments (shear zones) were identified within the West Ridge during the drilling and surface trenching programs. Shear zones
identified in the trenching programs by KP (2016a) and MR and Hydrometrics (Peet, 2016a,b,c) are named according to the trench they were encountered in. Key structural lineaments encountered in surface trenches within the West Ridge area are shown as a blue line in plan view on Figure 3.5. Shear zones encountered in drillholes are depicted in cross-section on Figures 3.6 through 3.10.

An example structural lineament in the West Ridge area, the T-19 shear zone, is shown in Photographs 3.2, 3.3 and 3.4. The surface expression of the T-19 shear encountered in the T-19 surface trench is shown in Photograph 3.2. The projection of the shear is consistent with shear zones encountered at depth in drillholes including the examples shown in Photographs 3.3 and 3.4 for DH16-02 and DH16-03, respectively. The T-19 shear is characteristic of the numerous east-west striking, steeply dipping structures that cross-cut the West Ridge. The photographs illustrate the alteration and friable and/or broken nature typical of West Ridge structural lineaments.
Photograph 3.3  Core photo of shear zone encountered at depth in DH16-02 (295 to 304 ft downhole) inferred to be the T-19 shear.

Photograph 3.4  T-19 structural lineament encountered at depth in DH16-03 (158 to 166 ft downhole) inferred to be the T-19 shear zone
Core orientation and downhole surveys were carried out in the three inclined drillholes of the Phase 2c drilling program (DH15-06, DH15-10, and DH15-14) and four inclined drillholes of the Phase 5 drilling program (DH16-02 to DH16-05) to investigate the discontinuities in the rock mass. Details of the core orientation and downhole surveys are provided in the memos in Appendix D (KP, 2016f,k). The following three major discontinuity sets were identified in the data set from the inclined drillholes:

- Set 1 – Dip 64°, Dip Direction 279°
- Set 2 – Dip 76°, Dip Direction 10°, and
- Set 3 – Dip 68°, Dip Direction 77°.

3.5 HYDROGEOLOGICAL CONDITIONS

3.5.1 Groundwater Levels

Groundwater levels in the area of the West Ridge are available at 92 monitoring locations as described in Section 3.2. Water levels range from 50 ft bgs within the West Ridge to less than 5 ft bgs at lower ground elevations adjacent to the YDTI. Piezometric elevations along the West Ridge typically range from 6,430 ft to 6,440 ft with elevations to the north and south of the ridge increasing to 6,460 ft and 6,480 ft, respectively. Piezometric conditions at the following monitoring sites along the West Ridge were identified to be lower than the surrounding water levels and in exception to the description above:

- Piezometric elevations of 6,372 ft to 6,395 ft at monitoring wells MW12-16, MW15-03, and drillholes DH15-06 and DH15-10. This piezometric low is termed the shallow piezometric low.
- Piezometric elevations of 6,322 to 6,342 ft at monitoring wells MW16-01, MW16-02D, and drillholes DH15-14 (VW1 and VW2) and DH16-01. This piezometric low is termed the deep isolated fracture system.

Time-series plots of water level elevation at each vibrating wire piezometer installed in the 2015 and 2016 drillholes are provided in Appendix E1. Time series plots of water levels at the West Ridge monitoring wells are provided in Hydrometrics (2017). Potentiometric contours across the study area as developed by Hydrometrics using water level data measured in drillholes and monitoring wells on October 26, 2016, are shown on Figure 3.12. Piezometric elevations and water level contours in the West Ridge are shown on Figure 3.13.
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

LEGEND:
- STRUCTURAL LINEAMENT
- CONCEPTUAL GROUNDWATER FLOW DIRECTIONS
3.5.2 Hydraulic Conductivity Testing

Hydraulic testing was conducted within the West Ridge drillholes and monitoring wells to assess the permeability of the subsurface materials and hydraulic connectivity between sites. Hydraulic testing to assess bedrock hydraulic conductivity included packer testing in drillholes and constant discharge tests in monitoring wells as described below.

- 113 falling head and constant head packer tests were completed in the geotechnical drillholes within the West Embankment and West Ridge as part of the 2015 and 2016 site investigations. Constant head and falling head tests were conducted at 20 to 60 ft downhole intervals. Five packer tests were conducted above the water table, five tests were conducted as open hole tests and 103 tests were successfully conducted in a sealed test interval below the water table.
- Three constant discharge tests and 10 slug tests were conducted in monitoring wells by Hydrometrics to estimate the bedrock hydraulic conductivity.

A summary of all hydraulic conductivity tests conducted along the West Ridge is provided in Table 3.4. A plot showing the hydraulic conductivity estimate versus test interval depth is provided on Figure 3.14. Results of tests conducted above the water table and as open hole tests are provided for qualitative purposes on the plot and are not included in the summary statistics. Hydraulic conductivity test results in the West Ridge range from less than $1 \times 10^{-8}$ m/s to $6 \times 10^{-6}$ m/s, with a geometric mean of $2 \times 10^{-7}$ m/s. Results of hydraulic conductivity tests conducted in shear zone material have a lower range of values than the tests conducted in the BQM material, which suggests shear zones are lower permeability structures. Higher hydraulic conductivity estimates are generally associated with test intervals that were characterized as highly to moderately fractured rock.

Additives were typically used to facilitate drilling at depths greater than 400 or 500 feet during the Phase 2c and Phase 5 drilling programs. Packer testing was discontinued once additives were used in a drillhole and as a result hydraulic conductivity estimates are not available for deeper portions of the drillholes. Observations of drill circulation loss were noted during the Phase 2c and Phase 5 drilling programs. Water circulation was lost during drilling in zones of highly fractured bedrock in drillholes DH16-06, DH15-10, DH6-14, DH16-02, DH16-03, and DH16-05.

<table>
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<tr>
<th>Lithology</th>
<th>Number of Tests</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Geometric Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz Monzonite</td>
<td>104</td>
<td>&lt;1E-08</td>
<td>6E-06</td>
<td>3E-07</td>
</tr>
<tr>
<td>Shear Zone/Dike</td>
<td>12</td>
<td>&lt;1E-08</td>
<td>1E-07</td>
<td>3E-08</td>
</tr>
<tr>
<td>Total Tests</td>
<td>116</td>
<td>&lt;1E-08</td>
<td>6E-06</td>
<td>2E-07</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Minimum, maximum, and geometric mean hydraulic conductivity values are summarized for packer test results from the Phase 2A, 2B, 2C and 5 site investigation programs as well as response test and constant discharge testing in monitoring wells (Hydrometrics, 2017).
2. Results of tests conducted above the water table or as open hole tests are not included in the summary statistics.
3. Test lithology is categorized as shear zone if at least half of the test interval is comprised of shear zone material and altered bedrock.
NOTES:
1. CLOSED MARKERS INDICATE RESULTS FROM SATURATED TEST INTERVALS, OPEN MARKERS INDICATE UNSATURATED TEST INTERVALS, AND DASHED LINES INDICATED OPEN HOLE TESTS.
2. HYDRAULIC CONDUCTIVITY ESTIMATES REPRESENT PACKER TESTS COMPLETED IN DRILLHOLES (KP, 2016b,c,f.k) AND RESPONSE TESTS AND CONSTANT DISCHARGE TESTS IN MONITORING WELLS (HYDROMETRICS, 2014 AND 2017)
3. THE LOWER LIMIT OF PACKER TESTING RESOLUTION IS 1E-8 m/s. VALUES LESS THAN 1E-8 m/s ARE SHOWN AS 1E-8 m/s.
4. DEPTHS ARE VERTICAL BELOW GROUND SURFACE.
5. TEST LITHOLOGY IS CLASSIFIED AS A SHEAR ZONE IF MORE THAN HALF OF THE INTERVAL IS COMPRISED OF SHEAR ZONE MATERIAL AND ALTERED BEDROCK.
Hydrometrics conducted hydraulic testing in monitoring well MW16-02D to assess hydraulic connectivity between monitoring sites and to characterize the extent of the deep isolated fracture system. The hydraulic testing included the following:

- Step-test conducted on July 25 to 27, 2016 to assess design pumping rates
- 14-day pumping test conducted from August 17 to 31, 2016 with water level recovery monitored over four weeks, and
- 7-day recharge test conducted from October 5 to 11, 2016 with water level recovery monitored over the following months.

The pumping test was conducted by maintaining the water level in the pumping well at a near constant depth and varying the pumping rate. Approximately 165,250 gallons were pumped from the MW16-02D well during the test at an average rate of 8.1 gpm. Response to the hydraulic testing was monitored using the network of monitoring wells and VWPs in drillholes in the West Ridge area. Response of the pumping test at adjacent monitoring sites included:

- Monitoring well MW16-01 and drillholes DH15-14 (VW1 and VW2) and DH16-01 exhibited significant responses to pumping and are inferred to be within the boundaries of the isolated system. Drawdown in the pumped well MW16-02D was 275 feet and in observation well MW16-01 located 300 feet away was 160 feet.
- Drillholes DH15-14 (VW3 and VW4) and DH15-05 (VW5) and monitoring wells MW16-02S and MW15-01 exhibited a muted response to the pumping test and are inferred to be outside the isolated system adjacent to structural boundaries that provide recharge to the system via leakage. The water level response outside the system ranged from approximately 1 ft of drawdown north of the system to approximately 5 ft of drawdown in DH16-05 VW5 located to the west.

Results of the pumping test indicate that the deep isolated fracture system is limited in aerial extent with low permeability shear zones located to the north, south and west of pumping well MW16-02D. Analysis of the water level drawdown and recovery data indicates a hydraulic conductivity and storativity on the order of $10^{-7}$ m/s (0.04 ft/day) and $10^{-5}$ m/s, respectively. These values, along with the distribution and magnitude of drawdown observed during the pumping test indicate a very low rate of flow or groundwater flux through the isolated fracture system.

A recharge test was conducted by adding water to monitoring well MW16-02D over a seven-day period starting on October 5, 2016. The objective of the test was to assess the hydraulic head response in the deep isolated fracture system to augmented recharge and to expedite recovery of water levels to an equilibrium state following the pumping test. Approximately 19,900 gallons were added to the deep isolated fracture system during the test at an average rate of 2.3 gpm. The water level in the recharge well was kept at a near constant level during the test. The water level at observation well MW16-01 located 300 feet away increased 50 feet. Results of the recharge test indicate that the hydraulic head within the isolated fracture system is very sensitive to small changes in recharge rates, with heads increasing 50 feet or more in response to the low recharge rates utilized in the test. The results also suggest that natural recharge rates, both prior to and after the augmented recharge test, may be on the order of one gpm or less. These results are consistent with the limited lateral extent and low hydraulic conductivity and storativity values derived from the long-term pumping test.
Detailed methodology and results of the pumping test and recharge test are presented in Hydrometrics (2017). Time series plots of water levels at monitoring sites that displayed a response to the pumping test and recharge test are provided in Appendix E2.

3.5.3 Structural Geology Model

3.5.3.1 Overview

Groundwater flow in the West Ridge area is influenced by the presence of structural lineaments. A structural geology model of the West Ridge was developed using the structural lineaments (shear zones) encountered during the drilling and trenching investigation programs in order to identify key structural lineaments that influence local hydrogeological conditions. The key objectives of the structural geology model were to identify shear zones that may:

1. Influence piezometric elevations in the shallow piezometric low identified in the West Ridge bedrock groundwater system in MW12-16, MW15-03, DH15-06 and DH15-10.
2. Influence piezometric elevations in the deep isolated fracture system first identified in DH15-14.
3. Influence piezometric elevations elsewhere within the West Ridge area.

Shear zones bounding the shallow piezometric low and the deep isolated fracture system within the West Ridge bedrock groundwater system are discussed in the sections that follow. Figures showing the projection of individual shears in the geological model relative to drillhole and surface trench data are presented in Appendix F.

3.5.3.2 Structural Boundaries of the Shallow Piezometric Low

The shallow piezometric low was identified in MW12-16, DH15-10, and DH15-06. Piezometric elevations within this region are locally depressed approximately 40 ft less than groundwater elevations in the rest of the West Ridge groundwater system. This groundwater low is interpreted to be structurally controlled to the north and south. Surface trenching completed in the vicinity identified two key shear zones that are inferred to isolate and limit recharge to the shallow piezometric low. The south bounding structure is the T-21 shear and the north bounding structure is the T-15 shear as discussed below.

T-21 Shear Zone (South Boundary): The T-21 shear zone was identified in the T-21 trench located just south of DH15-06. The shear is an east-west striking structure with a steep northward dip. Its projection corresponds with the southern boundary of the piezometric low and with a shear zone encountered in DH15-14. Piezometric elevation above the T-21 shear in DH15-14 (VW5) is approximately 6,395 ft while those below the shear (VW3 and VW4) measure piezometric levels of approximately 6,430 ft. This structural lineament is expected to limit groundwater recharge to the low from the south.

T-15 Shear Zone (North Boundary): The T-15 shear was encountered in surface trenches T-15, T-16 and T-25. The T-15 shear is an east-west striking structure with a northward dip ranging from 45 to 55 degrees based on surface trenching completed by MR. The projection of this lineament corresponds with a shear zone encountered at depth in DH15-10. Water levels to the north of the shear are approximately 6,410 ft while those to the south within the piezometric low are about 6,375 ft. As with the T-21 structure, the T-15 shear is expected to limit groundwater recharge to the piezometric low from the northern portion of the West Ridge.
3.5.3.3 Structural Boundaries of the Deep Isolated Fracture System

The deep isolated fracture system was first identified in DH15-14 and was investigated further during the 2016 site investigation programs completed by KP and Hydrometrics. Piezometric elevations within this system are locally depressed approximately 100 ft below groundwater elevations in the rest of the West Ridge groundwater system and are approximately 6,330 to 6,340 ft.

The extent of the deep isolated fracture system was assessed using observed hydrologic response at VWP sensors and monitoring wells in the vicinity of the piezometric low to a 14-day pumping test conducted by Hydrometrics (Hydrometrics, 2016). The following sites responded to the pumping test and are considered to be screened or located in the boundaries of the isolated system:

- MW16-02D (pumping well)
- MW16-01
- DH15-14 (sensors VW1 and VW2, located below approximately 6,040 ft elevation), and
- DH16-01 (the lower three VWPs).

A number of monitoring sites exhibited a muted response to the pumping test and are inferred to be outside the isolated system but located adjacent to structural boundaries that provide recharge to the system via leakage:

- DH15-14 (VW3 and VW4 above approx. 6,040 ft elevation)
- DH16-05 (VW5)
- MW16-02S, and
- MW15-01.

Details of the pumping test and additional discussion on the response to pumping observed in monitoring wells and drillholes is provided in Hydrometrics (2017). Drillhole and monitoring well locations are shown in plan view on Figure 3.5 and in cross-sections on Figures 3.6 to 3.10. The deep isolated fracture system is inferred to be spatially constrained to the north, south, west and above by structures, based on the response to the pumping test. These structures consist of projections of shear zones encountered in surface trenches and projections of shear zones encountered only in subsurface drillholes. Drillholes and monitoring wells exhibiting a significant response to the pumping test fall within the structural compartment created by these boundaries. The shear zones interpreted to act as boundaries to the isolated fracture system are shown on Figure 3.15 and are discussed below.

The shear zones that are thought to constrain the deep isolated fracture system may either act as boundaries to flow or movement along the shear may have created the more permeable and highly fractured zone of the deep isolated fracture system. Identifying the specific details of the bounding shears is not considered to be as important as characterizing the extent of the deep fractured system. The discussion below shows that the groundwater system is compartmentalized by structures and that a high density of structures exists in the West Ridge near DH15-14.
NOTES:
1. SHEAR ZONES IN THE IMAGE ARE COLORED AS FOLLOWS:
   T-19 SHEAR (NORTHERN BOUNDARY) = ORANGE/BROWN
   T-19 TWIN SHEAR (SOUTHERN BOUNDARY) = YELLOW
   7W SHEAR (WESTERN BOUNDARY) = GREEN
   3W SHEAR (UPPER BOUNDARY) = RED
2. THE APPROXIMATE EXTENT OF THE DEEP ISOLATED FRACTURE SYSTEM IS OUTLINED WITH A
   RED DASHED LINE.
**T-19 Shear Zone (Northern Boundary):** The T-19 shear zone is identified as the northern bounding structure of the deep isolated fracture system. This shear is an east-west striking structure that dips to the north between 45 and 55 degrees based on surface trenching completed by MR. The surface expression of the structure was identified in the T-13, T-14 and T-19 trenches and the resulting projection matches shear zones encountered at depth within a number of monitoring wells and drillholes. An absence of iron oxidation in drillholes and monitoring wells immediately below the T-19 shear suggests that this shear zone is a local barrier to meteoric recharge.

Results of the pumping test indicate that the T-19 shear acts as a competent flow boundary between the isolated fracture system and the West Ridge groundwater system above the shear to the north. The T-19 shear separates the significant response seen within the deep fracture system from muted recharge responses identified in DH15-14 (VW3 and VW4), MW15-01, MW16-02S, and DH15-07, which are interpreted to be caused by leakage through the boundaries from the surrounding West Ridge groundwater system.

**T-19 Twin (8W) Shear Zone (Southern Boundary):**

The T-19 Twin shear (8W) is inferred to bound the system to the south and is another east-west striking, north dipping structure. The surface expression of the T-19 Twin shear was identified in the T-20 trench and projection of the T-19 Twin shear is inferred to parallel the T-19 shear lineament. The presence of a shear paralleling the T-19 shear was also inferred based on subsurface geology and the shear zone encountered in DH16-02 at 487 ft downhole. The “8W” refers to the name assigned to the downhole shear and its initial projection at the start of the structural model development. The T-19 Twin shear projection intersects shear zones encountered in drillholes DH16-02, DH16-03, DH16-04, and DH16-05.

The T-19 Twin shear isolates VWP sensors that exhibit a response to the pumping test from VWPs located to the south in DH16-02, DH16-03 and DH16-04 where no pump test response was observed. Change in piezometric elevation across the inferred T-19 shear is not substantial and may suggest the shear zone is not a barrier to flow.

Diurnal fluctuations of piezometric elevation were observed in the lower VWP sensors (VW1 to VW4) in drillhole DH16-05. These diurnal fluctuations were caused by daily domestic pumping of Residential Well 196757 and indicate a hydraulic connection between DH16-05 and the residential well. The T-19 Twin shear separates the VWPs in DH16-05 that exhibit the fluctuations from those higher in the drillhole (VW5 through VW7) which do not. Pumping from the well stopped on Oct 1, 2016 for the winter. The end of pumping corresponds with the end of daily fluctuations observed in DH16-05.

**7W Shear Zone (Western Boundary):**

The pumping test suggests the presence of a western bounding structure that hydraulically separates DH16-05 from the pumping test well. Water levels were drawn down approximately 270 ft in the pumping well during the pumping test; however, the piezometric elevations in all but one of the VWPs in DH16-05 remained unaffected. Sensor VW5 in DH16-05 showed a delayed response to the test with a decrease in water level of approximately 5 ft. This sensor is interpreted to be situated west of a structure separating it from the deep isolated fracture system but within western extents of the T-19 and T-19 Twin shear projections. The response is thought to be caused by leakage into the fracture system through the structural boundary.
The 7W shear zone is the inferred western boundary to the isolated fracture system. The projection of the 7W shear was interpreted from the orientation of a shear zone encountered at approximately 544 ft downhole in DH16-03, which has a near vertical dip and a northwest-southeast strike. The 7W shear has not been located in surface trenches to date.

3W Shear Zone (Upper Boundary):

Piezometric head measured within the isolated fracture system is between 6,325 and 6,340 ft as compared to approximately 6,425 ft measured in the overlying West Ridge groundwater system. This marked change in piezometric elevation suggests the presence of an upper bounding structure that serves as a barrier to flow and restricts recharge to the underlying isolated fracture system.

The 3W shear zone is the inferred top boundary to the isolated fracture system. The projection of the 3W shear was interpreted from the orientation of a shear zone encountered at 445 ft downhole in DH16-03. The 3W shear is a shallow northward dipping and east-west trending shear zone that intersects both the T-19 and T-19 Twin shears in the isolated fracture system just above the observed decrease in piezometric elevations. The 3W shear projection corresponds well with shear zones encountered at depth within DH15-14, DH16-02 and MW16-01. The projected surface expression of the 3W shear is coincident with a lineament (Lineament 2) identified by Hydrometrics based on a separate lineament study developed using topographical feature analysis. The limited trenching efforts to date in this area have not yet identified the surface expression of this feature.

3.5.3.4 Lineament 1 Structural Boundary to West Ridge

An east-west striking structural lineament termed Lineament 1 was identified north of the West Ridge based on a lineament study conducted by Hydrometrics in 2012 (Hydrometrics, 2012). The study was based on review of available topographic and geological maps, inspection of residential well logs in the vicinity and evaluation of local surface topography. The lineament is thought to be a steeply north dipping structure extend eastward from the head of Bull Run Creek drainage and cross-cutting the West Ridge at its northern extent. A subtle surface expression of Lineament 1 was found in a historic test pit and the structure appeared to dip to the north. The inferred strike of Lineament 1 corresponds well with a discrepancy between piezometric elevations measured to the north and south as highlighted by Hydrometrics (2014) and discussed in Section 3.5.4. The location of Lineament 1 is shown on Figure 3.5.

3.5.4 Groundwater Flow System

The predominant groundwater regime of the West Ridge is referred to as the West Ridge bedrock groundwater system. A groundwater divide is located along the topographical ridgeline that separates groundwater flow to the east and west. Groundwater west of the divide is expected to discharge to local topographic lows west of the ridge and to Browns Gulch and Bull Run Creek regional lows at the base of the range to the west. Groundwater east of the ridgeline is expected to discharge to local topographic lows, such as the local drainages present upslope from the proposed West Embankment and to the alluvial deposits adjacent to and underlying YDTI. Groundwater potentiometric contours of the West Ridge bedrock groundwater system were developed by Hydrometrics based on water level data collected at drillholes and monitoring wells in October 2016 and were shown previously on Figure 3.13. A more detailed characterization of the hydrogeology of the West Ridge is available in Hydrometrics (2017).
Groundwater flow directions and piezometric elevations within the West Ridge bedrock groundwater system are influenced by the orientation and hydraulic behavior of numerous east-west striking shear zones that cross-cut the ridge. The shear zones discussed in Section 3.5.3 are generally inferred to impede groundwater flow and create a compartmentalized groundwater flow system that consists of a series of step-reductions in water table elevation across the structures. Lineament 1, the shallow piezometric low, and the deep isolated fracture system described in Section 3.5.3 are three such structure-controlled compartmentalized flow systems. A 3-dimensional view of the groundwater potentiometric contours, structural lineaments and the YDTI is presented on Figure 3.16.

The highest piezometric elevations in the West Ridge bedrock groundwater system exist north of the ridge and in the topographically broad southern end of the ridge. These regions are inferred to be the primary sources of groundwater recharge to the West Ridge groundwater system. Piezometric elevations to the north of Lineament 1 are higher than elsewhere in the system and are inferred to be elevated due to impedance of groundwater flow southward across Lineament 1. The observed groundwater elevation just north of this structure at monitoring well MW12-13 is approximately 6,470 ft while those measured south of Lineament 1 are approximately 6,435 ft. Piezometric heads within the southern end of the West Ridge are the highest measured on site and are up to 6,483 ft in monitoring well MW12-15. The relatively elevated piezometric elevations in the southern portion of the ridge are attributed to enhanced groundwater recharge resulting from the broad surface topography, the presence of permeable bedrock outcrops in the area and due to the potential influence of West Ridge geological structures down-gradient to the north (Hydrometrics, 2017).

Groundwater flow will typically be within weathered and competent BQM from groundwater recharge areas along topographic highs to down-gradient discharge locations. Preferential groundwater flow paths in bedrock may be present in fractured bedrock zones and in narrow brittle quartz veins. Two zones were identified to have hydraulic heads lower than the surrounding groundwater system: the shallow piezometric low and the deep isolated fracture system. The potential for these two regions to serve as a potential seepage pathways following continued filling of the YDTI justified additional study. Results of the hydrogeological characterization for each low hydraulic head system are described below.
NOTES:
1. SURFACE TOPOGRAPHY IS SHOWN WITH A 20 FT CONTOUR INTERVAL AND 100 FT INDEX CONTOURS.
2. THE WEST RIDGE RIDGELINE AND INFERRED GROUNDWATER DIVIDE ARE APPROXIMATELY
   COINCIDENT WITH THE ALIGNMENT OF MOULTON ROAD SHOWN ABOVE.
3. STRUCTURAL LINEAMENTS SHOWN ABOVE ARE COLOR CODED AS FOLLOWS:
   - LINEAMENT 1
   - T-15 SHEAR
   - T-19 TWIN SHEAR
   - T-21 SHEAR
   - 3W SHEAR
   - 7W SHEAR
   - T-19 SHEAR
4. DRILLHOLES AND MONITORING WELLS ARE SHOWN AS BLACK TRACES; RESIDENTIAL WELLS ARE
   SHOWN AS BLUE TRACES.
5. BLUE CONTOURS ARE POTENTIOMETRIC CONTOURS OF THE BEDROCK GROUNDWATER SYSTEM FOR
   OCTOBER 2016 AND WERE DEVELOPED BY HYDROMETRICS.
Shallow Piezometric Low

The shallow piezometric low exists in the centre of the West Ridge at the location of MW12-16 and MW15-03 and is apparent as the saddle in the groundwater contours shown on Figure 3.13. This piezometric low is structurally bounded to the north and south by two east-west striking structural lineaments (T-15 and T-21 shear zones, respectively) that are distanced approximately 450 ft apart. The water table elevation between the structural boundaries is approximately 6,378 ft and is depressed approximately 40 ft below the surrounding water table. The locally depressed groundwater elevations are interpreted to be a result of the structural boundaries impeding the flow of groundwater into the piezometric low from adjacent regions to the north and south. This effect is compounded by the relatively narrow surface topography of the West Ridge at this location, which is thought to limit meteoric recharge to the groundwater low and further reduce piezometric elevations within the low. The orientation of the structural lineaments and their position relative to the extent of the piezometric low are shown in cross-section on Figure 3.8.

Deep Isolated Fracture System

The deep isolated fracture system exists in the area that extends from monitoring well MW16-01 and the lower portion of drillhole DH15-14 eastward to include MW16-02D and drillhole DH16-01. Piezometric elevations within the deep isolated fracture system are approximately 80 to 100 ft lower than those in the overlying and surrounding West Ridge bedrock groundwater system. The deep fracture system is inferred to be structurally isolated from the surrounding West Ridge bedrock groundwater system by geological structures to the north, south, west and top of the system as discussed in Section 3.5.3. The extent of the deep isolated fracture system was estimated based on the results of a 14-day pumping test and observations of background water levels. Dimensions of the isolated fracture system are limited in aerial extent and are estimated to be approximately 1,800 ft in an east-west direction and up to 400 ft in a north-south direction. The system is estimated to narrow slightly to the east. The top of the system is shallowest near drillhole DH16-01 (200 ft bgs) located adjacent to the YDTI and deepens to the west to approximately 450 ft below ground surface in the vicinity of MW16-01 and DH15-14. Estimated dimensions reflect the distance between the shear zones inferred to bound the deep system. This dimension is considered to be an upper bound estimate since the fractured zone does not likely extend the full width between shear zones. Bounding shear zones and piezometric elevations in the deep isolated fracture system are shown on Figures 3.6 and 3.8.

Continuous drilling and hydraulic testing activity in the West Ridge since installation of the monitoring sites in the West Ridge has complicated the measurement of the background (static) water level in the deep isolated fracture system. Prior to drilling the 2016 monitoring wells and conducting the pumping test, the piezometric elevations at the DH15-14 VW1 and VW2 sensors in the fractured zone appeared to stabilize at 6,328 and 6,323 ft, respectively. These piezometric conditions indicated an upward gradient in the zone between two lowermost sensors located in DH15-14. Piezometric elevations within the isolated system at the time of writing this report continue to recover from the long-term pumping test and the recharge test, with piezometric elevations at DH15-14 VW1 and VW2 on December 1, 2016 at 6,338 and 6,351 ft, respectively. These present day water levels are higher than pre-testing levels and indicate a downward hydraulic gradient is present. The transition from an upward to downward hydraulic gradient between the lowermost sensors in
DH15-14 may indicate the system was previously recharged from below and is currently being recharged from above. Ongoing monitoring of piezometric elevations within the system will continue in order to determine the static background water level and confirm the source of recharge to the system. It is expected that recent drilling activity has influenced the groundwater flow regime in the vicinity of the deep isolated fracture system and the source of recharge to this zone.

The primary source of groundwater recharge to the isolated fracture system is inferred to be leakage through the north, west and overlying bounding structures. Reported piezometric levels at sensors immediately overlying the deep isolated fracture system in DH15-14 (VW3) and DH16-01 (VW4), located to the west in DH16-05, and located to the north in DH15-06 are higher than the reported water levels in the fracture system, indicating the surrounding groundwater system is a potential recharge source. The water levels in the deep isolated fracture system continued to rise after the recharge test. The rate of rise was attributed to an estimated natural rate of recharge to the system of 1 gpm or less (Hydrometrics, 2017). Results of the recharge test indicate that the hydraulic head within the isolated fracture system is very sensitive to small changes in recharge rates, with heads increasing 50 ft or more in response to the low recharge rates utilized in the test.

Water levels reported at the monitoring sites in the deep isolated fracture system on December 1, 2016 indicate that groundwater within the isolated fracture system flows east toward the YDTI. Reported water levels at sensor VW2 in DH15-14, MW16-01 and MW16-02D are 6,350 ft +/- 1 or 2 ft. The three lowest VWP sensors in DH16-01 report water levels at 6,327 ft which remain below the pond elevation (EL 6,340 ft) and indicate that the direction of flow within the fracture system is to the east. The horizontal hydraulic gradient calculated between MW16-02D (6,349 ft) and DH16-01 VW1 (6,327 ft) is approximately 0.02 ft/ft to the east. Groundwater discharge from the deep isolated fracture zone is expected to be to the alluvial deposits located beneath the YDTI or as southward flow along the edge of the YDTI. Groundwater from the isolated fracture system is not expected to discharge to the YDTI pond given the higher elevation of the pond. The hydraulic heads reported at VWP sensors in drillhole DH16-01 located adjacent to the pond remain 10 ft below the pond elevation, which suggests that the isolated fracture system does not have a strong hydraulic connection with the YDTI pond at its current pond elevation.
4 – EAST-WEST AND NORTH-SOUTH EMBANKMENT SITE CONDITIONS

4.1 GENERAL

There have been several investigations completed in the vicinity of the East-West and North-South Embankment areas spanning over five decades. The investigations were completed by several different engineering consultants in coordination with the mine operator of the time using a variety of methods. The information available from each program varies depending on the state of practice at the time and the records that remain intact. The timing of these investigations typically coincided with certain phases of project development. It is also possible that other site investigation work was carried out before MR took ownership of the property in 1986; however, only the known, available, and useful investigative studies are described in this report. This section of the report focuses on the information extracted from prior and more recent studies to develop the current site characterization supporting the design.

A summary of the geotechnical drillholes, standpipe piezometer installations, and test pits conducted during the investigations described below are summarized in Table 4.1. The locations are shown on Figures 4.1 and 4.2. Logs are separated by program and provided in Appendix A.

### Table 4.1 Drillhole, Standpipe Piezometers and Test Pit Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>Drillholes (1,4,5)</th>
<th>Standpipe Piezometers (2)</th>
<th>Test Pits (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East-West Embankment</td>
<td>62 T-1, 62 T-5, 62 T-6, SD-1, SD-3, DH-1, DH-3, DH-5, DH-6, DH-6A, DH-7 to DH-11, DH-13, 92-1, DH12-01B, DH12-03, DH12-04, DH15-S1 to DH15-S5</td>
<td>93-1 to 93-5, 94-1 to 94-12, 99-1 12-07, 12-10A, 14-01</td>
<td>TP14-01 to TP14-11</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>North-South Embankment</td>
<td>62 T-2, 62 T-3, 62 T-4, SD-2, MD-1, MD-2, DH-2, DH-4, DH-12, 92-2B, DH12-05A</td>
<td>94-B1 to 94-B5, 05-3, 05-4, 12-01 to 12-05, 12-06A, 13-01 to 13-03, 14-02</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>17</td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
1. Drillholes listed in this table do not include the CPT locations in the tailings mass, which will be described in Section 5.
2. Standpipe piezometers shown in this table were installed at the direction of Montana Resources personnel. Completion logs for 12-10A and 14-02 were not provided and are not included in the appendices.
3. Only test pits with known locations and completed test pit logs are shown in this table.
4. Drillhole logs for DH12-01B, DH12-03, DH12-04, and DH12-05A were not available in the factual data report (KP, 2013) that was prepared following the site investigation program. The drillhole information is instead included in a summary table in the body of that factual report.
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.
4. FOR LEGEND AND OVERALL GENERAL ARRANGEMENT SEE FIGURE 2.1.
NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 5 FEET.
3. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.
4. FOR LEGEND AND OVERALL GENERAL ARRANGEMENT SEE FIGURE 2.1.
The initial investigations were completed in the early 1960’s and were reported on by Dames and Moore in two reports (Dames and Moore, 1962 and Dames and Moore, 1963). These initial investigations were significant because they were completed at the outset of the development of the tailings facility, and provide the most reliable source of information related to the geotechnical and hydrogeological conditions prior to development. The reports include mapping that delineates the presence of and approximate boundaries for natural soils including recent stream deposits, outwash deposits, and residual soil (completely weathered BQM). Six boreholes (62 T-1 to 62 T-6) were advanced and various laboratory tests were performed on select soil samples to determine surface and sub-surface site conditions. Three boreholes were located in the vicinity of the current East-West Embankment (approximately Section 8+00 W) and three boreholes were located along the current North-South Embankment. The locations generally aligned historically with the embankment that formed the initial impoundment area.

M.K. Botz was retained by TAC in 1969 to supervise drilling beneath the South and Main leach pads to determine the condition of the asphalt pad beneath the Main leach pad and the water transmitting ability of the alluvium beneath both leach pads (Botz, 1969b). These historic leach pads are located downstream of the East-West and North-South Embankments. The South leach pad borders the Precipitation Plant to the north and comprises the lower downstream extent of the East-West Embankment with a bench elevation of approximately 5,950 ft. Nine drillholes (SD-1 to SD-3, MD-1 to MD-5, and MD-5A) were completed in the vicinity of the leach pads. Drillholes SD-1 to SD-3 are the closest and most relevant to the YDTI. The drillholes were advanced initially with a cable tool drill and cased with 6-inch steel casing during drilling. Core samples of the alluvium were collected by driving a 2.5-inch core barrel as the foundation was approached; generally, the bottom of the leach pad to 10 ft below the pad was cored. The recovered alluvium core was reportedly tested for shear strength and permeability; however, the results of this testing are not provided in the report.

The next phase of investigation was completed in 1980 by IECO (IECO, 1981). The IECO study of geotechnical and hydrological conditions was initiated in response to an inspection of the YDTI by the US Army Corps of Engineers (USACE, 1980). A summary of the USACE Inspection Report was provided in the Design Basis Report (KP, 2017a). Fourteen drillholes (DH-1 to DH-13, and DH-6A) were completed in the vicinity of the East-West and North-South Embankment areas with a focus on drilling along the embankment crest and the approximate maximum embankment section (approximately Station 3+00 W) at that time. The drillholes were typically advanced through the rockfill material and tailings into natural ground using a cable tool rig while obtaining samples and penetration resistance with several sampling devices.

Standpipe piezometers were installed upon completion in all fourteen drillholes to measure groundwater levels. The only piezometer that is still monitored from the IECO program is DH-11, which is located at the downstream toe of the East-West Embankment (Station 0+00). The remaining thirteen piezometers have been destroyed or buried. Laboratory testing was completed on selected samples of rockfill, natural soils, and tailings that were recovered during the drilling program and from surface sampling sites. The laboratory tests included index tests, and direct shear and triaxial tests to determine strength parameters for stability analyses. Selected samples were subjected to a series of cyclic-dynamic triaxial compression tests. Results of these cyclic triaxial tests were used in the IECO dynamic stability and liquefaction studies, by Harding Lawson Associates (HLA) in subsequent studies, and are also referenced by KP in current liquefaction analyses (KP, 2017b).
HLA completed two boreholes in 1992 (92-1 and 92-2B) using a mobile drill equipped with a casing advance system to compliment the studies performed by IECO and to further investigate the in-place density of the tailings at the upstream edge of the North-South Embankment (HLA, 1993). A review by HLA of the field investigation procedures and results obtained during the IECO investigation indicated that the low penetration N-values in key boreholes (DH-2 and DH-12) for the dynamic stability assessment could have resulted from heaving and disturbance of the tailings during drilling, sampling, and testing using the cable tool rig. HLA postulated that the measured values by IECO were not necessarily indicative of the in-place density of the tailings material. HLA performed additional field investigations to support a revised evaluation of the potential for soil liquefaction (HLA, 1993).

Borehole 92-1 was completed to calibrate a specialized hammer and to obtain a site-specific correlation between N-values measured using this hammer and those measured using the SPT technique (ASTM D1586-84). Borehole 92-2B was advanced adjacent to the location of DH-2 from the earlier IECO investigations. The tailings encountered consisted of uniformly graded, fine-grained, medium-dense silty sands with occasional lenses of cleaner sands. The corrected tailings N-values measured by HLA ranged from 10 to 20, compared to values of 0 recorded by IECO using the cable tool rig during the 1981 investigation.

MR installed standpipe piezometers in 1993 and 1994 to define and monitor the pore pressure conditions within the tailings embankment subsequent to the HLA study. Piezometers were constructed at seventeen locations in the East-West Embankment and five locations in the tailings basin upstream of the North-South Embankment. The piezometers were mostly completed above the alluvial foundation at the base of the embankment, except for those installed in the tailings basin. The results of the investigation were described in a report authored by MR entitled Yankee Doodle Tailings Impoundment Design and Construction in 1997, revised in 1999 (MR, 1999). Four of these piezometers (93-4, 94-5, 94-8, and 94-11) near the downstream toe of the East-West Embankment are currently still monitored.

The pore pressure monitoring network for the embankment was expanded by MR between 1999 and 2014. Eleven additional standpipe piezometers were installed in the vicinity of the North-South Embankment and four were installed in the East-West Embankment. All of these piezometers, except two that were damaged, are currently still monitored.

KP completed annual site investigation programs between 2012 and 2014 using the ODEX drilling technique with periodic SPT and CPT soundings (KP, 2013; KP, 2014; KP, 2016d). The CPT equipment and methods used during these programs allowed for measurement of penetration (tip) resistance, sleeve friction, and the dynamic pore pressure generated in the tailings. Pore pressure dissipation (PPD) testing was completed every 10 to 15 ft and seismic shear wave velocity measurements were completed every 3 ft. The primary focus of these programs was to collect field information to support geotechnical and hydrogeological characterization of the tailings mass adjacent to the embankment for use in updated liquefaction and stability assessments. The areas investigated were mainly along the East-West Embankment (between Station 18+00W and 53+00W) and along the North-South Embankment (between Station 13+00N and 18+00N) in the vicinity of historic drillholes by IECO and HLA (DH-2 and 92-2B). The tailings characterization is discussed in detail in Section 5 of this report.
Eleven test pits were excavated at the downstream toe of the East-West Embankment in 2014 as a preliminary investigation of the strength of the embankment fill in this area (KP, 2016d). This material had historically been exposed to acidic drainage, and was targeted during the investigation because it was expected to be an area of poor quality degraded rockfill that was easy to access. The exposed materials in the test pit walls and spoil piles were logged for geotechnical characteristics and samples were collected for laboratory index, density, and single stage consolidated undrained (CU) triaxial compression tests to determine the shear strength of the matrix fraction of the material over a range of confining stresses. The triaxial testing was completed on a remolded composite sample of the 1-inch minus fraction of the embankment fill material.

4.2 GEOLOGIC MODEL AND SITE INVESTIGATION PURPOSE

Historically, the YDTI has been developed by progressively placing rockfill material to construct and raise rockfill embankments. Information related to past embankment construction practices is available in the historic reports (Dames and Moore, 1963; IECO, 1981; MR, 1999). The early construction history of the YDTI dating from the early 1950s to 1971 was compiled to determine the geotechnical fabric of the downstream foundation materials incorporated in the initial lifts of the embankment (Applied Geological Services, 2017). The dumping sequence for the central pedestal area was compiled based on available aerial surveys, aerial and oblique photographs, planimetric progress maps, and engineering notes and memoranda dating back to 1954. The report by Applied Geological Services is included as Appendix G1.

Rockfill sourced from the Berkeley Pit was used to construct the embankment until 1982. Since the mid-1980s, rockfill material has been sourced from the Continental Pit for ongoing construction of the embankment. This material boundary gives rise to two sub units of rockfill material designated 'Berkeley Pit rockfill' and 'Continental Pit rockfill' as indicated on Figure 4.3. The limits of historic zones and lifts of the embankment (at Station 8+00W) are shown on Figure 4.3, including the approximate year of construction of each zone dating back to the early 1960s.

The construction chronology of the YDTI is presented in Appendix G2 through a series of aerial photographs taken from 1952 through 2015. Embankment construction began in 1962 with the construction of a downstream starter dike and an additional upstream starter dike added in 1963 to facilitate downstream embankment construction. Initial construction of the North-South and East-West Embankments that flank the facility began during this period. The location of the two starter dikes, the North-South Embankment and the East-West Embankment are visible on Figure G2.2, which shows the YDTI in 1966. Tailings deposition to YDTI began in August 1963. The depression between the upstream and downstream starter dikes was infilled between 1963 and 1970, with the first lift progressing eastward and subsequent infilling from all sides (upstream, downstream and from the west), as shown on Figure G2.2. An oxide leach pile was developed from 1970 through 1972 to the west of the central section, as shown on Figure G2.4. Numerous subsequent lifts to the facility in the central section, North-South and East-West Embankments were completed from the 1970s through 2015. In 2015, a rockfill surcharge (rockfill overlaying tailings beach material) was added to the facility as shown on Figure G2.12.

The rockfill used to construct the YDTI embankments comprises pit-run material end-dumped in 30 to 100 ft lifts by the mine haul fleet. Material segregation occurs as the rockfill is end-dumped from the crest of each lift. Finer particles tend to accumulate near the top of the lift while the coarser
material (cobbles and boulders) rolls further down the slope and accumulates at the toe. This results in the formation of a course segregated rubble layer at the bottom of each lift. In addition, compacted horizontal layers are commonly encountered at the top of the lifts where surfaces were subjected to haul truck traffic prior to and during placement of subsequent lifts. Ripping of these compacted lift surfaces has been commonly completed to enhance vertical infiltration.

As shown on Figures G2.4 through G2.12, historic lifts to the YDTI (1956 through 1978) were predominantly completed using a mix of upstream, downstream and lateral (across the dam face) end-dumping directions. More recent embankment construction was comprised predominantly of lateral end dumping along the axis of the embankment, as shown for example on Figure G2.10. A geotechnical conceptual model of the rockfill embankment material fabric resulting from the complex end-dumping history at the YDTI is shown for Section 8+00 on Figure G2.13 (Appendix G2). In addition, an inter-fingered material fabric of alternating fine-grained and coarse-grained layers exist within each lift. The presence of alternating coarse and fine layers is inferred to be a result of variable rockfill lithology (Herasymuik Et Al., 2006). An example of these inter-fingered inclined layers present in the Great Northern Dump located southeast of the YDTI is shown in Photograph 4.1. These layers dip at angle of repose and are oriented in the direction in which the rockfill was dumped. The complex, multi-directional end-dumping history at the YDTI embankments (as shown in Appendix G2) suggests that material fabric within the East-West Embankment is oriented in three primary directions – downstream, upstream and laterally (across the embankment face). Layers of the material fabric are inferred to be discontinuous between adjacent lifts variation in end-dump orientation and/or due to misalignment of coarse/fine layers between adjacent lifts.

Photograph 4.1 Inter-Bedded Rockfill Layering in the Great Northern Dump

A phased site investigation program was completed in 2015 to support the design of the YDTI. Phase 3 of the investigation program included five drillholes (DH15-S1 to DH15-S5) in the vicinity of the East-West Embankment (between Station 0+00 and 8+00W) at the downstream toe and extending from the crest of the embankment into the natural soils and bedrock beneath it. A sonic
drilling technique was chosen to provide continuous core recovery through the fill materials and natural soils. Drillholes were terminated at the bedrock contact. The 2015 site investigation focused on the maximum section of the East-West Embankment (Station 8+00W). The potential for variability in embankment materials across the site was considered in the shear strength characterization described in the sections that follow.

The following ‘expected’ conditions in the embankment were presumed at the outset of the 2015 drilling program due to the historic material sources and construction practices, and based on the findings of the previous investigations:

- A variation in gradation of the fill materials would be encountered with coarser materials at the base of the historic 50 ft lifts and finer material at the top.
- Coarse rubble zones of the historic lifts underlain at times by historic haul truck running surfaces, and where perched water with low pore pressures might be encountered.
- Inter-beded fine-grained and coarse-grained layers present within a lift.
- Deeper (older) lifts of fill material may have degraded over time due to chemical weathering.
- Groundwater would be encountered towards the base of the embankment.
- Embankment fill is underlain by alluvial sediments in the historic stream channel followed by completely weathered bedrock (residual soil) and intact bedrock.

Detailed geotechnical logging of the embankment fill materials was completed with the specific focus of identifying variation in strength characteristics, particle size, particle hardness and material weathering. The purpose was to understand and document the nature and variability of the fill materials supporting a characterization of the shear strength properties of the rockfill material for global stability analyses.
NOTES:

1. COORDINATE SYSTEM IS ANACONDA MINE GRID.

2. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.

3. PIEZOMETRIC LEVELS (W.L.) SHOWN USING OCTOBER 2016 READINGS UNLESS OTHERWISE INDICATED.

4. THERMISTOR TEMPERATURE PROFILE SHOWN ON FIGURE 4.6.

5. SOL STANDS FOR SETTING OUT LINE.

LEGEND:

- TAILINGS
- MINING PIT ROCKFILL
- CONTINUOUS ROCKFILL
- EMBANKMENT FILL
- SILTY SAND / ALLUVIUM
- THERMISTOR
- THERMISTOR STRING
- PIZZOMETRIC HEAD (FEET)
- VIBRATING WIRE PIZZOMETE
4.3 MATERIAL TYPES

4.3.1 General

The characterization of the shear strength of the materials described in the following sections focuses on the rockfill forming the embankment and the alluvium in the foundation. These two materials control the geotechnical stability of the impoundment. A high friction angle and high cohesion value consistent with the strength of massive BQM are applied to bedrock in the stability assessment (KP, 2017b), which has the effect of limiting critical slip surfaces to the embankment and alluvium. No adversely oriented joint sets are recognized that would comprise the strength of the underlying bedrock, and pervasive hydrothermal alteration is diminished to the North and East of the Berkeley Pit (MR, 1999).

Therefore, this section of the site characterization is focused on the rockfill and the alluvium, which is consistent with the historical assessments of stability for the YDTI. The geotechnical conditions of the tailings encountered in the impoundment are described in Section 5.

4.3.2 Rockfill

The shear strength characterization of the rockfill adopted for engineering stability analyses historically adopted a uniform drained friction angle between 35 and 38 degrees. The friction angle used in the analysis was supported by direct shear testing (Dames and Moore, 1962), triaxial shear testing (IECO, 1981), and field observations of the constructed embankments. MR prepared an inventory of embankment bench slopes based on 1994 aerial mapping. 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 (MR, 1999). The mapping is consistent with visual observations of current conditions.

Rockfill strength parameters have been assessed based on research by Thomas Leps (Leps, 1970). Leps compiled and analyzed published data for individual large scale triaxial tests on gravels and rockfill, and for comparison included his own research data on the shearing strength of sands. Leps developed a series of relationships from his analysis to show friction angle as a function of normal pressure. The Leps non-linear shear strength functions recognize that rockfill (and sand) can maintain a higher angle of friction at a lower confining pressure and lower friction angle at higher pressures. Leps recognized the following general trends in the data:

- Increasing relative density at a given normal pressure results in an increased friction angle.
- Improving the gradation of rockfill, provided it does not increase the fines content, increases the friction angle at a given normal pressure.
- A broad classification of particle strength may be meaningful based on the unconfined compressive strength of the particles as follows:
  - Weak rock particles ranging 500 to 2,500 psi
  - Average rock particles ranging 2,500 to 10,000 psi
  - Strong rock particles ranging 10,000 to 30,000 psi
- Increasing particle angularity results in an increased friction angle.
- Saturated particles are less strong than dry ones.
Several non-linear shear strength functions resulted from the analysis by Leps and the general trends in the data. Leps established a strength function for average rockfill, which represents his judgement of the median strength of all the tests analyzed. Relationships were provided for an upper boundary for well graded, high density rockfill with strong particles and a lower boundary for poorly graded, low density rockfill with weak particles. An angular sand function was also determined based on his research data.

The recent KP drilling investigations (KP, 2016g) collected continuous core samples of the embankment rockfill by sonic drilling from three benches of the East-West Embankment. The drillholes (DH15-S3, DH15-S4, and DH15-S5) were advanced to depths between 305 and 727 ft below the existing embankment crest elevation and reached natural ground beneath the embankment. The rockfill encountered was highly variable, and generally consisted of 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. The interpretation of the conditions encountered in each of these drillholes is included on Figure 4.4.

Figure 4.4 indicates the following information for each drillhole:

- Qualitative descriptions of particle size distribution with depth
- Alteration of the rockfill (BQM, supergene argillic, grey sericite, and leached capping)
- Zones of increased weathering
- Possible presence of historic haul road surfaces
- Moisture condition of the recovered core and iron oxide staining in sulphide materials, and
- The location of instrumentation and piezometric levels.
Figure 4.4

LEGEND:

ROK: WHITE QUARTZ VOLCANICS
SA: COMPACTION ANOMALY
MLD: WET ZONE, BILLY BALEY AND LOADED CAP
GS: GREY SANDSTONE

t: WATER LEVEL

NOTES:
1. ELEVATIONS BASED ON COMMON POINTS
2. ELEVATIONS AND SLOPES ARE APPROXIMATE UNLESS OTHERWISE NOTED.

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EAST - WEST EMBANKMENT
SECTION 4+50 W
DRILLHOLE INTERPRETATION

FIGURE 4.4
Drillhole DH15-S3 was advanced from the lower bench of the embankment from EL. 5,911 ft to a depth of 305 ft. The slope of the lower bench of the embankment is shown on Photograph 4.2. The drillhole intersected natural ground at a depth of approximately 288 ft (EL. 5,623 ft) and encountered 16 ft of alluvium and completely weathered bedrock before terminating in moderately weathered BQM bedrock. The surface of the embankment slope of this lower bench is oxidized to a light brown color and some minor rill and gully erosion is apparent from surface runoff where fine-grained materials are exposed on the slope.

Drillhole DH15-S3 encountered gravel, cobbles, and boulders within a matrix of silty to clayey sand. The upper 72 ft of the drillhole encountered an oxidized light brown colored rockfill consistent with the surface exposures along the slope depicted in Photograph 4.1. The middle portion of the drillhole encountered 125 ft of continuous sulphide rockfill, which was mostly dry and unweathered with clasts ranging from friable to competent. The sulphide rockfill in this area may be indicative of an area of the historic (South) leach pad that was infrequently leached or not leached at all. The remaining rockfill below a depth of 197 ft was highly variable containing a mix of sulphide and leached capping rockfill sources. Weathering and moisture increased below a depth of 240 ft (EL. 5,670 ft). One VWP was installed within the alluvium in the foundation upon completion of the drillhole and measures a piezometric elevation of 5,677 ft, which is approximately 54 ft above the natural ground surface within the embankment rockfill. The piezometric level is consistent with increased moisture in the rockfill core recovered at this depth.
Drillhole DH15-S4 was advanced from a bench at EL. 6,145 ft to a depth of 527 ft. The drillhole intersected natural ground at a depth of 494 ft (EL. 5,651 ft) and encountered 30 ft of alluvium and completely weathered bedrock before terminating in moderately weathered BQM bedrock. The material encountered consisted of gravel, cobbles, and boulders within a silty sand or sandy silt matrix with periodic finer grained (sandy clay) intervals. It is surmised that some degradation of the original larger sized rockfill has occurred since initial placement. Much of the recovered rockfill core consisted of friable, highly altered and weathered materials with slightly weathered materials interspersed. Weathering and moisture content appeared to increase below a depth of 395 ft (EL. 5,750 ft). A zone of leached capping material that has degraded to a hard silty sandy clay with some gravel and cobbles was encountered at a depth of 435 ft. Two VWPs were installed upon completion of the drillhole. One VWP is positioned in the alluvium in the foundation (EL. 5,625 ft) and measures a piezometric elevation of 5,720 ft, which is 69 ft above the natural ground surface. The second VWP is positioned in the rockfill 100 ft above (EL. 5,725 ft) and measures a piezometric elevation of 5,792 ft, indicating a downward gradient.

Drillhole DH15-S5 was advanced from the present day crest of the embankment at EL. 6,373 ft to a depth of 727 ft. Natural ground was intersected at a depth of 717 ft (EL. 5,656 ft) and encountered 9 ft of alluvium and completely weathered bedrock before terminating at the contact with moderately weathered BQM bedrock. The material consisted of varying mixtures of gravel, cobbles and boulders within a silty clayey sand matrix. The boundary between the Continental and Berkeley rockfill is at a depth of roughly 200 ft in the drillhole. No distinguishing characteristics are apparent near the interface of these two zones, or more broadly between the 200 ft thickness of Continental rockfill and 500 ft thickness of Berkeley rockfill. The rockfill encountered was highly variable in grain size distribution, geologic alteration of the rockfill, clast strength, and subsequent degradation since its initial placement. Four VWPs were installed in the rockfill of the embankment upon completion of the drillhole at EL. 6,216 ft, EL. 6,023 ft, EL. 5,848 ft, and EL. 5,666 ft. The bottom VWP is positioned just above the alluvium and measures a piezometric elevation of 5,772 ft, which is 116 ft above the natural ground surface. The remaining higher elevation VWPs report low piezometric levels and indicate a strong downward gradient in the embankment and likely perched water conditions.

Observations made during drilling did not support a differentiation of shear strength parameters by historic pit source. The weakest Leps (1970) relation of Angular Sand was conservatively assigned to all historic rockfill material for the stability analyses in recognition of the potential for site wide variability and long-term degradation of the rockfill material in closure. Use of this function may be somewhat conservative, particularly where historically the most durable rockfill materials have been placed. New rockfill material from the Continental Pit for future raises of the embankment was assigned the slightly stronger Lower Boundary relationship. Additional detail describing the stability analyses is provided in the Stability Assessment Report (KP, 2017b).

4.3.3 Surficial Materials

The surficial materials in the vicinity of the YDTI have historically been referred to generally as alluvium. The surficial materials located in the vicinity of the East-West and North-South Embankment areas were distinguished into two types, recent stream deposits and outwash deposits, as part of the amalgamated mapping described in Section 2.6.2. A description of the nature, variability and condition of these surficial materials prior to substantial development of the YDTI is provided below.
The recent stream deposits are present along the historic Silver Bow Creek channel and its tributaries. The developed channels were filled with dark brown well-sorted and moderately-loose sands and gravels with occasional isolated lenses of silt. The stream deposits are approximately 800 ft wide and up to 45 ft deep (Dames and Moore, 1963). The recent stream deposits generally underlie the existing East-West Embankment.

A broad band of alluvial and outwash material is located directly east of the historic Silver Bow Creek channel and forms a moderately-sloping plain adjacent to the East Ridge. The outwash deposits increase in depth to more than 80 ft in the southern portion of the site. Material consisting of silt to cobble sized particles make up the upper soils. The deeper soils are interbedded with firm silts, sandy loams and sands (Dames and Moore, 1963). The outwash materials generally underlie the existing North-South Embankment.

Botz (1969b) described the ground conditions encountered during drilling as consisting of interlayered mixtures of sand, silt, and clay, with some gravel and scattered rocks. The alluvium material appeared to be deposited by streams and was most probably derived from erosion of altered granitic rocks located to the west of the existing leach pads. Alluvium penetrated during drilling reportedly contained layers of relatively clean sand and thin layers of sandy-clay. Quartz, feldspar, and gold-colored mica were the most common mineral constituents in the alluvium. Gravel and rocks in the alluvium were of granitic composition. Bedrock beneath the alluvium consisted of weathered and altered granitic rock that was so weathered that the core samples were the only reliable means for distinguishing weathered bedrock from alluvium. Bedrock was encountered in drillhole SD-3 at a depth of 13 ft, and was not encountered in any other drill hole.

IECO sampled the alluvium as part of the investigative program and described the material as silty, clayey, and gravelly sands. The foundation materials were encountered in a dense to very dense condition. The alluvium was underlain by weathered quartz monzonite in the form of very dense clayey sand that graded into less weathered bedrock with depth (IECO, 1981). The descriptions provided in the IECO report are consistent with more recent findings by KP in the vicinity of the West Embankment (see Section 3).

KP encountered the alluvium below the embankment fill at depths down hole of between 288 ft and 717 ft (between EL. 5,623 ft and 5,656 ft). The alluvium was generally a dark brown to medium grey, dense to very dense, silty sand with gravel. Figure 4.5 shows photographs of the material recovered during sonic drilling in drillhole DH15-S3. The material grades from embankment fill through approximately 10 ft of alluvium into completely weathered bedrock that is very similar to the overlying alluvium, and then terminates in more competent grey BQM at the end of the drillhole.

Disturbed samples of alluvium were collected at approximately 720 ft depth in DH15-S5 near the top of natural ground. The grain size distribution of the alluvium was approximately 2% gravel, 80% sand, 15% silt, and 3% clay, which is consistent with the surficial materials encountered in the West Embankment area (see Section 3.4.2). Two consolidated undrained (CU) triaxial tests were completed on samples remolded to a dry density of 100 pcf. The results of the triaxial testing are shown in Table 4.2.
Table 4.2  CU Triaxial Testing of Remolded Alluvium Samples

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Effective Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Phi (degrees)</td>
</tr>
<tr>
<td>DH15-S5-25</td>
<td>29</td>
</tr>
<tr>
<td>DH15-S5-27</td>
<td>27</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Triaxial test results are included in Appendix D of Phase 3 Sonic Drilling Program Summary (KP, 2016g).

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

Direct shear tests were performed on undisturbed samples of natural soils by Dames and Moore (Dames and Moore, 1962 & 1963). Samples were typically tested at two surcharge pressures, and the results were plotted to illustrate the relationship between normal pressure and shearing strength. The results showed a wide range of variability, but the material was generally assumed to be non-cohesive with angles of internal friction in the range of 16 to 42 degrees (MR, 1999).

The shear strength characterization of the alluvium for the stability assessment (KP, 2017b) adopted a base case friction angle of 27 degrees (for overburden as described in that report) based on the recent CU triaxial testing completed on the remolded samples. A sensitivity analysis was also completed to determine the effect of decreasing the shear strength of the alluvium on the global stability of the embankment (KP, 2017b).
SITE CHARACTERIZATION REPORT

REPRESENTATIVE STRATIGRAPHIC PROFILE OF EMBANKMENT FOUNDATION MATERIALS

MONTANA RESOURCES, LLP

YANKEE DOODLE TAILINGS IMPOUNDMENT

SITE CHARACTERIZATION REPORT

REPRESENTATIVE STRATIGRAPHIC PROFILE OF EMBANKMENT FOUNDATION MATERIALS

FIGURE 4.5
4.4 MONITORING INSTRUMENTATION NETWORK

4.4.1 General

Piezometric conditions within the East-West and North-South Embankments are monitored using a network of VWPs and standpipe piezometers at the locations shown on Figures 4.1 and 4.2. These locations continue to be actively monitored, as discussed in the sections that follow.

4.4.2 Standpipe Piezometers

Seventeen (17) standpipe piezometers are actively monitored in the vicinity of the East-West and North-South Embankments. Piezometer details and recent calculated piezometric elevations for the active sites are presented in Table 4.3. Water levels in the wells are measured manually using a water level meter tape. Manual water level plots are provided as Appendix E3.

Table 4.3 East-West and North-South Embankment Active Standpipe Piezometers

<table>
<thead>
<tr>
<th>Standpipe Piezometer ID</th>
<th>Total Depth (ft bgs)</th>
<th>Screen Interval (ft bgs)</th>
<th>Piezometric Elevation (ft)</th>
<th>Date of Most Recent Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>DH-11</td>
<td>80.0</td>
<td>70.0</td>
<td>80.0</td>
<td>5671</td>
</tr>
<tr>
<td>MW93-4</td>
<td>45.9</td>
<td>35.3</td>
<td>44.9</td>
<td>5668</td>
</tr>
<tr>
<td>MW94-5</td>
<td>241.0</td>
<td>217.6</td>
<td>236.9</td>
<td>5672</td>
</tr>
<tr>
<td>MW94-8</td>
<td>61.6</td>
<td>51.5</td>
<td>60.7</td>
<td>5672</td>
</tr>
<tr>
<td>MW94-11</td>
<td>284.6</td>
<td>247.3</td>
<td>276.3</td>
<td>5677</td>
</tr>
<tr>
<td>MW12-01</td>
<td>618.5</td>
<td>552.9</td>
<td>612.5</td>
<td>5909</td>
</tr>
<tr>
<td>MW12-02</td>
<td>541.6</td>
<td>439.5</td>
<td>539.5</td>
<td>5972</td>
</tr>
<tr>
<td>MW12-03</td>
<td>597.7</td>
<td>489.5</td>
<td>589.5</td>
<td>5917</td>
</tr>
<tr>
<td>MW12-04</td>
<td>360.0</td>
<td>334.5</td>
<td>354.5</td>
<td>6064</td>
</tr>
<tr>
<td>MW12-05</td>
<td>159.5</td>
<td>131.5</td>
<td>151.5</td>
<td>Dry ²</td>
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<tr>
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<td>240.0</td>
<td>214.5</td>
<td>234.5</td>
<td>5914</td>
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<tr>
<td>MW12-07</td>
<td>65.5</td>
<td>40.0</td>
<td>60.0</td>
<td>6304</td>
</tr>
<tr>
<td>MW13-01</td>
<td>300.9</td>
<td>198.5</td>
<td>298.5</td>
<td>5936</td>
</tr>
<tr>
<td>MW13-02</td>
<td>260.0</td>
<td>194.5</td>
<td>254.5</td>
<td>5901</td>
</tr>
<tr>
<td>MW13-03</td>
<td>160.0</td>
<td>134.5</td>
<td>154.5</td>
<td>5954</td>
</tr>
<tr>
<td>MW14-01</td>
<td>716.0</td>
<td>634.5</td>
<td>714.0</td>
<td>5768</td>
</tr>
<tr>
<td>MW14-02</td>
<td>Installation details unavailable ¹</td>
<td></td>
<td></td>
<td>5985</td>
</tr>
</tbody>
</table>

NOTES:
1. Standpipe piezometers shown in this table were installed under the direction of MR personnel. Completion log for MW14-02 was not provided and is not included in Appendix A.
2. Dry to the bottom of MW12-05 screened zone at EL. 6,194 ft.
4.4.3 Vibrating Wire Piezometers

Five drillholes with a total of 13 VWP sensors were installed in the East-West Embankment during the 2015 Phase 3 site investigation program completed by KP. The installation details and piezometric data recorded on October 26, 2016 are presented in Table 4.4. Piezometric data are recorded at 12-hour intervals using a datalogger located at surface near each drillhole. Time-series plots of piezometric elevation at the sites are presented in Appendix E1.

Table 4.4 East-West and North-South Embankment Drillhole Summary

<table>
<thead>
<tr>
<th>Drillhole ID</th>
<th>VWP Sensor #</th>
<th>VWP Serial #</th>
<th>Sensor Depth</th>
<th>Sensor Elevation</th>
<th>Pressure on Sensor (ft water)</th>
<th>Piezometric Depth</th>
<th>Piezometric Elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft bgs)</td>
<td>(ft)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DH15-S1</td>
<td>VW1</td>
<td>1522035</td>
<td>66</td>
<td>5,609</td>
<td>60</td>
<td>60</td>
<td>5,669</td>
</tr>
<tr>
<td></td>
<td>VW2</td>
<td>1522037</td>
<td>36</td>
<td>5,639</td>
<td>33</td>
<td>33</td>
<td>5,672</td>
</tr>
<tr>
<td></td>
<td>VW3</td>
<td>1522039</td>
<td>22</td>
<td>5,653</td>
<td>15</td>
<td>14</td>
<td>5,667</td>
</tr>
<tr>
<td>DH15-S2</td>
<td>VW1</td>
<td>1522034</td>
<td>56</td>
<td>5,600</td>
<td>49</td>
<td>49</td>
<td>5,649</td>
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<tr>
<td></td>
<td>VW2</td>
<td>1522036</td>
<td>36</td>
<td>5,620</td>
<td>31</td>
<td>31</td>
<td>5,651</td>
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<tr>
<td></td>
<td>VW3</td>
<td>1522038</td>
<td>16</td>
<td>5,640</td>
<td>11</td>
<td>11</td>
<td>5,651</td>
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<tr>
<td>DH15-S3</td>
<td>VW1</td>
<td>1522033</td>
<td>300</td>
<td>5,611</td>
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<td>66</td>
<td>5,677</td>
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<tr>
<td>DH15-S4</td>
<td>VW1</td>
<td>1525992</td>
<td>520</td>
<td>5,625</td>
<td>98</td>
<td>95</td>
<td>5,720</td>
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<td></td>
<td>VW2</td>
<td>1522032</td>
<td>420</td>
<td>5,725</td>
<td>68</td>
<td>353</td>
<td>5,792</td>
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<tr>
<td>DH15-S5</td>
<td>VW1</td>
<td>1520142</td>
<td>707</td>
<td>5,666</td>
<td>108</td>
<td>106</td>
<td>5,772</td>
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<td></td>
<td>VW2</td>
<td>1405595</td>
<td>525</td>
<td>5,848</td>
<td>17</td>
<td>16</td>
<td>5,864</td>
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<td></td>
<td>VW3</td>
<td>1405596</td>
<td>350</td>
<td>6,023</td>
<td>7</td>
<td>6</td>
<td>6,029</td>
</tr>
<tr>
<td></td>
<td>VW4</td>
<td>1503822</td>
<td>157</td>
<td>6,216</td>
<td>0</td>
<td>Dry</td>
<td>&lt; 6,216</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Coordinates are in Anaconda Mine Grid. Collar survey was provided by Montana Resources.
2. Piezometric depth and elevations are presented for measurements recorded on October 26, 2016 unless otherwise noted.
3. Piezometric depth and elevation presented for DH15-S2 was recorded on October 22, 2016.
4. VW4 in DH15-S5 was intentionally installed at a shallower depth to confirm drained conditions.

4.4.4 Thermistor Installation

A thermistor was installed in DH15-S4 to monitor temperatures at discrete depths within the embankment in an effort to evaluate exothermic chemical alteration processes within the rockfill. The thermistor string has 20 thermistor sensors (nodes) installed at 20 ft intervals from ground level to 400 ft below ground surface. One of the thermistor nodes (Thermistor 9) stopped functioning immediately after installation. The embankment temperature profile with depth from the thermistor string is presented on Figure 4.6 and includes five temperature profiles since installation a little over a year ago. Ground temperatures will continue to be monitored at this location on an ongoing basis.
NOTES:
1. THERMISTOR NODE 9 (INSTALLATION DEPTH = 180 ft bgs) IS NOT FUNCTIONAL AND HAS BEEN OMITTED FROM THIS PLOT.
2. THE THERMISTOR DATA LOGGER WAS INSTALLED ON THE NOVEMBER 18, 2015.
3. ELEVATION VALUES PLOTTED ON THE VERTICAL AXIS ARE RELATIVE TO ANACONDA MINE GRID DATUM.
4. DEPTH VALUES PLOTTED ON THE VERTICAL AXIS ARE OF VERTICAL DEPTH BELOW THE SURFACE OF THE EMBANKMENT BASED ON AN EMBANKMENT ELEVATION OF 6,145 ft AT THE THERMISTOR LOCATION.
4.5 HYDROGEOLOGICAL CONDITIONS

Seepage is commonly observed as toe discharge at the base of the embankment and as perched flow on the top of the lower bench (known as the Number 10 Seep) as shown in Photograph 4.3 and indicated on Figure 4.7. Drilling and VWPs installed in the embankment rockfill indicate Number 10 Seep is well above the phreatic surface, which is located along the base of the embankment. The rockfill at and above the elevation of Number 10 Seep is predominantly unsaturated. As such, the presence of seepage at Number 10 Seep is inferred to be a result of perched drainage within the embankment rockfill. Figure G.7 (Appendix G) shows two possible pathways for perched seepage flow through the heterogeneous embankment to the location of Number 10 Seep. The first potential pathway may convey seepage from the North-South Embankment along an historic pipeline alignment and a buried haul truck ramp to the location of the seep. The second possible seepage pathway may convey seepage along a buried tailings pipe ramp through the central embankment section.

Both the toe discharge and Number 10 Seep flow are collected at Horseshoe Bend (HsB) and the Houligan Pond, and are incorporated into the Precipitation Plant.

Photograph 4.3 Number 10 Seep (EL. 5,920 ft)

The East-West and North-South Embankments are inferred to be well drained and largely unsaturated. The inferred phreatic surface within the embankment along Section 8+00W is shown on Figure 4.8. Piezometric elevations drop sharply from approximately 6,275 ft within the YDTI tailings beach to approximately 5,650 ft at the embankment toe as shown on Figure 4.3. The largest drop in piezometric elevation occurs between CPT14-01A and DH15-S5 as a result of the higher
permeability and free draining nature of the rockfill material relative to the tailings mass. VWP sensors installed in drillhole DH15-S5 installed in the embankment crest report hydraulic pressures that range from zero feet (unsaturated) at 157 ft bgs to 17 ft at a depth of 525 ft bgs. The strong downward gradient between the sensors and the minimal pressure reported at the sensors suggests perched water conditions are present at or between the sensors. Monitoring well MW14-01 is located south of drillhole DH15-S5 and is screened through the bottom 80 ft of the embankment. The water level reported in monitoring well MW14-01 is 5,769 ft, consistent with the piezometric elevation of the lowest sensor in drillholes DH15-S5. The phreatic surface within the embankment downstream of the crest sits deep within the embankment within the bottom 50 to 120 ft of rockfill, and is expected to vary depending on rockfill hydraulic conductivity and the original ground elevation along the base of the permeable embankment rockfill. Historic piezometric elevations recorded within embankment rockfill in the 1980’s indicate that the phreatic surface within the embankment has remained relatively constant elevation from 1980 through present day (2017). For example, a water level recorded at DH-3 along Section 8+00 (Figure 4.3) indicates that the phreatic surface is approximately 5,783 ft in February, 1981. Water levels recorded in October 2016 at MW14-01 and DH15-S5 nearby indicate a similar phreatic surface elevation of approximately 5,775 ft.

Hydraulic conductivity of embankment rockfill material was estimated from hydraulic response testing of standpipe piezometers installed within embankment rockfill. Testing was completed by Braun Intertec in 14 standpipes within the East-West and North-South Embankments (Braun Intertec, 1995). Testing results indicate that rockfill has a geometric mean hydraulic conductivity of 7.2x10^{-4} ft/s (2.2x10^{-4} m/s) and range between 2.3x10^{-4} ft/s (7.0x10^{-5} m/s) to 1.8x10^{-3} ft/s (5.5x10^{-4} m/s). These hydraulic conductivity measurements are representative of the bulk rockfill material surrounding the standpipe piezometer screen interval. Hydraulic conductivity of rockfill material is expected to vary depending on original rockfill lithology and based on spatial distribution of grain-size, as discussed in Section 4.2.

Perched water conditions are expected within the embankment rockfill, particularly within interbedded dipping fine layers present within the lifts and on historic road surfaces. Descriptions of an intermittent presence of increased moisture content in the upper and mid portions of the drillholes in the rockfill support this interpretation as presented previously on Figure 4.4 and support the conceptual model discussed in Section 4.2.

Flow of seepage from the East-West Embankment and North-South Embankment is inferred to follow the historic drainages that pre-existed construction of the YDTI. The inferred potentiometric contours and flow directions based on historic drainages within the YDTI are shown in comparison to the existing embankment footprints on Figure 4.9. The topography underlying the embankment suggests that seepage along the East-West and North-South Embankment alignments will flow towards the central embankment section following the historical surface topography and discharge from the embankment to the locations in HSB shown on Figure 4.6.

This interpretation is corroborated by historic interpretations by Dames and Moore from the early 1960s. Water generally flows southward and westward in the more permeable sands of the alluvial outwash deposits located at the eastern and southern margins of the site towards the flow line of historic Silver Bow Creek (Dames and Moore, 1963). Similarly, some small percentage of the water probably flows through the fractures in the bedrock. The groundwater flow regime and soil conditions related to historic Silver Bow Creek were considered particularly significant to project planning during
early development of the facility. An upstream drainage trench connecting to a drainage conduit through the embankment was designed for this groundwater discharge zone. The upstream drainage trench is now buried beneath the embankment fill. These drainage measures still contribute to flows at Horseshoe Bend via the half-round flat-bottom concrete culvert (also known as the Historic Drain).
NOTES:
1. COORDINATE SYSTEM IS ANACONDA MINING GRID.
2. DIMENSIONS AND ELEVATIONS ARE IN FEET, UNLESS NOTED OTHERWISE.
3. PIEZOMETRIC LEVELS (W.L.) SHOWN USING OCTOBER 2016 READINGS UNLESS OTHERWISE INDICATED.
4. THERMISTOR TEMPERATURE PROFILE SHOWN ON FIGURE 4.6.
5. SOL STANDS FOR SETTING OUT LINE.
6. THE LINE DEPICTING THE PHREATIC SURFACE INDICATES MOSTLY UNSATURATED CONDITIONS TO THE LEFT/ABOVE WHILE SATURATED CONDITIONS TO THE RIGHT/BELOW.
7. DISCONTINUOUS ZONES WITH VARIABLE OR PARTIAL SATURATION LEVELS DEPicted WITHIN THE HETEROGENEOUS EMBANKMENT ROCKFILL.
NOTES:
1. COORDINATE SYSTEM AND ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. CONTOURS ARE AT 20 ft INTERVALS AND REPRESENT THE 1956 TOPOGRAPHY OF THE DRAINAGE BASIN.
3. POTENTIOMETRIC CONTOURS REPRESENT OCTOBER 2016 PIEZOMETRIC ELEVATIONS AND WERE DEVELOPED BY HYDROMETRICS.
5 – TAILINGS IMPOUNDMENT GEOTECHNICAL CONDITIONS

5.1 GENERAL

Tailings within the YDTI were classified using index parameters from laboratory testing and based on in-situ data collected during CPT testing during the site investigation (SI) programs between 2012 and 2015. Characterization of tailings material properties, the spatial distribution of tailings within the YDTI and piezometric conditions are discussed in this section.

Four SI programs were completed between 2012 and 2015 to investigate the nature and distribution of tailings materials within the YDTI. The programs included seismic cone penetration testing (CPT) and advancement of geotechnical drillholes within the tailings mass. Tailings samples were collected to support the characterization of tailings material properties and spatial distribution of tailings material types within the YDTI. The pertinent work completed in each of the SI programs is summarized below.

The 2012 Geotechnical Site Investigation (KP, 2013) program was carried out from September 5 to September 24, 2012 and included:

- Drilling of eight geotechnical drillholes within the tailings mass, including:
  - Standard Penetration Tests (SPTs)
  - Geotechnical logging of collected SPT samples
- Completion of five CPTs in tailings, including:
  - Pore Pressure Dissipation (PPD) tests at selected depth intervals
  - Seismic shear wave velocity measurements with depth
- Installation of VWPs at three CPT locations (CPT12-03A, CPT12-04, and CPT12-04A) to monitor the pore water pressure in the tailings mass
- Laboratory testing of tailings samples

The 2013 Geotechnical Site Investigation (KP, 2014) program was carried out from October 1 to October 12, 2013 and included the following components:

- Advancement of two geotechnical drillholes within the tailings mass, including:
  - Standard Penetration Tests
  - Geotechnical logging of collected SPT samples
- Completion of six CPTs, including:
  - PPD tests at selected depth intervals
  - Seismic shear wave velocity measurements with depth
- Installation of settlement monitoring points on the drill pads
- Installation of VWPs at all six CPT locations in the tailings mass
- Laboratory testing of selected tailings samples

The 2014 Geotechnical Site Investigation (KP, 2016d) program was carried out from May 21 to June 2, 2014 and included the following components:

- Completion of six CPTs, including:
  - PPD tests at selected depth intervals
  - Seismic shear wave velocity measurements with depth
- Installation of VWPs at three CPT locations (CPT14-01A, CPT14-02, and CPT14-04) to monitor water levels within the tailings mass
Collecting tailings samples from the tailings beach and drill cuttings
Laboratory testing of test pit and tailings samples

The 2015 Phase 4 SI program was carried out from October 22 to Nov 16 with additional work to finalize VWPs installations completed from December 10 to 15, 2015. The program included the following components:

- Completion of eight CPTs within the tailings beach, along the southern margin of the tailings pond and beneath areas surcharged by rockfill, including:
  - PPD tests at selected depth intervals
  - Seismic shear wave velocity measurements with depth
- Installation of VWPs at six CPT locations in the tailings beach (CPT15-03, CPT15-04 and CPT15-05) and beneath the rockfill surcharge (CPT15-06, CPT15-07 and CPT15-08)
- Laboratory testing of three tailings Shelby samples collected from CPT15-01 at the southern margin of the supernatant pond

A summary of the geotechnical drillholes, CPTs, and monitoring wells completed in the tailings mass is shown in Table 5.1 and the locations are shown on Figure 5.1. Logs are separated by program and provided in Appendix A.
YANKEE DOODLE TAILINGS IMPOUNDMENT

NOTES:
1. COORDINATE GRID IS ANACONDA MINE GRID.
2. CONTOUR INTERVAL IS 25 FEET.

LEGEND:
- DRILLHOLE AND CPTS - KNIGHT PIESOLD 2015
- DRILLHOLE AND CPTS - KNIGHT PIESOLD 2012-2014
- STANDPIPE PIEZOMETERS - MR ENGINEERING DEPT.
## Table 5.1 Summary of Tailings CPT Soundings and Drillholes within YDTI

<table>
<thead>
<tr>
<th>Drillhole/ CPT Location</th>
<th>Easting (ft)</th>
<th>Northing (ft)</th>
<th>Elevation (ft)</th>
<th>Pre-Collar Depth (ft)</th>
<th>Total Depth (ft)</th>
<th>Instrumentation</th>
<th>Samples Collected</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT15-01</td>
<td>134,170</td>
<td>146,589</td>
<td>6,333</td>
<td>-</td>
<td>98.9</td>
<td>-</td>
<td>3 Shelby Tubes</td>
</tr>
<tr>
<td>CPT15-02</td>
<td>135,737</td>
<td>146,924</td>
<td>6,333</td>
<td>-</td>
<td>141.7</td>
<td>-</td>
<td>3 Shelby Tubes</td>
</tr>
<tr>
<td>CPT15-03</td>
<td>135,808</td>
<td>140,587</td>
<td>6,354</td>
<td>-</td>
<td>296.8</td>
<td>3 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT15-04</td>
<td>135,808</td>
<td>141,462</td>
<td>6,354</td>
<td>-</td>
<td>341.7</td>
<td>3 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT15-05</td>
<td>135,810</td>
<td>142,344</td>
<td>6,346</td>
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<td>330.5</td>
<td>3 VWP</td>
<td>-</td>
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<tr>
<td>CPT15-06</td>
<td>134,597</td>
<td>141,303</td>
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<td>49.2</td>
<td>218.9</td>
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<td>-</td>
</tr>
<tr>
<td>CPT15-07</td>
<td>133,663</td>
<td>142,371</td>
<td>6,370</td>
<td>53.3</td>
<td>218.3</td>
<td>2 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT15-08</td>
<td>132,987</td>
<td>143,084</td>
<td>6,370</td>
<td>59.9</td>
<td>183.3</td>
<td>2 VWP</td>
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<td>CPT14-01</td>
<td>135,709</td>
<td>140,374</td>
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<td>200</td>
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<tr>
<td>CPT14-01A</td>
<td>135,697</td>
<td>140,370</td>
<td>6,354</td>
<td>75</td>
<td>194.5</td>
<td>1 VWP</td>
<td>-</td>
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<tr>
<td>CPT14-02</td>
<td>131,585</td>
<td>144,245</td>
<td>6,352</td>
<td>75</td>
<td>194.5</td>
<td>1 VWP</td>
<td>-</td>
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<tr>
<td>CPT14-03</td>
<td>131,551</td>
<td>144,436</td>
<td>6,355</td>
<td>74</td>
<td>194.5</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT14-04</td>
<td>133,546</td>
<td>142,234</td>
<td>6,347</td>
<td>39</td>
<td>192</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CPT13-01</td>
<td>134,393</td>
<td>141,176</td>
<td>6,355</td>
<td>28</td>
<td>194.5</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT13-02</td>
<td>134,567</td>
<td>141,334</td>
<td>6,357</td>
<td>34</td>
<td>201.3</td>
<td>1 VWP</td>
<td>-</td>
</tr>
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<td>CPT13-03</td>
<td>133,531</td>
<td>142,223</td>
<td>6,348</td>
<td>28</td>
<td>206.6</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT13-04</td>
<td>133,644</td>
<td>142,393</td>
<td>6,349</td>
<td>31</td>
<td>210.5</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT13-05</td>
<td>132,803</td>
<td>142,965</td>
<td>6,349</td>
<td>38</td>
<td>112.4</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT13-06</td>
<td>132,989</td>
<td>143,119</td>
<td>6,350</td>
<td>37</td>
<td>164.9</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>DH13-07</td>
<td>133,546</td>
<td>142,234</td>
<td>6,355</td>
<td>28</td>
<td>194.5</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>DH13-09</td>
<td>132,897</td>
<td>143,043</td>
<td>6,350</td>
<td>35</td>
<td>145</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT12-01A</td>
<td>137,390</td>
<td>141,022</td>
<td>6,392</td>
<td>81</td>
<td>134</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>DH12-01B</td>
<td>137,474</td>
<td>141,080</td>
<td>6,394</td>
<td>78</td>
<td>92</td>
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</tr>
<tr>
<td>DH12-03</td>
<td>135,064</td>
<td>140,413</td>
<td>6,354</td>
<td>40</td>
<td>126</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CPT12-03</td>
<td>135,064</td>
<td>140,413</td>
<td>6,354</td>
<td>126</td>
<td>134</td>
<td>-</td>
<td>5 SPT Samples</td>
</tr>
<tr>
<td>CPT12-04</td>
<td>134,973</td>
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<td>206.6</td>
<td>1 VWP</td>
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<td>CPT12-04A</td>
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<td>140,773</td>
<td>6,354</td>
<td>108</td>
<td>212</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>CPT12-05</td>
<td>134,653</td>
<td>140,851</td>
<td>6,354</td>
<td>40</td>
<td>109</td>
<td>1 VWP</td>
<td>-</td>
</tr>
<tr>
<td>DH12-04</td>
<td>134,728</td>
<td>140,773</td>
<td>6,354</td>
<td>35</td>
<td>106</td>
<td>-</td>
<td>4 SPT Samples</td>
</tr>
<tr>
<td>DH12-05A</td>
<td>137,461</td>
<td>141,414</td>
<td>6,325</td>
<td>65</td>
<td>-</td>
<td>-</td>
<td>3 SPT Samples</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Coordinate system and elevations are based on the Anaconda Mine Grid.
2. Pre-collar depth is the depth required to reach tailings material when drilling through overlying embankment fill material.
5.2 GEOLOGIC SECTIONS

A conceptual model of the spatial distribution of tailings within the YDTI was developed from the available field data and an understanding of the long-term tailings deposition methods. Interpretation of CPT results including seismic shear velocity, piezocone tip and sleeve resistance, and normalized soil behavior (SBTn) classifications were used along with tailings laboratory test results. Tailings materials have been separated into three typical zones as follows based on this analysis:

1. Tailings sands comprising the tailings beach
2. Tailings sands beneath the rockfill surcharge, and
3. Tailings slimes in the vicinity of the supernatant pond.

The conceptual model is shown in plan view on Figure 5.2, and cross sections are shown on Figures 5.3 and 5.4.

5.3 MATERIAL TYPES

5.3.1 Depositional Processes in YDTI

The generalized spatial distribution of tailings within the YDTI, as shown on Figure 5.2 and Figure 5.3, is driven by the depositional dynamics and morphology present within the facility. The historical deposition of mine tailings in YDTI is analogous to alluvial fan and delta depositional processes in the natural environment. These processes can be used to explain the tailings beach morphology and the spatial variability in grain sizes within the YDTI. Alluvial fans and deltas are both depositional features formed where a stream undergoes a sudden decrease in energy, resulting in a decrease in sediment transport capacity. The types of environmental settings where these geomorphic features are formed results in certain generalized characteristics, as explained below.

An alluvial fan is a cone-shaped sediment deposition feature that forms where a stream emerges from a confined valley or canyon and flows out onto an unconfined plain. An alluvial fan forms by the deposition of sediment along the stream channel. The stream channel on an alluvial fan typically shifts course laterally back and forth across the fan over time and is often split into multiple branches at any given time, due to the increasing channel bed elevation and lack of lateral confinement. The grain size of deposited sediment typically decreases longitudinally down the fan, as the coarsest sediment carried by the stream is deposited at the fan apex, and successively finer sediment is deposited farther downstream as the stream gradient flattens and the stream energy diminishes.
TAILINGS MATERIAL DISTRIBUTION INTERPRETED FROM NORMALIZED SOIL BEHAVIOR TYPE (SBTn) CLASSIFICATION FROM CPT SOUNDINGS


3. ELEVATIONS ARE IN REFERENCE TO ANACONDA MINE GRID.

TAILINGS - SANDS (COARSER)

TAILINGS - SLIMES

NOTES:

1. TAILINGS MATERIAL DISTRIBUTION INTERPRETED FROM NORMALIZED SOIL BEHAVIOR TYPE (SBTn) CLASSIFICATION FROM CPT SOUNDINGS.


3. ELEVATIONS ARE IN REFERENCE TO ANACONDA MINE GRID.
NOTES:
1. TAILINGS MATERIAL DISTRIBUTION INTERPRETED FROM NORMALIZED SOIL BEHAVIOR TYPE (SBTn) CLASSIFICATION FROM CPT SOUNDINGS.
3. ELEVATIONS ARE IN REFERENCE TO ANACONDA MINE GRID.
4. SUPERNATANT POND ELEVATION REPRESENTS THE ELEVATION DURING THE PHASE 4 2015 SITE INVESTIGATION. THE CURRENT POND ELEVATION IS APPROXIMATELY 6340 ft.

LEGEND:
- TAILINGS - SANDS (COARSER)
- TAILINGS - SANDS (FINER)
- TAILINGS - SLIMES
- SUPERNATANT POND
- INFERRRED PHREATIC SURFACE
- ORIGINAL GROUND 1956
- FUTURE TAILINGS SURFACE
- 2010 TAILINGS SURFACE

SCALE A

MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
TAILINGS MATERIAL DISTRIBUTION
SECTION 1

FIGURE 5.3
NOTES:
1. TAILINGS MATERIAL DISTRIBUTION INTERPRETED FROM NORMALIZED SOIL BEHAVIOR TYPE (SBTn) CLASSIFICATION FROM CPT SOUNDINGS.
2. VWP DATA REPRESENTS READINGS TAKEN ON OCTOBER 26, 2016 EXCEPT: CPT15-05 WAS LAST DOWNLOADED ON JULY 1, 2016, AND CPT15-06 WAS LAST DOWNLOADED ON MAY 4, 2016.
3. ELEVATIONS ARE IN REFERENCE TO ANACONDA MINE GRID.
4. SUPERNATANT POND ELEVATION REPRESENTS THE ELEVATION DURING THE PHASE 4 2015 SITE INVESTIGATION. THE CURRENT POND ELEVATION IS APPROXIMATELY 6340 ft.

LEGEND:
TAILINGS - SANDS (COARSER)
TAILINGS - SANDS (FINER)
ROCKFILL
SUPERNATANT POND
INFERRED PHREATIC SURFACE
ORIGINAL GROUND 1956
2015 TAILINGS SURFACE
FUTURE TAILINGS SURFACE
MATCHLINE
REVIEWED
DATE
DESCRIPTION DRAWN
DESIGNED
REV
MONTANA RESOURCES, LLP
YANKEE DOODLE TAILINGS IMPOUNDMENT
TAILINGS MATERIAL DISTRIBUTION
SECTION 2
FIGURE 5.4
0 11 AUG'17 ISSUED WITH REPORT KTD MJC DDF
A delta is a sediment deposition feature that forms where a stream flows into a body of standing water (i.e. lake or sea). A delta may form as a downstream extension of an alluvial fan if a water body is present. Like alluvial fans, deltas also have characteristic longitudinal gradients and grain size distributions. The coarser fraction of the sediment load that reaches the standing water body deposits close to the edge of the water body in a set of relatively steep bedding layers called foreset beds. Finer sediment is transported farther out into the water body and settles out of suspension to form flatter bottomset beds. As the foreset beds advance outward into the water body over time, they build on top of the previously deposited bottomset beds. Bottomset beds can also become interlayered with the foreset beds if there are periods in which coarse sediment deposition ceases but finer sediment continues to settle out of suspension (i.e. foreset bedding ceases but bottomset bedding continues).

Tailings are transported to the YDTI in a slurry pipeline, which is analogous to a stream with a high sediment load. The slurry exits the pipeline through a discharge point onto the YDTI tailings beach. The discharge point is analogous to the point where a stream emerges from a confined valley. The coarser fraction of the tailings (sand sized material) deposit in a cone-shaped depositional feature, analogous to an alluvial fan, downslope from the pipeline discharge point. The tailings are roughly sorted by grain size as the slurry flows down the fan slope, with the coarsest sand particles deposited near the discharge point and finer sands deposited farther down the fan slope. However, this pattern is disturbed over time as the slurry channel shifts course laterally and goes through phases of aggradation and incision, processes that tend to create complex spatial patterns of grain size within the fan. Tailings were historically deposited in the YDTI using a multiple point slurry discharge system along the East-West Embankment. In 1986 the system was changed to use a single discharge point at the southernmost point of the YDTI. The use of a single discharge point has promoted the formation of a cone-shaped beach. The underlying tailings materials deposited using multiple discharge locations may have more complex spatial patterns of grain size because the location of coarsest sediment deposition (at the spigot or fan apex) varied over time.

The YDTI contains a supernatant pond of standing water, so the tailings fan or beach transitions into a delta along its lower boundary. The finer sands at the lower edge of the fan/beach extending into the pond are analogous to the foreset beds of a delta. Even finer silt and clay sized tailings particles are carried out into the pond in suspension. Some of this material settles out of suspension in the pond, creating fine-grained bottomset beds, or “slimes” deposits, while a very minor portion of the suspended material may be transported with decant water back to the mill. The differentiation between settled sediment and decanted sediment varies over time depending on tailings input rate to the pond (mill production rate), tailings grain size distribution (mill grinding), water volume in the pond, and water decant rate. The foreset beds of fine sand likely advanced out on top of fine-grained bottomset beds of slimes, and the foreset beds are likely inter-layered with pockets of slimes. Variability in YDTI pond level over time likely contributes to an even more complex spatial distribution of grain sizes within the fan/beach and delta. Channel incision probably occurred on the beach during periods of low pond level, resulting in sand transport far out into the pond to interlayer with the bottomset beds of slimes. Conversely, layers of slimes would have been laid down on the fan/beach during periods of high pond level.
5.3.2 Tailings Beach (Sands)

Tailings material within the beach are highly stratified with inter-bedded layers predominantly comprised of sands, sand mixtures, and silt mixtures. Normalized Soil Behavior Type (SBTn) and piezocone tip and sleeve resistance profiles from the CPT locations illustrate the numerous inter-bedded layers present in the tailing. Piezocone tip resistance along a section through the north-south centerline of the tailings beach (including data from CPT15-03, CPT15-04 and CPT15-05) ranges from approximately 10 to 280 tonnes per square foot (tsf) with sleeve friction ranging from 0.1 to 5 tsf. These values are indicative of loose to medium dense tailings materials. The SBTn classifications for the same profile suggest interbedded layers of sand, sand mixtures, and silt mixtures. A similar degree of inter-bedding and similar SBTn classifications are present in the CPT soundings from 2012 through 2014.

Classification of tailings material within the tailings beach was further developed through laboratory testing and geological logging of tailings samples collected as part of 2012, 2013 and 2014 SI programs. A total of 30 SPT samples were collected in 2012 and 2013. In addition, two bag samples were collected in 2014 from CPT14-02 and brass tube samples were collected from the tailings surface in three sample areas (KP, 2016d). Geological logs and particle size analyses for these samples show the tailings to be a mix of sand, silty sand, sandy silt and silt layers. Sand content in the samples ranges from approximately 9% to 85% with an average of approximately 65% sand. Fines content ranges from approximately 14% to 90% with an average of approximately 35% fines. These sample results generally support the SBTn-based material classifications from CPT soundings throughout the tailings beach.

Materials within the tailings beach generally become finer with increasing distance from the tailings discharge point, as shown previously on Figures 5.2 and 5.3. Seismic shear wave velocity measurements were recorded at 17 CPT locations within the tailings beach between 2012 and 2015. Measured shear velocities along the centerline of the YDTI are shown on Figure 5.5 as recorded in CPT15-03, CPT15-04 and CPT15-05 within the tailings beach and at CPT15-02 at the southern margin of the tailings pond. Shear wave velocities are generally highest in proximity to the tailings discharge. Shear wave velocities are comparatively highest in CPT15-03 indicating that coarser materials exist closest to the tailings discharge, as is expected. Shear velocities decrease in CPT15-04 and are then further reduced in CPT15-05 illustrating the presence of finer materials with distance from the discharge point. Similar trends are present in other data collected between 2012 and 2014.

The measured shear wave velocities are shown as Figure 5.5 also illustrate an increasing trend with depth indicating that tailings materials become denser with depth. In CPT15-03, CPT15-04 and CPT15-05, for example, the measured shear wave velocities increase with depth from approximately 450 ft/s to over 1,200 ft/s over the 300 ft deep test interval. Similar increasing trends are apparent in other CPT soundings. The highest measured shear wave velocities, nearly 1,600 ft/s, were encountered in CPT15-03, CPT15-04, and CPT12-05 below an elevation of 6,125 ft where there is a marked increase in shear wave velocity. This increase corresponds to a rise in tip/sleeve resistance encountered in the CPT soundings and a drop in the hydraulic head profiles measured from PPD testing. These three locations are the deepest of the testing programs and their findings indicate the presence of denser, consolidated tailings materials at depth within the YDTI.
NOTES:
1. SEISMIC VELOCITIES WERE MEASURED AT 3.3 ft (1 m) INTERVALS DURING CPT SOUNDING.
5.3.3 Tailings Beneath Rockfill Surcharge (Sands)

An objective of the 2015 Phase 4 SI was to investigate the geotechnical properties of tailings in an area of the tailings beach that has been surcharged with rockfill between Station 23+00 NW and Station 53+00 NW, approximately along the alignment of Figure 5.4. Material classification of tailings within the rockfill surcharge area relies on CPT testing completed in both the 2013 and 2015 SI programs. CPT13-01 through CPT13-06 were pre-collared through rockfill drill pads placed on the tailings surface prior to testing and are representative of a partially-loaded condition. The rockfill surcharge was placed over the entire area between these drill pads between 2014 and 2015. The surcharge was constructed progressively to allow pore pressures to dissipate between loading phases. CPT15-06, CPT15-07, and CPT15-08 were advanced at the same location as CPT13-02, CPT13-04 and CPT13-06, respectively. The thickness of the rockfill surcharge at CPT15-06, CPT15-07, and CPT15-08 during testing was approximately 49 ft, 53 ft and 60 ft, respectively. The surcharge has since been increased by approximately another 50 ft across this area to EL. 6,400 ft.

SBTn classifications from the 2013 and 2015 CPT soundings indicated that the tailings beneath the rockfill surcharge are stratified with inter-beded layers of sand mixtures and silt mixtures with some clay present at depth in CPT15-08 and CPT13-06, furthest from the tailings discharge point. These classifications suggest that tailings materials are of similar composition to tailings materials tested in the tailings beach.

A comparison of seismic shear wave velocities measured at the same locations during CPT testing in 2013 and 2015 is presented on Figure 5.6. Data series are color coded by location and data and are shown using a dashed line from 2013 and a solid line from 2015. Shear wave velocities measured in 2015 increased slightly with depth from approximately 800 ft/s to 1,100 ft/s over an interval of about 160 ft. As shown, the 2015 seismic velocities are comparably higher than those measured in the 2013 CPT soundings at the same locations where velocities ranged from approximately 600 ft/s 1,000 ft/s over a similar depth interval. The increase in seismic velocities from 2013 to 2015 is indicative of the densification of tailings materials in the region between a partially-loaded (drill pads over the CPT locations only) and fully-loaded (continuous rockfill) conditions in the rockfill surcharge area.

A comparison of data from the rockfill surcharge area (CPT15-06 and CPT15-07) with those from the unloaded tailings beach (CPT15-04 and CPT15-05, respectively) is presented on Figure 5.7. The data indicates that the shear wave velocities measured in 2015 are higher in the upper 20 to 30 ft of tailings beneath the rockfill surcharge than at locations in the tailings beach at a similar distance from the tailings discharge point. The increased seismic shear wave velocity is attributed to consolidation and densification of the tailings mass caused by the overlying rockfill surcharge. Shear wave velocities beneath this upper 20 to 30 ft interval are generally similar to those measured in the tailings beach.
NOTES:
1. SEISMIC VELOCITIES WERE MEASURED AT 3.3 FT (1 m) INTERVALS DURING CPT SOUNDING.
2. PRE-DRILLED CASING WAS INSTALLED THROUGH THE ROCKFILL MATERIALS OF THE EAST-WEST EMBANKMENT PRIOR TO CPT SOUNDING, AS SHOWN ON THE PLOTS ABOVE.
3. ELEVATIONS BASED ON ANACONDA MINE GRID

Increase in seismic velocity and tip resistance indicate that the SCPT encountered native ground beneath the tailings material.
NOTES:
1. SEISMIC VELOCITIES WERE MEASURED AT 3.3 FT (1 m) INTERVALS DURING CPT SOUNDING.
2. PRE-DRILLED CASING WAS INSTALLED THROUGH THE ROCKFILL MATERIALS OF THE EAST-WEST EMBANKMENT PRIOR TO CPT SOUNDING, AS SHOWN ON THE PLOTS ABOVE.
3. ELEVATIONS BASED ON ANACONDA MINE GRID

Seismic velocities are higher directly beneath rockfill loading as compared to CPT locations in tailings beach.

Seismic velocities become similar to CPT locations in tailings beach.

Increase in seismic velocity and tip resistance indicate that the CPT encountered native ground beneath the tailings material.
5.3.4 Tailings Pond (Slimes)

Tailings materials in the vicinity of the supernatant pond were investigated in 2015. CPT15-01 and CPT15-02 were advanced at the southern margin of the supernatant pond, as shown on Figure 5.1 with the objective of investigating the geotechnical properties and pore water pressure regime in the tailings slimes. The CPT tip resistance measurements in CPT15-01 and CPT15-02 are in the range of approximately < 1 to 75 tsf with one spike reaching 110 tsf. This range is indicative of finer tailings materials (slimes) that become slightly denser with depth. The SBTn classifications indicate that the tailings are inter-bedded layers of silt-mixtures and clay-mixtures. Measured shear wave velocities at CPT15-01 and CPT15-02 are lower than those measured in the tailings beach and generally increase with depth from approximately 100 ft/s to 750 ft/s. These are typical values for unconsolidated loose silt-clay tailings materials. Shear wave velocity profiles for CPT15-01 and CPT15-02 were shown previously on Figure 5.5.

Three Shelby samples of tailings slimes from CPT15-01 were analyzed in the laboratory to further characterize the material properties of tailings slimes. Results of grain size distribution testing indicate that the sampled materials range from approximately 2% to 48% sand (fine sand) with an average sand content of 26% (KP, 2016e). Sampled fines are predominantly comprised of silt (45% to 80%) with some clay (7% to 17%). The three samples are classified as sandy silt, silt, and silt with sand. These material classifications generally support the SBTn-based classifications from CPT sounding at CPT15-01 and CPT15-02 within the tailings pond.

Tailings slimes (interbedded silt-mixtures and clay-mixtures) are thought to be present throughout the current extent of the YDTI tailings pond. In addition, these deposits are thought to exist further South at increasing depth beneath the tailings beach forming a layer of fine tailings along the bottom of the YDTI, as illustrated on Figure 5.3. These deeper fine tailings deposits are the result of the depositional history in the YDTI and were deposited as the pond moved northward and upward with increasing tailings surface elevation (continued deposition). The presence of this layer could not be confirmed by the CPT testing completed in the tailings beach in 2015 as the depth of these deposits exceeded the capabilities of the testing equipment due to excessive rod flex during CPT sounding.

5.4 MOISTURE CONTENT

The moisture content of the tailings sands ranged from 12% to 39% and generally increased with distance from the tailings discharge point. DH13-09 is situated furthest from the tailings discharge and exhibits the highest measured moisture contents (ranging from 32% to 39% with an average of 37%). Relatively lower moisture contents (ranging from 26% to 35% with an average of 28%) are measured in DH13-07 which is located approximately 2,400 ft closer to the tailings discharge. The lowest moisture contents were measured in samples collected in 2012 in proximity to the tailings discharge. These sample locations are situated in close proximity to the tailings discharge and in an area that is well drained by the East-West Embankment. Samples within the tailings beach do not exhibit a discernable correlation between moisture content and depth beneath the tailings surface. The variability of moisture content within each drillhole is likely due to the influence of inter-layering within the tailings mass and the position over time of the meandering tailings stream within the impoundment.

The moisture content of tailings slimes was investigated through laboratory testing of three Shelby samples collected from CPT15-01 and range from 34% to 46%. The moisture content of tailings
slimes is generally higher than measurements in the tailings beach over the same depth interval. The highest measured moisture content in tailings slimes (46%) is from sample CPT15-01-02 which exhibits the highest fines content (98%) of the three samples. Moisture content data are limited and, as such, no clear trend with depth is apparent. Moisture in the tailings slimes is generally expected to decrease with depth due to the consolidation of tailings fines at depth.

5.5 CONSOLIDATION CHARACTERISTICS OF TAILINGS SLIMES

Laboratory testing was carried out to estimate the consolidation characteristics of the tailings slimes (KP, 2016e). Three tailings slime samples were obtained from CPT15-01 using a stationary piston (passive-suction) sampler and 3-inch Shelby tubes. One-dimensional consolidation testing was completed on tailings samples CPT15-01-01 and CPT15-01-03 to determine the pre-consolidation pressure ($P_c$) and compression index ($C_c$) and to assess the effects of consolidation on tailings void ratio and moisture content. Testing resulted in a $C_c$ of 0.24 and $P_c$ pressures of 12.6 ksf (CPT15-01-01) and 10.9 ksf (CPT15-01-03). Additional details are presented in the factual data memorandum (KP, 2016e).

5.6 SHEAR STRENGTH (TAILINGS SLIMES)

Electronic Vane Shear testing (eVST) was completed in 2015 at CPT15-01 and CPT15-02 within the tailings slimes. Testing suggests an average peak undrained shear strength of approximately 0.07 tsf (values range from 0.03 to 0.13 tsf) and an average remolded shear strength of 0.02 tsf (with a range from 0 to 0.04 tsf). The peak shear strength values from eVST testing in CPT15-01 are in agreement with the shear strengths calculated from the Ball Cone Penetration Testing (BCPTu) sounding. The remolded shear strength estimates from the eVST testing correspond well with the BCPTu remolded cycle testing. A comparison of shear strength values derived from BCPTu sounding, BCPTu cyclic remold testing and eVST testing at CPT15-01 is presented on Figure 5.8.
NOTES:
1. THE SHEAR STRENGTH ESTIMATES ABOVE WERE MEASURED FROM eVST TESTING, BCPTu SOUNDING AND BCPTu CYCLIC REMOLDED TESTING.
2. THE BCPTu BALL REACHED REFUSAL THREE TIMES ON HIGHER DENSITY TAILINGS LAYERS AND REQUIRED DRILL CASING TO BE WASHED THROUGH THE LAYER PRIOR TO RE-STARTING THE BCPTu SOUNDING.
3. ELEVATIONS BASED ON ANACONDA MINE GRID.
5.7 TAILINGS STATE CHARACTERIZATION

Tailings state characterization was completed to preliminarily evaluate liquefaction susceptibility of tailings materials within YDTI. The analysis relies on data collected during piezocone penetration testing (CPTu) to classify tailings material as either potentially dilative or potentially contractive based on CPTu-based soil behavior and excess pore pressure conditions measured during each CPT sounding. Tailings with susceptibility to liquefaction exhibit potentially contractive CPTu soil behavior and require that excess pore pressure – dynamic pore pressure in excess of static pore pressure – is present.

5.7.1 Characterization Methodology

Analyses of state parameter and normalized pore pressure difference were completed to highlight locations where potentially contractive soil behaviors and excess pore pressure conditions are indicative of potentially liquefiable tailings materials within the YDTI. The analyses were completed for the following three regions of the YDTI using data from eight CPT soundings completed during 2015 as listed in parentheses:

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

A state parameter (ψ) analysis was completed to delineate regions where potentially contractive tailings material types exist. In general, negative state parameters (ψ<0) are indicative of potentially dilative material types and positive state parameters (ψ>0) of potentially contractive material types (Winckler et. al., 2014). Jefferies and Been (2006) proposed ψ>-0.05 be used as division between potentially contractive and dilative materials specifically for tailings liquefaction analyses, rather than ψ>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 ψ > 0 and ψ > -0.05 criteria.

The state parameter assessment was supplemented with a pore pressure-based analysis to highlight regions where excess pore pressures are present along with potentially contractive tailings material types. Normalized pore pressure difference (ν) 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 ν (dynamic pressures exceeding static pressures) are potentially contractive while those with a negative ν (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.

5.7.2 Results of State Characterization

Figure 5.9 presents the resulting state parameter profiles for CPT15-03, CPT15-07 and CPT15-02 which are indicative of conditions within the tailings beach, beneath the rockfill surcharge and along the southern margin of the supernatant pond, respectively.
NOTES:
1. ELEVATIONS ARE BASED ON ANACONDA MINE GRID.
2. STATE PARAMETER IS CALCULATED AS THE DIFFERENCE BETWEEN IN-SITU VOID RATIO AND VOID RATIO AT CRITICAL STATE.
3. MATERIALS WITH A NEGATIVE STATE PARAMETER ARE POTENTIALLY DILATIVE AND THOSE WITH A POSITIVE STATE PARAMETER ARE POTENTIALLY CONTRACTIVE.
4. THE YELLOW-DASHED LINE SHOWS AN ALTERNATIVE AXIS DIVISION ($\psi = -0.05$) BETWEEN POTENTIALLY DILATIVE AND CONTRACTIVE MATERIALS AS PRESENTED IN WINCKLER ET. AL. (2014)
CPTs within the tailings beach generally have state parameters on the boundary of potentially dilative and contractive behavior depending on whether the \( \psi < -0.05 \) or \( \psi < 0 \) is used. State parameter plots suggest that potentially contractive material types exist in CPT15-03 and CPT15-04 between 6,275 and 6,200 ft elevation; however, tailings at these elevations do not exhibit excess pore pressures. Predominantly negative normalized pore pressure differences \((\bar{u})\) are present throughout the tailings profile suggesting that although the materials are on the boundary of potentially contractive or dilative based on state parameter, the excess pore pressures required for potentially contractive behavior are generally not present.

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

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

5.7.3 Summary of State Characterization Findings

Based on the normalized pore pressure difference, state parameter and combined analyses the following general conclusions can be made:

- Tailings materials in the tailings beach and beneath the rockfill surcharge are predominantly near to the boundary between potentially contractive or potentially dilative behavior. Much of the data do not satisfy the \( \psi < -0.05 \) criterion for dilative tailings material and some zones of potentially contractive \( (\psi > 0) \) materials exist within the tailings profiles.
- Tailings along the southern margin of the supernatant pond exhibit potentially contractive behavior and therefore have a higher susceptibility to liquefaction during shearing.

The liquefaction assessment for the YDTI is presented in the Stability Assessment Report (KP, 2017b).
5.8 MONITORING INSTRUMENTATION NETWORK

Thirty VWPs were installed at twenty-one CPT locations within tailings in the YDTI to monitor pore water pressure conditions within the tailings mass since 2012. Piezometric data have been collected from the installations on a continuous basis since installation, except during periods when the dataloggers were inactive due to damage or battery failure. Details of the installations and the most recent piezometric elevations available at the time of writing are summarized in Table 5.2. The locations of the VWP installations listed in Table 5.2 were shown previously on Figure 5.1. Time-series plots of piezometric head recorded by the VWPs are provided in Appendix E1.

Table 5.2 Summary of Vibrating Wire Piezometers in Tailings

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<th>VWP Location ID</th>
<th>Collar Elevation at Installation (ft)</th>
<th>VWP ID Number</th>
<th>Sensor Depth (ft bgs)</th>
<th>Sensor Elevation (ft)</th>
<th>Piezometric Depth (ft)</th>
<th>Piezometric Elevation (ft)</th>
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NOTES:
1. Piezometric data presented for CPT15-03, CPT15-04, and CPT15-07 were VWP readings from October 26, 2016.
2. Piezometric data presented for CPT13-01 through CPT13-06, CPT14-04, CPT15-05, and DH15-03A were VWP readings taken on July 10, 2016.
3. Piezometric data presented for CPT14-01A were VWP readings taken on October 21, 2016.
4. Piezometric data presented for CPT14-02 were VWP readings taken on March 15, 2016.
5. Piezometric data presented for CPT15-06 were VWP readings taken on May 4, 2016.
6. Piezometric data presented for CPT15-08 were VWP readings taken on October 28, 2016.
5.9 HYDRAULIC CONDUCTIVITY

Representative bulk hydraulic conductivity values for the main tailings deposit regions within the YDTI were estimated in order to characterize bulk hydraulic properties of the tailings mass. The analyses were completed for the following three regions of YDTI using data collected from the soundings listed in parentheses:

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

Representative hydraulic conductivity values were estimated using two separate methodologies that incorporate results of the SB Tn classification and PPD testing conducted in the 2015 CPT soundings. Both methods estimate a hydraulic conductivity value that is considered to be approximate but generally within the correct order of magnitude. SB Tn-based hydraulic conductivity estimates were calculated by relating the SB Tn soil index parameter ($I_c$), which is a function of normalized piezocone tip resistance and friction ratio, to hydraulic conductivity following the methodology presented by Robertson (2010). An SB Tn-based estimate of hydraulic conductivity is provided every 2 cm (0.78 inch), which is the recording interval of the sounding data acquisition system. The PPD-based methodology was used to check the range of the hydraulic conductivity values estimated using the SB Tn-based approach. The PPD-based methodology provides an estimate of the horizontal hydraulic conductivity at discrete depths within each sounding where PPD tests were completed. The PPD-based method is based on the simplified method developed by Parez and Fauriel and uses the $t_{50}$ time (the time required for 50% dissipation of excess pore pressure) to estimate a potential range of horizontal hydraulic conductivity values. Hydraulic conductivity estimates using the PPD-based approach represent the upper bound of the range as presented in Robertson (2015).

Geometric mean hydraulic conductivity values in each sounding determined using all hydraulic conductivity estimates for each SB Tn-based and PPD-based estimation method are presented in Table 5.3. Geometric mean hydraulic conductivity values for tailings beach material and tailings material located beneath the waste rock surcharge are of similar magnitude. Hydraulic conductivity is locally variable within the tailings profile, as expected, due to the presence of inter-layering material types as discussed in Section 5.3. There is reasonable agreement between the PPD-based and SB Tn-based geometric mean hydraulic conductivity estimates for soundings located in the tailings beach and beneath the rockfill surcharge. A representative hydraulic conductivity of the coarse tailings is between 0.003 ft/day ($1\times10^{-7}$ m/s) and 0.3 ft/day ($1\times10^{-6}$ m/s) based on the data. Hydraulic conductivity estimates for data from the three CPT soundings within the tailings beach exhibit no clear trend with depth or position in the beach. Hydraulic conductivity of tailings beneath the rockfill surcharge generally decrease with increasing distance from the tailings discharge point as finer tailings comprise a greater portion of the grain size distribution.
Table 5.3  Estimated Hydraulic Conductivity of Tailings Sands and Slimes

<table>
<thead>
<tr>
<th>Material Type</th>
<th>CPT Sounding</th>
<th>Geometric Mean Hydraulic Conductivity Estimate (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>SBTn-Based Hydraulic Conductivity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(ft/day)</td>
</tr>
<tr>
<td>Tailings Beach (Sands)</td>
<td>CPT15-03</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>CPT15-04</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>CPT15-05</td>
<td>0.2</td>
</tr>
<tr>
<td>Tailings beneath Rockfill Surcharge (Sands)</td>
<td>CPT15-06</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>CPT15-07</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>CPT15-08</td>
<td>&lt;0.1</td>
</tr>
<tr>
<td>Tailings Fines (Slimes)</td>
<td>CPT15-01</td>
<td>8.1E-04</td>
</tr>
<tr>
<td></td>
<td>CPT15-02</td>
<td>2.1E-03</td>
</tr>
</tbody>
</table>

NOTES:
1. Values represent the geometric mean of all hydraulic conductivity values calculated within the sounding using the method indicated.
2. Pore pressure dissipation (PPD) test completed in fine tailings at CPT15-01, CPT15-02 and CPT15-08 did not fully equilibrate and therefore the PPD-based hydraulic conductivity estimates are not valid for these soundings.
3. SBTn= Normalized Soil Behavior Type, PPD=Pore Pressure Dissipation.
4. The PPD-based method provides an estimate of the horizontal hydraulic conductivity.

Geometric mean hydraulic conductivity values for the tailings slimes located along the southern margin of the supernatant pond are estimated using only the SBTn-based method. The PPD-based hydraulic conductivity estimates are sensitive to the t_{50} dissipation time and an estimate of the equilibrium pore pressure (U_{eq}) is needed for the methodology to be valid. Pore pressures in PPD tests conducted in the finer materials present in CPT15-01, CPT15-02 and at depth within CPT15-08 were slow to dissipate and had not reached equilibrium when the tests were terminated. As a result, U_{eq} and t_{50} values for these soundings are not known and PPD-based hydraulic conductivity estimates for CPT15-01, CPT15-02 and CPT15-08 are not available. Estimates of bulk hydraulic conductivity using the SBTn-based method suggest a representative hydraulic conductivity for the tailings slimes is between 8x10^{-4} ft/day (3x10^{-9} m/s) and 2x10^{-3} ft/day (9x10^{-9} m/s). As expected, these hydraulic conductivity estimates are lower than the values estimated for the tailings material within the beach and beneath the rockfill surcharge.

5.10 TAILINGS PIEZOMETRIC CONDITIONS

Piezometric conditions within the tailings were investigated during CPT testing using PPD tests and are monitored on an on-going basis using a network of VWPs and data loggers installed within the YDTI as described in Section 5.8. The inferred location of the phreatic surface in the tailings mass and immediately surrounding the YDTI are shown in cross-section on Figures 5.3 and 5.4. Vertical profiles of equilibrium pore pressure and hydraulic head calculated using the results of PPD tests conducted during the 2015 CPT soundings are presented on Figures 5.10 and 5.11, respectively.

The elevation of the inferred phreatic surface (water table) within the tailings beach gradually decreases southward towards the East-West Embankment. The phreatic surface shown on Figure 5.3 decreases from the level of the supernatant pond (EL. 6,332 ft) at the southern margin of the
tailings pond near CPT15-01 and CPT15-02 to approximately 6,284 ft near CPT15-03. The water table contours and hydraulic head profiles at CPT15-03, CPT15-04 and CPT15-05 indicate the direction of flow within the tailings beach is downward and toward the embankment. The vertical hydraulic gradient calculated in each of the three soundings is slightly less than hydrostatic at elevations above 6,200 ft, and increases above hydrostatic between elevations of 6,125 ft and 6,200 ft. Hydraulic conditions are generally hydrostatic in the lower portion of CPT15-05, while closer to the embankment in CPT15-03 and CPT-04 the vertical hydraulic gradient increases up to 0.2 (ft/ft). The horizontal hydraulic gradient between the three soundings increases with depth.

The inferred phreatic surface in the tailings beneath the embankment rockfill is similar to that found in the unloaded tailings beach at a similar distance from the East-West Embankment, as shown on Figure 5.4. Results of PPD testing in the CPT soundings indicate the phreatic surface is lowest at CPT15-06 at an elevation of approximately 6,301 ft (23 ft beneath the rockfill) and increases to 6,322 ft and 6,323 ft at CPT15-07 and CPT15-08, respectively. The piezometric elevations at VWPs installed at similar depths in CPT15-06 and CPT15-07 increase with distance from the discharge point. The vertical hydraulic gradient in the three soundings in the tailings beneath the embankment rockfill is similar in character to the vertical hydraulic gradient in the soundings in the tailings beach.

Groundwater elevations in the YDTI pond footprint are controlled by the elevation of the supernatant pond. CPT15-01 and CPT15-02 are located along the southern margin of the tailings pond, and the slimes tailings surface at the time of CPT testing was approximately 1.5 ft below the pond level. The bathymetry of the tailings surface beneath the YDTI pond indicates that the pond is up to 90 ft deep in some areas. PPD tests completed during the soundings at CPT15-01 and CPT15-02 shown on Figure 5.10 indicate that pore pressures at these locations are slightly above hydrostatic in the upper portions of the holes and become less than hydrostatic with depth.
NOTES:
2. EQUILIBRIUM PORE PRESSURES ARE RESULTS FROM PPD TESTING CONDUCTED DURING THE CPT SOUNDINGS.
3. ELEVATIONS BASED ON ANACONDA MINE GRID.
4. PPD TESTS ATTEMPTED AT 26.9, 66.2 AND 98.9 ft bgs IN CPT15-01 DID NOT ACHIEVE AN EQUILIBRIUM PORE PRESSURE.
NOTES:
1. THE HYDRAULIC HEAD DATA PRESENTED ABOVE ARE DERIVED FROM EQUILIBRIUM PORE PRESSURES RESULTING FROM PPD TESTING CONDUCTED DURING THE CPT SOUNDINGS.
2. ELEVATIONS BASED ON ANACONDA MINE GRID.
6 – SUMMARY

Montana Resources completed a phased geotechnical and hydrogeological site investigation program throughout 2015 and 2016 commensurate with the information needs to support the design and permitting of the YDTI. The scope of the site investigation programs was adjusted and expanded as additional information related to the ground conditions became available. The findings of this site investigation work were integrated with information from previous site investigation work that was completed during various stages of design and construction of the YDTI between 1962 and the present.

There have been several investigations completed in the vicinity of the East-West and North South Embankment areas spanning over five decades. The investigations were completed by several different engineering consultants in coordination with the mine operator of the time using a variety of methods. The recent drilling investigations collected continuous core samples of the embankment rockfill by sonic drilling through the existing embankment to natural ground beneath. The rockfill encountered was highly variable, and generally consisted of 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. The variability encountered during the recent investigations, and recognition of the potential for site wide variability and long-term degradation after mine closure suggests that it would be appropriate to adopt conservative shear strength parameters for the rockfill in the static stability analyses.

The elevation of the phreatic surface within the tailings beach gradually decreases southward from the supernatant pond towards the East-West Embankment. The direction of flow within the tailings beach is downward and towards the embankment. The phreatic surface within the embankment downstream of the crest sits deep within the embankment within the bottom 50 to 120 ft of rockfill, and is expected to vary depending on rockfill hydraulic conductivity and the original ground elevation along the base of the permeable embankment rockfill. Perched water conditions are expected within the embankment rockfill, particularly on historic road surfaces. Flow of seepage from the East-West Embankment and North-South Embankment is inferred to follow the historic drainages that pre-existed construction of the YDTI. The topography underlying the embankment suggests that seepage along the East-West and North-South Embankment alignments will flow towards the central embankment section following the historical surface topography and discharge from the embankment to Horseshoe Bend.

Piezometric elevations along the West Ridge typically range from 6,430 ft to 6,440 ft with elevations to the north and south of the ridge increasing to 6,460 ft and 6,480 ft. Groundwater flow in the West Ridge area is influenced by the presence of structural lineaments. A shallow piezometric low exists in the centre of the West Ridge and is structurally bounded to the north and south by east-west striking lineaments. The locally depressed groundwater elevations are interpreted to be a result of the structural boundaries impeding the flow of groundwater into the piezometric low from adjacent regions to the north and south. This effect is compounded by the relatively narrow surface topography of the West Ridge at this location, which is expected to limit meteoric recharge to the groundwater low and further reduce piezometric elevations within the low.
7 – REFERENCES


Hydrometrics, Inc. (Hydrometrics, 1994). Results of Permeability Testing at the Yankee Doodle Tailings Dam. Helena, Montana.


8 – CERTIFICATION

This report was prepared and reviewed by the undersigned.

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Project Engineer

Prepared:  
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APPENDICES

Due to file size restrictions, the following appendix reports are not included in the copy of the Site Characterization Report posted to the website. To obtain copies of the following appendix reports, please contact:

DEQ- Hard Rock Mining Bureau
1520 E 6th Avenue,
Helena, MT 59601
(406) 444-4953

Appendix A Drillhole, Test Pit, and CPT Logs
- Appendix A1 Dames and Moore (1962 - 1963)
- Appendix A2 M.K. Botz (1969)
- Appendix A3 Golder Associates (1980)
- Appendix A6 Montana Resources (2005 - 2014)
- Appendix A7 Hydrometrics (2012)
- Appendix A8 Knight Piesold (2012 - 2014)
- Appendix A9 Hydrometrics (2015)
- Appendix A10 Knight Piesold (2015)
- Appendix A11 Hydrometrics (2016)
- Appendix A12 Knight Piesold (2016)

Appendix B Seismic Hazard Assessment (available online as separate document, provided per Draft EIS comments)

Appendix C Historic Geology Maps

Appendix D 2015 Geotechnical Site Investigation Memos
- Appendix D1 VA15-03370 - Phase 1A Memo Test Pit Program Summary
- Appendix D2 VA16-00184 - Phase 1A Addendum - Infiltration Testing
- Appendix D3 VA15-03524 - Phase 1B Memo Trenching Program Summary
- Appendix D4 2016_Additional Trenching_Memo_M.Peet
- Appendix D5 VA15-03317 - Phase 2A Geotechnical Drilling Program Summary
- Appendix D6 VA15-03525 - Phase 2B Geotechnical Drilling Program Summary
- Appendix D7 VA16-00012 - Phase 2c West Embankment Hydrogeological Drilling Program
- Appendix D8 VA16-00013 - Phase 3 Memo Sonic Drilling Program Summary
- Appendix D9 VA16-00014 - Phase 4 Tailings Impoundment SCPT Program Summary
- Appendix D10 VA16-00856 - Phase 5 West Embankment Hydrogeological Drilling Program
- Appendix D11 Additional Memos on Trenching by Michael Peet

Appendix E Water Level Plots
- Appendix E1 Vibrating Wire Piezometer (VWP) Time Series Plots
- Appendix E2 Select Pumping and Recharge Test vwp time series plots
- Appendix E3 Standpipe Piezometer Manual Measurements

Appendix F Structural Geology Modelling Figures

Appendix G YDTI Embankment Construction Chronology
- Appendix G1 Yankee Doodle tailings Impoundment Dam Center Pedestal Construction Detail (Applied Geological Services, LLC. Report)
- Appendix G2 YDTI Embankment Construction Chronology