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# Fairbanks Gold Mining, Inc.
# Barnes Creek Heap Leach Facility
# Report on Final Design
# Reissued in Final
# Revision 1

**March 27, 2017**

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| A | December 14, 2016 | Issued for Client Review | Mr. Tom Kerr | Mr. Jose Bolanos |
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## List of Revisions

This section provides a general overview of the revisions made by Knight Piésold and Co. (Knight Piésold) to the Revision 0 issuance of the Barnes Creek Heap Leach Facility (BCHLF) Final Design Report that resulted in the following issuance:

- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility, Report on Final Design, Reissued in Final, Revision 1, March 20, 2017* (DV101.00089.99)

The Revision 0 report was issued by Knight Piésold under the following title:

- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility, Report on Final Design, Issued in Final, Revision 0, February 7, 2017* (DV101.00089.99)

Minor modifications were made throughout the text to provide better flow, consolidate topics, address comments from the Alaska Department of Natural Resources (ADNR), and update areas of the design that were modified. Notable revisions to the Revision 0 issuance of the BCHLF Final Design Report are listed as follows. Notable revisions to Drawings include only those in going from Revision 0 to Revision 1.

**Executive Summary**

- Consolidated some redundant content (2nd to last paragraph pp ES-2)
- Clarified materials comprising the expected BCHLF foundation surface (sub-points to bullet 1 pp ES-3)
- Developed discussion (with revised sizing) of the Fish Creek storm water detention basin (2nd paragraph pp ES-6)
- Removed south perimeter outlet channel and indicated discharge from south perimeter diversion culverts to be discharged to the Fish Creek storm water detention basin (3rd paragraph pp ES-8 and last associated bullet point)
- Included closure strategy per discussions with ADNR (last paragraph)

**Section 2.0 – Site Conditions**

***2.5 Seismic Hazard Analysis***

***2.5.2 Recommended Seismic Design Parameters***

- Clarified that the BCHLF dam will not be considered a jurisdictional dam in the post-closure period (2nd and last paragraphs)

**Section 3.0 – Climate Study and Hydrologic and Hydraulic Analyses**

***3.2 Hydrologic and Hydraulic Analyses***

- Removed south perimeter outlet channel and indicated discharge from south perimeter diversion culverts to be discharged to the Fish Creek storm water detention basin (4th and 5th bullets pp 3-3)

- Added Fish Creek storm water detention basin (6th bullet pp 3-3)
- Clarified that the September rain-on-snow event typically controls with respect to overall volume; however, due to runoff ground conditions the January rain-on-snow event controls the required capacity of the Fish Creek detention basin (1st paragraph pp 3-4)

#### *3.2.1 Storm Water Incident to the Pad*

- Specified excess flows through the in-heap pond emergency spillway to be conveyed to the Fish Creek storm water detention basin (3rd paragraph)

#### *3.2.2 Hydraulic Analyses*

- Removed south perimeter outlet channel

### Section 4.0 – Geotechnical Investigation

#### *4.10 Geosynthetics Testing*

- Specified standard products used for geomembrane (for both GSE and Solmax products) and specified GSE TenDrain 300 as the geonet product that was tested (2nd paragraph)

#### *4.10.1 Geosynthetic Interface Testing*

- Specified plan and frequencies for on-going sampling and testing of prepared subbase material and indicated updated stability and deformation analyses to be conducted, as deemed necessary, based on the testing results (last paragraph)

#### *4.14 Geotechnical Recommendations and Construction Material Assumptions*

#### *4.14.1 Additional Geotechnical Investigations*

- Clarified that the listed investigations should feed into the planned on-going testing program for prepared subbase (discussed in Section 4.10.1) and recommended additional subbase sampling and interface testing be conducted prior to the BCHLF Stage 1 construction (last two sentences)

### Section 5.0 – Project Design Criteria

#### *Table 5.1 – Dam Hazard Classification and General Criteria*

- Clarified that the BCHLF dam will not be considered a jurisdictional dam in the post-closure period (last sentence Note 1)

#### *Table 5.3 – In-Heap Storage Pond Criteria*

- Clarified that the downstream slopes of the in-heap pond embankment vary and modified the associated Note 1

#### *Table 5.4 – Leach Pad Component Systems Criteria*

- Added drainage wells to be installed within the bottom of the transverse underdrain random fill trench (1st entry, 3rd paragraph)
- Clarified that three underdrain monitoring wells will be installed and extended into the bedrock surface beneath the valley bottom (2nd entry)
- Clarified that the number of PCMS channels will be based on assessment of the heap development and under leach area such that the active leach area will contribute flows to a minimum of two PCMS channels (3rd entry)
- Added new Note 2 to specify geomembrane product and renumber subsequent notes
- Modified Note 4 to specify geonet product

***Table 5.6 – Geotechnical Analyses Criteria***

- Modified Geomembrane Tensile Properties (last entries in table)
- Clarified that the BCHLF dam will not be considered a jurisdictional dam in the post-closure period (last sentence Note 3)

**Section 6.0 – Leach Pad Design**

***6.2 Grading Plan and Foundation Preparation***

- Clarified that questionable zones of material shall be removed and replaced per the BCHLF CQA/QC Manual and Earthwork Specifications (5th bullet point pp 6-2)

***6.4 Groundwater Underdrains***

- Clarified that three underdrain monitoring wells will be installed and extended into the bedrock surface beneath the valley bottom (1st full paragraph pp 6-4)
- Included reference to Drawing presenting FGMI's existing wells in the vicinity of the Fish Creek stockpile area (2nd full paragraph pp 6-4)
- Modified number of drilled-in drainage wells (from 5 to 7) to be installed within the bottom of the transverse underdrain random fill trench (last sentence, 3rd full paragraph, pp 6-4)
- Clarified that piezometers are not required in the underdrains (2nd to last paragraph of section, pp 6-6)

***6.5 In-Heap Storage Pond Embankment***

- Clarified that excavation and backfilling of the Fish Creek stockpile area has already been completed and formed the Fish Creek storm water detention basin (3rd paragraph pp 6-7)

***6.6 Leach Pad Liner System***

***6.6.1 General Liner System***

- Specified GSE or Solmax standard product for all 80-mil DST LLDPE geomembrane (1st paragraph)

***6.6.3 Process Component Monitoring System***

- Reorganized content
- Clarified that monitoring of discharge from the PCMS will be addressed in FGMI's O&M Manual (last sentence, 1st paragraph)
- Included assessment for location of PCMS channels (2nd and 3rd paragraphs)
- Modified PCMS channel widths based on 22-ft roll-width of geomembrane (1st paragraph, pp 6-12)
- Modified estimated leakage based on new PCMS channel widths (2nd paragraph, pp 6-12)
- Modified PCMS channel capacity (4th paragraph, pp 6-12)
- Modified PCMS sumps (last two paragraphs, pp 6-12)

***6.7 Overliner and Solution Collection Pipework***

- Clarified that installation of solution collectors and headers that are intended to overlay PCMS channels must follow the PCMS channel horizontal setting out data (last sentence, 1st paragraph)

***6.9 In-Heap Storage Pond***

***6.9.3 Emergency Spillway***

- Clarified flow through the spillway is directed to the Fish Creek storm water detention basin (1st paragraph)
- Moved discussion of Fish Creek storm water detention basin to Section 6.10 (3rd to last paragraph)

### *6.10 Fish Creek Storm Water Detention Basin*

- Added new section and renumber subsequent sections
- Added Table 6.7 and renumbered subsequent tables in Section 6.0

### *Table 6.7 Fish Creek Storm Water Detention Basin Inflow Design Flood Volumes*

- Replaced previous Table 6.7 with new content
- Moved loading schedule to Table 6.8

### *6.11 Perimeter Roads, Surface Water Diversions, and Culverts*

### *6.11.5 South Perimeter Diversion Culverts*

- Removed south perimeter outlet channel and specified that the culverts will discharge directly to the Fish Creek storm water detention

### *6.11.6 South Perimeter Diversion Culverts Reinforced Slope*

- Removed south perimeter outlet channel and renumbered this section to follow Section 6.11.5

### *6.12 Ore Loading, Heap Access, and Culverts*

### *Table 6.8 Conceptual Lift-by-Lift Heap Loading Schedule*

- Renamed from Table 6.7

### *6.13 Instrumentation*

- Added reference to Drawings 140 and 560 showing FGMI's existing wells in the vicinity of the Fish Creek stockpile area
- Modified overliner piezometers (added piezometers and renumbered)

### *6.14 Drainage at Closure*

- Added discussion of Fish Creek storm water detention basin
- Added that it is understood that FGMI's intended closure strategy will be to create a landform that will be reclaimed under the Reclamation Standard (AS 27.19), that the post-closure waste management will be managed per ADEC requirements, and that the impoundment function of the dam will no longer be necessary and the dam will be abandoned in place after in-heap rinsing is complete.

## Section 7.0 – Slope Stability and Settlement Analyses

### *7.1 Stability Analyses*

### *7.1.2 Material Properties*

- Added reference to Section 4.10.1 for further discussion of the basis for prepared subbase interface shear strengths used (1st sentence, 3rd paragraph)
- Indicated updated stability and deformation analyses to be conducted, as deemed necessary, based on updated testing results (last sentence, 3rd paragraph)

### *7.2 Settlement Analyses*

### *7.2.1 Introduction and Methodology*

- Modified 1st point, last sentence, 1st paragraph

### *7.2.4 Settlement Analysis Results*

- Modified section naming (to Sections A, C, and D) to be consistent with the settlement calculation package and the geotechnical report

- Revised maximum settlement result for Section A to 16.3 ft
- Added statement on sensitivity assessment of lateral boundary conditions

#### *7.2.4.1 Uniaxial Strains*

- Modified last sentence of 1st paragraph

#### *7.2.4.2 Biaxial Strains*

- Renamed from axisymmetric
- Renamed sections (to Sections A, C, and D)
- Replaced comparison of strains with GRI GM17 to comparison with Peggs, et al., 2005

#### *7.2.5 Settlement Conclusions*

- Replaced comparison of strains with GRI GM17 to comparison with Peggs, et al., 2005

### Section 9.0 – Construction Material Take-Off Quantities

- Listed basis for changes in MTO quantities (1st paragraph)
- Revised quantities, added two entries, added notes 5 through 8 to end of table
  - Bolded entries – values were changed
  - Blue-shaded boxes – values were not changed
  - Grey-shaded boxes – description was modified
  - Yellow-shaded boxes – new entry

### Section 10.0 – Considerations for Follow-Up Engineering

- Modified closure design discussion (1st paragraph, 3rd bullet)
- Moved discussions associated with the Fish Creek storm water detention basin to Section 6.10
- Modified discussion associated with excavation of the stockpile areas (2nd paragraph, 1st bullet)
- Clarified that additional prepared subbase testing will be required throughout construction (last paragraph pp 10-2)

### Section 11.0 – Summary

- Modified for changes implemented to the body of the report text
- Included closure strategy per discussions with ADNR (last paragraph)

### Section 12.0 – References

- Updated references to include Peggs, et al. 2005

### Figures

#### *Figure 3.01 Stage 1 Water Management Plan*

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Modified Note 2 to indicate discharge into the Fish Creek storm water detention basin

***Figure 3.02 Stage 2 Water Management Plan***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Modified Note 2 to indicate discharge into the Fish Creek storm water detention basin

***Figure 3.03 Stage 3 Water Management Plan***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Modified Note 2 to indicate discharge into the Fish Creek storm water detention basin

***Figure 3.04 Stage 4 Water Management Plan***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Modified Note 2 to indicate discharge into the Fish Creek storm water detention basin

***Figure 3.05 Stage 5 Water Management Plan***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Modified Note 2 to indicate discharge into the Fish Creek storm water detention basin

***Figure 3.06 Stage 6 Water Management Plan***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Modified Note 2 to indicate discharge into the Fish Creek storm water detention basin

***Figure 3.07 Ultimate Heap Configuration Watershed Plan***

- Modified Fish Creek stockpile area topography to include storm water detention basin

***Figure 6.02 PCMS Channels and Under Leach Area Assessment, Stage 2, Lift 4***

- Replaced Figure 6.02 with new content
- Fish Creek storm water detention basin moved to Drawings 560 and 562

***Figure 6.03 PCMS Channels and Under Leach Area Assessment, Stage 3, Lift 6***

- Replaced Figure 6.03 with new content and renamed previous figure to Figure 6.10

***Figure 6.04 PCMS Channels and Under Leach Area Assessment, Stage 4, Lift 8***

- Replaced Figure 6.04 with new content and renamed previous figure to Figure 6.11

***Figure 6.05 PCMS Channels and Under Leach Area Assessment, Stage 5, Lift 10***

- Replaced Figure 6.05 with new content and renamed previous figure to Figure 6.12

***Figure 6.06 PCMS Channels and Under Leach Area Assessment, Stage 6, Lift 12***

- Replaced Figure 6.06 with new content and renamed previous figure to Figure 6.13

***Figure 6.07 PCMS Channels and Under Leach Area Assessment, Stage 6, Lift 14***

- New figure

***Figure 6.08 PCMS Channels and Under Leach Area Assessment, Stage 6, Lift 16***

- New figure

***Figure 6.09 PCMS Channels and Under Leach Area Assessment, Stage 6, Lift 18***

- New figure

***Figure 6.10 Staged Pad Areas versus In-Heap Storage Pond Elevations***

- Renamed from Figure 6.03

***Figure 6.11 Staged Pad Areas versus In-Heap Storage Pond Volumes***

- Renamed from Figure 6.04

***Figure 6.12 In-Heap Storage Pond Filling Curve and Stage 6 (Final) Storage Components***

- Renamed from Figure 6.05

***Figure 6.13 Ore Heap Filling Curve***

- Renamed from Figure 6.06

***Figure 7.01 Slope Stability Sections Location Plan***

- Removed non-essential information (external channels and culverts)
- Removed south perimeter outlet channel

***Figure 7.02 Settlement Sections Location Plan***

- Renamed previous Sections B and C to Sections C and D, respectively to match settlement calculation package and Geotechnical report
- Added Note 2 discussing the omission of Section B from the settlement analyses because it is cut along the crest of the in-heap storage pond embankment

**Drawings**

***101-89-99-000 Cover Sheet and Index***

- Modified dates and revisions of applicable drawings

***101-89-99-130 Site Geologic Sections, Sheet 1 of 2***

- Corrected location of Section B callout in Section A

***101-89-99-140 General Arrangement Plan (Topographic Map of Dam Site)***

- Modified Fish Creek stockpile area topography to include storm water detention basin
- Added Note 8 (regarding 3rd monitoring well with location to be determined) and Note 9 (regarding drainage wells to be constructed within the base of the transverse underdrain random fill trench)
- Adjusted the end of the longitudinal underdrain random fill trench to match Drawing 200
- Added FGMI's existing monitoring and dewatering wells in the vicinity of the Fish Creek stockpile area

***101-89-99-142 In-heap Storage Pond Embankment Profile and Cross-Sections***

- Modified Fish Creek stockpile area topography to include storm water detention basin
- Modified Note 5 (regarding well depths and to refer to Drawing 200 for more information) and Note 6 (regarding number of drainage wells and completion depths)
- Added Note 7 regarding 3rd monitoring well with location to be determined

- Corrected locations of planned underdrain monitoring wells (MW-01 and MW-02) in Section A and extended the depths of the underdrain monitoring wells to approximate bedrock contact

***101-89-99-144 In-heap Storage Pond Embankment General Details***

- Added Note 4 specifying geomembrane and geonet products

***101-89-99-200 Underdrain System and PCMS Plan***

- Modified Fish Creek stockpile area topography to include storm water detention basin
- Added two additional PCMS channels and renumbered
- Modified Note 11 and setting out data (regarding well depths)
- Added Note 14 (regarding 3rd monitoring well with location to be determined), Note 15 (regarding FGMI's existing monitoring and dewatering wells in the vicinity of the Fish Creek stockpile area), and Note 16 (regarding drainage wells to be constructed within the base of the transverse underdrain random fill trench)

***101-89-99-202 Underdrain Random Fill Profiles and Horizontal Control Data***

- Modified, and increased size of, profile for the transverse underdrain random fill trench (for better clarity)
- Added two additional drainage wells
- Added Notes 4 and 5

***101-89-99-204 PCMS Channel Profiles and Horizontal Control Data, Sheet 1 of 2***

- Added new PCMS channel 1 and renumbered others
- Renamed to Sheet 1 of 2
- Moved previous PCMS channel 3 to new Drawing 205

***101-89-99-205 PCMS Channel Profiles and Horizontal Control Data, Sheet 2 of 2***

- Added new drawing for PCMS channels 4 and 5

***101-89-99-206 Underdrain System and PCMS Sections and Details, Sheet 1 of 8***

- Added Section A (underdrain random fill trench) from Drawing 220 and reduced its scale
- Added Notes 2 through 9 (Notes 2 through 6 from Drawing 220, Note 7 specifying geomembrane)
- Renamed to Sheet 1 of 8 (previously 1 of 7)
- Modified Details 1 and 2 for revised profile of transverse underdrain random fill trench, and added two additional drainage wells
- Added stationing for drainage wells

***101-89-99-210 Underdrain System and PCMS Sections and Details, Sheet 2 of 8***

- Replaced Note 3 to specify geomembrane and geonet
- Modified number of PCMS header pipes in Section A
- Renamed to Sheet 2 of 8

***101-89-99-215 Underdrain System and PCMS Sections and Details, Sheet 3 of 8***

- Added Note 8 specifying geomembrane and geonet
- Added Note 9 for PCMS collector pipe slot arrangement
- Modified number of PCMS header pipes in Details 1 and 2
- Renamed to Sheet 3 of 8

***101-89-99-220 Underdrain System and PCMS Sections and Details, Sheet 4 of 8***

- Moved underdrain random fill trench section to Drawing 206
- Removed previous Notes 6 and 8 (specifically associated with underdrain random fill trench)
- Added Note 9 (specifying geomembrane and geonet) and Notes 10 and 11
- Widened PCMS channel and sump
- Added neoprene sheet on PCMS sump bottom
- Added Detail 2 (PCMS collector pipe slot arrangement)
- Renamed to Sheet 4 of 8

***101-89-99-222 Underdrain System and PCMS Sections and Details, Sheet 5 of 8***

- Added new drawing to clarify PCMS sump pipe boot and header pipe transition

***101-89-99-225 Underdrain System and PCMS Sections and Details, Sheet 6 of 8***

- Added two additional PCMS header pipes and modified spacing and setting out data for discharge points
- Added Note 11 specifying geomembrane and geonet
- Renamed to Sheet 6 of 8

***101-89-99-230 Underdrain System and PCMS Sections and Details, Sheet 7 of 8***

- Added Note 14 specifying geomembrane
- Renamed to Sheet 7 of 8

***101-89-99-235 Underdrain System and PCMS Sections and Details, Sheet 8 of 8***

- Modified PCMS channel (Section A and B) to use full geomembrane roll width
- Added Note 7 specifying geomembrane
- Renamed to Sheet 8 of 8

***101-89-99-300 In-heap Storage Pond LCRS Plan***

- Added Note 12 specifying geomembrane and geonet
- Modified Fish Creek stockpile area topography to include storm water detention basin

***101-89-99-310 In-heap Storage Pond LCRS Sections and Details, Sheet 1 of 2***

- Added Note 15 specifying geomembrane and geonet
- Removed extrusion welds to steel pipe in Detail 2
- Adjusted LCRS sump and setting out data

***101-89-99-320 In-heap Storage Pond LCRS Sections and Details, Sheet 2 of 2***

- Added Note 12 specifying geomembrane and geonet
- Fixed underdrain random fill trench beneath LCRS sump in Section A (to properly reflect the 12”-thick layer of site grading fill overlying the trench and the approximate position of the trench)

***101-89-99-400 Stage 1 Grading Plan***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin

***101-89-99-401 In-Heap Storage Pond Grading Plan***

- Modified Fish Creek stockpile area topography to include storm water detention basin

***101-89-99-410 Staged Grading Plan thru Stage 6***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin

***101-89-99-430 Stage 1 Perimeter Road Profile and Horizontal Control Data***

- Added two additional PCMS sumps in profile and renumbered others

***101-89-99-500 Solution Collection Pipework and Instrumentation Plan***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Added PCMS channel locations
- Modified Note 13 to specify that solution headers overlying PCMS channels shall follow the PCMS channel horizontal control data
- Added overliner layer piezometers and renumbered others
- Modified solution collection pipework around PCMS channel 4

***101-89-99-516 Inclined Solution Extraction Wells Sections and Details, Sheet 1 of 3***

- Added Note 14 specifying geomembrane and geonet
- Modified Fish Creek stockpile area topography to include storm water detention basin

***101-89-99-518 Inclined Solution Extraction Wells Sections and Details, Sheet 3 of 3***

- Added Note 16 specifying geomembrane and geonet

***101-89-99-520 Solution Pipework and Liner System Sections and Details***

- Added Note 8 specifying geomembrane and geonet

***101-89-99-530 Pad Perimeter Sections and Details, Sheet 1 of 3***

- Added Note 14 specifying geomembrane and geonet

***101-89-99-532 Pad Perimeter Sections and Details, Sheet 2 of 3***

- Removed south perimeter outlet channel
- Added Note 13 specifying geomembrane

***101-89-99-534 Pad Perimeter Sections and Details, Sheet 3 of 3***

- Added Note 14 specifying geomembrane and geonet

***101-89-99-540 South Perimeter Diversion Culverts Plan and Profile***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography
- Replaced Note 5 to indicate discharge into the Fish Creek storm water detention basin

***101-89-99-542 South Perimeter Diversion Culverts Sections and Details, Sheet 1 of 2***

- Removed south perimeter outlet channel
- Modified outlet from culverts in Section C and Detail 1
- Replaced Note 3 to indicate discharge into the Fish Creek storm water detention basin

***101-89-99-550 In-heap Storage Pond Spillway and Outlet Channel Plan and Profile, Stage 1***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Added two additional PCMS header pipes

***101-89-99-551 In-heap Storage Pond Spillway and Outlet Channel Plan, Stage 2***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin
- Added two additional PCMS header pipes

***101-89-99-552 In-heap Storage Pond Spillway Sections and Details, Sheet 1 of 3***

- Added two additional PCMS header pipes

***101-89-99-554 In-heap Storage Pond Spillway Sections and Details, Sheet 2 of 3***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography to include storm water detention basin

***101-89-99-556 In-heap Storage Pond Spillway Sections and Details, Sheet 3 of 3***

- Added callouts for Fish Creek storm water detention basin in Section A

***101-89-99-560 Fish Creek Storm Water Detention Basin Plan***

- Added new drawing

***101-89-99-562 Fish Creek Storm Water Detention Basin Typical Sections***

- Added new drawing

***101-89-99-600 Stage 1 Schematic Heap Configuration***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography

***101-89-99-602 Stage 2 (EL 1640) Schematic Heap Configuration***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography

***101-89-99-604 Stage 3 (EL 1730) Schematic Heap Configuration***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography

***101-89-99-606 Stage 4 (EL 1820) Schematic Heap Configuration***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography

***101-89-99-608 Stage 5 (EL 1950) Schematic Heap Configuration***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography

***101-89-99-610 Stage 6 (Final) Schematic Heap Configuration***

- Removed south perimeter outlet channel
- Modified Fish Creek stockpile area topography

***101-89-99-720 Instrumentation Sections and Details***

- Modified locations for piezometers 23 through 48
- Added piezometers 49 through 58
- Added piezometer models and figure references to table

**Appendices**

***Appendix D-1 Biaxial Strain at Liner System Calculation***

- Renamed from "Axisymmetric Strain at Liner System Calculation"

***Appendix E Peggs, et al., 2005, Assessment of Maximum Allowable Strains in Polyethylene and Polypropylene Geomembranes***

- Replaced previous Appendix E "GRI Test Method GM17"

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## Executive Summary

The Fort Knox mine is owned and operated by Fairbanks Gold Mining Inc. (FGMI). The mine is located on the United States Geological Survey Livengood (A-1) SW quadrangle map in Township 2N, Range 2E of the Fairbanks Meridian (USGS, 1992a). The approximate GPS coordinates are latitude 65º00'N, longitude 147º20'W.

In March 2011, FGMI began investigating alternatives for expanding the heap leach ore capacity at Fort Knox and initially contemplated a new heap leach facility in the Barnes Creek valley; the drainage adjacent to and southwest of the existing Walter Creek Heap Leach Facility (WCHLF) and upstream of the Tailing Storage Facility (TSF). Concept plans for the Barnes Creek Heap Leach Facility (BCHLF) were developed but were complicated by the presence, at that time, of the Barnes Creek low-grade ore stockpile that occupied most of the valley. This prompted attention to shift away from development of the BCHLF and instead towards increasing the WCHLF storage capacity. However, since 2011, FGMI has removed a significant amount of the low-grade ore stockpile by relocating it to the WCHLF for processing, which has opened the Barnes Creek valley for the BCHLF. Based on this, the identification of additional ore reserves at the Fort Knox mine, and the shorter haul distance from the open pit to the BCHLF, FGMI decided to reconsider the BCHLF under more attractive development and haulage conditions.

In 2015, at the request of FGMI, Knight Piésold completed the BCHLF Prefeasibility Study (PFS) design to support initial internal planning and permitting efforts. The results were encouraging and this led to a detailed design effort in 2016 to produce final permitting and construction level plans and specifications together with operating requirements. The detailed design is presented in this report to support continued permitting efforts and subsequent construction of the BCHLF.

The BCHLF has been designed with an arrangement similar to the WCHLF. This is in part due to the success of the construction and operation of the WCHLF, and the gained knowledge of the materials and their behavior in it. The BCHLF will be a valley-fill heap leach pad with an in-heap solution storage pond. The BCHLF will extend up the valley from the existing Barnes Creek conveyor causeway, and has been designed with six distinct and progressively developed stages (Stages 1 through 6). Following FGMI's planned removal of most of the remaining Barnes Creek low-grade ore stockpile, the maximum relief across the BCHLF pad footprint is expected to be on the order of 720 feet (ft), from the low-point of the in-heap storage pond (1450 feet above mean sea level, fmsl), to the high-point along the upper perimeter of Stage 6 (2170 fmsl).

Geotechnical site investigations were conducted in 2011 and 2016 to support the initial and final designs of the BCHLF. The investigations primarily focused on subsurface conditions to verify the nature of the Barnes Creek conveyor causeway fill, the causeway foundation, and the foundation beneath the Barnes Creek low-grade ore stockpile. Since the causeway is already in place, a key goal was to assess the ability of its foundation and lower fill to convey groundwater flows under it. These flows will be intercepted and directed to the upstream toe of the causeway in underdrains that will be installed beneath the BCHLF liner system. The groundwater flows will be passed through the lower portion of the causeway fill and its foundation to the Fish Creek stockpile area where they will be intercepted by the pit dewatering wells. To enhance the ability to adequately pass these flows under the causeway, the groundwater underdrains will include a lateral extension along the upstream toe of the causeway for better distribution. Additionally,

drilled-in drainage wells will be completed in the base of the lateral underdrain extension to maximize the opportunity for the groundwater flows to intersect interbedded layers of higher conductivity colluvial materials that were found to exist beneath the conveyor causeway.

A second key goal was to assess the compressibility of the small amount of low-grade ore and underlying site stripping spoils and original native soils, including some organic material, that will be left in place in the valley bottom. The objective was to confirm that the integrity of the liner system, to be constructed above these materials, will be adequately maintained when the BCHLF is developed. Thirdly, the investigations provided information on the strength of the materials to be left in place under the BCHLF and in the causeway fill for analyses of the BCHLF stability. The design has been completed using the findings of the geotechnical investigation and provides for adequate groundwater flow passage, liner integrity preservation and overall stability of the BCHLF. Key take-aways from the geotechnical investigations include:

- The Fairbanks Schist bedrock underlying the valley is fractured, but comprises interlocked blocks of competent bedrock that are expected to form a sound foundation for the BCHLF
- The existing conveyor causeway embankment is expected to form a stable downstream toe for the BCHLF, and a stable containing embankment for the in-heap pond for the BCHLF
- Interbedded layers of higher hydraulic conductivity colluvial material near the base of the conveyor causeway are expected to adequately convey groundwater flows from beneath the BCHLF to the Fish Creek stockpile area, particularly with the flows distributed along the upstream toe of the causeway by the lateral underdrain extension and the drilled-in drainage wells to intersect the interbedded colluvial layers
- Remnants of the low-grade ore stockpile, as well as the excavation spoils and native soils that are planned to be left in place, exhibit (1) adequate stiffness such that settlement and deformation of the liner is expected to be well within tolerable limits, and (2) adequate strength such that the stability of the BCHLF will meet appropriate factors of safety

The geotechnical investigations have quantified the nature and expected performance of the various materials that will comprise the foundation for the BCHLF, and work has been done to identify and confirm the adequacy of construction materials; however, the work completed to date has not yet sufficiently defined the sources of prepared subbase for all stages of the BCHLF. The prepared subbase under the WCHLF has been constructed using select and processed weathered Fairbanks Schist, which is prevalent in the area. A similar approach is planned for the BCHLF. However, the site of the proposed BCHLF is predominantly disturbed and the underlying weathered schist that would be used appears to be contaminated in places with the overlying natural topsoil that was not removed prior to development of the Barnes Creek low-grade ore stockpile. While sufficient prepared subbase material will be available for the BCHLF Stage 1 construction, it is uncertain how much suitable material will be available for the subsequent stages. On-going investigations will be needed to identify those sources or alternate materials. A key priority should be made to effectively using, and stockpiling for future use, all potential prepared subbase material encountered during development of the BCHLF.

At its final configuration, through Stage 6 development, the BCHLF ore heap will reach a maximum height of approximately 800 ft from toe to crest, and will provide a storage capacity of approximately 124.4 M cubic yards (cy), or 210.4 million tons (Mt) based on an average in-place ore dry density of 125.3 pounds per cubic-foot (pcf). This is the same overall density as in the WCHLF since the ore is expected to be very similar. A maximum 500-ft vertical ore thickness over the pad liner system has been adopted also to remain consistent with the WCHLF. Slope stability analyses of the full build out Stage 6 heap were conducted under both static and seismic loading conditions using parameters developed from the BCHLF geotechnical investigation and from previous analyses of similar materials for the WCHLF development, and the results provided Factors of Safety and estimated deformation values that are within appropriate limits for the facility. Settlement analyses, conducted to assess the effect of constructing the BCHLF over the existing materials that will be left in place, indicate that flow reversal and localized low-points are not expected to develop along the pad liner system and that the estimated foundation settlements are well within tolerable limits that could affect the integrity of the BCHLF liner system.

The overall contributing catchment area to the BCHLF is approximately 760 acres; however, a significant portion of this is upstream of a current and future waste dump within the upper reaches of the valley and will not produce surface runoff to the BCHLF. Approximately 516 acres of the overall upstream catchment is expected to contribute surface runoff to the BCHLF, and at its final (Stage 6) configuration the BCHLF will occupy about 290 acres of this contributing area. The approximate pad areas associated with the staged development of the BCHLF are presented in Table ES.1.

**Table ES.1**
**Barnes Creek Heap Leach Facility Staged Pad Areas (2-dimensional)**

| Description | Stage | | | | | | Total |
|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | |
| Elevation (1) | 1545 | 1640 | 1730 | 1820 | 1950 | 2165 | |
| Area (M ft$^2$) | | | | | | | |
| Incremental | 2.08 | 2.10 | 2.02 | 2.00 | 2.47 | 1.91 | 12.58 |
| Cumulative | 2.08 | 4.19 | 6.20 | 8.20 | 10.67 | 12.58 | |
| Area (acres) | | | | | | | |
| Incremental | 47.83 | 48.32 | 46.26 | 45.93 | 56.72 | 43.76 | 288.82 |
| Cumulative | 47.83 | 96.15 | 142.41 | 188.33 | 245.06 | 288.82 | |

Note: 1. The stage elevations represent the approximate low points at the northeastern end of each respective stage perimeter road (i.e. the road along the upper edge of each stage's perimeter).

The Barnes Creek valley is predominantly disturbed and covered with waste dumps, a remnant low-grade ore stockpile, and haul roads. Small pockets of native ground and forested areas remain on the upper northeast side-slope and in the far western reaches of the valley. Following FGMI's planned removal of the Barnes Creek low-grade ore stockpile material, development of the BCHLF will generally include construction procedures and components like those used on the WCHLF. The construction procedures and components will generally include the following (and will be built during the stages identified in parentheses):

1. Removing the Barnes Creek stockpile material to develop a competent foundation (Stages 1 through 6)
   - Approximately 0 to 15 vertical ft of the stockpile material is planned to be left in place in portions of the valley bottom and will form part of the leach pad foundation surface
   - Other portions of the leach pad basin will be over-excavated to remove low-grade ore and the resulting excavation surface will form part of the leach pad foundation surface
   - The waste rock dump at the northwest end of the Barnes Creek valley will remain beneath the BCHLF and is comprised of loose-dumped mine waste rock up to approximately 250-ft thick
2. Clearing and stripping areas of native ground to a competent foundation (Stages 5 and 6)
3. Regrading the leach pad basin slopes to 2.5H:1V (horizontal:vertical) or flatter (Stages 1 through 6)
4. Installing groundwater underdrains at the interface of the pad foundation and overlying site grading fill, or on the regraded basin slopes (Stages 1 through 6)
5. Constructing an in-heap storage pond embankment fill, at the downstream toe of the facility, against the upstream face of the existing conveyor causeway to provide a kind and stable surface for the pad liner system to be installed against (Stage 1)
6. Installing a process component monitoring system (PCMS) immediately below the pad liner system (Stages 2 through 6, PCMS sumps and outlet header pipes to be constructed during Stage 1)
7. Placing a moisture conditioned and compacted low-permeability prepared soil sub-grade (prepared subbase) layer over the regraded basin slopes (Stages 1 through 6)

8. Installing an 80-mil double-side textured LLDPE geomembrane liner directly over, and in intimate contact with, the prepared subbase to form a composite liner (Stages 1 through 6). This liner is considered the secondary liner within the in-heap storage pond (Stage 1) and the primary liner above the in-heap storage pond (Stages 2 through 6)
9. Installing a leachate collection and recovery system (LCRS), comprised of drain aggregate in the valley bottom and a tri-planar geonet on the valley side slopes, over the composite liner within the in-heap storage pond (Stage 1)
10. Placing a non-woven geotextile layer and then a second moisture conditioned and compacted low-permeability prepared subbase layer over the LCRS drain aggregate in the bottom of the in-heap pond. If overliner drain aggregate (minus 2-inch crushed mill reject) is used within the LCRS in lieu of LCRS drain aggregate (minus 1/2-inch crushed mill reject), a layer of tri-planar geonet will also be placed to separate the overliner aggregate from the non-woven geotextile (Stage 1)
11. Installing a second 80-mil double-side textured LLDPE geomembrane liner, considered the primary liner, over the LCRS and low-permeability layer within the in-heap storage pond (Stage 1)
12. Constructing and installing the vertical and inclined solution extraction wells (Stages 1 and 2)
13. Placing a solution collection system comprised of free-draining aggregate (overliner material) with encapsulated perforated pipes over the primary liner (Stages 1 through 6)
14. Constructing an emergency spillway and outlet channel through the crest of the in-heap storage pond embankment to avoid overtopping of the embankment during the 100-yr/24-hr design storm event (Stage 1)
15. Constructing roads, diversion channels, and culvert crossings around the staged perimeters of the BCHLF (Stages 1 through 6)
16. Installing monitoring instrumentation (Stages 1 through 6)

In part, due to the Fort Knox location where cold weather conditions are experienced for significant periods of each year, and to remain consistent with the operational success of the WCHLF, the BCHLF has been designed with an in-heap pond for storage of process solution and storm water. It will be contained behind an approximate 95-ft-high in-heap pond embankment, with a crest elevation of 1545 fmsl, that will be developed on the upstream side of the existing Barnes Creek conveyor causeway.

FGMI currently plans to deliver a total solution flow of 16,000 gallons per minute (gpm) to the heap; however, to allow flexibility for future possible increases the BCHLF solution collection system has been designed to accommodate a 50-percent increase to 24,000 gpm. The maximum unit rate of solution applied to the heap will be 0.005 gpm/ft$^2$. The overliner solution collection system has been designed to collect and convey the process fluid from the base of the heap to the in-heap storage pond while limiting the maximum hydraulic head acting above the leach pad composite liner system to 1 ft (outside the confines of the pond).

The leach solution within the in-heap pond will be removed by a series of extraction wells located near the upstream toe of the in-heap storage pond embankment:

- Three inclined solution extraction wells will be installed on top of the overliner layer and will be extended up the northeast corner of the in-heap pond to the crest of the in-heap pond embankment. They will be used for solution collection during initial operations until the permanent vertical wells can be brought into operation (i.e. until before ore placement is completed to near the collar elevations of the vertical wells). Each of the three inclined wells has been designed to accommodate a 4,000 gpm capacity submersible pump such that a total of up to 12,000 gpm may be extracted. Because the inclined wells will be primarily used for the initial operations, and will be installed on top of the overliner layer, they will not be directly connected to the leach pad solution collection system. However, to improve hydraulic performance, the bottoms of the three wells will be interconnected with a manifold assembly and two perforated solution collectors will be extended from the manifold assembly out into the pond.
- For the long-term operation of the facility, five vertical solution extraction wells will be constructed as large diameter pumping sumps within the in-heap storage pond. Each well will include a platform base installed

at the bottom of the in-heap pond, at approximate elevation 1450 fmsl, just upstream of the impounding embankment toe. The bottom of each well will be connected directly to the leach pad solution collection header pipes to promote a high level of hydraulic efficiency. The vertical wells will be extended from their bases up to elevation 1644 fmsl, which will be 4-ft above the Lift 4 bench on the front face of the heap. Each well has been designed to accommodate a 4,000 gpm pump such that at 16,000 gpm a total of four wells will be in service, with the fifth well available for backup use.

Based on FGMI's experience with the WCHLF to date, any restriction of solution flow through the voids in the ore and into the wells is expected to be negligible, with likely only a small drawdown of the pond solution level at the wells. The direct connectivity of the vertical wells to the solution collection system, the dedicated solution collectors for the inclined wells, and the high hydraulic conductivity of the ore around the wells are expected to promote good hydraulic performance of the extraction system.

Smaller pumping wells will also be installed for the LCRS beneath the in-heap pond. The LCRS pump cans will consist of two inclined pipes running up the upstream face of the in-heap pond embankment. Two 250 gpm pumps will be operated within the LCRS pump cans, as needed, from the start of leaching operations within the in-heap pond.

To support the BCHLF final design, an updated climate study and operational period process water balance were completed. Based on seasonal ground conditions and associated infiltration, January rain-on-snow events were found to produce the largest design peak flows from surface runoff (i.e. for channel and culvert hydraulic design); however, the September rain-on-snow events were typically found to produce the largest overall volumes (i.e. the 100-yr/24-hr design storm storage component within the in-heap pond). The primary reason for the difference is that the runoff from the January event (smaller overall depth) assumes frozen ground conditions (higher runoff coefficient). These data and the staged footprint limits for the BCHLF have been used to size the in-heap pond storage components. Above the maximum normal operating levels, the following three primary components are accounted for within the in-heap pond:

- 100-year/24-hour (yr/hr) design storm event (September rain-on-snow)
  The design storm event storage components associated with Stage 1 to the Stage 6 of the BCHLF were estimated to vary between 5.2 and 31.2 M gallons, respectively.
- 24-hr emergency draindown due to loss of power or pumps
  The emergency draindown storage component was calculated to be approximately 23.0 M gallons (based on the total solution supply rate of 16,000 gpm acting over the 24-hr period).
- Freeboard allowance
  A 5-ft (vertical distance) freeboard allowance was included as a contingency against overtopping of the dam. The volume within the 5-ft freeboard allowance was calculated to be approximately 13.0 M gallons, while the freeboard volume up to the spillway invert (3 ft below the crest of the in-heap pond embankment) was calculated to be 5.1 M gallons.

To maintain sufficient capacity within the in-heap pond for the required storage components and the 5-ft freeboard allowance, the BCHLF operational period process water balance suggests that the following neutralization treatment capacities will be required by the dates indicated:

- 250-gpm capacity starting March 2020
- 2,800-gpm capacity by 2021 with a gradual increase to 3,800-gpm capacity by 2026

The maximum normal operating levels that should be honored within the BCHLF in-heap pond during operations are:

- Stage 1 – 1527.8 fmsl
- Stage 2 – 1525.3 fmsl
- Stage 3 – 1522.7 fmsl

- Stage 4 – 1520.1 fmsl
- Stage 5 – 1516.8 fmsl
- Stage 6 – 1514.0 fmsl

The total estimated capacity of the in-heap storage pond is 122.2 M gallons up to the crest of the impounding embankment. This is the storage capacity within the ore voids based on an average ore porosity of 21.8 percent, which is predicted to apply at the time of loading conditions associated with the ultimate heap arrangement at Stage 6. In comparing the maximum normal operating level for each stage of the BCHLF with the proposed pond minimum normal operating level, 1494 fmsl, a large degree of operational flexibility between the two levels will be provided.

In the very unlikely event that the heap surface becomes frozen and a major thawing and runoff event follows, it is possible that the spillway may come into service. The spillway has been designed to safely pass the flow from a completely frozen and impermeable heap surface, at the Stage 6 full build out arrangement, receiving runoff from the winter 100-yr/24-hr design storm event. The spillway will be located on the northeast abutment of the in-heap pond embankment. Solution exiting through the spillway will be directed safely downstream to the Fish Creek storm water detention basin that has been formed between the excavated downstream slope of the over widened conveyor causeway embankment and FGMI's backfilling of the Fish Creek stockpile area. The storm water detention basin will receive any flows discharged through the BCHLF emergency spillway and from external surface runoff diverted around the facility. The design storage requirement for the basin was calculated to be approximately 4.93 $Mft^3$, based on the 100-yr/24-hr January rain-on-snow design storm event. While waters collected in the detention basin are expected to rapidly infiltrate beneath the Fish Creek stockpile area and be picked up by dewatering wells in the area, FGMI has indicated that the volume of water within the detention basin will be controlled via direct pumping from the basin as needed. Provisions for pumping of waters collected in the Fish Creek storm water detention basin will be included in FGMI's O&M Manual.

During the very unlikely condition that the heap surface is frozen, storm water runoff routed off the heap would be required to pass beneath the haul ramps (along the southern heap perimeter) that are planned to access the heap for loading the facility. Four in-heap culvert crossings are included in the BCHLF design and have been sized to pass the peak flow runoff associated with the 100-yr/24-hr design storm event acting on the fully loaded BCHLF Stage 6 heap configuration.

Initial construction access and ore haulage into the bottom of the in-heap storage pond will be conducted from the upper (west) end of the valley. The Stage 1 haul road has been designed to originate from the existing crusher platform and terminate at the upper end of the Stage 1 pad area. Loading of Stages 1 and 2 are expected to originate from the crusher platform. Loading of Stage 3 and beyond will utilize access points, from the existing waste dump haul road, that will be constructed to every other lift of the ore heap.

Due to the upward-expanding nature of a valley-fill facility, the initial lifts of ore will provide smaller storage capacities than those developed in later stages. The initial heap loading will utilize a reduced rate between 70,000 and 120,000 tons per day (tpd), and is planned to begin in September and end in November of 2019. During years 2 and 3, heap loading will be conducted between July and November but at the full production loading rate of 123,000 tpd. After year 3, when additional pad area is available, loading of the ore heap is expected to occur approximately 8 months, or 244 days, out of each year (April through November) at the same loading rate of approximately 123,000 tpd, or roughly 30 M tons per annum (tpa). It is estimated that loading of the BCHLF will be completed in July 2027, approximately 8 years after the start of loading. The currently planned lift-by-lift loading schedule for the Barnes Creek ore heap is summarized in Table ES.2.

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**Table ES.2**
**Conceptual Lift-by-Lift Heap Loading Schedule**

| Stage | Completion Date | Lift | Lift Top Elev (ft) | Loading Date | | Avg Loading Rate (tpd) | Volume (cy) | | Storage (tons) | | Days to Load |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Start | End | | Incremental | Cumulative | Incremental | Cumulative | |
| 1 | 31-Aug-19 | 0 | 1450 | base of in-heap pond | | | | | 0 | | 0 |
| | | 1 | 1490 | 1-Sep-19 | 13-Sep-19 | 70,000 | 549,439 | 549,439 | 929,404 | 929,404 | 13.3 |
| | | 2 | 1540 | 14-Sep-19 | 30-Sep-19 | 70,000 | 692,026 | 1,241,465 | 1,170,596 | 2,100,000 | 16.7 |
| | | 2 | 1540 | 1-Oct-19 | 18-Oct-19 | 120,000 | 1,282,653 | 2,524,118 | 2,169,671 | 4,269,671 | 18.1 |
| | | 3 | 1590 | 19-Oct-19 | 30-Nov-19 | 120,000 | 3,044,739 | 5,568,857 | 5,150,329 | 9,420,000 | 42.9 |
| | | 3 | 1590 | 1-Jul-20 | 1-Jul-20 | 120,000 | 71,800 | 5,640,657 | 121,453 | 9,541,453 | 1.0 |
| 2 | 1-Oct-20 | 3 | 1590 | 2-Jul-20 | 7-Jul-20 | 123,000 | 471,499 | 6,112,155 | 797,563 | 10,339,016 | 6.5 |
| | | 4 | 1640 | 8-Jul-20 | 16-Sep-20 | 123,000 | 5,163,652 | 11,275,807 | 8,734,575 | 19,073,591 | 71.0 |
| | | 5 | 1690 | 17-Sep-20 | 30-Nov-20 | 123,000 | 5,416,554 | 16,692,361 | 9,162,373 | 28,235,964 | 74.5 |
| | | 5 | 1690 | 1-Jul-21 | 7-Jul-21 | 123,000 | 581,087 | 17,273,448 | 982,937 | 29,218,901 | 8.0 |
| 3 | 1-Oct-21 | 5 | 1690 | 8-Jul-21 | 17-Jul-21 | 123,000 | 711,773 | 17,985,221 | 1,203,999 | 30,422,900 | 9.8 |
| | | 6 | 1740 | 18-Jul-21 | 7-Nov-21 | 123,000 | 8,184,116 | 26,169,336 | 13,843,841 | 44,266,741 | 112.6 |
| | | 7 | 1790 | 8-Nov-21 | 30-Nov-21 | 123,000 | 1,648,324 | 27,817,661 | 2,788,223 | 47,054,964 | 22.7 |
| | | 7 | 1790 | 1-Apr-22 | 30-Jun-22 | 123,000 | 6,687,333 | 34,504,994 | 11,311,958 | 58,366,922 | 92.0 |
| 4 | 1-Oct-22 | 7 | 1790 | 1-Jul-22 | 17-Jul-22 | 123,000 | 1,203,575 | 35,708,568 | 2,035,907 | 60,402,829 | 16.6 |
| | | 8 | 1840 | 18-Jul-22 | 30-Nov-22 | 123,000 | 9,851,400 | 45,559,968 | 16,664,135 | 77,066,964 | 135.5 |
| | | 8 | 1840 | 1-Apr-23 | 9-Apr-23 | 123,000 | 718,557 | 46,278,525 | 1,215,475 | 78,282,438 | 9.9 |
| | | 9 | 1890 | 10-Apr-23 | 26-Aug-23 | 123,000 | 10,061,970 | 56,340,495 | 17,020,326 | 95,302,764 | 138.4 |
| 5 | 1-Oct-23 | 9 | 1890 | 27-Aug-23 | 18-Sep-23 | 123,000 | 1,706,229 | 58,046,724 | 2,886,171 | 98,188,936 | 23.5 |
| | | 10 | 1940 | 19-Sep-23 | 30-Nov-23 | 123,000 | 5,255,551 | 63,302,275 | 8,890,028 | 107,078,964 | 72.3 |
| | | 10 | 1940 | 1-Apr-24 | 10-Jul-24 | 123,000 | 7,364,718 | 70,666,993 | 12,457,789 | 119,536,752 | 101.3 |
| | | 11 | 1990 | 11-Jul-24 | 30-Nov-24 | 123,000 | 10,377,589 | 81,044,583 | 17,554,211 | 137,090,964 | 142.7 |
| | | 11 | 1990 | 1-Apr-25 | 24-Apr-25 | 123,000 | 1,806,282 | 82,850,865 | 3,055,416 | 140,146,380 | 24.8 |
| 6 | 1-Oct-24 | 11 | 1990 | 25-Apr-25 | 28-Apr-25 | 123,000 | 295,500 | 83,146,364 | 499,852 | 140,646,232 | 4.1 |
| | | 12 | 2040 | 29-Apr-25 | 30-Sep-25 | 123,000 | 11,273,543 | 94,419,907 | 19,069,762 | 159,715,994 | 155.0 |
| | | 13 | 2090 | 1-Oct-25 | 30-Nov-25 | 123,000 | 4,366,983 | 98,786,890 | 7,386,970 | 167,102,964 | 60.1 |
| | | 13 | 2090 | 1-Apr-26 | 9-Jun-26 | 123,000 | 5,124,697 | 103,911,587 | 8,668,681 | 175,771,645 | 70.5 |
| | | 14 | 2140 | 10-Jun-26 | 25-Sep-26 | 123,000 | 7,865,431 | 111,777,018 | 13,304,770 | 189,076,415 | 108.2 |
| | | 15 | 2190 | 26-Sep-26 | 30-Nov-26 | 123,000 | 4,752,179 | 116,529,197 | 8,038,549 | 197,114,964 | 65.4 |
| | | 15 | 2190 | 1-Apr-27 | 18-Apr-27 | 123,000 | 1,367,618 | 117,896,816 | 2,313,395 | 199,428,358 | 18.8 |
| | | 16 | 2240 | 19-Apr-27 | 7-Jun-27 | 123,000 | 3,616,301 | 121,513,117 | 6,117,154 | 205,545,512 | 49.7 |
| | | 17 | 2290 | 8-Jun-27 | 5-Jul-27 | 123,000 | 2,037,337 | 123,550,454 | 3,446,258 | 208,991,770 | 28.0 |
| | | 18 | 2340 | 6-Jul-27 | 17-Jul-27 | 123,000 | 852,921 | 124,403,375 | 1,442,758 | 210,434,528 | 11.7 |

Perimeter roads and surface water diversion channels have been included in the design for each stage of the BCHLF. They have been divided into two categories; inter-stage and ultimate. Ultimate perimeter roads and diversion channels will wrap around the final pad footprint, while the inter-stage perimeter roads/diversions will be located just above each stage of the pad development through Stage 5.

Each inter-stage perimeter road has been sized and graded to convey the January 100-yr/24-hr design storm runoff, from the upstream catchment areas, along the road corridor. The surface runoff diverted by the inter-stage roads will preferentially be conveyed along the north side of the facility to avoid routing additional flows toward the open pit and the crusher building southwest of the facility. The ultimate perimeter channel, located predominantly around the northeast and east sides of the BCHLF, will collect and convey the runoff from the inter-stage perimeter roads (and any runoff from smaller catchment areas above the final pad footprint) and discharge the flows to the Fish Creek stockpile area. The ultimate perimeter diversion channel will be riprap-lined and has been sized to pass the peak flow associated with the January 100-yr/24-hr design storm event.

Perimeter diversion culverts have been included along the south side of the facility to pass flows collected by the existing waste dump haul road (located on the southwest side of the BCHLF) and discharge them to the Fish Creek storm water detention basin. These culverts are intended to reduce the current flows diverted by the existing waste dump haul road to the open pit. The south perimeter diversion culverts will be constructed during the initial BCHLF development, and will be constructed along the following alignment:

- beneath the existing waste dump haul road near the southwest corner of the BCHLF,
- down the south abutment of the BCHLF (partially below the ultimate perimeter road and partially on native ground outside the ultimate facility footprint),
- above the crusher conveyor outlet tunnel (supported by a gabion-reinforced fill slope),
- beneath the existing conveyor causeway access road adjacent to the existing crusher building, and
- into the Fish Creek storm water detention basin.

The south perimeter diversion culverts have been sized to pass the peak flows associated with the January 100-yr/24-hr design storm event. Gabion reinforced slopes will be required to support the culverts crossing above the conveyor outlet tunnel.

Incrementally staged construction material take-off (MTO) quantity estimates were completed for all identified components of the BCHLF specifically within Knight Piésold's work scope and are included in this report. Unit rates and cost estimates associated with the MTOs will be completed by FGMI and are not included in this report.

Although closure design was not included in Knight Piésold's work scope and is not provided, it is understood that FGMI's intended closure strategy will be to create a landform that will be reclaimed under the Reclamation Standard (AS 27.19), that the post-closure waste management will be managed per ADEC requirements, and that the impoundment function of the dam will no longer be necessary and the dam will be abandoned in place after in-heap rinsing is complete.

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# Fairbanks Gold Mining, Inc.
# Barnes Creek Heap Leach Facility
# Report on Final Design
# Reissued in Final
# Revision 1

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CONSULTING

# Fairbanks Gold Mining, Inc.
# Barnes Creek Heap Leach Facility
# Report on Final Design
# Reissued in Final
# Revision 1

## Section 1.0 - Introduction

### 1.1 Project Background

Fairbanks Gold Mining, Inc. (FGMI) owns and operates the Fort Knox Mine (Fort Knox) located approximately 26 miles northeast of Fairbanks, Alaska. The mine is located on the United States Geological Survey Livengood (A-1) SW quadrangle map in Township 2N, Range 2E of the Fairbanks Meridian (USGS, 1992a). The approximate GPS coordinates are latitude 65º00'N, longitude 147º20'W. FGMI is contemplating the development of a second heap leach pad at Fort Knox that will be termed the Barnes Creek Heap Leach Facility (BCHLF).

In March 2011, FGMI began investigating alternatives for expanding the leach ore capacity at Fort Knox, and initially contemplated a new heap leach facility in the Barnes Creek valley, which is in an adjacent drainage to the southwest of the existing Walter Creek Heap Leach Facility (WCHLF) and upstream of the Tailing Storage Facility (TSF). Concept plans for the BCHLF were developed at that time, but were complicated by the presence of low-grade ore stockpiles that would need to be removed from the Barnes Creek valley. This prompted a shift in attention to increasing the WCHLF storage capacity and the continued development of that facility.

Since 2011, FGMI has relocated a significant amount of the low-grade ore stockpiles to the WCHLF and thereby opened the Barnes Creek valley to facilitate construction. Based on this, the identification of additional ore reserves at the Fort Knox mine, and the shorter haul distance to Barnes Creek, FGMI decided to revisit development of the BCHLF under more attractive and economic conditions.

In 2015, at the request of FGMI, Knight Piésold completed the BCHLF Prefeasibility Study (PFS) design to support internal planning and initial permitting efforts.

The Final design components presented herein are provided to support continued permitting efforts and development of the BCHLF.

### 1.2 Previous Work

Previous work conducted by Knight Piésold for Fort Knox was used to support and develop the designs presented herein where applicable. This work includes, but may not be limited to, the following:

- *Fort Knox Project, Design of Tailing Storage Facility and Water Reservoir (Design Submittal No. 3)*, by Knight Piésold and Co., February 7, 1994 (1169E)
- *Fairbanks Gold Mining, Inc. Fort Knox Project, Conveyor Crossing Foundation Report*, by Knight Piésold and Co., February 7, 1997 (1169G)
- *Fairbanks Gold Mining Inc., Walter Creek Valley Fill Heap Leach Facility, Design Report, Revision 2, Issued for Construction*, by Knight Piésold and Co., March 27, 2007 (DV101.00089.13)
- *Fairbanks Gold Mining Inc., Fort Knox Mine, Walter Creek Valley Fill Heap Leach Facility, Stage 1 Construction Completion Report*, Rev 0, by Knight Piésold and Co., July 2010 (DV101.00089.29, DV-10-0316)

- *Fairbanks Gold Mining, Inc., Walter Creek Valley Fill Heap Leach Facility Expansion, Report on WCHLF Expansion Final Design, Issued for Client Review, Revision A*, by Knight Piésold and Co., April 9, 2015 (DV101.00089/71, DV-15-0287)
- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility, Report on Prefeasibility Study Design, Issued in Final, Revision 0*, by Knight Piésold and Co., September 8, 2015 (DV101.00089/79, DV15-0839)
- *Fairbanks Gold Mining, Inc., Fort Knox Project, Tailing Storage Facility Expansion, 2016 Deposition Plan*, by Knight Piésold and Co., April 22, 2016 (DV101.00089/93, DV-16-0315)
- Fairbanks Gold Mining, Inc., Fort Knox Project, Barnes Creek Heap Leach Facility Final Design, Seismic Hazard Review, by Knight Piésold and Co., July 11, 2016 (DV101.00089/99, DV16-0553)
- Alaska Dam Safety Program, Hazard Potential Classification and Jurisdictional Review for Fort Knox Barnes Creek Heap Leach Facility Dam, submitted by Knight Piésold and Co. June 14, 2016, approved by ADNR October 17, 2016 (DV101.00089/99) (see Appendix A)
- Fairbanks Gold Mining, Inc., Fort Knox Project - BCHLF Final Design, Operational *Period Process Water Balance, Rev A*, by Knight Piésold and Co., October 28, 2016 (DV101.00089/99, DV16-0842)
- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility Final Design, Climatic/Hydrologic Characterization Report, Issued in Final, Revision 0*, by Knight Piésold and Co., December 14, 2016 (DV101.00089/99, DV16-1033)
- *Fairbanks Gold Mining, Inc., Fort Knox Project, Barnes Creek Heap Leach Facility, Report on Final Design Level Geotechnical Investigations, Issued in Final, Revision 0*, by Knight Piésold and Co., January 17, 2017 (DV101.00089/99, DV17-0056)
- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility, CQA/QC Manual, Reissued in Final, Revision 1*, by Knight Piésold and Co., March 27, 2017 (DV101.00089/99, DV17-0338) (CQA/QC Manual)
- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility, Technical Specifications for Earthwork, Pipework, and Concrete Construction, Reissued in Final, Revision 1*, by Knight Piésold and Co., March 27, 2017 (DV101.00089/99, DV17-0338) (Earthwork Specifications)
- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility, Technical Specifications for Geosynthetics, Reissued in Final, Revision 1*, by Knight Piésold and Co., March 27, 2016 (DV101.00089/99, DV17-0338) (Geosynthetics Specifications)

## 1.3 Scope of Report

This report presents the BCHLF final design and specifically accounts for the following:

- Updated seismic hazard assessment
- Updated climate study and hydrologic analyses
- Geotechnical investigations and laboratory testing
- In-heap storage pond embankment
- Leach pad staging
- Stage 1 haul road (for initial construction and loading of the facility)
- Underdrain system and process component monitoring system (PCMS)
- Underdrain monitoring/pumping wells
- Liner and solution collection systems including leachate collection and recovery system (LCRS)
- Staged in-heap storage pond volume components
- In-heap storage pond emergency spillway and outlet channel
- Pregnant solution extraction wells (inclined and vertical)
- Surface water management plan
- Schematic staged heap configurations (with on-heap haul ramp) and loading schedule

- Instrumentation and monitoring plan
- Slope stability and seismic induced deformation analyses
- Settlement analyses
- Operational period process water balance model (stand-alone)
- Technical Specifications for earthworks and geosynthetics
- Staged construction material take-off (MTO) quantities
- Final design drawings

The following items were excluded from the BCHLF final design presented herein:

- In-Heap Storage Pond Dam Break Analyses

  Based on the configuration of the BCHLF in-heap storage pond embankment (combined with the existing conveyor causeway embankment), a dam break that would result in the rapid, uncontrolled release of solution from the facility is considered highly unlikely.

  The possibility of a traditional dam break is further reduced because the in-heap storage pond embankment and existing conveyor causeway will be buttressed by the Fish Creek stockpile area (planned to be excavated and backfilled higher than the causeway)
- Haul Road and Solution Pipe Corridor Design

  The existing waste dump haul road along the southwest side of the planned BCHLF footprint will be utilized for loading the ore heap. Other than the initial Stage 1 haul road presented herein, no additional haul roads are expected, nor are they accounted for in Knight Piésold's BCHLF design.

  A solution pipe corridor will be required for the pregnant and barren pipelines from the BCHLF to the mill site. The design of this structure was not included in Knight Piésold's work scope.
- Solution Application System Design
- Unit Rates and Cost Estimates
- Closure Design

  While a potential, preliminary closure concept is discussed herein, closure design was not included in Knight Piésold's work scope and is not provided.

## 1.4 Limitations and Disclaimer

This report titled *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility, Report on Final Design, Reissued in Final, Revision 1* has been prepared by Knight Piésold for the exclusive use of FGMI. No other party is an intended beneficiary of this report or the information, opinions, and conclusions contained herein. Any use by any party other than FGMI, except for the sole purpose of regulatory review and approval, of any of the information, opinions, or conclusions is the sole responsibility of said party. The use of this report shall be at the sole risk of the user regardless of any fault or negligence of FGMI or Knight Piésold.

The information and analyses contained herein have been completed to a level of detail commensurate with the objectives of the assignment and in light of the information made available to Knight Piésold at the time of preparation. This report and its supporting documentation have been reviewed and/or checked for conformance with industry-accepted norms and applicable government regulations. Calculations and computer simulations have been checked and verified for reasonableness, and the content of the report has been reviewed for completeness, accuracy, and appropriateness of conclusions. To the best of the information and belief of Knight Piésold, the information presented in this report is accurate to within the limitations specified herein.

This report is Knight Piésold pdf file: *DV17-0313-BCHLF-Final_Design-Rev1.pdf*. Any reproductions or modifications of this report are uncontrolled and may not be the most recent revision.

# Section 2.0 - Site Conditions

## 2.1 Climate

The climate near the Fort Knox site is sub-arctic and typically cold and dry, although summers can be wet. The mean annual precipitation (i.e. rainfall and snowfall, in water-equivalent) is approximately 17.3 inches and the mean annual potential evaporation is approximately 20.5 inches. The coldest month is January with an average minimum temperature of approximately minus (-) 5 degrees Fahrenheit (°F) and the warmest month is July with an average maximum temperature of 69°F.

## 2.2 Geomorphology

The BCHLF is located within the western valley of the Walter Creek/Barnes Creek/Fish Creek drainage basin. The topography in the area consists of rounded, moderately sloping hills and ridges, and broad, flat-bottomed valleys. The topography is approaching maturity, apparently never having been glaciated. Weathering is deep and intense in certain areas. Discontinuous permafrost has generally been found in isolated locations along the south sides of the valley bottoms but appears to be less prevalent in the upper reaches of the valleys.

The BCHLF is planned for development in the Barnes Creek valley located immediately southwest of the WCHLF. The valley is predominantly disturbed and covered with mine waste rock dumps, a low-grade ore stockpile and haul roads. Based on preloading of the native foundation materials with placement of the low-grade ore stockpile and waste rock dumps within the planned BCHLF footprint, any localized permafrost that might have existed near the planned heap foundation level will likely have melted and consolidated. Small pockets of forested areas, with a variety of ground covers, remain on the upper northeast side-slope and in the far western reaches of the valley. The undisturbed ground surfaces within the Barnes Creek valley are typically overlain by a 1-ft average thickness of organic topsoil.

The exposed valley slopes on the northeast side of the Barnes Creek valley are moderate, ranging from 4H:1V to 5H:1V on average.

## 2.3 Vegetation

The upper limits of the Barnes Creek valley, above the areas disturbed by the low-grade ore stockpile and waste rock dump development, are forested in small pockets of undisturbed native ground. The vegetation is similar to that found within the Walter Creek valley. Native ground surfaces that are comprised of well-drained soils are vegetated mainly with white spruce, paper birch, and quaking aspen. Moderately well-drained soils support vegetation similar to that found on well-drained soils; however, black spruce and willows are also found. Horsetail and grasses typically cover the existing ground surfaces in these areas. Additional ground cover consists of mosses, lichens, Labrador tea, cranberry, blueberry, shrubs of bog birch and tussocks of cotton grass.

## 2.4 Geology

### 2.4.1 Regional Geology

#### 2.4.1.1 Tectonic Framework

The project site is located within the Yukon-Tanana Upland, a physiographic province bordered on the north by the Kaltag-Tintina fault and on the south by the Denali fault. The area is underlain by crystalline rocks of the Yukon-Tanana metamorphic complex.

The Kaltag-Tintina and Denali faults are major strike-slip faults that have experienced large-scale right-lateral displacement. The Kaltag-Tintina fault, which shows as much as 260 miles of right-lateral

movement, appears to be inactive. In contrast, the Denali fault appears to be currently active, although there is evidence that the movement may be shifting to the Totschunda and Fairweather faults (Plafker, Hudson, Burns, & Rubin, 1978).

Within the Yukon-Tanana Upland, the oldest structural grain trends to the northwest, but a superimposed northeast trend becomes dominant at the Fort Knox site. The northeast trend apparently is associated with the emplacement of granitic plutons and the development of associated ore deposits. Several northeast-trending fault systems transect the Yukon-Tanana block and exhibit left-lateral offsets.

The Fort Knox deposit is located within the Fairbanks Mining District in the northeastern part of the Yukon-Tanana Upland. The district is bounded on the northwest by the valley of the Chatanika River. The southeast boundary is approximately parallel to the Chena River valley and its northern tributary.

At least two episodes of folding have been recognized in the district. The first episode is synmetamorphic and consists of overturned to recumbent folds with northwest-trending axes. The second episode folded the metamorphic units into a series of broad, northeast-trending open folds. Local small-scale folds, faults, and shears are also present.

#### 2.4.1.2 Yukon-Tanana Metamorphic Complex

The Yukon-Tanana Upland consists largely of metamorphic rocks formerly mapped as the Birch Creek Schist Formation but which are now generally referred to as the Yukon-Tanana metamorphic complex.

In the Fairbanks Mining District, the complex can be subdivided into four metamorphosed stratigraphic groups: the Chatanika terrain, the Fairbanks Schist unit, the Chena River sequence and the Birch Hill sequence. The Fairbanks Schist unit, of greenschist facies metamorphic grade, is the predominant rock assemblage in the district and at the Fort Knox site. Lithologies include muscovite-quartz schist, micaceous quartzite, biotite-muscovite-quartz schist, and massive brown and gray quartzite. These are interpreted to be meta-sedimentary with minor meta-volcanic rocks. The Cleary sequence is an assemblage of meta-sedimentary and meta-volcanic rocks located near the center of the Fairbanks Schist unit. The sequence includes calcareous greenschist, chlorite schist, and interlensed potassium feldspar-quartz schist and muscovite quartz schist. The latter two rock-units host most of the known base and precious metal-sulfide deposits in the district. The Cleary sequence also contains areas of chlorite schist and impure marble. The Birch Hill sequence occurs in a narrow band in the southern part of the district and consists of phyllite, micaceous calc-schist, calc-amphibolite and quartzite.

The Chatanika terrane is a unit exposed in the northern part of the district and includes high-grade metamorphic rocks. Structurally above the Fairbanks Schist unit is the Chena River sequence, consisting of amphibolite, tremolite marble and coarse-grained garnet muscovite schist. The Chatanika terrane, the Fairbanks Schist unit, the Chena River sequence and the Birch Hill sequence are all believed to be in thrust-fault contact.

#### 2.4.1.3 Intrusive Rocks

Two dominant intrusive rock types are found within the district. The first consists of northeasterly-trending masses of hornblende-bearing granodiorite found in the northern and southwestern parts of the district. The second consists of porphyritic quartz monzonite and granite at Gilmore Dome and near Twin Creek. Ages of the plutons range from 118 to 60 M years and appear to cluster around two ages, approximately 90 and 60 M years.

Larger plutons such as the Pedro Dome and Gilmore Dome plutons have associated contact metamorphic zones in the adjoining metamorphic rocks. These zones can be up to 1,600 ft in width.

### 2.4.2 Local Geology

Much of the local geology within the BCHLF footprint is obstructed from view due to the presence of waste rock dumps and the low-grade ore stockpile within the valley, sparse bedrock outcrops, and vegetative cover. To evaluate the local geology present at this site, both regional and local geologic maps and past sub-surface study reports were reviewed. The following is a list of these documents:

- Summary Report on 1991 Condemnation Drilling Program, *Memorandum to Arne Bakke*, dated November, 1 1991 (Adams, D, Kulash, W, Deal, F, & Lear, K)
- *Preliminary Geologic Map of the Fairbanks Mining District*, from the Alaska Division of Geological and Geophysical Surveys (ADGGS, 1996), at a 1:63,360 scale
- *Fairbanks Gold Mining, Inc. Fort Knox Project, Conveyor Crossing Foundation Report*, dated February 7, 1997 (Knight Piésold, 1997)
- *Barnes Creek Geology* mapping performed by Alaska Exploration Group (AEG, 2011) at a 1:500 scale

A local geologic map based on interpretation of the above information is shown on Drawing 120.

#### 2.4.2.1 Man-Made Fills

Placer–mine dredge tailings are mapped within the valley bottom just downstream of the BCHLF footprint. It is anticipated that this material will primarily be comprised of reworked creek deposits consisting of gravel to cobble sized material. The extent of this material is interpreted from the regional 1996 ADGGS map, and as such is anticipated to vary and could potentially extend farther up the valley within the facility footprint.

During 1995 and 1996, a conveyor causeway was constructed crossing the mouth of the Barnes Creek valley, just west of the confluence with Fish Creek. Excavations for the construction of this structure ranged from several feet to 15 ft in depth. The excavated material primarily consisted of a 2-ft-thick layer of frozen organic silt overlain by topsoil and underlain by alluvial sands and gravels. The high-water-content topsoil and organic silts were not suitable for use as fill and were placed in a spoil stockpile upstream of the causeway within the Barnes Creek valley. The location and extent of this stockpile is not known precisely, but it is anticipated to be present upstream of the planned toe of the BCHLF in-heap pond embankment.

Since the start of mining operations at the Fort Knox Mine, mine waste rock and low-grade ore have been dumped within the Barnes Creek valley and the confluence with the Fish Creek drainage. Subgrade preparation was generally not performed in these areas, and the dumped material was typically placed directly on the native ground surface. It is estimated that the thickness of the dumped material within the BCHLF footprint varied from zero to a couple of hundred feet; however, FGMI has been actively removing the low-grade ore for processing on the WCHLF in anticipation of the planned BCHLF development.

#### 2.4.2.2 Quaternary Deposits

Geologic units from the Quaternary Period include organic silt, Fairbanks loess and retransported silt, described as follows:

- Organic silts on the order of 2-ft thick were encountered during construction of the conveyor causeway. While this deposit is not shown on the reviewed maps, it is anticipated to be present in other low-lying bog/wetland areas that are now likely covered by the man-made fill deposits discussed in Section 2.4.2.1. The silt deposits are typically organic rich with ground ice present in some north facing areas.
- The Fairbanks Loess (Qef) is found within the valley bottom and along the southwest facing slopes of the Barnes Creek valley. This deposit typically consists of unconsolidated, well-sorted silt.
- The Retransported Silt (Qer) is present within the valley bottom and originates from the deposits higher up on the valley slopes. These deposits consist predominately of silty sands and gravels with the potential presence of ground ice.

Bedrock on the valley side slopes is covered with generally 0 to 15 ft of overburden soils that may include the Fairbanks Loess, Retransported Silt, residual soil weathered from the Fairbanks Schist and/or colluvial mixtures; however, locally these may be up to 30-ft thick. While not shown on the geology maps, subsurface studies indicate that the Retransported Silt overlies dense, silty sands and gravels within the valley bottom. Theses underlying materials are likely from alluvial deposition within Barnes Creek.

#### 2.4.2.3 Bedrock

The mapping indicates the BCHLF area is underlain by the late Precambrian to early Paleozoic, polymetamorphic Fairbanks Schist unit. Condemnation drilling and the 2011 Alaska Exploration Group map indicate that relative percentages of mineralogical constituents of the bedrock vary on a local scale, but, within the planned BCHLF footprint, primarily consist of either quartz muscovite schist or quartzite. Subsurface conditions encountered during previously performed drilling programs indicate that the bedrock varies locally from unaltered to slightly altered, improving with depth. The degree of fracturing across the site is variable; however, it appears to correlate to a higher degree of fracturing at closer proximities to the inferred faults. The Fairbanks Schist has more than sufficient strength to support the proposed facility.

#### 2.4.2.4 Structural Geology

The most prominent folding feature in this area is associated with an unnamed syncline, the axis of which cross-cuts the northern portion of the planned facility footprint. The syncline is generally orientated in an east-west direction with the flanks dipping down to the north and south. The angle of dip is unclear based on the information reviewed; however, Knight Piésold does not anticipate that this geologic structure will be detrimental to the planned BCHLF. The 2011 Alaska Exploration Group map includes local foliation measurements. On average, foliation within the schist has a northeast-to-southwest strike and dips in a southerly direction at angles varying from 20 to 70 degrees but in general at an angle of 50 degrees.

A series of northeast-southwest-trending faults extend across the Fort Knox site. These faults are identified as high-angle faults, which by definition is a fault with a dip greater than 45 degrees. As shown on Drawing 120, the syncline identified within the planned facility footprint has been offset by these faults by as much as 3,500 ft, which is indicative of strike-slip movement along these structures. On the 1996 ADGGS map, a fault has been inferred at the downstream toe of the planned BCHLF. This fault appears to be associated with the regional northeast-to-southwest-trending fault system. Additionally, the 2011 Alaska Exploration Group map indicates an inferred northwest-southeast trending fault that extends through the center of the Barnes Creek Valley and dips to the northeast at an angle of 55 degrees. Since these structures are dipping at relatively high angles, it is not anticipated that they will daylight to provide a zone of weakness along which failure of the planned BCHLF could occur.

Numerous, relatively thin shear zones have been mapped and appear to follow foliation and/or joint orientation. The 1991 condemnation drilling program has identified south-dipping shear zones in-filled with quartz, while the 1995/1996 construction of the conveyor crossing exposed shear zones in which intense weathering had resulted in the alteration of the bedrock into highly plastic argillic clay. It is anticipated that, while additional unmapped shear zones are likely present within the footprint of the proposed facility, they will not have a detrimental effect on the slope stability of the BCHLF.

## 2.5 Seismic Hazard Analysis

### 2.5.1 Background of Project Seismic Hazard Evaluation

The initial seismic hazard evaluation for the Fort Knox site was completed as part of the original design for the TSF and water supply reservoir (Knight Piésold, 1994). Based on work by Thenhaus (1985), Pulpan (1988), and others, the recommended design earthquake at the time was a magnitude M7.5 event, which was estimated to produce a peak horizontal ground acceleration (PHGA) of 0.27g at the Fort Knox site. This corresponded to an event with a 10-percent probability of exceedance in a 50-year (yr) design life (i.e. a 475-yr recurrence interval). These data were used in the design and analysis of several subsequent raises to the TSF, including those constructed through 2006.

More recently, the seismic design parameters for analyzing the facilities at the Fort Knox site were reassessed based on:

- Expansion of the national seismic hazard maps that was completed (USGS, 1999) and updated (USGS, 2007) to cover Alaska in addition to the contiguous lower 48 states and provided additional data relative to the site
- Changes to common practice to address two different design earthquake events for impounding facilities, including:
  - Operating basis earthquake (OBE) where the facility is expected to remain fully operational under the influence of the OBE with little or no damage
  - Maximum design earthquake (MDE) where the facility may experience damage at a level that may inhibit routine operation of the facility, but no catastrophic failure (i.e. uncontrolled release of reservoir contents) occurs

Depending on the type and size of structure, and the potential consequences of a failure, the MDE will be a significant event for the site, up to consideration of the maximum credible earthquake (MCE), while a lesser design earthquake event is typically considered for the OBE. As summarized in Table 2.1, the Alaska Department of Natural Resources guidelines (ADNR, 2005) specify a range of recurrence interval events for both the OBE and MDE depending on the hazard classification of a facility.

**Table 2.1**
**State of Alaska Guidelines**
**Hazard Classification versus Recurrence Interval**

| Hazard Classification | Recurrence Interval | |
|---|---|---|
| | OBE | MDE |
| I (High) | 150 to >250 years | 2,500 years to MCE |
| II (Significant) | 70 to 200 years | 1,000 to 2,500 years |
| III (Low) | 50 to 150 years | 500 to 1,000 years |

The location of the site (latitude 65º00'N, longitude 147º20'W) was confirmed with the Livengood (A-1) SW quadrangle topographic map published by the USGS (1992a) and GoogleEarth™. The USGS website, which presents the results of the probabilistic seismic hazard assessments upon which the nationwide hazard maps are based, makes the underlying data available for download. The extracted data for the Fort Knox site is summarized in Table 2.2.

**Table 2.2**
**2007 Probabilistic Seismic Hazard**

| Probability of Exceedance | Recurrence Interval (years) | PHGA (g) |
|---|---|---|
| 50% in 75 years | 108 | 0.08 |
| 20% in 50 years | 224 | 0.12 |
| 10% in 50 years | 475 | 0.18 |
| 5% in 50 years | 975 | 0.25 |
| 2% in 50 years | 2,475 | 0.37 |
| 1% in 50 years | 4,975 | 0.47 |

### 2.5.2 Recommended Seismic Design Parameters

The BCHLF in-heap pond embankment will be considered an impounding structure due to the containment of process leach solution (PLS). During the short-term operational period, when the TSF is in place immediately downstream with a large impounding capacity, the BCHLF will be considered a low hazard consequence structure (Dam Hazard Classification III, see Appendix A), and per Table 2.1 and Table 2.2 the OBE and MDE were selected as 108-year and 975-year recurrence interval events, respectively, with the relevant seismic design parameters summarized in Table 2.3.

The BCHLF closure will be similar to that planned for the WCHLF. When the residual fluid within the BCHLF meets adequate quality standards, the fluid will be drained from the base of the facility by drilling a series of holes through the heap and underlying liner system and into the underdrain random fill. In this case, there will be no differential impoundment of waters upstream of the embankment and no post-closure risk of an uncontrolled release of process solution from the closed facility. As such, the BCHLF in-heap pond embankment will not be considered as a jurisdictional dam in the post-closure period. Notwithstanding this, Dam Hazard Classification II was adopted for the post-closure design standard for the BCHLF due to the long period of exposure of the facility, and per Table 2.1 and Table 2.2, the 2,475-yr recurrence interval event was adopted as the MDE with the relevant seismic design parameters summarized in Table 2.3.

A more detailed discussion of the seismic design parameter development for the Fort Knox site is presented under separate cover (Knight Piésold, 2016a). In summary, the design earthquake events presented in Table 2.3 were used to analyze the seismic stability of the BCHLF in its final heap configuration through the Stage 6 development presented herein.

**Table 2.3**
**Seismic Design Parameters**

| Loading Condition | Recurrence Interval | Design Event |
|---|---|---|
| OBE (during operations) | 108 years | M7.5 generating PHGA=0.08g |
| MDE (during operations) | 975 years | M7.5 generating PHGA=0.25g |
| MDE (after closure) | 2,475 years | M7.5 generating PHGA=0.37g |

To clarify, the BCHLF Dam is currently:

- Classified as Class III (3) for the operational period
- Designed to Dam Hazard Classification III (3) standards for the operational period
- Designed to Dam Hazard Classification II (2) standards for the post-closure period

The seismic design parameters selected for the OBE and MDE, for both the operational and post-closure periods, fall within the ranges of the values listed for Dam Hazard Classification III (3) and II (2) in the ADNR guidelines. The BCHLF in-heap pond embankment will not be considered a jurisdictional dam in the post-closure period (see Section 6.14).

# Section 3.0 - Climate Study and Hydrologic and Hydraulic Analyses

## 3.1 Climate Study

The basis for the most recent iteration of the climatic and hydrologic analyses for Fort Knox was originally performed by Knight Piésold to support development of the site-wide water balance. After the original submittal of the analyses and results, Knight Piésold updated certain portions of the datasets that were required for continued modifications and updates to the site-wide water balance model. The latest climatic and hydrologic analyses and results are presented in the Knight Piésold report:

- *Fairbanks Gold Mining, Inc., Barnes Creek Heap Leach Facility Final Design, Climatic/Hydrologic Characterization Report, Issued in Final, Revision 0*, by Knight Piésold and Co., December 14, 2016 (DV101.00089/99, DV16-1033)

In addition to the site-wide water balance, relevant results of the analyses were also used to support the civil design of water management structures associated with the project. The final design of the BCHLF water management structures (i.e. in-heap storage pond, diversion channels, culverts, and emergency spillway), and the BCHLF operational period process water balance, have been based on the updated climatic and hydrologic information provided in the above document, as follows:

- Precipitation, temperature, evaporation, surface runoff, and infiltration are the general climatic parameters used as input to the BCHLF operational period process water balance. The process water balance is used to help define operating levels and required storage components within the BCHLF in-heap pond
- Recurrence interval storm events (precipitation), temperatures, and snow accumulation are the general climatic parameters used as input to the hydrologic analyses conducted to support the design of the BCHLF water management structures and to help define operating levels and required storage components within the BCHLF in-heap pond

Because the process leach solution and the precipitation incident to the BCHLF is designed to be contained within the in-heap storage pond, no open bodies of water are included in the current design. As such, evaporation from water surfaces and the formation and melting of ice have not been utilized. These climatic parameters, included as part of the overall site climatic update and presented in the Climatic/Hydrologic Characterization Report (Knight Piésold, 2016b), are not discussed herein. Table 3.1 summarizes the site design average annual climatic/hydrologic parameters used in the current BCHLF design.

**Table 3.1**
**Site Design Average Annual Climatic/Hydrologic Parameters**

| Parameter | Natural Ground | Disturbed Ground | Heap Ore/ Waste Rock | Wet Surfaces |
|---|---|---|---|---|
| Precipitation (in) | 17.3 | 17.3 | 17.3 | 17.3 |
| Rain (in) | 12.0 | 12.0 | 12.0 | 12.0 |
| Snow (in) | 5.3 | 5.3 | 5.3 | 5.3 |
| Minimum Temperature (°F) | 22.0 | 22.0 | 22.0 | 22.0 |
| Maximum Temperature (°F) | 38.1 | 38.1 | 38.1 | 38.1 |
| Potential Evaporation (in) | 20.5 | 20.5 | 20.5 | 20.5 |
| Evaporation (in) | 7.7 | 7.9 | 4.8 | 18.5 |
| Runoff (in) | 6.0 | 7.0 | 4.0 | NA |
| Infiltration (in) | 3.6 | 2.3 | 8.4 | NA |

Table 3.2 presents the site-design rain, and rain-on-snow, depths per month resulting from the latest climatic and hydrologic analyses. The site design rain-on-snow was estimated considering the 10-yr snow accumulation and the 10-yr maximum average daily temperature.

**Table 3.2**
**Site Design Rain and Rain-on-Snow Depths per Month**

| Return Period (yrs) | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **24-hour Event Precipitation** | | | | | | | | | | | | |
| 2 | 0.4 | 0.3 | 0.2 | 0.3 | 0.5 | 1.0 | 1.1 | 1.0 | 0.8 | 0.4 | 0.4 | 0.3 |
| 10 | 0.9 | 0.5 | 0.4 | 0.6 | 0.9 | 2.1 | 2.0 | 1.7 | 1.4 | 0.7 | 0.9 | 0.6 |
| 50 | 1.3 | 0.8 | 0.7 | 1.0 | 1.4 | 3.3 | 3.0 | 2.4 | 2.1 | 1.0 | 1.3 | 0.9 |
| 100 | 1.5 | 0.9 | 0.8 | 1.2 | 1.6 | 3.8 | 3.4 | 2.7 | 2.3 | 1.1 | 1.5 | 1.0 |
| **24-Hour Event Snowmelt During Rain** | | | | | | | | | | | | |
| 2 | 1.0 | 0.4 | 0.4 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.6 | 1.3 | 0.6 | 0.4 |
| 10 | 1.1 | 0.4 | 0.4 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.6 | 1.3 | 0.6 | 0.4 |
| 50 | 1.2 | 0.4 | 0.5 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.6 | 1.4 | 0.6 | 0.4 |
| 100 | 1.2 | 0.4 | 0.5 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.6 | 1.4 | 0.6 | 0.4 |
| **24-Hour Event Rain-on-Snow** | | | | | | | | | | | | |
| 2 | 1.4 | 0.7 | 0.6 | 0.3 | NA | NA | NA | NA | 2.4 | 1.7 | 1.0 | 0.7 |
| 10 | **2.0** | 0.9 | 0.9 | 0.6 | NA | NA | NA | NA | **3.1** | 2.0 | 1.5 | 1.0 |
| 50 | 2.5 | 1.1 | 1.1 | 1.0 | NA | NA | NA | NA | 3.7 | 2.4 | 1.9 | 1.3 |
| 100 | **2.7** | 1.2 | 1.2 | 1.2 | NA | NA | NA | NA | **4.0** | 2.5 | 2.1 | 1.4 |

Notes: 1. The values in Table 3.2 have been converted to snow-water equivalents, where applicable.
2. Snowmelt was calculated in accordance with the United States Army Corps of Engineers *Runoff from Snowmelt, Engineer Manual 1110-2-1406* (USACE, 1998).
3. Snowmelt values are limited to the 10-year snow accumulation values, where applicable.
4. The cells shaded green represent the greatest depths per calendar month, corresponding to either rain-on-snow depths (September through April) or rain depths (May through August).

As discussed in Knight Piésold's Climatic/Hydrologic Characterization Report (Knight Piésold, 2016b), based on seasonal ground conditions and associated infiltration, the January rain-on-snow events control with respect to peak flows from surface runoff (i.e. for channel and culvert hydraulic design); however, the September rain-on-snow events control with respect to overall volume (i.e. the 100-yr/24-hr design storm storage component within the in-heap pond).

## 3.2 Hydrologic and Hydraulic Analyses

The surface water management plan for the BCHLF has been developed for controlling and safely directing upstream runoff around the facility, and for containment and conveyance of precipitation incident to the pad. More specifically, the BCHLF surface water management plan accounts for the following:

- Stage 1 haul road diversion channel
  - This channel is located adjacent to the Stage 1 haul road, that will originate from the existing crusher platform and generally run downhill to the upper end of the Stage 1 leach pad, and will discharge conveyed flows to the Stage 1 north inter-stage perimeter road

- Inter-stage perimeter road diversion requirements
  - As with the WCHLF, the BCHLF inter-stage perimeter roads will be used to convey surface runoff from upstream catchments to the BCHLF ultimate perimeter diversion channels
- Ultimate perimeter road diversion channels
  - These channels are sited primarily along the north/northeast perimeter of the BCHLF and along the north abutment leading uphill from the in-heap pond embankment
- Waste dump haul road diversion channel
  - This channel is located adjacent to FGMI's existing waste dump haul road which runs along the south/southwest perimeter of the BCHLF
- South perimeter diversion culverts
  - These culverts are generally sited along the south abutment of the BCHLF and will convey flows from the waste dump haul road diversion channel, and temporarily from the Stage 2 south inter-stage perimeter road, to the Fish Creek storm water detention basin
- In-heap storage pond emergency spillway and outlet channel
  - The emergency spillway is sited in the northeast corner of the BCHLF in-heap storage pond. Downstream of the spillway concrete weir section, the spillway transitions to the spillway outlet channel. The outlet channel will generally convey flows from the emergency spillway and the north perimeter diversion channel to the Fish Creek storm water detention basin.
- Fish Creek storm water detention basin
  - The storm water detention basin is sited along the downstream side of the existing conveyor causeway embankment and will receive any flows discharged through the BCHLF emergency spillway and from external surface runoff diverted around the facility (i.e. from the ultimate perimeter road diversion channels and the south perimeter diversion culverts).
- In-heap culvert crossings
  - In-heap culvert crossing Number (No.) 1 is sited near the southern corner of the Stage 2 leach pad. This culvert crossing will pass upstream flows beneath the envisioned heap access road that will be used for maintenance and operation of the vertical solution extraction wells.
  - In-heap culvert crossings Numbers (Nos.) 2, 3, and 4 (to accommodate heap access for loading of Stages 3, 4, and 5 of the BCHLF, respectively) are sited along the southwest perimeter of the BCHLF. These culvert crossings will pass the flows from upstream catchment areas beneath the heap-access haul road as it is progressively relocated up the existing waste dump haul road.

The following water management plans were developed to support the BCHLF designs:

- Water management plans for the inter-stage perimeter diversions (i.e. the inter-stage perimeter roads associated with Stages 1 through 5) are presented on Figures 3.1 through 3.5, respectively
- The water management plan for the ultimate perimeter diversion channel is presented on Figure 3.6
- The water management plan for the final (Stage 6) heap configuration (for sizing the in-heap storage pond emergency spillway, outlet channel, and in-heap culvert crossings) is presented on Figure 3.7

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) curve number (CN) approach (NRCS, 1986) was used with the USACE Hydrologic Engineering Center Hydrologic Modeling System software (HEC-HMS) kinematic wave routing techniques to estimate flood hydrographs and peak flows produced by 24-hr storm events for various return periods (USACE, 2010). The NRCS Type I storm hyetograph (rainfall distribution), modified to account for snowmelt where applicable, was adopted as representative for modeling storms events at the Fort Knox site.

Both summer storm (rainfall only) and rain-on-snow events were evaluated to determine which produces the larger peak flow for use in design. The January rain-on-snow events control with respect to peak flows from surface runoff (i.e. for channel and culvert hydraulic design); however, the September rain-on-snow

events typically control with respect to overall volume (i.e. the 100-yr/24-hr design storm storage component within the lined in-heap pond). Due to runoff conditions (i.e. frozen ground versus infiltration during summer storm events), the January rain-on-snow event was found to control the required capacity of the Fish Creek storm water detention basin.

### 3.2.1 Storm Water Incident to the Pad

Storm water incident to the pad is defined as the runoff produced, including snowmelt, by precipitation falling directly on the surface of the ore heap and within the lined leach pad basin.

Most storm water incident to the BCHLF leach pad will infiltrate into the heap and be conveyed as subsurface flow through the relatively coarsely graded and highly porous and permeable ore (Knight Piésold, 2017a) to the in-heap storage pond. Any small amount of water that occurs as surface runoff will flow in the perimeter offset between the toe of the heap and the pad perimeter berm. Any surface flow will be conveyed downslope to the relatively small impoundment area between the upstream slope of the in-heap pond embankment and the front face of the heap, where it will typically infiltrate the heap. This swale (1540 fmsl bottom elevation) is formed between the crest of Lift 2 and the toe of Lift 3, and will provide an estimated temporary storage capacity of approximately 0.10 M cubic feet ($ft^3$), or 0.7 M gallons, below the in-heap pond spillway invert elevation of 1542 feet above mean sea level (fmsl).

Ice formation on the surface of the heap is not expected to significantly impede the infiltration of melt water due to the coarse gradation and relatively large percentage of interstitial openings at the surface of the heap. If frozen conditions prevent heap infiltration, a larger portion of surface runoff will be conveyed to, and temporarily stored in, the swale at the front toe of the BCHLF heap. Any excess runoff would flow through the emergency spillway, located on the northeast abutment of the in-heap pond embankment, and be conveyed (along with surface runoff external to the leach pad) to the Fish Creek storm water detention basin. Flows collected within the Fish Creek detention basin (see Section 6.10) are expected to infiltrate the coarse backfill material and be intercepted by the existing pit dewatering wells and then pumped to the TSF for containment or use as makeup. Based on the current BCHLF configurations, the potential leach pad runoff volume under frozen conditions is estimated to be 2.83 M $ft^3$ (January 100-yr/24-hr rain-on-snow event, 2.70 inches, multiplied by the 2-dimensional area of the final pad (through Stage 6), 12.60 M square feet ($ft^2$)) in comparison to the 0.10 M $ft^3$ capacity of the swale (below the spillway invert). Therefore, the excess runoff volume that would occur as overflow through the BCHLF emergency spillway would be approximately 2.73 M $ft^3$. Based on the design assumptions, this overflow runoff should be considered as non-contact since it theoretically would be separated from the leach pad process solution by the frozen exterior slopes of the heap. The in-heap storage pond emergency spillway has been designed to pass 220 cubic feet per second (cfs). This is the peak flow runoff resulting from the 100-yr/24-hr January rain-on-snow event acting on the final pad footprint, accounting for the Stage 6 heap configuration, and based on little to no attenuation within the in-heap pond swale.

If the heap surface is not frozen, the storm water incident to the pad will infiltrate to the in-heap storage pond which has sufficient capacity to contain the September 100-yr/24-hr rain-on-snow storm event. The volume components of the in-heap storage pond are discussed in Section 6.9.4.

### 3.2.2 Hydraulic Analyses

The hydraulic design of channels was completed using the Manning formula, for normal flow conditions, and the general discharge formula as follows:

$$V = k/n\ (R_h^{2/3})\ (S^{1/2}),\ \text{and}$$
$$Q = A\ V$$

The general discharge formula can be re-written as follows (Chow, 1959):

$$Q = k/n\ (A\ R_h^{2/3})\ (S^{1/2})$$

Where: $Q$ = the volumetric discharge (cfs)
$A$ = the cross-sectional area of the flow ($ft^2$)
$k$ = a conversion constant equal to 1.486 for U.S. customary units
$n$ = the Gauckler-Manning coefficient ($s/ft^{1/3}$)
$R_h$ = the hydraulic radius (ft)
$S$ = the slope of the water surface or the linear hydraulic head loss (ft/ft)

Manning's n values were estimated based on the channel erosion protection utilized (i.e. soil-lined, riprap, grouted riprap, etc.). As requested by FGMI, the inter-stage perimeter roads were not designed with adjacent diversion channels, but rather the surface runoff from upstream areas is generally intended to be conveyed by the road cross-section.

The surface water management structures associated with the BCHLF were sized per the following:

- Inter-stage perimeter road diversions
  - Capacity – sized to accommodate the peak flow runoff from all contributing upstream watersheds, resulting from the 100-yr/24-hr January rain-on-snow event
    - Flow conveyed along the road running-width with up to 1 ft of flow depth adjacent to the geomembrane-lined road safety berm

    The inter-stage perimeter road diversions were designed with minus 1.0-percent grades and typical 2-percent cross-falls. Minus 20-percent cross-falls were utilized for the north inter-stage perimeter road shoulder widths (i.e. the hill-side width of the road corridor in excess of 16 ft).
  - Erosion protection – unlined road surface comprised of weathered schist/gravel, some cobbles, and colloidal silts
- Ultimate perimeter road diversions
  - Capacity – sized to accommodate the larger peak flow runoff resulting from the 100-yr/24-hr January rain-on-snow event acting on:
    - all contributing upstream watersheds with no freeboard, or
    - contributing upstream watersheds external to the Stage 6 pad footprint, with 1 ft of freeboard

    The ultimate perimeter road diversion channels were designed with grades varying between 1 and 25 percent.

    The inter-stage perimeter roads were configured to divert all collected runoff from upstream areas to the north ultimate perimeter diversions and the south perimeter diversion culverts. As the BCHLF is expanded, the contributing areas (external to the lined pad area, but internal to the final footprint through Stage 6) will decrease to zero. The difference between the 100-yr/24-hr peak flows (i.e. the flows from external catchments and the flows from internal plus external catchments) will be temporarily accommodated within the ultimate perimeter channel freeboard.
  - Erosion protection – riprap, sized for the larger peak flow runoff resulting from:
    - the 10-yr/24-hr January rain-on-snow event acting on all contributing upstream watersheds, or
    - the 100-yr/24-hr January rain-on-snow event acting on contributing upstream watersheds external to the final (Stage 6) pad footprint
- Stage 1 haul road diversion
  - Capacity – sized to accommodate the peak flow runoff from all contributing upstream watersheds, resulting from the 100-yr/24-hr January rain-on-snow event
  - Erosion protection – riprap, sized for the design capacity peak flow

- South perimeter diversion culverts
  - Capacity – based on the same criteria as the ultimate perimeter road diversion channels

    The south perimeter diversion culverts were designed with grades varying from 0.5 to 40 percent
  - Erosion protection – based on the same criteria as the ultimate perimeter road diversion channels
- In-heap culvert crossings
  - Capacity – sized to accommodate the peak flow runoff from all contributing upstream watersheds, within the final (Stage 6) pad footprint, resulting from the 100-yr/24-hr January rain-on-snow event (based on the Stage 6 heap configuration)
- In-heap storage pond emergency spillway and outlet channel
  - Weir section
    - Capacity – sized to accommodate the peak flow runoff from all contributing upstream watersheds within the final (Stage 6) pad footprint, resulting from the 100-yr/24-hr design storm event (based on the Stage 6 heap configuration), with 1 ft of freeboard
    - Erosion protection – reinforced concrete
  - Outlet channel
    - Capacity – sized to accommodate the worst-case combined peak flow from the attenuated spillway discharge plus the discharge from the north ultimate perimeter diversion channel, resulting from the 100-yr/24-hr design storm event, with 1 ft of freeboard (assessed for the Stage 1 and Stage 6 configurations)

    The spillway outlet channel was designed with a 3 percent grade and includes a low-level culvert crossing beneath a conceptual drive-thru crossing.
    - Erosion protection – grouted riprap, sized for the design capacity peak flow

# Section 4.0 - Geotechnical Investigation

## 4.1 Introduction

To date, the Barnes Creek valley at Fort Knox has been utilized for storage of mine waste rock and low-grade ore from the open pit. A conveyor system and causeway (mean elevation 1545 fmsl) cross the lower reaches of the valley for delivery of ore from the primary crusher to the mill. Since its construction, the upstream and downstream slopes of the conveyor causeway were covered by the Barnes Creek and Fish Creek low-grade ore stockpiles, respectively. Most of the low-grade ore, from both stockpiles, is planned to be removed for processing and to facilitate construction of the BCHLF. Once the Fish Creek stockpile material is removed, FGMI intends to backfill the area downstream of the causeway with mine waste rock to approximately elevation 1600 fmsl.

The existing conveyor causeway is envisioned to partially form the impounding embankment for the BCHLF in-heap storage pond. During its construction, the causeway foundation was cleared of vegetation and unsuitable soils, and those materials were placed upstream in the base of the Barnes Creek valley. The low-grade ore was then stockpiled directly on top of the spoil piles and unstripped native ground. Removal of these spoil piles, and any other topsoil and unsuitable material from beneath the base of the Barnes Creek stockpile, will likely be difficult due to an estimated groundwater elevation of 1452 fmsl near the toe of the planned BCHLF, which is above the original native ground surface.

Geotechnical site investigations were conducted in 2011 and 2016 to support the final design of the BCHLF. The investigations primarily focused on subsurface conditions to verify the nature of the Barnes Creek conveyor causeway fill, the causeway foundation, and the foundation beneath the Barnes Creek low-grade ore stockpile. A key goal of the investigations was to assess the in-situ material's ability to convey groundwater flows, expected to emanate from beneath the planned BCHLF, through and below the existing causeway. In fact, the conveyor causeway was specifically designed as a flow-through embankment to allow surface runoff within the Barnes Creek drainage to pass through the embankment and into the TSF (Knight Piésold, 1994). Evaluation of surface and subsurface conditions consisted of site reconnaissance, rotary drilling and rock coring, material sampling, and laboratory testing. Soil samples collect during the 2011 and 2016 site investigations were sent to Knight Piésold's geotechnical laboratory in Denver, Colorado. Other testing, was completed on material from potential borrow sources for construction of the BCHLF. This included some work completed on materials that were utilized elsewhere on site that will also be a part of the BCHLF construction.

Details of the various investigations are included under separate cover (Knight Piésold, 2017a). It should be noted that the geotechnical investigations completed to date have quantified the nature and expected performance of the various materials that will comprise the BCHLF; however, investigations have not been sufficiently detailed to identify available quantities of the various materials that will be borrowed locally, especially the prepared subbase. As part of the preparation for each stage of construction, further geotechnical site investigations are recommended to establish sufficient quantities and quality of prepared subbase for the work.

## 4.2 Surface Conditions

The BCHLF valley is predominantly disturbed and covered with mine waste rock dumps, a low-grade ore stockpile and haul roads. Small pockets of forested area, with a variety of ground covers, remain on the upper northeast side-slopes and in the far western reaches of the valley. The exposed valley slopes on the northeast side are moderate, ranging from 4H:1V to 5H:1V on average. Following FGMI's planned removal of the Barnes Creek low-grade ore stockpile, the maximum relief across the BCHLF footprint is expected to be on the order of 720 ft, from the base of the in-heap pond (1450 fmsl) to the northwest perimeter of the Stage 6 leach pad (2170 fmsl).

Some localized areas of permafrost were required to be removed from the base of the Walter Creek valley during the initial stages of that pad construction. The limited boreholes included with the 2011 Barnes Creek site investigation did not encounter permafrost, but it is possible that some as-yet unidentified permafrost will need to be removed from Barnes Creek. This will be assessed in the field during construction.

As of August 2014, the tailing deposit had reached an estimated elevation of 1525 fmsl in the area just below and to the east of the BCHLF. This tailing level will continue to rise until the end of milling operations and is currently anticipated to reach an approximate maximum elevation of 1554 to 1558 fmsl along that edge of the TSF.

## 4.3 Subsurface Investigation

The 2011 and 2016 site investigations and associated laboratory testing program consisted of test pitting and soil boring through the existing low-grade stockpile and existing native ground, rock coring into the underlying bedrock, in-situ material testing, recovered material logging, and sampling for subsequent geotechnical laboratory testing as summarized in the following sections.

### 4.3.1 Test Pitting

A total of twenty-two test pits were excavated within the Barnes Creek valley and targeted potential prepared subbase borrow areas to include: (1) some native material exposed by low-grade ore removal in the bottom of the Barnes Creek valley (i.e. the lower borrow area), and (2) several undisturbed areas on the southwest side of the ridge separating the Barnes and Walter Creek valleys (i.e. the upper borrow area).

### 4.3.2 Soil Boring

Eight soil borings were completed during the 2011 site investigation with two more completed in 2016. Boring Nos. B-01 through B-03 and B-09 were advanced through the conveyor causeway embankment. Boring Nos. B-04 through B-06 and B-10 required the driller to advance the holes through substantial thicknesses of low-grade ore or mine waste rock before the underlying soils were encountered. Boring Nos. B-07 and B-08 were located at the edge of the Barnes Creek low-grade stockpile such that the native materials were exposed at the ground surface.

Drill cuttings were logged and occasional bulk disturbed samples were collected to assess the nature and consistency of the stockpiled low-grade ore, mine waste rock, and the underlying foundation soils. Conventional soil samplers included a 3.0-inch-outside-diameter modified California barrel sampler that captured samples in 2.5-inch-diameter by 6.0-inch-long brass liners and a 2.0-inch-outside-diameter split spoon sampler that collected small disturbed samples. The standard penetration test (SPT) results were adjusted to account for the variations from the standard equipment. After collection, material samples were appropriately packaged, sealed, and labeled and then shipped to Knight Piésold's geotechnical laboratory in Denver, Colorado for subsequent testing.

### 4.3.3 Rock Coring

Rock coring into the underlying Fairbanks Schist was completed at Boring Nos. B-01 through B-04 and B-06. The borings were advanced into bedrock using HQ3 triple tube methods. The resultant core was logged for geotechnical properties such as recovery, rock quality designation (RQD), rock strength, weathering/alteration, fracture frequency/spacing and description (i.e. shape and roughness), and the core was photographed in the HQ3 splits before being boxed and labeled for storage on site.

### 4.3.4 In-Situ Testing

Soil drilling at Boring No. B-02 was interrupted periodically to complete falling head, in-situ permeability tests of the surrounding material. The casing was advanced to the target depth and retracted to leave a

3.0-ft-long test interval. Prior to the test, a Level TROLL 700 data-logger was suspended within the casing below the level of the surrounding groundwater table at a depth of 123.0 ft to measure pressure head variance observed during the test. To initiate the test, the drill casing was filled with water above the level of the surrounding groundwater table, and the falling water level within the casing was recorded over time with the data logger. The resultant data were analyzed to assess the saturated hydraulic conductivity of the surrounding soil. A comparison of the soil boring log for Boring No. B-02 to the permeability test results indicated that the five in-situ permeability tests were actually conducted below the limits of the causeway foundation excavation within the underlying alluvial gravels. The shallowest test indicated a relatively high saturated hydraulic conductivity of $1.2x10^{-2}$ cm/sec, which would be pervious enough to help pass underdrainage flows from beneath the BCHLF liner toward the downstream TSF. The other deeper in-situ permeability tests revealed a decreasing conductivity with depth as would be expected with increasing density with depth and the variable depositional environment under which those sediments were emplaced that produced interbedded finer-grained and coarser-grained layers. Similar test work was completed in Boring No. B-09 to assess the saturated hydraulic conductivity of the conveyor causeway embankment itself. In-situ testing down Boring No. B-09, in the base of the causeway embankment, suggested a hydraulic conductivity of $5x10^{-4}$ cm/sec, with lab results indicating numbers as high as $1x10^{-1}$ cm/sec on similar schistose waste rock.

The most common in-situ penetration test, the SPT, involves driving an unlined, 2.0-inch outside-diameter, split spoon sampler a total of 18 inches (i.e. three 6-inch increments) using a 140-pound hammer with a 30-inch drop. During the test, the number of hammer blows necessary to advance the sampler over each 6-inch increment is recorded. Sampler refusal is defined as reaching a blow count of 50 during any one of the 6-inch increments. For tests that do not encounter refusal, the sum of the last two incremental blow counts comprises the SPT blow count or N-value. The SPT blow count provides an indication of the relative density of coarser-grained materials and may be correlated with many other geotechnical material properties. Blow count data for in-situ penetration testing were recorded on 6-inch increments during collection of those samples; however, since the sampling involved non-standard equipment (i.e. heavier hammers with longer drops, and in some cases larger diameter samplers), the collected data was adjusted in accordance with guidelines provided by Lacroix and Horn (1973) to provide approximate SPT blow counts. In examining the results of the in-situ penetration testing:

- Three tests were completed in the conveyor causeway fill. Two tests refused, and one yielded an approximate SPT blow count of 47.
- Twenty-six tests were completed in the native soils. Nineteen tests refused, and the other seven yielded approximate SPT blow counts between 31 and over 100.
- Fourteen tests were completed in the highly-weathered bedrock. Eleven tests refused, and the other three yielded approximate SPT blow counts between 41 and 97.

Per the guidelines provided by Peck, Hanson and Thornburn (1974), these results are representative of dense to very dense materials with approximate friction angles in excess of 36 degrees.

## 4.4 Geotechnical Laboratory Testing

After their receipt from the field, samples were inventoried, and specific testing was assigned. The testing was completed at the Knight Piésold geotechnical laboratory in Denver, Colorado. Numerous sets of index testing were conducted to provide a broad-based characterization of the nature and extent of the materials encountered beneath and adjacent to the Barnes Creek low-grade ore stockpile and the conveyor causeway as well as the materials that will comprise the BCHLF itself. From that initial suite of index testing, samples were selected for additional geotechnical testing to provide characterization of the shear strength, compressibility and permeability of those materials. Each of the tests conducted was completed in general accordance with the applicable standard procedures of the American Society for Testing and Materials (ASTM, 2015a and 2015b) and the Unites States Bureau of Reclamation (USBR, 1998 and 1990). Detailed geotechnical laboratory test results from the testing programs in 2011 and 2016 (as well as applicable results from testing of site specific materials from other aspects of the work at Fort Knox) are included under

separate cover (Knight Piésold, 2017a). The balance of the samples was retained either on site or at the laboratory in Denver, Colorado for use in future testing, if required. Materials of interest include:

- Leach Ore – comprises mineralized granitic material that will either be placed on the completed heap leach pad for processing or will remain as loose-dumped low-grade ore from the stockpile that will be abandoned below the heap leach pad as part of the BCHLF basin grading. In either case, the leach ore was placed loose and, thus, can be expected to be relatively more compressible compared to the other materials that are part of the BCHLF.
- Drain Aggregate – comprises mill reject (i.e. granitic material rejected by the crushing and grinding processes in the Fort Knox mill) that will be processed to yield both 2-inch minus and 1/2-inch minus materials for placement in the process solution collection system and groundwater underdrains. Since the drain aggregate is relatively strong and will only exist in thin layers within the BCHLF construction, it is typically neglected from slope stability and settlement analyses. It was important, however, to confirm that the drain aggregate will maintain a sufficiently high saturated hydraulic conductivity to function as intended.
- Prepared Subbase – comprises locally available finer-grained soil that when placed and compacted provides a saturated hydraulic conductivity of less than $1.0 \times 10^{-5}$ cm/sec as part of the composite liner system underlying the BCHLF. The prepared subbase may include aeolian silt, residual soil weathered from the Fairbanks schist or a colluvial mixture of the two. While generally low to non-plastic, the prepared subbase can achieve the desired conductivity if the material is sufficiently well graded with a high enough fines content (and it is placed and compacted properly).
- Site Grading Fill and Mine Waste Rock – generally comprise schistose material, although there is some non-mineralized granitic material present on site. Site grading fill is material that is locally available as cut-to-fill within the BCHLF basin or that will be borrowed from mine waste rock production. In either case, it is generally schistose material that will be placed and compacted as engineered fill and, thus, is expected to be relatively less compressible compared to the other materials that are part of the BCHLF and its foundation. In the context of the design and analysis of the BCHLF, site grading fill includes the existing conveyor causeway embankment. Mine waste rock comprises similar schistose material that is loose dumped and, thus, can be expected to be relatively more compressible like the leach ore.
- Foundation Soils – comprise aeolian silt, residual soil weathered from the Fairbanks schist, creek gravels, and colluvial mixtures. This includes the organic soil and excavation spoil known to have been left under the Barnes Creek low-grade ore stockpile upstream of the conveyor causeway embankment that will remain in place after the planned construction of the BCHLF. Where sufficiently fine-grained and inorganic, the foundation soils may serve as prepared subbase borrow.
- Foundation Bedrock – comprises the Fairbanks schist that underlies much of the project vicinity. Since the schist is strong and stiff relative to the foundation soils and other materials comprising the BCHLF, it will have little influence on the slope stability and settlement evaluations of the BCHLF. Where assessed elsewhere on site (i.e. at the TSF), the behavior of the highly-weathered and heavily-fractured schist was not found to be controlled by geologic structure (i.e. fractures, foliation, etc.). Consequently, its behavior has been assessed as a fractured rock mass, and those data (Knight Piésold, 2016c) have been adopted for evaluations of the BCHLF.

In addition to the natural materials described above, testing sought to assess potential geosynthetics for the BCHLF construction (i.e. both GSE- and Solmax-manufactured 80-mil double-side textured (DST) linear low density polyethylene (LLDPE) geomembrane and GSE-manufactured geosynthetic drain layers) as discussed in Section 4.10.

## 4.5 Index Testing

Particle size distribution analyses (i.e. hydrometer and/or sieve analyses, ASTM D422), were conducted on numerous samples of each of the material types listed above. Atterberg limits (ASTM D4318) were also run on samples of the finer-grained materials. The combination of these index tests allowed the samples

to be classified (ASTM D2487) according to the unified soil classification system (USCS), which is commonly used to group soils that exhibit similar engineering properties and behavior.

The leach ore and drain aggregate comprise relatively clean gravels that can be expected to be strong, stiff and freely draining. The potential prepared subbase borrow sources yielded material that were slightly too coarse compared to the desired specification range; however, the results of a test fill indicated that the friable schist particles comprising the residual soil will break down on handling (i.e. excavation, hauling, placement, and compaction) to yield a material with the desired particle size distribution. Although the site grading fill and mine waste rock were found to contain a significant gravel fraction, they exhibited a much higher fines content than the leach ore and drain aggregate due to the friability of the schist relative to the granitic materials. The foundation soils encountered comprise non-plastic to low-plasticity, relatively well-graded, coarse-grained soils (i.e. silty sands with gravel and silty gravels with sand). Of the foundation soils samples tested, several on the coarser side of the gradation band are representative of the highly-weathered bedrock, while the native soils are somewhat less coarse-grained.

## 4.6 Specific Gravity Testing

Specific gravity testing (ASTM C127 and/or D854) was completed on a limited number of samples from each of the material types to facilitate assessment of the phase relationships (i.e. the relationships between moisture content, unit weight, void ratio, and degree of saturation), which are necessary to interpret other testing including compressibility and permeability as well as to develop material properties for subsequent geotechnical engineering analyses.

## 4.7 Moisture Content and Unit Weight Testing

FGMI provided two data sets relative to the moisture content of the leach ore, site grading fill and mine waste rock. These included: (1) results of moisture content testing of samples from sonic drilling of the WCHLF in April and July 2013 and (2) dig face moisture content testing of ore from the Fort Knox and True North open pits from 2001 to 2005. The moisture contents from the WCHLF drilling were parsed into three categories including: (1) material that had been placed but not leached (i.e. the as-received moisture content of the leach ore), (2) material that had been placed and had been under leach long enough to reach steady state conditions (i.e. the under leach moisture content of the leach ore), and (3) material that had been placed, leached and allowed to draindown for an extended period of time (i.e. the draindown moisture content of the leach ore). The 2001 to 2005 dig face moisture content data from the Fort Knox open pit were include in the assessment of the as-received moisture content of the leach ore. Unlike the Fort Knox ore that is granitic in origin, the True North ore is schistose in origin, so the 2001 to 2005 dig face moisture content data from the True North open pit were assessed as indicative of the natural moisture content of schistose materials to include the site grading fill and mine waste rock. Since the data were not normally distributed, the median values, rather than the mean values, were selected as representative of the various material types and conditions. These included:

- Leach ore as-received moisture content – 2.6 percent by weight
- Leach ore under leach moisture content – 5.4 percent by weight
- Leach ore as-received moisture content – 4.0 percent by weight
- Site grading fill and mine waste rock natural moisture content – 7.3 percent by weight

The natural moisture content of the schistose material was also consistent with draindown moisture contents from schistose ore samples from the Gil-Sourdough deposit, which is another satellite deposit to Fort Knox like True North.

Natural moisture content (ASTM D2216) was measured for twenty-one samples of the foundation soils including several samples of the highly-weathered bedrock. In addition to the natural moisture content testing, sixteen of the samples (i.e. those in modified California sampler brass liners), were also tested for in-situ dry unit weight (ASTM D2937). The samples indicated dry unit weights ranging between 78.6 and

131.1 pounds per cubic foot (pcf) with median value of 120.3 pcf. Natural moisture contents for the foundation soils above the groundwater table ranged between 2.3 and 6.9 percent by weight with a median value of 5.1 percent by weight.

## 4.8 Compaction Testing

Potential borrow materials that would be placed and compacted as engineered fill (i.e. prepared subbase and site grading fill) were subject to modified Proctor compaction (ASTM D1557) testing. The results of those compaction tests provide insight as to the behavior of the materials when placed and compacted by modern methods in earthworks construction.

## 4.9 Triaxial Shear Strength Testing

Specimens of the leach ore, prepared subbase, site grading fill and foundation soils were subject to isotropically consolidated undrained triaxial shear strength testing with pore pressure measurements (ASTM D4767). In each test, three test specimens were consolidated under a range of effective confining stresses, which were expected to be representative of the stresses induced under the weight of the proposed BCHLF, before shearing. The leach ore specimens were placed loose and allowed to consolidate under the effective confining stress before shearing to simulate the loose placement of the leach ore in the field. The prepared subbase specimens were prepared at 93 percent of their modified Proctor maximum dry density (MDD) and at their optimum moisture content (OMC) plus 3 percent by weight to represent conditions anticipated during construction of the BCHLF.

Testing of the Monte Cristo Creek weathered schist was originally conducted for design of the TSF. Since the mine waste rock from that portion of the Fort Knox pit is known to be some of the most highly weathered that the mine produces, strength test results on the Monte Cristo Creek weathered schist were deemed to provide a conservative assessment of the site grading fill at the BCHLF. Those specimens were prepared at a density representative of that realized in the field during the construction of the TSF. This comprises about 93 percent of the modified Proctor MDD, while the actual specification for the site grading fill at the BCHLF requires a higher (i.e. 95 percent of modified Proctor MDD) density, which would yield an even higher shear strength.

In 2011, material from one of the finer-grained bulk disturbed samples of the foundation soils was selected for subsequent shear strength testing. To provide a conservative (lower) assessment of the shear strength of the native soils, test specimens were remolded at the lowest in-situ dry unit weight measured during the 2011 investigation (i.e. 106.6 pcf, at a moisture content of 6.9 percent by weight). The sample selected for shear strength testing had the lowest gravel content (percent retained on a No. 4 sieve) and one of the highest fines contents (percent passing a No. 200 sieve) of the samples tested. In 2016, a series of three relatively undisturbed specimens of the foundation soils (i.e. modified California sampler brass liners) were also subject to triaxial shear strength testing.

Because of its lower density, the effective stress friction angle of the leach ore was about 38 degrees, while the compacted site grading fill was 43 degrees. Despite the fact that the granitic material is generally thought to be stronger because the individual particles are harder and more durable, the effect of proper placement and compaction on the strength of the resultant fill is apparent. No specific testing was completed on the mine waste rock, because the strength of loose dumped schistose material can be approximated as a friction angle of 36 degrees based on the inclination of angle of repose slopes at the existing waste rock dumps on site.

The prepared subbase exhibited a friction angle of 36 degrees, which far exceeds the strength of the subbase or geosynthetic interfaces discussed in Section 4.10, therefore the shear strength of the composite liner system underlying the BCHLF will be controlled by the associated interface strengths and not the internal strength of the prepared subbase. The foundation soils exhibited friction angles of 32 and 38 degrees for the remolded and relatively undisturbed specimens, respectively. Given those results and an approximate friction angle of 36 degrees based on the SPT results discussed in Section 4.3.4, a

conservative friction angle of 32 degrees was selected to represent the strength of the foundation soils. Since that exceeds the strength of the liner system interfaces, the slope stability of the BCHLF will be largely unaffected by the shear strength of the foundation soils. The same is true of the foundation bedrock for which the rock mass strength is represented by a Hoek-Brown failure envelope from an assessment of the Fairbanks schist conducted for the TSF (Knight Piésold, 2016c).

## 4.10 Geosynthetics Testing

Within the portion of the BCHLF above the crest of the in-heap pond embankment, the facility will be lined with a single 80-mil DST LLDPE geomembrane on prepared subbase. The shear strength of the single lined system will be controlled by the strength of the subbase versus geomembrane interface (i.e. the subbase interface). Within the portion of the BCHLF below the crest of the in-heap pond embankment, the facility will be lined with a double 80-mil DST LLDPE geomembrane on prepared subbase with the two geomembranes separated by a geosynthetic drain layer. The shear strength of the double lined system will be controlled by the strength of the geosynthetic drain layer versus geomembrane interface (i.e. the geosynthetic interface).

Geomembranes from two different suppliers, GSE and Solmax (both standard products), were considered. The geosynthetic drain layers considered included either GSE TenDrain 300 geonet or a GSE geocomposite (which includes the geonet with geotextile heat bonded to each side). The shear strength and puncture resistance of the single- and double-lined systems were assessed as discussed in the following sub-sections.

### 4.10.1 Subbase Interface Testing

Apart from work on the BCHLF, a significant body of data exists from liner interface direct shear strength testing of prepared subbase materials originating in the Barnes Creek valley that were utilized in the construction of the WCHLF. In addition to that historic data, liner interface direct shear testing (ASTM D5321) was completed to assess the shear strength of two subbase interfaces, i.e., prepared subbase versus GSE 80-mil DST LLDPE geomembrane and prepared subbase versus Solmax 80-mil DST LLDPE geomembrane. The tests involved placement of prepared subbase material (blended to match the gradation of the field-constructed test fill discussed in Section 4.5) at 93 percent of the modified Proctor MDD and OMC plus 3 percent by dry weight against the wetted face of the geomembrane. Each test specimen was fully consolidated to a designated confining stress before initiating shear.

In evaluating the results of the recently completed liner interface direct shear strength testing as compared to historic testing of materials from the Barnes Creek valley that were utilized in the construction of the WCHLF, the recent testing produced a lower interface strength than the historic testing. The historic testing includes a larger number of individual test points and includes samples of material that were placed and compacted as part of the full-scale WCHLF construction (i.e. they were recovered from beneath the WCHLF Stage 5 geomembrane before that area was loaded with leach ore). As such, the historic testing was believed to be more representative of the behavior anticipated beneath the BCHLF and was used in analyses of the BCHLF. Recently tested material, which was made to replicate the test fill gradation discussed in Section 4.5, was heavily processed and due to a shortage of material had been reused from prior index, compaction, triaxial shear, and permeability testing. It is very likely, given the friable nature of the weathered schist, that the particles had degraded thereby yielding a different gradation and a less angular material that would likely exhibit a weaker interface strength, which is consistent with the results as noted above. Additional samples of freshly screened prepared subbase should be tested in the lead up to the BCHLF construction to confirm the strengths used in the design from the historic testing.

As noted in the BCHLF CQA/QC Manual (Knight Piésold, 2017d), as part on on-going CQA/QC, samples of prepared subbase material shall be collected and shipped to an off-site geotechnical laboratory for more-advanced geotechnical testing (i.e. permeability and interface shear strength). A minimum of six (6) 5-gallon buckets shall be sampled for each prepared subbase borrow area (or for every 25,000 cubic yards of prepared subbase, whichever is greater) that will be used for each year's construction. Three (3) of

every six (6) buckets shall be shipped to the off-site laboratory for testing, while the other three (3) buckets shall be kept on site as backup. Updated stability and deformation analyses shall be conducted, as deemed necessary, based on the results of additional testing prior to the BCHLF construction and based on the planned on-going testing program.

### 4.10.2 Geosynthetic Interface Testing

Liner interface direct shear testing (ASTM D5321) was completed to assess the shear strength of four geosynthetic interfaces (i.e. geonet and geocomposite versus GSE 80-mil DST LLDPE geomembrane, and geonet and geocomposite versus Solmax 80-mil DST LLDPE geomembrane). The geonet clearly provides better shearing resistance against the 80-mil DST LLDPE geomembrane than the geocomposite at the high stress levels considered, therefore the geocomposite was eliminated from further consideration, and GSE geonet will be utilized as the geosynthetic drain layer regardless of the geomembrane supplier ultimately selected.

### 4.10.3 Liner Integrity Testing

Liner system puncture resistance is evaluated by liner integrity testing (LIT). In that procedure, a cross-section of the liner system and overlying solution collection system is constructed within a rigid wall permeameter, and the liner system is loaded to a vertical effective confining stress that is equivalent to the anticipated maximum height of leach ore to be placed above the liner. After loading, the liner coupon or coupons are exhumed, visually inspected and vacuum box tested for leaks. A pass or fail result is assigned depending on whether any punctures are identified or not.

A series of six LIT were completed for the BCHLF, including:

- GSE single geomembrane
- GSE double geomembrane with GSE geonet
- GSE double geomembrane with GSE geocomposite
- Solmax single geomembrane
- Solmax double geomembrane with GSE geonet
- Solmax double geomembrane with GSE geocomposite

In each test, the geosynthetics were underlain by prepared subbase that was placed and compacted at 93 percent of the modified Proctor MDD, and at OMC plus 3 percent by weight, and the geosynthetics were overlain by 2-inch minus drain aggregate with the larger particles adversely oriented to the geomembrane surface. Each of the liner systems considered passed the LIT.

## 4.11 Compressibility Testing

Settlement analyses of the BCHLF comprise evaluation of the vertical deformations of material beneath the composite liner system under the weight of the leach ore placed above the liner system. These analyses require evaluation of the compressibility of the material types involved including the leach ore, site grading fill, mine waste rock, foundation soils, and foundation bedrock. Such testing included:

- Rigid wall permeability testing (USBR 5600 and/or 5605) of the leach ore
- Load-density testing (USBR 5600 and/or 5605) of the site grading fill and mine waste rock with the procedure modified to evaluate dry unit weight and void ratio as a function of vertical effective confining stress without the associated permeability testing
- Consolidation testing (ASTM D2435) of the foundation soils

Given the resultant test data, compressibility (void ratio versus effective confining stress) and, in some cases, conductivity (void ratio versus saturated hydraulic conductivity) relationships were developed for the materials in accordance with the forms suggested by Abu-Hejleh and Znidarcic (1994), which comprise:

Compressibility: $e = A(\sigma' + Z)^B$
Conductivity: $K = Ce^D$

where: e - void ratio
σ´ - effective confining stress
K - saturated hydraulic conductivity
A, B, Z, C and D - curve fitting parameters

## 4.12 Permeability Testing

Permeability testing of the various material types included rigid wall permeability testing of the more permeable materials (USBR 5600 and/or 5605) and flexible wall permeability testing of the less permeable materials (ASTM D5084) under a range of effective confining stresses representative of conditions anticipated in the field. The flexible wall permeability tests were commonly conducted in conjunction with the triaxial shear strength testing discussed in Section 4.9.

## 4.13 Results

In support of the final design of the BCHLF, Knight Piésold has compiled and analyzed: (1) geotechnical site investigation and laboratory testing data from programs that were conducted in 2011 and 2016, (2) information from evaluations of other facilities on site that involve the same or similar materials, and (3) available historic information about the design and construction of the conveyor causeway. Given that background, several significant conclusions can be drawn and recommendations made regarding the suitability of the site and borrow sources for construction of the proposed BCHLF. Those follow below:

- Geologic mapping near the BCHLF indicates that the Barnes Creek valley is underlain by the Fairbanks Schist. A thin layer of residual and transported soils overlay the upper valley slopes, and the mapping also indicates retransported loess and reworked creek gravels within the footprint of the BCHLF along the valley bottom. The drilling program confirmed the presence of the Fairbanks Schist throughout the vicinity of the BCHLF along with limited thicknesses of native soils.
- The Fairbanks Schist includes a wide variety of metamorphic rocks including quartz muscovite schist, quartzite, quartzite grit, marble, chlorite schist, amphibolite, and magnetite-rich biotite schist. The Fairbanks Schist is composed of nearly 90 percent quartzite and quartz muscovite schist. The RQD and fracture frequency data contained in the rock coring logs and the core box photographs both attest to the variable, but generally well foliated and highly fractured, nature of the Fairbanks Schist (i.e. the Fairbanks Schist typically exhibits low RQDs and high fracture frequencies). Nevertheless, it must be understood that, while the schist bedrock is fractured and hence pervious within certain intervals, the formation comprises interlocked blocks of competent bedrock that would form a sound foundation for the proposed BCHLF.
- Construction records indicated, and investigations confirmed, that the conveyor causeway embankment was constructed of mine waste rock that was placed and compacted as a competent engineered fill on an appropriately prepared bedrock and dense alluvium foundation. As such, the conveyor causeway should exhibit adequate strength and stiffness to help serve as the in-heap pond embankment for the proposed BCHLF.
- In-situ and laboratory permeability testing of the conveyor causeway fill and underlying foundation soils yielded results that varied with the nature of the soils involved and the variable depositional environment in which they were emplaced. Nevertheless, there are interbedded layers of higher conductivity material that should adequately convey groundwater underdrain flows downstream from beneath the BCHLF liner system through and/or below the conveyor causeway embankment toward the TSF. The groundwater

drain system from beneath the BCHLF (the transverse underdrain random fill trench, see Section 6.4) will be configured to distribute its flows along the upstream toe of the conveyor causeway embankment. Drilled-in drainage wells will be constructed in the base of the transverse underdrain random fill trench to maximize the opportunity for the groundwater flows to intersect one of the higher conductivity layers.

- The borings at the upstream toe of the causeway, and at the center of the low-grade ore stockpile, located the causeway excavation spoil and native soils that were left in the Barnes Creek valley and that will remain under the proposed BCHLF. Because of the long-term preloading provided by the weight of the Barnes Creek low-grade ore stockpile, these materials do not represent a layer of compressible material that could result in excessive settlement under the proposed BCHLF or a laterally continuous layer of weak material that could adversely impact the slope stability of the facility. The regraded Barnes Creek valley comprising the remnants of the low-grade ore stockpile (underlain by the excavation spoil and native soils) exhibits adequate strength and stiffness to serve as the foundation for the proposed BCHLF.
- Groundwater was encountered in the alluvium at the bottom of the Barnes Creek valley with a gradient indicating flow down the Barnes Creek valley toward the TSF. Construction may need to address the possible presence of shallow groundwater in the vicinity of the low point in the BCHLF construction (i.e. the bottom of the LCRS sump beneath the in-heap pond). There is also fairly shallow bedrock on the southwest side of the original Barnes Creek valley in the vicinity of the LCRS sump.
- The saturated hydraulic conductivity of the leach ore is expected to remain well above the leach application rate of 0.005 gallons per minute (gpm) per square-foot ($ft^2$) assuming vertical infiltration at unit hydraulic gradient. This indicates that the heap is expected to remain freely drained throughout its operation.
- The minus 2-inch and minus 1/2-inch drain aggregate maintain significant hydraulic capacity even under large confining stresses; thus, those material sources should continue to provide adequate borrow for the solution collection system and groundwater underdrains.
- By increasing compaction to 93 percent of the modified Proctor MDD and compacting wet of the modified Proctor OMC, the prepared subbase borrow material assessed in the BCHLF geotechnical investigation program meets the desired maximum saturated hydraulic conductivity of $1.0x10^{-5}$ cm/sec.
- A detailed and complete set of geotechnical material properties were developed for subsequent limit equilibrium slope stability analyses and geomechanical finite element settlement analyses of the proposed BCHLF.

## 4.14 Geotechnical Recommendations and Construction Material Assumptions

The following material descriptions are based on the BCHLF Technical Specification for Earthworks (Knight Piésold, 2017b). The reader is advised to refer to that document for additional information on the materials listed herein.

### 4.14.1 Additional Geotechnical Investigations

The geotechnical investigations completed to date have quantified the nature and expected performance of the various materials that will comprise the BCHLF; however, investigations have not been sufficiently detailed to identify available quantities of the various materials that will be borrowed locally, especially the prepared subbase. As part of the preparation for each subsequent stage of construction, it is recommended that FGMI conduct additional geotechnical investigations to establish sufficient quantity and quality of prepared subbase for the work. These investigations should feed into the planned on-going testing program, as discussed in Section 4.10.1, that shall be utilized to demonstrate that acceptable geomembrane versus prepared subbase interface shear strength is achieved. It is recommended that additional prepared subbase sampling and interface testing be conducted prior to the Stage 1 BCHLF construction.

### 4.14.2 Excavation

In general, excavation of the remnant stockpiled materials, overburden soils, and any underlying weathered bedrock needed to shape and prepare the foundation for the BCHLF, is expected to be possible with conventional heavy excavation equipment.

It is expected that, once the low-grade ore stockpile is removed to the planned base of the BCHLF, some seeps and springs will be exposed on the foundation slopes. These will be intercepted and conveyed beneath the BCHLF in groundwater underdrains that will convey the flows to the base of the in-heap storage pond embankment and conveyor causeway.

As the BCHLF development continues up the valley, additional seeps are anticipated to be encountered and will be intercepted by the groundwater underdrains. Surface runoff from upstream catchment areas will typically be diverted around the facility.

### 4.14.3 Unsuitable Materials

Unsuitable materials such as topsoil, organic matter, permafrost, potential wet soils from the drainage bottoms, or debris will be removed and wasted where exposed by the excavation of the low-grade ore stockpile or the grading of the BCHLF basin. Similar materials that have been well-consolidated under the pre-load comprising the weight of low-grade ore stockpile (but are not exposed by the excavation or grading) may be left in place without adverse impact on the anticipated performance of the planned facility. Lenses of permafrost, if encountered, will be removed and replaced with site grading fill if the permafrost is judged to have sufficient ice content to affect construction.

### 4.14.4 Rough Grade Surface

The rough grade surface will generally be the surface exposed following removal of topsoil and unsuitable material. In some cases, additional excavation within the pad footprint area will be required to establish the rough grade surface. rough grade surface will generally underlie the leach pad underdrain random fill, underdrains, PCMS channels, site grading fill, prepared subbase, and other components directly adjacent to the leach pad (i.e. base of pad perimeter roads, channels, spillway, etc.).

The rough grade surface shall consist of soils or rock exposed in the bottom of required excavations and beneath stripped areas. There is no specific gradation requirement for the soils or rock exposed. The finished surface shall be generally smooth and suitable for the planned construction.

### 4.14.5 Site Grading Fill and Road Random Fill

Site grading fill will be used on a variety of locations, including shaping the heap leach pad basin. Site grading fill (or random fill) will be placed to backfill areas from which permafrost or unsuitable soils have been removed within the leach pad footprint. It will also be used to cover areas of exposed bedrock, to construct portions of the ultimate pad perimeter berms, and may be used for road construction (road random fill) and backfilling of excavations associated with culvert crossings.

Where contractor-based equipment is used to construct the larger zones of fill required to develop the BCHLF leach pad basin, site grading fill shall be used. Where larger mine haul trucks are used to construct the larger zones of fill, random fill may be used.

The on-site silty sands and gravel, weathered bedrock, and select mine waste rock are expected to be suitable for use as site grading fill and road random fill. The maximum particle size shall be no larger than two-thirds of the lift thickness, and not more than 50 percent of the material shall pass the No. 200 sieve.

Based on previous testing of the overburden materials at the site, an effective friction angle of 43 degrees should be achieved for compacted schistose mine waste rock at a compacted density equivalent to 95 percent of the modified Proctor MDD.

### 4.14.6 Underdrain Trench and Solution Extraction Well Drain Aggregate

Underdrain trench drain aggregate will be used in the groundwater underdrains (except the underdrain random fill trenches) to be constructed beneath the leach pad liner system.

Solution extraction well drain aggregate will be used in the pervious, protective zones to be constructed around the vertical and inclined solution extraction wells.

The material for underdrain trench and solution extraction well drain aggregate shall be provided by FGMI as uncrushed mill reject. The material shall be hard, durable granitic rock that is free of organic matter and soft, friable particles.

### 4.14.7 Random Fill

Random fill will be used to construct the upstream face of the in-heap storage pond embankment and may also be used in areas of the leach pad basin construction requiring significant depths of fill.

Material for random fill will typically consist of mine waste rock from the open pit mine or stockpiles in the Barnes Creek valley. The random fill material may consist of granitic rock and/or schist. The gradation may vary, and the resulting fill shall generally be composed of gravel- to boulder-size rocks. The maximum particle size shall generally not exceed two-thirds of the lift thickness.

### 4.14.8 Underdrain Random Fill

Underdrain random fill will be used to construct the main trench drain beneath the BCHLF in-heap storage pond and beneath the upstream toe of the in-heap storage pond embankment. All underdrain random fill will be overlain by a layer of geotextile, as specified on the Drawings, and a 1-foot minimum compacted thickness of site-grading fill.

Underdrain random fill will consist of hard, durable granitic rock with minor amounts of schist. The gradation may vary, but in general, the resulting fill shall be granular and composed of gravel- to boulder-size rocks. The maximum particle size shall generally not exceed two-thirds of the lift thickness; however, larger size rock may be included if it will not adversely affect the overall intent of the underdrain random fill. The maximum amount of minus 4-inch material in the underdrain random fill shall be approximately 5 percent.

### 4.14.9 Prepared Subbase

Prepared subbase material will be used to form the low-permeability soil layer of the leach pad composite liner system that will lie directly beneath the leach pad geomembrane liner. Prepared subbase will be used on the interior faces of the ultimate pad perimeter berm, for the entire construction of all inter-stage perimeter berms, and to backfill portions of the Stage 1 geosynthetics termination trench to mitigate external waters from entering the LCRS.

The soils utilized to produce prepared subbase may vary in composition, but their suitability for use will be dependent on the final product's (post-processing) material gradation and the approval of the Engineer. Prepared Subbase shall be well-graded, contain only limited amounts of large particles, and shall be free of organic matter. Screening of the borrow material to produce suitable Prepared Subbase should be expected.

The gradation requirements for prepared subbase material are presented in Table 4.1.

**Table 4.1**
**Gradation Limits for Prepared Subbase**

| Sieve Size | Percent Passing |
|---|---|
| 2-inch | 100% |
| 1-inch | 70-100% |
| 3/8-inch | 55-100% |
| No. 4 | 48-90% |
| No. 40 | 32-65% |
| No. 200 | 25-45% |

4.14.10 Leachate Collection Recovery System Drain Aggregate

LCRS drain aggregate may be used to construct the in-heap storage pond LCRS for the portions of the pond with less than 10-percent slopes (i.e. the pond floor).

LCRS drain aggregate shall consist of minus 1/2-inch granitic crushed mill reject supplied by FGMI.

As designated on the Drawings, the pond floor LCRS may be constructed using overliner drain aggregate if the top surface of the LCRS aggregate layer is overlain by geonet to provide better uniform support of the overlying filter fabric (geotextile) and prepared subbase.

4.14.11 Overliner and Process Component Monitoring System Drain Aggregate

Overliner drain aggregate will be used to construct the overliner layer above the primary geomembrane liner throughout the leach pad basin. The overliner layer encapsulates the leach pad solution collection system pipework. Overliner drain aggregate will also be used to backfill the drilled-in drainage wells to be located beneath the lateral underdrain random fill trench drain.

As designated on the Drawings, overliner drain aggregate may also be used to construct the pond floor LCRS in lieu of LCRS drain aggregate if the top surface of the LCRS aggregate layer is overlain by geonet to provide better uniform support of the overlying filter fabric (geotextile) and prepared subbase.

PCMS drain aggregate will be used in the construction of the PCMS channels outside the limits of the in-heap storage pond.

Overliner and PCMS drain aggregate shall consist of minus 2-inch crushed mill reject material provided by FGMI. The material shall be hard, durable granitic rock that is free of organic matter and soft, friable particles.

4.14.12 Piezometer Sand

Piezometer Sand will be placed around the vibrating wire piezometers as indicated on the Drawings and shall consist of clean Ottawa Sand (or equivalent) with less than 3 percent passing the No. 200 sieve.

4.14.13 Termination Trench Backfill

Termination trench backfill will be used for infilling the geosynthetics anchor trenches as designated on the Drawings.

Termination trench backfill shall generally consist of site grading fill; however, the maximum particle size shall be no larger than 2 inches, and not more than 50 percent of the material shall pass the No. 200 sieve. Material similar to prepared subbase is ideal for this application, but there is no low-permeability

requirement for termination trench backfill. Screening of site grading fill may be required to achieve suitable termination trench backfill.

### 4.14.14 Spillway Foundation Fill

Spillway foundation fill will be used to develop the final grades and elevations of the in-heap storage pond emergency spillway prior to placement of the spillway reinforced concrete, and shall consist of minus 1/2-inch granitic crushed mill reject supplied by FGMI.

### 4.14.15 Riprap and Grouted Riprap

Riprap (and that used for grouted riprap) lining for diversion channels shall be granitic, hard, angular, durable, and reasonably well-graded rock free of overburden, organic, deleterious, or other objectionable material. Rounded rock, in general, is not acceptable. The unprocessed material will be sourced from on-site rock excavations or mine waste rock dumps.

### 4.14.16 Gabion Rockfill

Gabion rockfill will be used for backfilling of gabion baskets used for the construction of gabion walls for steepened fill slopes as shown on the Drawings.

Gabion rockfill shall be granitic rock from stockpile, or as indicated by FGMI, and shall be hard, durable, reasonably well graded, and free of overburden, organic, deleterious, or other objectionable material. FGMI will provide the unprocessed material from rock excavations or mine waste rock dumps. Gabion rockfill shall have a predominant rock size (approximately 85 percent by weight) of 4 to 12 inches.

### 4.14.17 Wearing Course

Wearing course will be used to surface un-paved roads, as directed by FGMI or designated on the Drawings, to promote improved traction on the travel surfaces. Wearing course shall consist of relatively sandy, well-graded gravel with some fines. The material may be produced as reject material from crushing and screening operations on-site, or it may be sourced from approved existing stockpiles or natural borrow areas.

### 4.14.18 Pipe Bedding Material

Pipe bedding material shall consist of fairly sandy, well-graded gravel with some fines. The material may be produced from crushing and screening operations on-site, or it may be sourced from approved existing stockpiles or natural borrow areas. Pipe bedding material will have a maximum liquid limit not greater than 35, and a plasticity index of between non-plastic and 12.

### 4.14.19 Pipe Backfill Material

Pipe backfill shall consist of on-site silty sands and gravels. Pipe backfill will contain less than 50 percent passing the No. 200 sieve, and the maximum particle size within 1 ft of the pipes (culverts) shall be 2 inches. In areas over 1 ft away from the pipes (culverts), the maximum particle size shall be no larger than two-thirds of the lift thickness.

# Section 5.0 - Project Design Criteria

The criteria presented herein is based on the criteria used for the BCHLF PFS (Knight Piésold, 2015b) and for the WCHLF Expansion final design (Knight Piésold, 2015a), but with slight modifications based on site-specific requirements of the BCHLF. A large portion of the developed criteria have been successfully incorporated in the design and construction of the WCHLF and are expected to translate well to the similar valley-fill BCHLF. Comments received from the ADNR on the BCHLF design have also been considered.

As part of the BCHLF final design, a "Result" column has been added to select design criteria tables, where deemed appropriate. In many instances, these items are not considered "design criteria", but rather the results of the design.

The source codes below may be referenced in the design criteria tables located on the following pages:

| Source Code | Source | Source Code | Source |
|---|---|---|---|
| KP | Knight Piésold | C | Calculated Value |
| FGMI | Fairbanks Gold Mining, Inc. | M | Measured Value |
| A | Assumed Value | SoA | State of Alaska |

The following tables and general design criteria categories are included:

- Table 5.1 – Dam Hazard Classification and General Criteria
- Table 5.2 – Leach Pad Capacity, Geometry, and Access Criteria
- Table 5.3 – In-Heap Storage Pond Criteria
- Table 5.4 – Leach Pad Component Systems Criteria
- Table 5.5 – Ore Properties Criteria
- Table 5.6 – Geotechnical Analyses Criteria
- Table 5.7 – Loading and Leaching Criteria
- Table 5.8 – Spillway, Surface Water Diversions, and Culverts Criteria

Applicable notes are located at the bottom of each table unless otherwise specified.

**Table 5.1**
**Dam Hazard Classification and General Criteria**

| Item | Design Criteria | Source |
|---|---|---|
| Dam Hazard Classification (11 AAC 93.157, **see Note 1**) | | |
| During operation | Class III (low) hazard potential | KP, FGMI |
| Post-closure | Class II (significant) hazard potential | KP, FGMI |
| General Earthwork Slopes (**see Note 2**) | | |
| Cut Slopes | 1.5H:1V (max) | KP |
| Fill Slopes | 2H:1V (max) | KP |

Notes: 1. The post-closure Dam Hazard Classification for the BCHLF is based on the classification adopted for the WCHLF dam (i.e. the classification of the Fort Knox TSF). To clarify, the BCHLF Dam is currently:
- Classified as Class III (3) for the operational period
- Designed to Dam Hazard Classification III (3) standards for the operational period
- Designed to Dam Hazard Classification II (2) standards for the post-closure period

The seismic design parameters for the OBE and MDE earthquakes for the operational and post closure - periods seismic design parameters selected for the BCHLF fall within in the middle-to-upper ranges of the values listed for Dam Hazard Classification III (3) and II (2) in the ADNR guidelines. The BCHLF in-heap pond embankment will not be considered a jurisdictional dam in the post-closure period.

2. The above noted cut and fill slopes, external to the leach pad basin, are followed as general maximum values (i.e. the steepest slopes allowed) unless site-specific conditions indicate otherwise or slope reinforcement is incorporated. Criteria specific to the leach pad basin are described in Table 5.2.

**Table 5.2**
**Leach Pad Capacity, Geometry, and Access Criteria**

| Item | Design Criteria | Result | Source |
|---|---|---|---|
| Leach Pad | | | |
| Storage capacity | To be maximized | 124.4 M $yd^3$<br>210.4 Mt | M |
| Lined pad area | Based on available area | 12.58 M $ft^2$<br>288.82 ac | M |
| Basin slope | 2.5H:1V (max) | | KP |
| Ore Heap | | | |
| Height (toe to crest) | Based on pad area and geometry | 800 ft | M |
| Overall slope | 3H:1V (max) | | KP |
| Individual lift slope | 1.3H:1V (angle of repose) | | FGMI |
| Nominal lift height | 50 ft (settled) | | FGMI |
| Initial lift height within in-heap storage pond | Based on embankment and freeboard | ~40 ft | M |
| Inter-lift bench width | 85 ft (typ) (to maintain overall heap slope) | | C |
| Thickness | 500 ft (max) (vertical above pad liner system) | | KP, FGMI |
| Setback | 36.5 ft (min) (from pad perimeter berm crest to toe of heap) | | C |
| Pad Perimeter Road | | | |
| Running width | 16 ft (min) | | FGMI |
| Grade | 25 percent (max) | | FGMI |
| Cross-fall | 2 percent (typ) away from the leach pad | | KP |
| Pad berm height | 3 ft (typ) | | KP |
| Pad berm side-slopes | 2.5H:1V (max) interior to the leach pad basin<br>1.5H:1V (max) exterior to the leach pad basin | | KP |
| Inter-Stage Perimeter Road | | | |
| Running width | 16 ft (min) | | KP, FGMI |
| Grade | 1 percent (min) | | KP, FGMI |
| Cross-fall | 2 percent (typ) away from the leach pad (typ) | | KP |
| Pad berm height | 1.5 ft (typ) | | KP |
| Pad berm side-slopes | 2.5H:1V (max) interior to the leach pad basin<br>1.5H:1V (max) exterior to the leach pad basin | | KP |
| Safety berm height | 1.5 ft (typ) | | KP |
| Safety berm side-slopes | 1.5H:1V | | KP |
| On-Heap Haul Road | | | |
| Running width | 100 ft (min, two way) | | FGMI |
| Grade | 10 percent (max) | | FGMI |
| Turning radius | 60 ft (min) | | FGMI |
| Safety berm height | 6 ft (typ) | | FGMI |
| Safety berm side-slopes | 1.7H:1V (approximate based on 20-ft base) | | C |
| Ultimate Tailing Elevation in Fish Creek Drainage | 1556 fmsl | | KP, FGMI |

**Table 5.3**
**In-Heap Storage Pond Criteria**

| Item | Design Criteria | Result | Source |
|---|---|---|---|
| In-Heap Storage Pond Embankment | | | |
| Crest elevation | Based on storage requirements | 1545 fmsl | KP, FGMI |
| Slope | 2.5H:1V (upstream)<br>varies (downstream) (**see Note 1**) | | KP |
| Crest width | 50 ft (min)(safety berms, pipeline, and access road) | ~200 ft | KP, FGMI |
| Embankment Crest Access Road | | | |
| Running width | 25 ft (min) | | KP, FGMI |
| Cross-fall | 2 percent (typ) away from the leach pad | | KP |
| Safety berm height | 3 ft (typ) (**see Note 2**) | | KP |
| Safety berm side-slopes | 1.5H:1V (max) (**see Note 2**) | | KP |
| In-Heap Storage Pond | | | |
| Base elevation | 1450 fmsl (low-point) | | FGMI |
| Minimum operating depth | 44 ft (**see Note 3**) | 28.6 M gal | KP, FGMI |
| *pump submergence* | *24 ft* | 24 ft | FGMI |
| *well drawdown contingency* | *~20 ft* | ~20 ft | KP, FGMI |
| Design storm event | | | |
| *Stage 1* | *100-yr/24-hr September rain-on-snow (4.0 in)* | 5.2 M gal | KP |
| *Stage 6* | *100-yr/24-hr September rain-on-snow (4.0 in)* | 31.2 M gal | KP |
| Emergency draindown | 24-hr duration (at 16,000 gpm application rate) | 23 M gal | KP |
| Freeboard | 5 ft (to remain consistent with WCHLF) | 13 M gal | FGMI |
| Capacity (designed) | Designed configuration | 124.2 M gal | C |
| Operational flexibility | (**see Note 4**) | | |
| *Stage 1* | | 54.3 M gal | C |
| *Stage 6* | | 28.3 M gal | C |
| Leachate collection and recovery system (LCRS) sump | Low-point in pond corner filled with drain aggregate and equipped with pump cans for evacuation of collected fluids (pumps, accessories, and electrics by others) | | |
| *depth* | *6 ft* | | KP |
| *side-slopes* | *2.5H:1V (max)* | | KP |

Notes:
1. As a modification to an existing structure, the overall crest width of the in-heap pond embankment is significantly greater than required by the design criteria. The low-grade ore has already been excavated from the downstream slope of the existing embankment; resulting in slopes varying from 1.3H:1V to 2H:1V. These slopes do not impact the integrity of containment within the in-heap storage pond or the integrity of the conveyor alignment. The Fish Creek storm water detention basin, which will be utilized for the operational period of the BCHLF, is formed in-part by the downstream slope of the existing conveyor causeway embankment. Inspection and maintenance of the downstream slopes will be required for the life of the facility. As the storm water detention basin will not be required for the BCHLF post-closure, it is expected that the basin will be backfilled as part of the BCHLF closure activities.
2. The embankment crest access road will be confined by two berms. The pad perimeter berm (**see Table 5.2**) will be located on the upstream side of the access road, while the safety berm will separate the access road from the conveyor access corridor. The safety berm will be a rockfill berm to promote surface runoff through the berm and away from the leach pad.
3. The 24-ft pump submergence and 20-ft drawdown contingency were adopted from the WCHLF. These approximate values were confirmed through discussions with FGMI and FGMI's designated pump supplier.
4. The design storm event volume and operational flexibility are inversely related, and will fluctuate as the pad area is progressively expanded. The operational flexibility is also impacted by seasonal/probabilistic fluctuations resulting from the BCHLF process water balance.

**Table 5.4**
**Leach Pad Component Systems Criteria**

| Item | Design Criteria | Source |
|---|---|---|
| Groundwater Underdrain System | An underdrain random fill trench will be developed below the lowest portion of the BCHLF by excavating into the floor of the in-heap storage pond and backfilling with coarse drainage aggregate. This underdrain trench will be constructed to the upstream toe of the existing conveyor causeway. A second underdrain random fill trench will be constructed along the upstream toe of the causeway to allow collected groundwater flows to spread and percolate through the base of the causeway.<br>Main and lateral underdrain trenches will be excavated in the main topographic drainages, and at select locations up the valley side slopes. All underdrain trenches will connect to the underdrain random fill trenches, and collected flows will be monitored and conveyed downstream for pumping.<br>Drainage wells will be installed within the bottom of the transverse underdrain random fill trench to promote connectivity with what is expected to be interbedded layers of higher conductivity material beneath the conveyor causeway embankment. The drainage wells will be located primarily in the bottom of the valley and shall be drilled-in approximately 30 ft into the colluvial materials underlying the original (i.e. pre-conveyor causeway) ground surface.<br>Pumping of groundwater flows will typically be conducted using the existing pit dewatering wells (**see Note 1**) | KP, FGMI |
| Underdrain Monitoring Wells | Three underdrain monitoring wells will be installed in close-proximity to the existing conveyor causeway to allow monitoring of groundwater levels and sampling for assessment of water quality emanating from beneath the facility. The wells will be extended to approximately 5 to 10 ft into the bedrock surface beneath the valley bottom. | KP, FGMI |
| Process Component Monitoring System (PCMS) | Geomembrane-lined trenches backfilled with drainage aggregate encapsulated in geotextile. The PCMS trenches will be located beneath the pad liner system above the confines of the in-heap storage pond, and aligned beneath select main solution headers. The number of PCMS channels will be established based on assessment of the expected heap development and under leach area such that the under leach area will contribute flows to a minimum of two PCMS channels. Just above the in-heap pond boundary, the PCMS trenches will transition to solid HDPE pipes for conveyance of collected flows to the monitoring point for discharge into the northeast corner of the in-heap storage pond. | KP, FGMI |

**Table 5.4**
**Leach Pad Component Systems Criteria**

| Item | Design Criteria | Source |
|---|---|---|
| Leach Pad Liner System (within the in-heap storage pond limits) | From bottom to top:<br>• 12-inch-thick prepared subbase ($k < 1\times10^{-5}$ cm/sec)<br>• 80-mil DST LLDPE secondary geomembrane ($k < 1\times10^{-11}$ cm/sec) (**see Note 2**)<br>• LCRS<br>– LCRS sump (**see Table 5.3**)<br>– Pond slopes < 10 percent (pond floor)<br>• 3-ft-thick LCRS gravel layer with perforated pipes reporting to LCRS sump<br>• 8 oz/sy non-woven geotextile<br>• 12-inch-thick prepared subbase ($k < 1\times10^{-5}$ cm/sec) (**see Note 3**)<br>– Pond slopes > 10 percent<br>• Tri-planar geonet (**see Note 4**)<br>• 80-mil DST LLDPE primary geomembrane ($k < 1\times10^{-11}$ cm/sec) (**see Note 2**)<br>• 36-inch-thick overliner drainage blanket (aggregate) encapsulating solution collection pipework (36-inch minimum cover over collection headers > 12-inch-dia) | KP, FGMI |
| Leach Pad Liner System (outside the in-heap storage pond limits) | From bottom to top:<br>• 12-inch-thick prepared subbase ($k < 1\times10^{-5}$ cm/sec)<br>• 80-mil DST LLDPE primary geomembrane ($k < 1\times10^{-11}$ cm/sec) (**see Note 2**)<br>• 36-inch-thick overliner drainage blanket (aggregate) encapsulating solution collection pipework (36-inch minimum cover over collection headers > 12-inch-dia) | KP, FGMI |
| Solution Collection Pipework System | Series of perforated CPT (Type SP) collectors and headers (12- and 24-inch-dia) fed by 4-inch-dia laterals. The solution collection pipework will be encapsulated within the overliner drainage blanket for protection from heap loading. The pipes will convey collected solution to the in-heap storage pond and solution extraction wells. The solution collection headers will be plumbed into the bases of each vertical solution extraction well.<br>The combined pipework and overliner system will limit the maximum hydraulic head acting above the leach pad composite liner to 1 ft (outside the limit of the in-heap storage pond). | KP |
| Overliner drain aggregate permeability | $1.0\times10^{-1}$ cm/sec (**see Note 5**) | KP |

**Table 5.4**
**Leach Pad Component Systems Criteria**

| Item | Design Criteria | Source |
|---|---|---|
| Vertical Solution Extraction Wells | Five 36-inch-dia vertical in-heap wells, each equipped with a 4,000 gpm pump and accessories, for long-term process solution recovery. The bases of the wells will be interconnected and plumbed to the perforated CPT solution collection headers. The vertical wells will be progressively extended with the heap development and are planned to daylight near the Lift 4 crest (EL 1640 fmsl). The vertical wells will not be utilized until installation is fully completed. Up to four of the vertical wells are expected to be operated at any given time to achieve the planned 16,000 gpm total solution flow. | KP, FGMI |
| Inclined Solution Extraction Wells | Three 30-inch-dia inclined in-heap wells, each equipped with a 4,000 gpm pump and accessories, for short-term process solution recovery. These wells will reside directly above the leach pad overliner drainage blanket and will be operated for the interim period until the vertical extraction wells are completed. The bases of the inclined wells will be interconnected and plumbed to two sections of perforated CPT pipe along the toes of the in-heap pond to improve hydraulic performance. Once the permanent vertical wells are commissioned, the inclined wells will not typically be operated. | KP, FGMI |

Notes:
1. All groundwater flows will be impounded (subsurface) in the Fish Creek stockpile area and migrate toward the open pit. The pit dewatering wells are supported by designated backup power supply as a priority in FGMI's operation plan.
2. All 80-mil double-side textured LLDPE geomembrane shall be GSE or Solmax standard product or Engineer approved equivalent.
3. A 12-inch-thick layer of prepared subbase will be placed over the LCRS gravel layer (separated by a non-woven geotextile) to mitigate seepage into the LCRS and to serve as a cushioning layer such that the leach pad primary geomembrane is not sandwiched between two layers of aggregate (i.e. LCRS gravel below and overliner aggregate above).
4. Due to constructability concerns, the LCRS sand layer incuded in the BCHLF PFS was replaced with a tri-planar geonet on the in-heap pond slopes greater than 10 percent (similar to the WCHLF). Based on laboratory testing, geonet alone provides a higher interface shear strength than geocomposite (i.e. geonet faced with geotextile) and was selected for the design. The tri-planar geonet shall be GSE TenDrain 300 or Engineer approved equivalent.
5. Based on testing conducted for the BCHLF in 2016 (Knight Piésold, 2017a), the overliner drain aggregate (minus 2-inch crushed mill reject) is expected to achieve an average permeability of $3.1 \times 10^{-1}$ cm/sec.

**Table 5.5**
**Ore Properties Criteria**

| Item | Design Criteria | | Result | Source |
|---|---|---|---|---|
| Average In-Place Dry Density | 123.0 pcf (**see Note 1**) | | 125.3 pcf | KP, FGMI |
| Permeability (**see Note 2**) | $1.8\times10^{-1}$ cm/sec (estimated) | | | KP, FGMI |
| Specific Gravity (**see Note 2**) | 2.68 | | 2.685 | KP, FGMI |
| Gradation (**see Note 2**) | Sieve Size | Percent Finer | | FGMI |
| | 12-inch | 91 to 98 | | |
| | 6-inch | 71 to 82 | | |
| | 3-inch | 51.4 to 80 | | |
| | 1-inch | 34.3 to 66.1 | | |
| | 0.75-inch | 26.9 to 62.6 | | |
| | 0.375-inch | 17.8 to 52.2 | | |
| | No. 4 | 12.5 to 43.5 | | |
| | No. 10 | 8.7 to 34.8 | | |
| | No. 60 | 3.4 to 15.7 | | |
| | No. 200 | 1.8 to 7.9 | | |
| Gravimetric Moisture Content (**see Note 3**) | | | | |
| Natural | 2.0 | | 2.6 | KP, FGMI |
| Under Leach | 8.0 | | 5.4 | KP, FGMI |
| Residual | 6.4 | | 4.0 | KP, FGMI |
| Porosity (**see Note 4**) | Varies from 36 percent under low load to 20 percent under 500-ft load (the average porosity of the ore within the WCHLF in-heap storage pond was estimated to be 23 percent) | | 21.8 | C |

Notes:
1. An average in-place dry density of 123.0 pcf was considered in the 2011 BCHLF proof of concept study and was retained for the PFS. Testing conducted by Knight Piésold for the BCHLF Final Design resulted in an estimated average settled dry density of 125.3 pcf.
2. The design criteria ore material properties were adopted from work conducted for the WCHLF. The result values are based on the test results from the 2016 BCHLF geotechnical investigations, where applicable.
3. The "Result" ore moisture contents (natural, under leach, and residual) were used in the BCHLF design.
4. The "Result" ore porosity is the average porosity within the BCHLF in-heap pond, and was calculated based on the site specific loading conditions of the BCHLF and work conducted for the WCHLF (Knight Piésold, 2017a).

**Table 5.6**
**Geotechnical Analyses Criteria**

| Item | Design Criteria | Result | Source |
|---|---|---|---|
| Static Factors of Safety (**see Note 1**) | | | |
| Failure through heap only | 1.3 (min) | | KP |
| Failure through heap and in-heap pond embankment | 1.5 (min) | | KP |
| Seismic-Induced Deformations | acceptably low deformations to mitigate damage to pad liner system (**see Note 2**) | varies | KP |
| Peak Horizontal Ground Acceleration (PHGA, **see Note 3**) | | | |
| OBE (during operation) | 108 yr recurrence interval (50 percent chance of exceedance in 75-yr period) | 0.08 g | KP |
| MDE (during operation) | 975 yr recurrence interval (5 percent chance of exceedance in 50-yr period) | 0.25 g | KP |
| MDE (post-closure) | 2,475 yr recurrence interval (2 percent chance of exceedance in 50-yr period) | 0.37 g | KP |
| Phreatic Surface within In-Heap Pond (normal) | | | |
| Operational Condition | 1540 fmsl (avg) | | KP |
| Post-closure Condition | 1560 fmsl (estimated) | | KP, A |
| Ultimate Tailing Elevation in Fish Creek Drainage | 1556 fmsl (**see Note 4**) | | KP, FGMI |
| Geomembrane Tensile Properties (for settlement analyses) | | | |
| Maximum Allowable Multiaxial Strain | 8% (Peggs, et al., 2005) | | KP and ADNR |

Notes:
1. The static factors of safety (FS) proposed for the BCHLF have been used for the WCHLF and many other projects Knight Piésold has been involved on worldwide. The FS of 1.5 is a commonly accepted value (recommended by the Canadian Dam Association and others) for long-term water impounding structures. The FS of 1.3, for failures through the heap alone, is another commonly accepted industry standard (recommended by the State of Nevada).
2. Seismic-induced deformations that would affect the pad liner system are considered based on the OBE and MDE only during operation of the facility. Seismic-induced deformations due to the MDE post-closure are not deemed pertinent because the pad liner system will be penetrated during closure and damage to the liner system will be inconsequential.
3. Based on the updated seismic hazard assessment discussed in Section 2.5. To clarify, the BCHLF Dam is currently:
   - Classified as Class III (3) for the operational period
   - Designed to Dam Hazard Classification III (3) standards for the operational period
   - Designed to Dam Hazard Classification II (2) standards for the post-closure period

   The seismic design parameters for the OBE and MDE earthquakes for the operational and post closure - periods seismic design parameters selected for the BCHLF fall within in the middle-to-upper ranges of the values listed for Dam Hazard Classification III (3) and II (2) in the ADNR guidelines. The BCHLF in-heap pond embankment will not be considered a jurisdictional dam in the post-closure period.
4. The ultimate tailing elevation in the Fish Creek drainage, based on the April 2016 TSF deposition plan, was used to estimate the potential post-closure phreatic condition within the BCHLF.

**Table 5.7**
**Loading and Leaching Criteria**

| Item | Design Criteria | Source |
|---|---|---|
| Expected Dates of Operation | | |
| Start of Loading | September 1, 2019 (estimated) | KP, FGMI |
| End of Loading | July 17, 2027 (estimated) | KP, FGMI |
| End of Leaching (final ore) | TBD | FGMI |
| Final Rinsing and Draindown | TBD | FGMI |
| Loading Rate (**see Note 1**) | 123,000 tpd (Apr though Nov, 244 days/year, 30 M tpa) | FGMI |
| Leach Cycle | 60 days | FGMI |
| Solution Application Rate | 0.005 gpm/ft$^2$ | FGMI |
| Total Solution Flow to Heap | | |
| General design | 16,000 gpm (**see Note 2**) | FGMI |
| Solution header design | 24,000 gpm (**see Note 2**) | FGMI |
| Area Under Leach | 3.2 M ft$^2$ | C |
| Method of Application | Drip emitters (buried in winter) | FGMI, KP |

Notes: 1. The loading and leaching criteria were used to develop the operational period process water balance discussed in Section 8.0. Based on the BCHLF geometry and estimated times required for construction, as discussed in Section 6.12, the heap loading rate will vary for the first few years of operation.

2. To allow flexibility for potential future expansion of the facility, the hydraulic design of the solution collection headers included a minimum factor of safety of 1.5 in their ability to accommodate a maximum total solution flow of 24,000 gpm to the heap. All other applicable elements of the BCHLF were designed with the currently planned 16,000 gpm total solution flow.

**Table 5.8**
**Spillway, Surface Water Diversions, and Culverts Criteria**

| Item | Design Criteria | Result | Source |
|---|---|---|---|
| In-Heap Storage Pond Emergency Spillway | | | |
| Weir design flow | 100-yr/24-hr January rain-on-snow peak flow from all contributing upstream watersheds within the final (Stage 6) pad footprint, with 1 ft of freeboard | | KP |
| Weir erosion protection | Reinforced concrete | | KP |
| Outlet channel design flow | 100-yr/24-hr January rain-on-snow peak flow from the worst case combined discarge of the attenuated spillway plus the north ultimate perimeter diversion channel, with 1 ft of freeboard (assessed for the Stage 1 and Stage 6 configurations) | | KP |
| Outlet channel erosion protection | Grouted riprap, sized based the design capacity peak flow velocity | | KP |
| Permanent Diversion Channels (**see Note 1**) | | | |
| Capacity | The larger of the 100-yr/24-hr January rain-on-snow peak flow from:<br>• all contributing upstream watersheds with no freeboard, or<br>• contributing upstream watersheds external to the final pad footprint, with 1 ft of freeboard | | KP |
| Erosion protection | Riprap and/or grouted riprap, sized based on the larger peak flow velocity from:<br>• the 10-yr/24-hr January rain-on-snow event acting on all contributing upstream watersheds, or<br>• the 100-yr/24-hr January rain-on-snow event acting on contributing upstream watersheds external to the final pad footprint | | KP |
| Temporary Diversion Channels (inter-stage perimeter roads) (**see Note 2**) | | | |
| Capacity | 100-yr/24-hr January rain-on-snow peak flow from all contributing upstream watersheds<br>Flow conveyed along the road running-width with up to 1 ft of flow depth adjacent to the geomembrane-lined road safety berm | | KP, FGMI |
| Erosion protection | Unlined road surface comprised of weathered schist/ gravel, some cobbles, and colloidal silts (consistent with WCHLF) | | FGMI |
| General Criteria (diversion channels and spillway outlet channel) | | | |
| Side-slopes | 2H:1V (typ)<br>1.5H:1V (max) when lined with grouted riprap<br>5H:1V (max) for drive-thru crossings | | KP |
| Freeboard | 1 ft (min) unless otherwise noted | | KP |
| Grade | minus 0.5 percent (min) | | KP |
| Culvert Crossings | | | |
| Design Storm | 100-yr/24-hr peak flow (permanent culverts upstream of major facilities) | | KP |
| Freeboard | 1 ft (min) | | KP |

Notes: 1. Permanent diversion channels are considered as those with an operational life greater than 3 years or that will be left in place post-closure.
2. Temporary diversion channels are considered as those with an operational life less than 3 years. The BCHLF inter-stage perimeter roads for the BCHLF will be designed to provide temporary diversion of surface water runoff from upstream catchments.

# Section 6.0 - Leach Pad Design

## 6.1 General

Based on Knight Piésold's experience with the design, construction, and historical performance of the WCHLF (which has performed up to, or better than, the original design expectations) the BCHLF final design has been completed using a similar approach.

At the planned final (Stage 6) configuration, the BCHLF ore heap will reach a maximum height of approximately 800 ft from toe to crest. As with the WCHLF, the maximum vertical ore thickness above the BCHLF pad liner system will be limited to 500 ft. Potential increases to the maximum ore thickness may be considered in the future, but will be subject to appropriate testing, analyses, and approval by the ADNR.

## 6.2 Grading Plan and Foundation Preparation

The site of the planned BCHLF is predominantly disturbed. In recent past, it has been covered with waste dumps, stockpiled low-grade ore, and haul roads. Small pockets of forested areas, with a variety of ground cover, remain on the upper northeast valley side-slope and in the far western reaches of the valley. The exposed native ground slopes on the northeast side of the valley are moderate, ranging from 4H:1V to 5H:1V on average. Based on the BCHLF site investigations, the undisturbed areas contain approximately 1 ft of surficial topsoil underlain by 2 to 15 ft of dense, silty sands and gravels that have resulted from weathering of the schist bedrock. These materials are typically underlain by less weathered, hard, competent schist.

Most of the stockpiled low-grade ore within the Barnes Creek valley will be removed to facilitate construction of the BCHLF; however, small, varying thicknesses of the stockpile material will be left in place and form portions of the BCHLF foundation. As discussed in Section 4.0, testing of the remnant stockpile, waste dump, and underlying native ground materials demonstrates them to be heavily consolidated and suitable for the BCHLF foundation. Significant additional settlement of the materials is not anticipated.

Relatively small and localized pockets of permafrost were encountered in the bottom of the Walter Creek valley during construction prepared for the WCHLF. Similar permafrost conditions were likely originally in place within the Barnes Creek valley; however, the heavy disturbance of the valley floor associated with placing the low-grade stockpile and waste dumps can be expected to have caused any small zones of ice to melt. No ice was encountered during site investigations conducted for the BCHLF. In the unlikely event permafrost is encountered while preparing the BCHLF site, it will be removed and replaced with appropriate fill material.

FGMI's planned excavation surface is shown in plan on Drawing 200. Although the planned excavation of the low-grade ore includes benching with steep temporary slopes, the excavation surface will be regraded to produce more uniform slopes that will range from approximately 2.5H:1V to 5H:1V on the northeast side of the valley, and from approximately 1.4H:1V to 4H:1V on the northwest to southwest sides. The steeper areas of the planned excavation surface are generally where the low-grade ore was stockpiled against previously placed waste dump and haul road fills.

Based on the BCHLF geotechnical investigations discussed in Section 4.0, groundwater levels within the valley bottom appear to be typically lower than the planned excavation levels. However, it is expected that localized excavations beneath the in-heap storage pond will encounter perched groundwater levels that will require dewatering efforts during construction. It is also expected that these localized areas of perched groundwater will be sufficiently lowered with the installation of the planned underdrain system discussed in Section 6.4.

Consistent with the WCHLF pad construction to date, the following foundation preparation activities will be conducted, where applicable, for development of the BCHLF:

- Trees will be removed from the foundation area (Stages 5 and 6 on the northern valley side-slopes)
- The foundation area will be cleared and grubbed (Stages 5 and 6 on the northern valley side-slopes)
- All topsoil, zones of permafrost, and highly organic or soft soils will be removed to stockpiles and replaced with appropriate fill material (where applicable)
- The native ground silty sands and gravels will be conformed to meet the required pad foundation grades (where applicable)
- Following the planned excavation of the low-grade ore stockpile, any encountered zones of material that are questionable in consolidation and/or strength shall be removed and replaced with appropriate fill material per the BCHLF CQA/QC Manual (Knight Piésold, 2017d) and the Earthwork Specifications (Knight Piésold, 2017b)
- Any rock outcrops will be leveled, smoothed, and covered with a minimum 3-ft-thickness of cover soil
- The leach pad basin will be regraded such that the resulting slopes, that will not receive additional fill, are 2.5H:1V or flatter to facilitate liner construction

The BCHLF will extend up the valley from the existing Barnes Creek conveyor causeway, and has been designed with six distinct stages (Stages 1 through 6). The maximum relief across the BCHLF footprint is expected to be on the order of 720 ft, from the base of the in-heap storage pond (1450 fmsl) to the upper perimeter of the Stage 6 pad area (2170 fmsl).

The overall contributing catchment area to the BCHLF is roughly 760 acres (2-dimensional); however, a significant portion of this is upstream of the current and future waste dump within the valley and will not produce surface runoff to the BCHLF. Approximately 516 acres of the overall upstream catchment is expected to contribute surface runoff to the BCHLF, and at its final (Stage 6) configuration the BCHLF will occupy about 290 acres of this contributing area. The approximate pad areas associated with the staged development of the BCHLF are presented in Table 6.1.

**Table 6.1**
**Barnes Creek Heap Leach Facility Staged Pad Areas (2-dimensional)**

| Description | Stage | | | | | | Total |
|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | |
| Elevation (1) | 1545 | 1640 | 1730 | 1820 | 1950 | 2165 | |
| Area (M ft$^2$) | | | | | | | |
| Incremental | 2.08 | 2.10 | 2.02 | 2.00 | 2.47 | 1.91 | 12.58 |
| Cumulative | 2.08 | 4.19 | 6.20 | 8.20 | 10.67 | 12.58 | |
| Area (acres) | | | | | | | |
| Incremental | 47.83 | 48.32 | 46.26 | 45.93 | 56.72 | 43.76 | 288.82 |
| Cumulative | 47.83 | 96.15 | 142.41 | 188.33 | 245.06 | 288.82 | |

Notes: 1. The stage elevations represent the approximate low points at the northeastern end of each respective stage perimeter road (i.e. the road along the upper edge of each stage's perimeter).

The BCHLF general arrangement is presented on Drawing 140 (and on Drawings 400 and 410 for the Stages 1 and 6 grading plans, respectively), while typical grading sections through the pad basin area are presented on Drawing 420. It should be noted that the staged delineations have been developed to support the design of surface water management structures (i.e. channels and culverts) and to establish guidelines

for maintaining available storage capacity within the in-heap pond. The staged delineations have also been designed to approximate 2 M $ft^2$ areas that are thought to be reasonable for annual liner production at the site. Construction deviations from the exact staged delineations presented herein, do not necessarily constitute a contradiction of the design intent.

## 6.3 Base Platform

A traditional base platform will not be constructed for the BCHLF. As with the WCHLF, the BCHLF in-heap storage pond will be developed to allow for initial ore leaching in a submerged "vat-like" manner, therefore a traditional base platform is not required to provide initial leaching area. It is currently planned that the Stage 1 facility will be constructed by the end of August 2019 to allow loading and vat leaching of the initial ore over the following winter months.

## 6.4 Groundwater Underdrains

Based on the BCHLF geotechnical investigations discussed in Section 4.13, groundwater levels within the valley bottom appear to be typically lower than the excavation levels required to develop the facility. However, it is expected that localized excavations beneath the in-heap storage pond will encounter perched groundwater levels and require dewatering efforts during construction. An underdrain system will be installed beneath the BCHLF to intercept groundwater encountered in the valley and convey it to the base of the existing conveyor causeway (which will form the core of the BCHLF in-heap pond embankment) for percolation under that structure.

The disturbed and covered nature of the valley, as shown on Figure 6.1, have not allowed specific assessment of groundwater flow conditions beneath the planned facility footprint. FGMI has indicated the pit dewatering wells downstream of the planned BCHLF typically operate with a combined rate of up to 1,800 gpm. While this pumping rate includes both groundwater flows from the Barnes Creek valley and seepage from the downstream TSF, it has been adopted as a probable maximum groundwater flow for the BCHLF design. Comparatively, the maximum flow measured as part of the original site characterization for the WCHLF was estimated as 1,495 gpm. This value was measured as surface water flow at the confluence of the three main drainages within the Walter Creek basin. The WCHLF was designed to intercept most of this flow and divert it around the upper perimeter of the facility; however, the underdrains for each of the three main drainages beneath the WCHLF were designed to pass a third of this flow, or approximately 500 gpm. For the BCHLF final design, more main underdrain trenches have been included outside the bottom of the valley (as shown on Drawing 200), and are spaced to allow flexibility for finger drains to be efficiently located to capture field-observed seeps. The final location of each underdrain may be adjusted in the field to better suit actual conditions during construction. The main underdrain trenches in the upper end of the valley have been sized to pass a minimum of 500 gpm each, while the main underdrains crossing beneath the flatter bottom of the BCHLF in-heap storage pond were sized with minimum capacities of 300 gpm. Reductions in flows captured and conveyed by the underdrains should be realized as the leach pad is progressively expanded and the areas contributing infiltration to groundwater decrease.

The groundwater management system for the BCHLF will consist of large underdrain trenches (termed underdrain random fill trenches) located in the valley bottom and along the upstream toe of the existing conveyor causeway fill. These large drain trenches will be fed by main underdrain trenches to be installed up the valley side-slopes. Additional finger underdrains will extend from the main underdrains to capture flows from isolated seeps and springs identified within the leach pad basin during construction. The underdrain system will intercept groundwater flows beneath the facility and route them to the upstream toe of the existing conveyor causeway where they will be spread within the transverse underdrain random fill trench to promote drainage beneath the base of the causeway fill. In-situ permeability testing conducted as part of the BCHLF geotechnical investigation program, see Section 4.3.4, suggests the causeway fill and interbedded layers of underlying higher-conductivity alluvial materials will adequately pass the estimated groundwater flows. Drilled-in drainage wells will be constructed in the base of the transverse underdrain random fill trench to maximize the opportunity for the groundwater flows to intersect one of the

higher conductivity layers. Downstream of the conveyor causeway, the groundwater flows will be intercepted by FGMI's dewatering wells beneath the Fish Creek stockpile area.

Three underdrain monitoring wells will be installed to approximately 5 to 10 ft into the bedrock surface beneath the valley bottom. Two of the planned monitoring wells are sited to be installed through the crest of the existing conveyor causeway. The third monitoring well will be located in the vicinity of the two currently planned wells; however, as discussed with the ADEC, the location of the third monitoring well will be determined prior to construction. The locations of the planned underdrain monitoring wells have been (or will be) established based on the pre-conveyor causeway valley bottom and based on interpretation of the local geology from geotechnical investigations in the area. All groundwater flows will typically be impounded (subsurface) in the Fish Creek stockpile area and migrate toward the open pit where they will be intercepted by the pit dewatering wells. The pit dewatering wells are supported by designated backup power supply as a priority in FGMI's operation plan. The proposed underdrain monitoring wells will allow for sampling of groundwater flows and measuring the water level through the in-heap pond embankment.

The underdrain system layout is shown on Drawing 200, the underdrain random fill trench profiles are provided on Drawing 202, and sections and details are presented on Drawings 206 through 235. The planned underdrain monitoring wells are shown in plan on Drawing 200 and in profile on Drawing 142. FGMI's existing wells in the vicinity of the Fish Creek stockpile area are shown on Drawings 140 and 560.

The underdrain random fill trenches have been designed as 6-ft-deep trapezoidal trenches with 20-ft bottom-widths and 1H:1V side-slopes. The average longitudinal slope along the excavated valley bottom is approximately 10 percent; however, a flatter 1.7-percent longitudinal grade will be developed beneath the floor of the in-heap storage pond. The primary underdrain random fill trench will be excavated into the valley bottom, beneath the planned pad grading surface, to the upstream toe of the existing conveyor causeway. A secondary, transverse underdrain random fill trench will be constructed along the upstream toe of the causeway to distribute the groundwater flows for conveyance through and beneath the conveyor causeway embankment. As noted in Section 4.3.4, interbedded layers of higher conductivity alluvial materials exist beneath the conveyor causeway fill. Seven (7) 24-inch-diameter drainage wells, backfilled with underdrain drain aggregate, will be completed to approximately 30 ft beneath the base of the transverse underdrain random fill trench to maximize the opportunity for the groundwater flows to intersect one of the higher conductivity layers.

The underdrain random fill trenches will be backfilled with material that will consist of hard, durable granitic rock with a minor amount of schist. In general, the underdrain random fill shall be granular and composed of gravel- to boulder-size rocks to provide a high level of flow capacity. Per the Earthwork Specifications (Knight Piésold, 2017b), the material in the uppermost 0.5 foot of the underdrain random fill shall not exceed 6 inches in size. The top surface of the underdrain random fill material will be overlain with a layer of 8-ounce per square-yard (oz/sy) non-woven geotextile followed by a 12-inch-thick layer of site grading fill. Per the Geosynthetics Specifications (Knight Piésold, 2017c), GSE 8-oz/sy non-woven geotextile meets or exceeds all requirements of GRI Test Method GT13(a) for Class 1 (High Survivability).

The main underdrains, located immediately upstream of the larger underdrain random fill trenches, are designed as large 3-ft-deep by 12-ft-wide trenches that will be constructed up the valley side-slopes and within the defined natural drainages throughout the leach pad basin. The main underdrains will be backfilled with pervious uncrushed mill reject and overlain with a layer of 8 oz/sy non-woven geotextile followed by a 12-inch-thick layer of site grading fill. Additional underdrain finger trenches, 3-ft-deep by 3-ft-wide, will be installed as required to intercept isolated groundwater seeps identified during construction within the pad footprint. For the purposes of construction MTO quantities, the additional finger trenches were assumed to be equal to approximately 15-percent of the main underdrain trench quantities.

The underdrain random fill trench and main underdrain flow capacities were analyzed based on the Leps equation for flow through rockfill (Leps, 1973). The basic equation assumes turbulent flow and can be expressed as follows:

$$V_v = W\, m^{0.5}\, i^{0.54}$$

Where: $V_v$ = the average velocity of water in the voids of the rockfill
$W$ = the empirical constant for a given rockfill material, depending primarily on the shape and roughness of the rock particles, and on the viscosity of water
$m$ = the hydraulic mean radius
$i$ = the hydraulic gradient

A practical determination of $m$ is reliable for clean, mono-sized rock but is very uncertain for a well-graded or nonhomogeneous rockfill. According to Wilkins and others (Leps, 1973), the empirical constant $W$, when expressed in inch-second units, varies from about 33 for crushed gravel to about 46 for polished marbles. The information shown in Table 6.2 was originally presented by Leps, and illustrates the variation of $m$ and $Wm^{0.5}$ based on rock size. The data presented is for mono-sized rock assuming a specific gravity of 2.87, a void ratio of 1.0, and a $W$ value of 33.

**Table 6.2**
**Typical Flow-through-Rockfill Data for Mono-sized Rock, from Leps**

| Rock size (inches) | $m$ (inches) | $m^{0.5}$ $(\text{inches})^{1/2}$ | $W\, m^{0.5}$ (inches/sec) |
|---|---|---|---|
| 3/4 | 0.09 | 0.30 | 10 |
| 2 | 0.24 | 0.49 | 16 |
| 6 | 0.75 | 0.87 | 28 |
| 8 | 0.96 | 0.98 | 32 |
| 24 | 3.11 | 1.76 | 58 |
| 48 | 6.43 | 2.54 | 84 |

From information originally supplied by FGMI, the uncrushed mill reject material was determined to have porosity of 0.26, and an average mean diameter ($D_{50}$) of 0.475 inches. Testing of minus 2-inch crushed mill reject (Knight Piésold, 2017a) resulted in an approximate 0.78-inch $D_{50}$. Based on the current BCHLF Earthwork Specifications, $D_{50}$ values of 1 inch and 8 inches were assumed for the underdrain drain aggregate and underdrain random fill, respectively. Based on larger grain size, the porosity of the underdrain drain aggregate and underdrain random fill will be greater than 26 percent; however, an average porosity of 26 percent has been maintained for both materials in the analyses presented herein. The values for $W\, m^{0.5}$ for the 1- and 8-inch $D_{50}$ material, based on a best-fit-line equation representing the data presented in Table 6.2, were estimated to be 11.32 and 32 inches/sec, respectively.

Along the valley bottom, the primary underdrain random fill trench considers both 1.7- and 10-percent slopes, while a minimum 5.2-percent hydraulic gradient was considered for the transverse underdrain random fill trench along the upstream toe of the conveyor causeway. Design of the main underdrain trenches consider slopes varying from 1.0 to 40 percent.

The flow capacities of the underdrain random fill and main underdrain trenches were determined for the various hydraulic gradients using $V_v$, from the Leps equation and the following formula:

$$Q = A\, V_v\, n$$

Where: $Q$ = the total volume of flow per unit of time
$A$ = the area of the flow cross-section
$V_v$ = the average velocity of water in the voids of the rockfill
$n$ = the porosity of the material through which the flow is being conveyed

Tables 6.3 and 6.4 present the estimated full-flow capacities of the underdrain random fill trenches and main underdrains, respectively, for the hydraulic gradients analyzed.

**Table 6.3**
**Estimated Full-Flow Capacities for Underdrain Random Fill Trenches at various Hydraulic Gradients**

| Hydraulic Gradient, *i* (%) | Area ($ft^2$) | Full Flow Capacity (gpm) |
|---|---|---|
| 1.7 | 156 | 5,377 |
| 5.2 | 156 | 9,835 |
| 10 | 156 | 14,000 |

Even at the shallow 1.7-percent grade, the estimated 5,377 gpm flow capacity of the primary underdrain random fill trench in comparison with the assumed maximum flowrate of 1,800 gpm demonstrates the trench should have sufficient capacity to accommodate groundwater flows beneath the planned facility. The longitudinal flow capacity of the transverse underdrain random fill trench along the upstream toe of the conveyor causeway is somewhat inconsequential as its primary function will be to facilitate lateral flows beneath the base of the existing conveyor causeway.

**Table 6.4**
**Estimated Full-Flow Capacities for Main Underdrain Trenches at various Hydraulic Gradients**

| Hydraulic Gradient, *i* (%) | Area ($ft^2$) | Full Flow Capacity (gpm) |
|---|---|---|
| 1 | 36 | 329 |
| 10 | 36 | 1,142 |
| 20 | 36 | 1,661 |
| 33 | 36 | 2,177 |
| 40 | 36 | 2,415 |

The design intent of the groundwater underdrains is to mitigate the build-up of hydraulic pressure beneath the BCHLF liner system by separating the groundwater flows from the base of the leach pad. Although intended to prevent the build-up of hydraulic pressure beneath the BCHLF liner system, the more specific intent is to promote drainage of groundwater flows beneath the facility and downstream. Piezometers are not required in the underdrains because there is no effect on the dam safety even if the groundwater level were to rise.

The underdrain system will also act as a global water quality monitoring system for the BCHLF. Generally, groundwater flow directions follow surface topography and, since the underdrains will be located in the main drainages beneath the leach pad, any minor amounts of process solution that pass through the pad liner system will be intercepted by the underdrain system and report to the underdrain monitoring well and the pit dewatering wells.

## 6.5 In-Heap Storage Pond Embankment

The BCHLF in-heap storage pond embankment will be formed using the existing conveyor causeway as its core element. The conveyor causeway separates the Barnes Creek low-grade ore stockpile (upstream of the causeway) from the Fish Creek stockpile area (downstream). Due to prior loading against the conveyor causeway by both the Barnes Creek and Fish Creek stockpiles, no significant future deformations of the causeway fill are expected as a result of the planned BCHLF development. The existing conveyor causeway is configured with a crest elevation of approximately 1545 fmsl and a nominal overall width of 200 to 250 ft.

FGMI intends to remove most of the low-grade ore from the Barnes Creek stockpile, on the upstream side of the existing conveyor causeway, for processing and to facilitate construction of the BCHLF. The Barnes

Creek stockpile will be excavated to an approximate low-point elevation of 1430 fmsl near the upstream toe of the existing conveyor causeway to allow construction of the underdrain random fill trenches discussed in Section 6.4. At its upstream toe, the original conveyor causeway foundation was developed with a low-point elevation of approximately 1412 fmsl. The stockpile material will be excavated to the approximate angle-of-repose slope of the existing conveyor causeway prior to conforming the area to the required grading of the BCHLF in-heap storage pond. As discussed in Section 7.1.7, in its interim condition the upstream face of the exposed conveyor causeway will have a potential for surficial sloughing. While this will not compromise the safety of the overall embankment, appropriate precautions and monitoring should be incorporated during construction. Notwithstanding this, it is recommended that cut slopes no steeper than 1.5H:1V be incorporated.

After completion of the planned Barnes Creek stockpile excavation, the exposed slope of the conveyor causeway embankment will be flattened to a 2.5H:1V slope with the placement of a wedge of site grading fill to form the upstream face of the BCHLF in-heap pond embankment. Random fill may be used if the material and placement methods are in accordance with the Earthwork Specifications. The in-heap storage pond embankment has been designed with a crest elevation of 1545 fmsl to match the approximate elevation of the existing causeway. The low-point within the in-heap storage pond will be constructed at approximately 1450 fmsl, in the southern corner of the pond. The overall crest width of the conveyor causeway embankment (over 200-ft wide) will be retained to support the existing conveyor that will remain in place and to provide access to the crest of the in-heap pond embankment (for access to the tops of the underdrain monitoring wells, inclined LCRS pump cans, and inclined solution extraction wells) and the emergency spillway.

On the downstream side of the existing causeway embankment, low-grade ore within the Fish Creek stockpile has been removed for processing, and the stockpile area has generally been backfilled with mine waste rock to an elevation higher than the existing conveyor causeway (i.e. to approximately 1610 fmsl). Immediately downstream of the conveyor causeway, the Fish Creek storm water detention basin has been formed between the causeway downstream slope and FGMI's backfill within the Fish Creek stockpile area. As discussed in Section 7.1.7, FGMI's excavation to angle-of-repose slopes on the downstream side of the conveyor causeway embankment will likely result in small bench-scale instabilities. FGMI's planned slopes will not compromise the integrity of the completed in-heap storage pond embankment or the existing conveyor. Safety precautions should be incorporated into future construction planning associated with the Fish Creek stockpile area.

## 6.6 Leach Pad Liner System

### 6.6.1 General Liner System

Based on the overall excellent performance of the WCHLF liner system, the BCHLF liner system has been designed to be similar. The system will be a composite liner consisting of an 80-mil DST LLDPE geomembrane installed directly over and in intimate contact with a 12-inch-thick prepared subbase soil layer that will have a hydraulic conductivity of less than $1\times10^{-5}$ cm/sec. The area under the in-heap storage pond will include a secondary 80-mil DST LLDPE geomembrane liner with an LCRS between the primary and secondary liners. The portion of the BCHLF outside the in-heap storage pond will be constructed with a PCMS in select locations beneath the pad composite liner system. All 80-mil DST LLDPE geomembrane shall be standard product as produced by GSE or Solmax, or the Engineer approved equivalent.

LLDPE geomembrane was selected for use because it is more flexible and compliant than high-density polyethylene (HDPE), giving it a slightly higher interface shear strength with the prepared subbase to improve overall heap stability. LLDPE geomembrane also provides the necessary chemical, ultraviolet (UV) light, and puncture resistances required for the facility. HDPE is typically used for facilities where long-term exposure to UV rays is expected; but the exposure time for the BCHLF geomembrane will typically be only on the order of two years or less. The small areas on top and exterior portions of the leach pad ultimate perimeter berm will be exposed for the life of the facility, but these areas will not be subject to solution flow.

A 36-inch-thick overliner (drain aggregate) layer containing a network of perforated pipes will be placed over the primary geomembrane liner to protect it from puncture by coarse ore particles and to provide a high-permeability layer to assist in (1) the removal of solution, and (2) the reduction of hydraulic head acting on the geomembrane liner. Vibrating wire piezometers, as discussed in Section 6.13, will be located within the leach pad overliner layer to monitor hydraulic head acting on the leach pad primary LLDPE liner.

The BCHLF general arrangement and the Stages 1 and 6 grading plans are presented on Drawings 140, 400, and 410, respectively, while the Stage 1 in-heap pond grading plan is presented on Drawing 401. The approximate delineations for Stages 3 through 5 are also presented on the Stage 6 grading plan (Drawing 410). Typical sections and details of the pad liner system are presented on Drawings 520 through 534.

### 6.6.2 In-Heap Storage Pond Liner System and LCRS

Beneath the in-heap storage pond, a double liner system including an LCRS will be used. The LCRS will serve as a monitoring and collection system for any process solution passing through the primary liner and will provide a hydraulic break between the primary and secondary 80-mil LLDPE liners. Thus, while the hydraulic head acting on the primary liner at the base of the in-heap pond will be the depth of process solution, the head on the secondary liner will be significantly less.

The in-heap storage pond LCRS plan is presented on Drawing 300, while typical sections and details are presented on Drawings 310 and 320.

The in-heap pond liner system will include a 12-inch-thick prepared subbase layer, with a hydraulic conductivity of less than $1\times10^{-5}$ cm/sec, overlain by an 80-mil DST LLDPE secondary geomembrane. The secondary liner will be overlain by the LCRS, which in turn will be overlain by an 80-mil DST LLDPE primary geomembrane. Leakage through the primary geomembrane will report to the LCRS and be and conveyed to the LCRS sump for removal. The LCRS sump will be located near the upstream toe of the in-heap pond embankment, in the southern corner of the pond.

On the side slopes of the in-heap storage pond, the LCRS will consist of GSE TenDrain 300 (or equivalent) tri-planar geonet (without geotextile facing) sandwiched between the primary and secondary geomembranes. In the areas of the pond with slopes shallower than 10-percent, the LCRS will consist of a 36-inch-thick layer of LCRS drain aggregate encapsulating a series of 4-inch-diameter perforated corrugated polyethylene tubing (CPT) (Type SP) lateral drains and 8-inch-diameter perforated CPT (Type SP) toe drains. A layer of 8 oz/sy non-woven geotextile will be placed over the top of the LCRS drain aggregate and the geotextile will be overlain by another 12-inch-thick layer of prepared subbase. The perforated collection pipes and drain aggregate have been included across the flatter grades of the pond floor to improve drainage within the LCRS and to reduce the potential for hydraulic head build-up. The LCRS drain aggregate (minus 1/2-inch crushed mill reject) may be replaced with overliner drain aggregate (minus 2-inch crushed mill reject) if a layer of tri-planar geonet is installed between the top surface of the aggregate and the overlying non-woven geotextile.

With the two low-permeability components in intimate contact (the pond primary geomembrane liner and the underlying prepared subbase placed above the LCRS), the composite liner on the pond floor is intended to combine the hydraulic, physical, and endurance advantages of the two materials to result in a more effective system.

The potential leakage rate through the in-heap storage pond primary geomembrane liner that will contribute flows to the LCRS was based on the methods described in “Leakage through Liners Constructed with Geomembranes” (Giroud & Bonaparte, 1989) and "Equations for Calculating the Rate of Liquid Migration Through Composite Liners Due to Geomembrane Defects" (Giroud, 1997). Typical liner performance evaluations base the size and number of defects on the quality of liner installation. For “excellent-quality” liner installation, the typical approach considers one 0.1-square-centimeter ($cm^2$) defect per acre, while a conservative estimate (“good-quality” liner installation) may consider one 1-$cm^2$ defect per acre (Giroud et al., 1994). The recommended equations (Giroud, 1997) for *Good Contact* (for the pond floor) and *Absolute*

*Maximum* (for the pond side slopes) ground-contact conditions were used to estimate the potential leakage to the LCRS. These equations are listed as follows:

*Good Contact:*

$$Q/A = n\ 0.976\ C_{qo}\ [1 + 0.1(h/t_s)^{0.95}]\ d^{0.2}\ h^{0.9}\ k_s^{0.74}$$

Where:
- $Q$ = the leakage rate through the holes in the geomembrane, $m^3$/sec
- $A$ = the considered geomembrane area, m2
- $n$ = the number of defects per considered geomembrane area (A)
- $C_{qo}$ = the contact quality factor (0.21 for Good Contact and circular defect)
- $h$ = the hydraulic head on top of the geomembrane, m
- $t_s$ = the thickness of the low-permeability soil component of the composite liner, m
- $d$ = the diameter of the circular defect, m
- $k_s$ = the hydraulic conductivity of the low-permeability soil, m/s

*Absolute Maximum:*

$$Q = C_B\ a\ (2gh_w)^{0.5}$$

Where:
- $Q$ = the leakage rate through the holes in the geomembrane, $m^3$/sec
- $C_B$ = 0.6, dimensionless coefficient for sharp edged orifice
- $a$ = the total area of holes in the geomembrane, $m^2$
- $g$ = acceleration due to gravity, m/$sec^2$
- $h_W$ = the liquid depth acting on the geomembrane (head), m

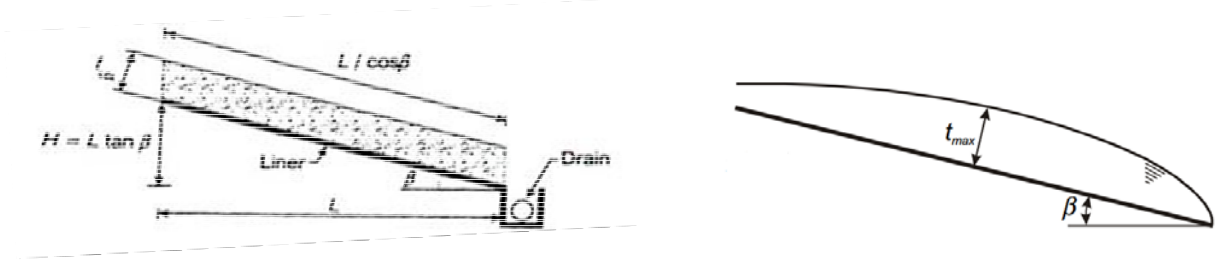
Two operating levels within the in-heap storage pond were used to evaluate the potential leakage into the LCRS: (1) the maximum normal (normal) operating level during Stage 1, 1527.7 fmsl, and (2) the embankment crest elevation of 1545 fmsl. The in-heap storage pond operating levels are discussed further in Section 6.9.4.

Case 1, considering the 1527.7 fmsl normal operating level within the pond, accounts for average liquid depths of 56.8 and 30.3 ft along the pond floor and side-slopes, respectively. The pond floor area was estimated to be 9.1 acres, while the area of the pond side-slopes up to 1527.7 fmsl was estimated to be 21.5 acres. The estimated leakage rate through the in-heap pond primary liner to the LCRS, for the Case 1 condition, was calculated to be approximately 281 gpm for good-quality liner installation.

Case 2, considering the 1545 fmsl embankment crest elevation (i.e. the maximum possible level within the in-heap storage pond), accounts for average liquid depths of 70.3 and 36.9 ft. The pond floor area was estimated to be 9.6 acres, while the area of the pond side-slopes up to the embankment crest elevation was estimated to be 29.6 acres. The estimated leakage rate through the in-heap pond primary liner to the LCRS, for the Case 2 condition, was calculated to be approximately 427 gpm for good-quality liner installation.

The floor of the in-heap storage pond (and the LCRS) will be sloped with a 2-percent grade toward the southwest corner where the LCRS sump will be located. A series of 4-inch-diameter perforated CPT (Type SP) lateral drains have been included across the floor of the LCRS to facilitate the collection and transport of potential leakage flows to the LCRS sump. Spacing of the lateral drains was evaluated using Giroud's equation (Giroud, Zornberg, & Zhao, 2000) for calculating the head acting on the liner.

Giroud's equation is shown graphically and mathematically (metric units) as follows:




$$t_{max} = j\frac{\sqrt{\tan^2\beta + 4q_h/k} - \tan\beta}{2\cos\beta}L$$

Where:
- ß = the basin slope between the pipes (degrees)
- $q_h$ = the design rate of liquid supply over a horizontal surface in terms of an equivalent velocity (m/s)
- $L$ = the drainage length of the lateral drain (m)
- $K$ = the permeability of the lateral drain (m/s)
- $t_{max}$ = the maximum head on the liner (m)
- $j$ = the numerical modifying factor which depends on the basin slope (set equal to 1 for conservative solution)

The LCRS drain aggregate (minus 1/2-inch crushed mill reject) was estimated to have an approximate permeability of $2.0\times10^{-3}$ cm/sec, while the overliner drain aggregate (minus 2-inch crushed mill reject) will have a minimum permeability of $1.0\times10^{-1}$ cm/sec. Based on the estimated leakage through only the pond floor to the LCRS (*Good Contact* and good-quality liner installation), and based on a target maximum hydraulic head of 1 ft acting on the LCRS secondary geomembrane, minimum pipe spacings of 38 and 1,018 ft were estimated to be required for the usage of LCRS drain aggregate and overliner drain aggregate, respectively. It is recommended that the overliner material be used for the LCRS, along with the previously described layer of tri-planar geonet, with lateral drains spaced every 105 ft. If LCRS drain aggregate is used, the lateral drain spacing should be reduced to 38 ft.

The LCRS sump is configured with an approximate 45 ft by 53 ft top perimeter, and will be 6-ft-deep with 2.5H:1V side-slopes. The sump will be filled with LCRS drain aggregate. Monitoring and return of solution collected within the LCRS will be accomplished using submersible pumps that will be lowered down two (2) 18-inch-diameter steel pump-cans. The pump-cans will be installed to run perpendicular up the interior face of the in-heap pond embankment, between the pond primary and secondary geomembranes, from the bottom of the LCRS sump to the embankment crest. The bottom 15 ft of each pump-can, within the sump, will be slotted (with the slot sizing designed to avoid migration of the drain aggregate into the pipes) to promote the ingress of solution. The bottom of the LCRS sump will be covered with a protective layer of conveyor belting (or similar) to reduce the potential for damage from the pump-cans. A layer of geosynthetic clay liner (GCL) will be installed within the LCRS sump, immediately above the 12-inch-thick prepared subbase, to further mitigate the potential for leakage from the LCRS.

Based on the LCRS sump geometry and an assumed porosity of 0.40 for the LCRS drain aggregate, the sump will have the capacity to collect approximately 19,594 gallons. Table 6.5 presents estimated times to fill and drain the LCRS sump considering good-quality liner installation and a recommended total pumping capacity of 350 to 450 gpm (likely two 250-gpm capacity pumps). It is estimated that 2 to 4 pump cycles per day will be required to maintain the LCRS in a relatively evacuated state.

**Table 6.5**
**Times to Fill and Drain LCRS Sump considering Recommended Outflow Pumping Rate**

| Level of Liner Installation Quality Control | "Excellent" | |
|---|---|---|
| Case | 1 | 2 |
| Pond Operating Level, fmsl | 1527.7 | 1545 |
| Sump Capacity, gallons | 19,593.7 | 19,593.7 |
| Inflow Leakage Rate, gpm | 281.2 | 427.4 |
| Time to Fill, minutes | 69.7 | 45.8 |
| Outflow Pumping Rate, gpm | 350.0 | 450.0 |
| Time to Drain, minutes | 284.7 | 866.3 |
| Pump Cycles per Day | 4 | 2 |

Comparatively, maximum pumping rates of approximately 4 gpm have been recorded from the WCHLF LCRS sump. This rate is lower than the estimated potential leakage rates for the BCHLF, and suggests that the quality of liner installation at the WCHLF was excellent. While construction of the BCHLF will strive to produce an end-product of similar or better quality, this is not guaranteed, therefore it is recommended to plan for the potentially higher leakage rates and pumping requirements as shown in Table 6.5.

If the design flow to the LCRS sump is exceeded, it does not mean that there has been a release to the environment, but rather that flows through the primary liner are larger than those used as a basis for the LCRS design. Vibrating wire piezometers, as discussed in Section 6.13, will be located within the LCRS drain aggregate to monitor the hydraulic head acting on the in-heap pond secondary geomembrane liner.

### 6.6.3 Process Component Monitoring System

The BCHLF PCMS has been designed to provide an indication of the leach pad liner system's performance upstream of the in-heap storage pond. More large-diameter solution headers have been incorporated into the BCHLF design, compared to the WCHLF; however, the number of solution header pipes does not specifically govern the number of required PCMS channels. Five PCMS channels will be located beneath select solution collection headers to monitor areas of the pad liner system where solution flow is expected to be present on a more consistent basis during operation. If leakage occurs through the pad geomembrane liner beneath the monitored solution collection headers, it will be collected in the PCMS channels and conveyed to the monitoring and discharge point that will be located in the northeast corner of the in-heap pond. Flows collected within the PCMS will be discharged into the double-lined in-heap storage pond. Monitoring of discharge from the PCMS will be addressed in FGMI's Operation and Maintenance (O&M) Manual.

The BCHLF PCMS channels have been located based on assessment of schematic heap configurations and the typical under leach area. Heap configurations representing every other lift of development, starting with Lift 4, were evaluated by superimposing FGMI's target 3.2 M $ft^2$ under leach area. PCMS channels were located such that a minimum of two channels will be located beneath the active under leach area at any time. The assessment of PCMS channels and under leach area is presented on Figures 6.02 through 6.09.

The PCMS is shown in plan on Drawing 200, profiles of the five PCMS channels are presented on Drawings 204 and 205, and sections and details are presented on Drawings 206 through 235. The PCMS channels are also shown superimposed beneath select solution collectors and headers on Drawing 500.

Above the confines of the in-heap storage pond, the PCMS channels will be constructed beneath the leach pad liner system to follow the planned orientations of the selected main solution collection headers. The "V"-shaped PCMS channels will be formed with a 6-inch-thick layer of prepared subbase overlain by an

80-mil DST LLDPE geomembrane to contain collected leakage from the pad liner system. The PCMS channels have been designed to facilitate construction and utilize a full roll-width of geomembrane (approximately 22 ft). The PCMS channels will have an approximate 21-ft top-width and 2-ft depth, and will be backfilled with PCMS drain aggregate for conveyance of flows. The top width of each PCMS channel will extend approximately 9 ft on either side of the largest solution collection header, which is deemed sufficient to meet the design intent of monitoring for potential leakage. To promote the concentration of flows around the main solution collection headers, directly above the PCMS channels, 1-ft-deep by 21-ft-wide "V"-shaped channels will be incorporated into the leach pad grading plan in these locations.

The potential leakage through the leach pad composite liner system to the PCMS channels was estimated based on the methods discussed in Section 6.6.2 (Giroud and Bonaparte, 1989) (Giroud, 1997) considering *Good Contact* between the liner and subbase and good-quality liner installation (one 1-$cm^2$ hole per acre of liner). The longest PCMS channel, approximately 2,762 ft, was used to model the largest contributing area. The hydraulic head acting above the leach pad composite liner system, including above the PCMS channel locations, is designed to be limited to 1 ft (outside the limit of the in-heap storage pond). Based on these assumptions, and a channel top width of approximately 21 ft, an estimated leakage rate of 6.6 gallons per day (gpd) (approximately 0.005 gpm) is expected to occur through the composite liner system to the longest PCMS channel.

Based on discussions with FGMI, small discharges from the WCHLF center PCMS channel (Valley 2) were measured between for an approximate 4-day period in August of 2014. The chemistry of the average 0.029 gpm flow that was measured suggested that the actual flow emanating from the leach pad into the PCMS channel was a minor component of the overall flow. It is surmised that external waters may have entered the upper end of the PCMS channel during what was a very wet summer at the Fort Knox site.

The flow capacity of the PCMS channel drain aggregate was estimated in accordance with the method discussed in Section 6.4 (Leps, 1973). Testing of minus 1/2-inch crushed mill reject (Knight Piésold, 2017a) resulted in an approximate 0.12-inch $D_{50}$. Based on a porosity of 26 percent, and considering a minimum slope of 8 percent, the estimated minimum capacity of each PCMS channel is approximately 196 gpm; well in excess of the estimated potential leakage rate of 0.005 gpm due to 1 ft of hydraulic head.

Just above the limit of the in-heap storage pond, the flows within each PCMS channel will be transitioned through a small lined sump and into a 4-inch-diameter, solid-wall HDPE (SDR 17) header pipe that will direct the flows to the discharge/monitoring point in the northeast corner of the in-heap storage pond.

The PCMS sumps will be located beneath the Stage 1 perimeter road. The typical PCMS sump configuration is presented on Drawing 220. Each sump will be approximately 10-ft long (parallel to the PCMS channels) by 12-ft wide at its crest, 2-ft deep, and configured with 1.5H:1V side slopes. The sumps will be lined with 80-mil DST LLDPE geomembrane, as extensions of the PCMS channels, and backfilled with PCMS drain aggregate. A 10-ft long section of slotted HDPE pipe will be extended from the base of each sump upstream into the connected PCMS channel to train flows into the solid-wall PCMS header pipe. The solid-wall header pipes will be booted through the sides of the lined sumps. The pipe boots shall be prefabricated as shown on Drawing 222, and shall be connected to the solid-wall header pipes (also shown on Drawing 222) prior to placement in the field. The sump liner and pipework shall be installed such that the slotted sections of pipe can be placed flush with the bottoms of the sumps to promote effective drainage from each sump.

The PCMS sumps, header pipes, and approximate 5-ft-long sections of slotted HDPE pipe will be constructed with the Stage 1 leach pad. To prevent migration of fines from the overlying materials that will form the Stage 1 inter-stage perimeter road, the PCMS sumps will be overlain with one layer each of 8 oz/sy non-woven geotextile and 80-mil geomembrane. A protective steel plate will also be placed over each sump to mitigate damage during Stage 2 construction activities when the sumps will be uncovered and connected to the upstream PCMS channels.

The PCMS header pipes will typically be installed at approximate 1-percent grades, beneath the Stage 1 perimeter road. Based on Manning's equation, the full-flow capacity of each PCMS header pipe will be approximately 90 gpm; much greater than the estimated potential leakage rate that will report to the longest PCMS channel. Each header pipe will be installed with two-heat trace and RTD loops (primary and secondary backup). The RTD and heat trace loops will be installed from the discharge/monitoring point upstream for an approximate length of 70 ft. After 70 ft, the header pipes will be overlain by approximately 28 ft of cover material and thus should be protected from freezing. The PCMS discharge/monitoring point will be located outside the planned footprint of the ore heap, at the upper edge of the northeast corner of the in-heap storage pond, to allow sampling and monitoring of the flows as they enter the pond.

Monitoring of the hydraulic head acting on each PCMS channel liner is not deemed necessary, therefore vibrating wire piezometers are not included in the BCHLF design. As discussed herein, the potential leakage and head-on-liner are expected to be minimal and the capacity of each PCMS channel is many times larger than the estimated potential leakage rate.

## 6.7 Overliner and Solution Collection Pipework

The leach pad composite liner system will be overlain by a 36-inch-thick overliner layer containing a network of solution collection pipes. The solution collection pipework plan is presented on Drawing 500, while sections and details of the collection system are shown on Drawings 520 through 534. As noted on Drawing 500, installation of the solution collectors and headers is typically required to be within 20 ft of the locations shown in plan; however, installation of the collectors and headers that are intended to overlay PCMS channels (Section 6.6.3) must follow the PCMS channel horizontal setting out data.

Based on liner integrity testing conducted to support the BCHLF design, see Section 4.10.3, minus 2-inch crushed mill reject has been confirmed for use as overliner material. The overliner layer will serve three primary purposes:

1. To reduce the hydraulic head acting on the pad liner system
2. To protect the pad liner system from damage during ore placement
3. To increase the rate of process solution recovery

The overliner layer is intended to be placed and extended in a controlled manner preceding the placement of overlying ore material. Placement of the 3-ft-thick overliner layer, without it being buttressed by overlying ore, should generally be limited to a 10-ft vertical height above the toe of placed ore.

The combination of the overliner layer and solution collection pipework is designed to collect and convey process leach solution from the pad area to the in-heap storage pond, and solution extraction wells, while limiting the maximum hydraulic head acting above the leach pad composite liner system to 1 ft (outside the limit of the in-heap storage pond). The solution pipework within the overliner layer will consist of perforated, smooth-interior, CPT (Type SP). Generally, 12- and 24-inch-diameter solution collectors and headers, respectively, are planned for installation within the BCHLF basin area.

Based on the currently planned 16,000 gpm solution flow to the BCHLF, and the unit-area application rate of 0.005 gpm/ft$^2$, the under leach area is expected to be limited to 3.2 M ft$^2$. Reduction of collector and header pipe sizes, that could be realized as contribution areas decrease up the valley slopes, has been avoided with the BCHLF design to provide flexibility for potential future expansion of the facility. Although 16,000 gpm is the currently planned solution flow to the BCHLF, the BCHLF solution collection headers have been designed to accommodate 24,000 gpm of total solution flow to the facility with the same unit-area application rate of 0.005 gpm/ft$^2$. The pipe flow capacities were evaluated using the modified general discharge formula as discussed in Section 3.2.2. Analyses of the solution collection system accounted for tributary areas, slopes, collection pipe sizes, estimated reductions in pipe flow-carrying capacities due to heap loading, and a minimum hydraulic capacity factor of safety of 1.5. The reduced cross-sectional areas of the header pipes were estimated based on the normal stresses induced from the

500-ft maximum thickness BCHLF heap configuration and empirical testing of pipe deflections due to heap loading for other similar projects.

The empirical testing was conducted between 1991 and 2000 at the United States Bureau of Reclamation (USBR) in Denver, Colorado. Materials (subbase, geomembrane, perforated CPT solution pipework, and drainage aggregate) were placed within a large reinforced steel frame to simulate typical cross-sections through the leach pad liner system. The USBR's universal testing machine, equipped with a hydraulic ram capable of imposing a load of 2.27 million kilograms, was used to impart normal stresses simulating the loading of a 150-m-thick heap (approximately 500 ft). In each test, the CPT pipes were equipped with "string-plot" deformation gauges such that the reduction in cross sectional area could be measured. Automatic readings were conducted every minute throughout the tests, and at every load increment equivalent to 30 m of additional heap, the loads were held constant for a period of 10 minutes to allow for settlement of materials and deflection of the CPT pipework. The maximum load of approximately 413 psi (simulating the 150-m-thick heap) was held for 24 hours to monitor additional deflections of the CPT. After the loading sequence was completed, the geomembrane and CPT pipe were uncovered and inspected for defects. The resulting vertical and horizontal deflections of the CPT pipes under load were used to develop best-fit trendline curves correlating heap height to the percentage of original flow area for the ADS 100-mm- (~4-inch-), 300-mm- (~12-inch-), and 600-mm- (~24-inch-) diameter CPT that were tested. Select information from Knight Piésold's empirical testing of the 600-mm-diameter ADS N-12 CPT is included in Appendix B (Knight Piésold, 2000).

A series of 4-inch-diameter perforated CPT (Type SP) solution laterals, encapsulated within the overliner layer drain aggregate, are also included in the leach pad solution collection system design. The solution laterals cover the areas between the solution collectors and header pipes, in a herringbone pattern, to facilitate the collection and transport of process fluid to the main collection pipes and ultimately to the BCHLF in-heap storage pond. Spacing of the solution laterals was evaluated using Giroud's equation (Giroud, Zornberg, & Zhao, 2000) as discussed in Section 6.6.2.

The BCHLF basin is comprised of four relatively distinct sloping areas as follows:

- 10-percent slopes – along the leach pad basin floor
- 10- to 20-percent slopes – along the upper northwestern and northeastern portions of the pad
- 25-percent slopes (on average) – along most of the northeastern basin side-slopes
- 40-percent slopes – along the southwestern and western basin side-slopes

Based on the $1.0\times10^{-1}$ cm/sec permeability of the overliner drain aggregate and setting $t_{max}$ equal to 1 ft, required pipe spacings of approximately 30, 35, 55, and 85 ft were calculated for the above noted slopes, respectively, and were used in the BCHLF design.

Although the design of the BCHLF solution collection pipework system follows commonly accepted mining industry practice, the anticipated deformations for the 12- and 24-inch-diameter pipes under the planned loading exceed those commonly accepted for CPT in standard civil engineering applications. Significant pipe deformations, and even buckling of the pipe haunches, may be experienced. To reduce the potential for such deformations, care will be taken to work the overliner drain aggregate under the pipe haunches and the drain gravel will be compacted along the main solution collection pipes to a distance equal to four pipe diameters on either side of the pipes. In the event a pipe is sufficiently deformed or crushed such that the free flow of solution is impeded, the overliner drain aggregate will provide local flow-carrying capacity to convey the solution around the choke point. In addition, the ore itself has excellent drainage properties. The ore to be placed on the BCHLF is expected to be run-of-mine, competent, coarse-grained granite (identical to the material placed on the WCHLF) that has a high flow-carrying capacity and will promote rapid drainage of process solution from the heap leach pad. During placement, the ore should naturally segregate to form a more pervious layer directly above the overliner. This ore layer will have the capacity to pass large flows and will serve as a tertiary method to transport process solution if needed. In summary,

the WCHLF has three redundant systems for solution transport to the in-heap storage pond and extraction wells, which include:

1. The solution collection pipework
2. The overliner drain layer
3. The competent and coarse grained ore (expected to form immediately above the overliner layer)

## 6.8 Solution Extraction Wells

### 6.8.1 General

Once process solution reports to the in-heap storage pond, it will be drawn laterally to a series of extraction wells located near the upstream toe of the in-heap pond embankment (along the southeastern edge of the pond) for removal. Vertical extraction wells will be progressively installed with the ore heap development; from the base of the in-heap storage pond at roughly elevation 1450 fmsl to where they will daylight on the Lift 4 heap bench at approximately elevation 1640 fmsl. For the initial, short-term, vat-leach operation, inclined solution extraction wells will be installed up the northeast side-slope of the in-heap pond, directly adjacent to the abutment of the in-heap storage pond embankment. The inclined extraction wells will daylight at the embankment crest just south of the in-heap pond emergency spillway.

The vertical solution extraction wells are presented in plan on Drawing 500. Sections and details of the vertical extraction wells are shown on Drawings 510, 512 and 514. The inclined wells are shown on Drawings 516, 517 and 518.

All work related to the solution extraction pumps, pipelines, and corridors are by Others.

### 6.8.2 Inclined Extraction Wells

Three (3) inclined solution extraction wells will be utilized for initial vat-leach operation of the BCHLF until the vertical extraction wells are completed. The inclined vertical wells will extend from the base of the in-heap storage pond to the embankment crest. It is envisioned that 4,000 gpm submersible pumps will be installed and operated in the three (3) wells to meet the 12,000 gpm solution extraction rate that is targeted for the initial period of operation.

Design of the inclined wells considered the maximum vertical effective stress that is anticipated to be imparted on the pipes based on the ultimate heap configuration and the in-heap storage pond filled to its minimum operating level of 1494 fmsl. Potential expansion of the ultimate heap configuration will not have an appreciable effect on the inclined extraction wells if the 500-ft maximum heap thickness is maintained.

The inclined solution extraction wells will typically be constructed using solid steel casing sections (ASTM 139 high strength low alloy (HSLA) steel, A606 Type 4) with a nominal outside diameter (OD) of 30 inches and a minimum wall thickness of 0.562 inches.

The bottom 20 ft of each well casing will incorporate Super Flo shutter screens, as manufactured by Roscoe Moss (or an approved equivalent). Roscoe Moss states that their Super Flo shutter screen product provides a crushing resistance of 1.6 times that of the blank pipe from which it was produced due to the corrugating effects. The shutter screens will be produced from 0.4375-inch-thick HSLA steel (ASTM 139, A606 Type 4) to form a 29-inch OD product.

The inclined extraction wells will be installed on top of the 36-inch-thick leach pad overliner layer, up the northeast side-slope of the in-heap storage pond and adjacent to the northeast abutment of the in-heap pond embankment. The base of the wells will tie into a fabricated steel manifold as shown on the drawings. Three-hundred- (300-) ft segments of 24-inch diameter perforated CPT (Type SP) solution collectors will be inserted into the ends of the well manifold assembly with a minimum 4-ft overlap to mitigate potential separation of the headers from the wells. The perforated CPT collectors will extend, on the surface of the

leach pad overliner layer, along the toes of the northeastern and southeastern pond slopes to enhance the hydraulic connectivity to the base of the inclined wells. The junction of the perforated CPT collectors to the well manifold assembly will be wrapped with a minimum of two (2) layers of 8 oz/sy non-woven geotextile to mitigate the ingress of overliner material into the well manifold assembly.

A 12-ft depth of extraction well drain aggregate, extending approximately 30-ft radially around the base of the inclined wells, will surround the slotted section of the inclined wells and promote the ingress of solution. Outside the extraction well drain aggregate surround zone, a minimum 3-ft cover of additional overliner material will be placed and compacted above and around the inclined well casings and solution collectors for protection during ore placement.

### 6.8.3 Vertical Extraction Wells

Five (5) vertical extraction wells, similar to those used at the WCHLF, will be installed for the long-term operation of the BCHLF. The vertical extraction wells will be installed near the upstream toe of the in-heap storage pond, upstream of the in-heap pond LCRS sump, with base elevations ranging from approximately 1451.3 to 1454.3 fmsl. The vertical wells will be progressively raised immediately ahead of ore placement until their ultimate collar elevations (1644 fmsl) are reached just above the top of Lift 4 elevation (1640 fmsl) for the BCHLF. Because continued construction of the vertical wells will be required until their full profile is developed, the vertical wells will likely be brought into service after the initial vat leaching operation. It is envisioned that 4,000 gpm submersible pumps will be installed and operated in four (4) of the five (5) wells to meet the 16,000 gpm solution extraction rate that is targeted for the long-term operation of the BCHLF. One additional vertical well is included in the design to allow for operational flexibility and servicing of other pumps, when needed.

Design of the vertical solution extraction wells accounted for the maximum lateral stresses and compressive dragdown forces that are estimated will be imparted on the pipes based on the ultimate heap configuration and the in-heap storage pond filled to its minimum operating level of 1494 fmsl. Potential expansion of the ultimate heap configuration will not have an appreciable effect on the inclined extraction wells if the 500-ft maximum heap thickness is maintained.

To reduce the compressive, longitudinal loading on the vertical wells due to settlement and dragdown of the surrounding leach ore, compression fittings were included in the design to provide strain-compatibility between the leach ore and the steel. The compression fillings effectively reduce the overall axial loading, that would act on the well casing below the fitting, by the magnitude of the dragdown forces that have accumulated above the fitting. To further mitigate the dragdown effects on the well casings, a layer of 80-mil smooth LLDPE geomembrane will be wrapped around the vertical wells, as shown on Drawing 512.

The vertical solution extraction wells will typically be constructed using solid steel casing sections (ASTM 139 HSLA steel, A606 Type 4) with a nominal outside diameter (OD) of 36 inches and a minimum wall thickness of 0.4375 inches.

Perforated sections will be incorporated in the construction of the vertical wells, as shown on Drawing 512, to increase the ingress of solution. The perforated sections will be produced from 0.5-inch-thick HSLA steel (ASTM 139, A606 Type 4) to form 36-inch OD well casings. These blank well casings will be torch-cut by the manufacturer (Roscoe Moss, or approved equivalent) in accordance with the slotting requirements shown on Drawing 514. Reductions in the blank pipe crushing strength, equivalent to the percentage of pipe area removed by the torch-cut slots, has been accounted for in the design of the vertical extraction well perforated sections.

The base of each vertical well will be founded on a reinforced steel plate underlain by protective layers of conveyor belting and geomembrane that will rest directly on top of the in-heap pond primary geomembrane liner. Thirty- (30-) inch-diameter steel pipe manifolds will be fabricated at the base of each vertical well to allow direct connection with the 24-inch-diameter perforated CPT header pipes of the leach pad solution collection system. The CPT solution headers will be installed within the steel pipe manifolds with a minimum

4 ft of overlap to mitigate potential separation of the headers from the wells. The interface of the CPT pipes with the steel manifolds will also be wrapped with a minimum of two (2) layers of 8 oz/sy non-woven geotextile to mitigate the ingress of overliner material into the well bases.

Once the base plates with manifolds, the CPT pipework connecting to each well, and the initial well casing sections are in place, the exterior of each well will be backfilled with approximate 5-ft-thick lifts of extraction well drain aggregate. These 5-ft-thick lifts will extend 20-ft radially outward from each well, and will be placed uniformly around the well casings to maintain the vertical alignments of the wells. Subsequent placement of ore locally around the drain aggregate surrounds will be conducted in relatively small, uniform lifts such that the vertical alignments of the wells remain true. Additional well casing sections and drain aggregate will be placed to keep the well construction just ahead of the ore lift level being placed. Following complete installation of the vertical extraction wells, the pumps will be set. Ultimately, the height of the wells will be about 194 ft.

Dragdown forces from settlement are not expected to be large along the well casings due to the incorporation of compression fittings and the relatively small compacted lifts in the drain aggregate surround zones. While the WCHLF well casings did experience deformations, it is believed these deformations were more associated with the uneven placement of surrounding drain aggregate and ore, rather than dragdown forces. Placement of ore and drain aggregate surround zones around the BCHLF vertical extraction wells will be conducted in a more uniform manner to mitigate this potential for deformation of the wells.

### 6.8.4 Minimum Operating Level and Theoretical Supply Rate to Vertical Wells

The approach to potential drawdown associated with the BCHLF vertical extraction wells is based on the experience and lessons learned from the WCHLF. Performance of the extraction wells for the WCHLF has exceeded all analyses of potential drawdown that were performed. Since the BCHLF well design and ore properties are relatively identical to the WCHLF, little drawdown of the pond level is expected during normal operation. As there are practical limitations associated with determining the hydraulic conductivity of large particle-size material, such as the ore, further assessment of the potential drawdown for the BCHLF was not deemed necessary.

The pumps planned for use in the BCHLF extraction wells will be identical to those used in the WCHLF. Identical to the WCHLF design, a minimum 24-ft depth of submergence from the base of the wells has been incorporated in the BCHLF design, along with a 20-ft contingency for potential drawdown. As such, the recommended initial minimum operating level for the BCHLF in-heap storage pond has been set at 1494 fmsl; however, this level may be lowered based on experienced gained during operation of the facility and subject to confirmation that sufficient strength is maintained in the solution extraction wells.

To illustrate the theoretical solution supply rate to the bases of the BCHLF vertical extraction wells, primarily dictated by their direct connections with the leach pad solution collection pipework, the potential flows through the submerged solution collection headers were analyzed assuming gravity conditions and using the Manning formula and the general discharge formula as follows:

$$V = k/n\ (R_h^{2/3})\ (S^{1/2})\text{, and}$$
$$Q = A\ V$$

Using the relationship, $A = R_h P$, the general discharge formula can be re-written as follows:

$$Q = P\ k/n\ R_h^{5/3}\ S^{1/2}$$

Where:
$Q$ = the volumetric discharge (cfs)
$A$ = the cross-sectional area of the flow ($ft^2$)
$P$ = the wetted perimeter (ft)
$k$ = a conversion constant equal to 1.486 for U.S. customary units
$n$ = the Gauckler-Manning coefficient ($s/ft^{1/3}$)
$R_h$ = the hydraulic radius (ft)
$S$ = the slope of the water surface or the linear hydraulic head loss (ft/ft)

Twenty-four- (24-) inch-diameter perforated CPT (Type SP) smooth-interior, dual-walled solution headers will be connected to the bases of each vertical extraction well. Significant empirical testing of this pipe has been conducted by Knight Piésold under simulated heap loading conditions, and those results were utilized in the WCHLF original design to evaluate the anticipated pipe performance during operations. Based on simulated loading from an approximate 250-ft-thick height of ore overlying the solution collection headers upstream of the vertical solution extraction wells, it is estimated that approximately 80 percent of the 24-inch-diameter CPT flow area will remain functional; resulting in an equivalent diameter of 1.8 ft. Based on this equivalent diameter, the wetted perimeter and hydraulic radius were calculated assuming full-flow conditions. Additional information utilized as input to the modified discharge formula was determined as follows:

- A Gauckler-Manning coefficient of 0.012 was adopted for smooth-interior CPT pipe
- The full flow capacity of each solution collection header was assessed considering approximate 1.5-percent (minimum) longitudinal slopes across the floor of the in-heap pond (the actual longitudinal slopes are designed to be slightly greater than this)

The maximum potential flow through each solution collection header was calculated, based on the above conditions, to be on the order of 23.1 cfs (10,378 gpm) under full-flow gravity conditions. This demonstrates that adequate supply of solution to the base of each well is expected to be provided considering the 4,000 gpm target pumping rate for each well.

## 6.9 In-Heap Storage Pond

### 6.9.1 General

In part, due to the Fort Knox project location that experiences cold weather conditions for significant periods each year, and to remain consistent with the operational success of the WCHLF, the BCHLF has been designed with in-heap storage of process solution and storm water incident to the leach pad. Process solution storage within the ore's interstitial pore spaces reduces the risk of freezing-related problems associated with open ponds and cold weather. An approximately 95-ft-high embankment, with a crest elevation of 1545 fmsl, is intended to be developed as an extension of the existing conveyor causeway to impound the BCHLF in-heap storage pond.

### 6.9.2 Freeboard Evaluation

A 5-ft freeboard allowance within the BCHLF in-heap storage pond (from the embankment crest, 1545 fmsl, to elevation 1540 fmsl) has been adopted to remain consistent with the WCHLF design. The general approach to evaluating the freeboard for the BCHLF was based on Section 3.3 of the FEMA document *Selecting and Accommodating Inflow Design Floods for Dams* (2013), which includes:

1. Wind-generated wave run-up,
2. Landslide-generated waves and/or displacement of reservoir volume,
3. Settlement of the embankment and foundation not included in the crest camber, and

4. Effects of possible spillway and/or outlet works malfunction during routing of the inflow design flood.

The calculations to confirm the adequacy of the BCHLF in-heap storage pond freeboard were conducted based on the methodology "Freeboard criteria and guidelines for computing freeboard allowances for storage dams" published by the USBR (1992). Because the in-heap pond will be fully contained within the heap, except under extreme conditions, the USBR methodology was applied to the relatively small swale area (along the front toe of the heap within the in-heap pond). In completing the analyses, the pond maximum water surface (MWS) and normal water surface elevations were assumed to be equal to the crest elevation of the in-heap storage embankment (elevation 1545 fmsl) to conservatively overestimate the geometric parameters used in the analyses. The top of the embankment impervious zone was considered as the same elevation.

According to the USBR methodology, two values for freeboard (minimum and normal) are typically calculated, and are defined as follows:

5. The minimum freeboard is defined as the vertical distance from the top of the impervious zone of the dam to the MWS elevation. The minimum freeboard should be the greater of:
   a. Three feet (3 ft), or
   b. The sum of the wave run-up and wind setup that would be generated by the average winds that would be expected to occur during the 100-yr/24-hr rain-on-snow event for the case where this event determines the assumed MWS.
6. The normal freeboard is defined as the vertical distance from the top of the dam's impervious zone to the normal water surface elevation, and should be sufficient to protect the dam from wind-generated waves that would be caused by the highest velocity sustained winds that could reasonably occur. The USBR suggests a wind velocity of 60 to 100 miles per hour (mph) for this.

The wave run-up and wind setup will be negligible because the majority of the pond is located within the ore heap. The wind velocity used to calculate minimum freeboard is typically less than the wind velocity used to determine normal freeboard. The lowest wind velocity typically considered when calculating minimum freeboard is that which has an exceedance probability of 10 percent. Aside from the allowable differences in wind velocity, the calculations for minimum and normal freeboard are identical. For the freeboard analysis discussed herein, only the maximum wind velocity of 100 mph was utilized.

According to the USBR methodology, the sum of the wave run-up and wind setup was determined for comparison with a 3 ft minimum freeboard. The wave run-up was calculated per the following equation:

$$R = \frac{6.49F^{0.5}}{0.4 + \left( \frac{6.49F^{0.5}}{5.12\left[0.559(169.87F)^{0.33}\right]^2} \right)^{0.5} \cot\theta}$$ (Equation 7, USBR 1992)

Where: $R$ = the wave run-up (ft)
$F$ = the fetch (miles)
$\theta$ = the angle of dam slope (degrees)

The fetch is defined as the average horizontal distance in the general direction of the wind over water, over which the wind acts to generate waves. A fetch distance of 0.19 miles (approximately 500 ft) was used for the in-heap storage pond based on the length of the pond parallel to the embankment crest. The angle of the upstream face of the in-heap pond embankment was taken as 21.80 degrees, representing the 2.5H:1V slope. Substituting these values into the above equation, the wave run-up for the in-heap storage pond during a 100-mph sustained wind was calculated to be 1.94 ft.

Wind setup was calculated using the following equation:

$$S = \frac{V^2 F}{1400 D} \quad \text{(Equation 8, USBR 1992)}$$

Where: $S$ = the wind setup (ft)
$V$ = the design wind velocity over water (mph)
$F$ = the fetch (miles)
$D$ = the average water depth along fetch length (ft)

The average depth along the fetch length for the in-heap storage pond was determined to be approximately 2 ft, and therefore the wind setup was calculated to be 0.26 ft.

Based on the calculations, the required in-heap storage pond freeboard was calculated to be 2.20 ft; therefore, the USBR guideline minimum freeboard of 3 ft governs.

Notwithstanding this assessment, a 5 ft freeboard allowance has been adopted for the BCHLF in-heap storage pond.

### 6.9.3 Emergency Spillway

An emergency spillway has been included with the BCHLF in-heap storage pond to accommodate overflow from the facility without overtopping the embankment. The emergency spillway will be located on the northeast abutment of the in-heap storage pond embankment and will lead to an outlet channel that will convey overflow from the facility to the Fish Creek storm water detention basin (Section 6.10) immediately downstream. The spillway outlet channel will also receive and convey flows diverted around the BCHLF by the north perimeter diversion channel (Section 6.11.2).

The in-heap storage pond has been sized to store the direct precipitation associated with the 100-yr/24-hr design storm event. If the BCHLF in-heap pond storage capacity is exceeded during an extreme event, excess water will flow through the emergency spillway. As discussed in Section 3.2.1, most storm water incident to the BCHLF will infiltrate into the heap and report to the in-heap storage pond. Any small amount of water that occurs as surface runoff will flow in the perimeter offset between the toe of the heap and the pad perimeter berm and be conveyed downslope to the small swale (1540 fmsl bottom elevation, which coincides with the bottom of the in-heap pond freeboard) along the front toe of the heap. Since the ore heap is highly pervious, it is likely that design flows entering the swale will infiltrate directly into the heap; however, at times frozen-ground conditions may inhibit this. In the unlikely event the surface of the heap is frozen, the entire runoff from the winter 100-yr/24-hr design storm event will be routed off and around the heap to the swale, and the excess volume will be conveyed through the emergency spillway and outlet channel.

Below the 1542 fmsl invert elevation of the in-heap pond emergency spillway, the swale will provide an estimated temporary storage capacity of approximately 0.10 M $ft^3$, or 0.7 M gallons. The excess runoff volume that would flow through the spillway under frozen conditions due to the January 100-yr/24-hr storm event was estimated based on the heap hydrology as 2.73 M $ft^3$, or 20.4 M gallons. Based on the design assumptions, this overflow runoff should be considered as non-contact since it theoretically would be separated from the leach pad process solution by the frozen exterior slopes of the heap.

The in-heap storage pond emergency spillway has been designed to pass the 220 cfs peak flow resulting from the January 100-yr/24-hr design storm event acting on the final (Stage 6) footprint of the BCHLF. The emergency spillway will be lined with 6-inch-thick reinforced concrete and has been configured with a 3-ft depth (invert elevation 1542 fmsl), a 25-ft bottom width, and 5H:1V side-slopes to accommodate light vehicle traffic through the spillway. An approximate 2-ft depth within the spillway is required to pass the design peak flow, thus 1 ft of freeboard to the embankment crest (1545 fmsl) is included. The in-heap

storage pond geomembrane liner will be connected to the upstream edge of the spillway concrete weir section using stainless steel batten bars and anchor bolts embedded in the concrete.

Downstream of the concrete weir section, the emergency spillway outlet channel will be lined with 9-inch nominal diameter ($D_n$) grouted riprap and will transition to a 3-ft bottom width by 4.25-ft-deep channel with 1.5H:1V side-slopes. The channel transition will occur over the first 50 ft of the outlet channel alignment; however, the channel grouted riprap erosion protection will be extended to a minimum elevation of 1543 fmsl. An 8 oz/sy non-woven geotextile will be installed beneath the grouted riprap to mitigate erosion of the channel subgrade if cracking of the grouted riprap occurs. The outlet channel has been designed with a 3-percent grade and will pass the 305 cfs maximum peak flow resulting from the January 100-yr/24-hr design storm event acting on all upstream catchments (Stage 6 configuration).

The spillway outlet channel will pass under the existing conveyor, located along the crest of the in-heap pond embankment, and allows for a minimum 5-ft clearance from the extents of channel excavation and the existing conveyor foundation supports. The adequacy of this clearance will be confirmed in the field during construction of the channel.

Downstream of the conveyor alignment, a drive-thru crossing has been included in the spillway outlet channel design. The geometry of the drive-thru crossing is similar to that of the emergency spillway reinforced concrete weir section, but consists of an approximate 28-ft bottom width, and 5H:1V side-slopes. The outlet channel drive-thru crossing is intended to accommodate light vehicle traffic on the existing access road along the downstream edge of the conveyor causeway. The drive-thru crossing bottom width was dictated by the inclusion of two 36-inch-diameter solid HDPE (SDR 17) low-level culverts that are intended to minimize the periods during which flow through the drive-thru section will occur. As requested by FGMI, the drive-thru crossing has been designed without erosion protection. Significant erosion of the drive-thru crossing should be expected during storm events that result in flow through the crossing; however, this will not affect the integrity of the BCHLF in-heap pond embankment. Maintenance of the spillway outlet channel and drive-thru crossing will be required for the life of the facility. Downstream of the drive-thru crossing, the outlet channel will transition in size, and discharge flows to the Fish Creek storm water detention basin discussed in Section 6.10.

Between the reinforced concrete weir section and the outlet channel drive-thru crossing, the outlet channel erosion protection and non-woven geotextile will be underlain by a layer of 80-mil DST LLDPE geomembrane to mitigate the potential for seepage and saturation of the embankment fill, particularly around the existing conveyor supports. The outlet channel geomembrane liner will be connected to the downstream edge of the spillway concrete weir section using stainless steel batten bars and anchor bolts embedded in the concrete. The outlet channel geomembrane liner will also be booted to the inlets of the two culverts associated with the low-level drive-thru crossing.

The in-heap pond emergency spillway and outlet channel is presented in plan on Drawings 550 and 551, while sections and details are presented on Drawings 552, 554, and 556.

### 6.9.4 In-Heap Pond Storage Components

The BCHLF in-heap pond will be predominantly filled with leach ore material. Solution within the in-heap pond will generally be stored in the interstitial pore spaces between the ore particles. The available solution storage capacity was estimated considering the settlement and consolidation that will occur as a result of the heap overlying the ore placed within the BCHLF in-heap pond.

Consolidation testing of the ore material placed within the WCHLF has suggested the ore porosity will vary from 36 percent under low load to 20 percent under the anticipated load due to the 500-ft maximum heap thickness (Knight Piésold, 2007). Based on consolidation testing of the ore, and the geometry of the WCHLF heap and in-heap pond, Knight Piésold estimated an average porosity of *23 percent* for the ore material within the pond. Based on third-party review of the WCHLF expansion design, and questioning of the in-heap pond porosity and associated storage capacity, FGMI agreed to conduct a field draindown test

on the WCHLF to collect data that would allow an independent check of the theoretical porosity that was based on laboratory testing. In the field draindown test, conducted in September 2016, solution flow to the heap and from the in-heap pond was halted for approximately 4 hours. Leading up to the draindown test, the solution flowrate to the heap was held relatively constant. Over the test period, the solution level within the pond rose due to the draindown from the heap. The change in pond level was correlated to a change in overall volume within the pond. The volume of solution collected in the pond was based on the total solution flow multiplied by the approximate 4-hr test duration. Using the volume of solution collected and the overall change in volume, the resulting average porosity for the WCHLF in-heap pond was calculated as *22.6 percent*, which compared well with the previously estimated value (*less than 2-percent difference*) and provided support to Knight Piésold's previously adopted approach.

Based on the approach used previously for the WCHLF, and the site-specific geometry of the BCHLF, the average porosity of the ore material within the BCHLF in-heap pond is estimated to be 21.8 percent.

As noted in Section 6.8.4, a minimum 24-ft depth of pump submergence and an approximate 20-ft contingency for potential drawdown (as measured from the base of the wells) has been incorporated in the BCHLF design to remain consistent with the WCHLF. As such, the recommended initial minimum operating level for the BCHLF in-heap storage pond has been set at 1494 fmsl; however, this level may be lowered based on experienced gained during operation of the facility and subject to confirmation that sufficient strength is maintained in the solution extraction wells.

The recommended minimum operating depth represents the assumed minimum pond level that will not lead to cavitation of the pumps. The pond maximum normal operating level represents the maximum elevation the pond should be operated at any stage of the BCHLF development to maintain the necessary capacity for the required in-heap pond storage components. The difference between the maximum normal and minimum operating levels represents the operational flexibility within the in-heap pond. This operational flexibility will decrease with the progressive development of the BCHLF.

The BCHLF in-heap pond storage components have been based on the proposed pad development presented herein up to the currently defined Stage 6 footprint. Above the maximum normal operating levels, the following three primary components are accounted for within the in-heap pond:

- 100-yr/24-hr design storm event

  The 2016 updated climate analyses, discussed in Section 3.0, indicate the 100-yr/24-hr storm event that governs overall volume is due to the September rain-on-snow condition and is estimated to produce 4.0 inches

  The pond storage volume associated with the 100-yr/24-hr design storm event (September rain-on-snow) was estimated to vary between 5.2 and 31.2 M gallons, for the Stage 1 through ultimate (Stage 6) configurations of the BCHLF development, respectively
- 24-hr emergency draindown due to loss of power or pumps

  Emergency draindown requirements have been conservatively estimated by multiplying the draindown duration by the solution flowrate supplied to the heap (i.e. 24 hours at the supply flowrate). An emergency draindown condition could occur if the power supply is interrupted or if pumps are temporarily rendered inoperable; both of which would result in reduced pumping capacity from the in-heap pond. Unlike the WCHLF, and as requested by FGMI and the ADNR, no recirculation of solution has been considered for the BCHLF emergency draindown condition.

  Based on the solution supply rate of 16,000 gpm, and considering no recirculation, the 24-hr draindown volume was estimated to be approximately 23.0 M gallons
- Freeboard allowance

  The freeboard allowance is a vertical distance typically included as a contingency against overtopping of a dam. A 5-ft freeboard depth from 1540 to 1545 fmsl, has been adopted for the BCHLF in-heap storage pond to remain consistent with the WCHLF. Within the BCHLF in-heap pond freeboard:

- 1540 fmsl is the bottom of the in-heap pond freeboard. This is the maximum water level prior to encroachment into the freeboard allowance. The bottom of the in-heap pond freeboard allowance (elevation 1540 fmsl) coincides with the bottom of the swale formed along the front toe of the heap (between the crest of Lift 2 and the toe of Lift 3) and will allow for visual monitoring of encroachments into the pond freeboard allowance.
- 1542 fmsl is the invert of the in-heap pond emergency spillway. During extreme conditions, flow will begin to pass through the spillway when water levels within the pond rise above this elevation. This could be from the design storm event acting on frozen ground conditions, or it could be due to extreme storm events above those established for the in-heap pond design criteria (i.e. greater than the 100-yr/24-hr storm event).
- 1544 fmsl is the bottom of the freeboard within the emergency spillway. This is the design water surface level within the emergency spillway when the design storm event (100-yr/24-hr January rain-on-snow acting on frozen ground) results in discharge through the emergency spillway.
- 1545 fmsl is the crest of the embankment. This is the maximum water level within the BCHLF in-heap pond prior to overtopping of the dam.

The volume within the 5-ft freeboard allowance was estimated to be approximately 13.0 M gallons, while the volume (within the freeboard) prior to discharge (i.e. from 1540 to 1542 fmsl) was estimated to be approximately 5.1 M gallons.

Figure 6.10 presents the stage-by-stage relationship of the BCHLF pad areas versus the maximum allowable solution levels in the in-heap pond associated with these components, while Figure 6.11 presents the same relationship but with the components represented by volumes. Figure 6.12 presents the in-heap pond filling curve along with the in-heap pond components associated with the Stage 6 configuration of the BCHLF. The stage-by-stage pond storage components and freeboard allowance, presented graphically on Figures 6.10 through 6.12, are summarized in Table 6.6 on the following page.

The most important findings from the assessment of the BCHLF in-heap pond storage components and freeboard allowance, as shown in Table 6.6, are the maximum normal operating levels that should be monitored and honored. These are the stage-by-stage pond levels that should not be exceeded under normal operating conditions such that the required pond capacities for the design storm event, draindown, and freeboard allowance are always available if needed.

By way of summary, the stage-by-stage in-heap storage pond maximum normal operating levels are:

- Stage 1 – 1527.8 fmsl
- Stage 2 – 1525.3 fmsl
- Stage 3 – 1522.7 fmsl
- Stage 4 – 1520.1 fmsl
- Stage 5 – 1516.8 fmsl
- Stage 6 – 1514.0 fmsl

In comparing the maximum normal operating levels for each stage of the BCHLF development with the pond minimum operating level, 1494 fmsl, a large degree of operational flexibility is expected. The minimum expected operating flexibility once the BCHLF is constructed through the Stage 6 configuration presented herein is 20-ft.

**Table 6.6**
**In-Heap Pond Storage Component and Freeboard Allowance Summary**

| Stage | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2-Dimensional Pad Area (1) | 2.08 M ft$^2$ | | 4.19 M ft$^2$ | | 6.20 M ft$^2$ | | 8.20 M ft$^2$ | | 10.67 M ft$^2$ | | 12.58 M ft$^2$ | |
| **Component** | **Elev (fmsl)** | **Volume (M gal)** | **Elev (fmsl)** | **Volume (M gal)** | **Elev (fmsl)** | **Volume (M gal)** | **Elev (fmsl)** | **Volume (M gal)** | **Elev (fmsl)** | **Volume (M gal)** | **Elev (fmsl)** | **Volume (M gal)** |
| Maximum Normal Operating Level (2, 3) | 1527.8 | 81.0 | 1525.3 | 75.8 | 1522.7 | 70.8 | 1520.1 | 65.8 | 1516.8 | 59.7 | 1514.0 | 54.9 |
| 100-yr/24-hr Storm Event (3) | 1527.8 to 1530.2 | 5.2 | 1525.3 to 1530.2 | 10.4 | 1522.7 to 1530.2 | 15.4 | 1520.1 to 1530.2 | 20.4 | 1516.8 to 1530.2 | 26.5 | 1514.0 to 1530.2 | 31.2 |
| 24-hr Emergency Draindown (4) | 1530.2 to 1540.0 | 23.0 | 1530.2 to 1540.0 | 23.0 | 1530.2 to 1540.0 | 23.0 | 1530.2 to 1540.0 | 23.0 | 1530.2 to 1540.0 | 23.0 | 1530.2 to 1540.0 | 23.0 |
| Freeboard Allowance (to spillway invert) | 1540.0 to 1542.0 | 5.1 | 1540.0 to 1542.0 | 5.1 | 1540.0 to 1542.0 | 5.1 | 1540.0 to 1542.0 | 5.1 | 1540.0 to 1542.0 | 5.1 | 1540.0 to 1542.0 | 5.1 |
| Freeboard Allowance (above spillway invert) | 1542.0 to 1545.0 | 7.9 | 1542.0 to 1545.0 | 7.9 | 1542.0 to 1545.0 | 7.9 | 1542.0 to 1545.0 | 7.9 | 1542.0 to 1545.0 | 7.9 | 1542.0 to 1545.0 | 7.9 |
| Total | | 122.22 | | 122.22 | | 122.22 | | 122.22 | | 122.22 | | 122.22 |

Notes: 1. The pad areas shown represent the cumulative 2-dimensional areas through each stage of the proposed BCHLF pad development, including FGMI's envisioned ultimate potential arrangement which includes expansion up to the valley ridgeline to the northwest and northeast of the currently defined Stage 6 footprint.

2. The Maximum Normal Operating Level represents the maximum elevation the pond should be operated at any stage of the BCHLF development to maintain the necessary capacity above it for the 100-yr/24-hr Storm Event and 24-hour Emergency Draindown components, in addition to maintaining 5 ft of freeboard below the crest of the in-heap pond embankment. The storage volumes presented for the Maximum Normal Operating Level are the total volumes from the bottom of the in-heap storage pond (elevation 1450 fmsl) to the elevations shown.

3. While the pond components associated with the Emergency Draindown and Freeboard allowance are assumed to remain constant throughout the life of the facility, the storage components associated with the Maximum Normal Operating Level and the 100-yr/24-hr Storm Event are inversely related to each other. As the pad area grows with each stage of development, the 100-yr/24-hr Storm Event component increases and reduces the Maximum Normal Operating Pond Level. It is expected that with on-going operational experience, FGMI will be able to readily adapt to this reducing level.

4. The 24-hr Emergency Draindown component provides storage for a condition where the power supply is interrupted or if pumps are temporarily rendered inoperable; both of which would result in reduced pumping capacity from the in-heap pond and its subsequent filling. Unlike the WCHLF, and as requested by FGMI and the ADNR, no recirculation of solution has been considered for the BCHLF emergency draindown condition. The BCHLF emergency draindown component is instead based on a drainage rate equal to the full solution supply rate of 16,000 gpm for entire 24-hr period.

## 6.10 Fish Creek Storm Water Detention Basin

Downstream of the BCHLF in-heap storage pond emergency spillway discussed in Section 6.9.3, the spillway outlet channel will discharge flows to the Fish Creek storm water detention basin.

The Fish Creek storm water detention basin has been formed between the excavated downstream slope of the over widened conveyor causeway embankment and FGMI's backfilling of the Fish Creek stockpile area. The resulting detention basin is comprised of upstream slopes varying from 2.0H:1.0V to 1.3H:1.0V, and downstream slopes of approximately 1.3H:1.0V.

The storm water detention basin will receive any unlikely overflow from the in-heap emergency spillway, and the surface runoff external to the BCHLF that is diverted around the facility. The backfilling of the stockpile area consisted predominantly of coarse-grained mine waste, and as such any flows reporting to the detention basin are expected to infiltrate beneath the backfilled stockpile area and be intercepted by existing dewatering wells in the area.

Nonetheless, the required capacity of the storm water detention basin was assessed to provide full storage of the worst-case runoff while considering:

- The Stage 1 and Stage 6 configurations of the BCHLF development (Figures 3.1 and 3.6) as these configurations comprise the extreme catchment conditions with respect to areas internal and external to the BCHLF that will report to the detention basin
- The monthly-basis runoff calculated from the 100-yr/24-hr precipitation (rainfall and snowmelt)
- The volume of the in-heap pond swale (discussed in Section 6.9.3) under winter, frozen-ground conditions

The calculations associated with the Fish Creek storm water detention basin inflow design flood volumes are summarized in Table 6.7. The design storage requirement for the basin was calculated to be approximately 4.93 $Mft^3$. This value does not account for reductions in the required detention volume due to the rate of infiltration within the detention basin. While waters collected in the detention basin are expected to rapidly infiltrate beneath the Fish Creek stockpile area and be picked up by dewatering wells in the area, FGMI has indicated that the volume of water within the detention basin will be controlled via direct pumping from the basin as needed (i.e. if collected waters do not infiltrate rapidly enough). Provisions for pumping of waters collected in the basin will be included in FGMI's O&M Manual.

The Fish Creek storm water detention basin is presented in plan on Drawing 560, along with FGMI's existing wells in the immediate vicinity, while typical sections through the basin are presented on Drawing 562. The stage-storage table presented on Drawing 562 indicates that the required detention volume (approximately 4.93 $Mft^3$) will be achieved with filling of the basin to approximately 1530.43 fmsl. It also shows a suggested maximum storage capacity of approximately 7.11 $Mft^3$ that may be realized with filling of the basin to 1540 fmsl.

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**Table 6.7**
**Fish Creek Storm Water Detention Basin Inflow Design Flood Volumes**

| BCHLF Stage | Area ID [1] | Winter Storm Contributing Area [2, 3] (ac) | Winter Storm Curve Number (CN) [4] | Summer Storm Contributing Area [2, 5] (ac) | Summer Storm Curve Number (CN) [6] |
|---|---|---|---|---|---|
| 1 | *External* | *469* | *100* | *469* | *75* |
| | *Internal* | *48* | *100* | *NA* | *NA* |
| | **Total** | **516** | **100** | **469** | **75** |
| 6 | *External* | *228* | *100* | *228* | *75* |
| | *Internal* | *289* | *100* | *NA* | *NA* |
| | **Total** | **516** | **100** | **228** | **75** |

| In-Heap Pond Swale Volume [11] (ac-ft) | In-Heap Pond Swale Volume [11] ($ft^3$) |
|---|---|
| | |
| 2.3 | 98,852 |
| | |
| | |
| 2.3 | 98,852 |
| | |

| Parameter | Jan | Feb | Mar | Apr [8] | May | Jun | Jul | Aug | Sep [10] | Oct | Nov | Dec | Max | Max ($ft^3$) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rain-on-Snow (CN = 100) [7] | | | | Rain (CN = 75) [9] | | | | | Rain-on-Snow (CN = 100) [7] | | | | |
| ***100-yr/24-hr Rain (in)*** | *1.5* | *0.9* | *0.8* | *1.2* | *1.6* | *3.8* | *3.4* | *2.7* | *2.3* | *1.1* | *1.5* | *1.0* | *3.8* | |
| ***100-yr/24-hr Snowmelt (in)*** | *1.2* | *0.4* | *0.5* | *0.0* | *0.0* | *0.0* | *0.0* | *0.0* | *1.6* | *1.4* | *0.6* | *0.4* | *1.6* | |
| **100-yr/24-hr Runoff (in)** | **2.7** | **1.2** | **1.2** | **1.2** | **0.2** | **1.5** | **1.2** | **0.7** | **1.6** | **2.5** | **2.1** | **1.4** | **2.7** | |
| **Stage 1 [11] 100-yr/24-hr Runoff Volume (ac-ft)** | **113.2** | 51.4 | 50.4 | 47.9 | 7.9 | 59.1 | 48.4 | 29.2 | **63.7** | 106.0 | 89.6 | 57.4 | **113.2** | **4,929,784** |
| **Stage 6 [11] 100-yr/24-hr Runoff Volume (ac-ft)** | **113.2** | 51.4 | 50.4 | 47.9 | 3.9 | 28.7 | 23.5 | 14.2 | **30.9** | 106.0 | 89.6 | 57.4 | **113.2** | **4,929,784** |

Notes:
1. External area: Area outside of footprint, Internal area: Area inside of footprint
2. Contributing areas from: Fairbanks Gold Mining, Inc. Barnes Creek Heap Leach Facility Report on Final Design Issued in Final Revision 0 (February 7, 2017)
3. Total winter storm contributing area will consist of both external and internal areas based on the assumption that the heap will be frozen (i.e., 100% runoff, no infiltration)
4. Assumed frozen ground runoff curve number (CN)
5. Total summer storm contributing area will consist only of external area based on the assumption that the BCHLF in-heap storage pond will capture rainfall/runoff via infiltration
6. Assumed unfrozen ground Antecedent Runoff Condition (ARC) II CN
7. It was assumed that frozen ground conditions are present from October through April. Rain-on-snow runoff values include 100% runoff from rain and snowmelt
8. April has insignificant snow accumulation values but frozen ground conditions and thus, its runoff values include 100% rain only
9. It was assumed that unfrozen ground conditions are present from May through September. Rain runoff values account for losses, per the stated CN
10. September has snow accumulation values but unfrozen ground conditions, and thus its runoff values include rain and snowmelt and account for losses, per the stated CN
11. Runoff from winter storm events, internal to the BCHLF lined footprint and that will report to the Fish Creek basin, have been reduced by the swale volume. The swale is located along the front toe of the heap just below the invert of the emergency spillway

## 6.11 Perimeter Roads, Surface Water Diversions, and Culverts

Perimeter roads and surface water diversions have been included in the design for each stage of the planned BCHLF pad construction. The perimeter roads and surface water diversions are broken into two categories, inter-stage and ultimate. Ultimate perimeter roads and channels wrap around the ultimate pad footprint (through Stage 6), while the inter-stage perimeter roads and channels are located along the upstream edge of each stage of pad development where subsequent stages will be extended further up the valley.

### 6.11.1 Inter-Stage Perimeter Roads and Diversions

Along the upper edge of each BCHLF stage of development, excluding the Stage 6 pad area (considered the final stage of development herein), inter-stage perimeter roads will provide construction and maintenance access and serve to divert upstream runoff.

The inter-stage roads have typically been configured with minimum 1.0-percent grades from high-points in the northwest end of the leach pad. Each road has been designed with a 16-ft running width, a 6-inch-thick layer of wearing course, and typically includes a 2-percent cross-fall away from the geomembrane-lined pad safety berm. Wearing course has been include in the design to improve travel conditions on the road. Because the inter-stage roads are likely to require frequent maintenance that will result in wasting of the wearing course material, at FGMI's option wearing course may be eliminated from the inter-stage roads.

All inter-stage perimeter roads include a minimum 4-ft-wide shoulder on the upstream side of the road, resulting in an overall minimum width of 20 ft to facilitate the leach pad construction. During each stage of pad development, cut and fill earthworks will be required to form the required new perimeter roads and to fill in the previous inter-stage roads. Along the south and west sides of the valley, significant fill will be required to form the leach pad basin; however, it is envisioned that the basin grading will be progressively developed only to the levels required to support the pad stage being constructed. The "additional" fills required for the southern and western basin slopes (i.e. external to the pad footprint and inter-stage perimeter road) will be graded with minimum 10-percent slopes from the existing valley side-slope down toward the inter-stage perimeter roads.

A 1.5-ft-high safety berm will typically be constructed along the pad-side edges of the inter-stage perimeter roads to prevent accidental ingress of vehicles onto the pad. The pad safety berm will be constructed of prepared subbase material, and the leach pad geosynthetics (geomembrane and geonet, where applicable) will be installed up and over the berm and be anchored beneath the pad perimeter road. After backfilling the inter-stage perimeter road for each subsequent stage of development, the geomembrane liner associated with the new pad area will extend down and over the previous stage's safety berm (with approximately 3 ft of overlap into the previous stage pad area) and be extrusion welded to the previous stage's geomembrane liner.

The inter-stage roads are considered temporary because they will eventually be regraded, backfilled, and incorporated into the footprint area of each subsequent stage of pad construction. Due to this temporary nature, and to remain consistent with the WCHLF design requested by FGMI, the peak flows associated with the 100-yr/24-hr winter design storm event acting on upstream watersheds will typically be accommodated within soil-lined inter-stage road corridors in lieu of dedicated channels. Flows diverted by the inter-stage perimeter roads will typically be conveyed to permanent diversion channels, as discussed in Section 6.11.2, that will be constructed along the ultimate pad perimeter.

One exception to the general design basis for the inter-stage roads is the road along the south side of Stage 1. To limit surface runoff to the southern end of the in-heap pond embankment, the southern Stage 1 perimeter road is designed with a flat longitudinal slope, without a safety berm, and with a 2-percent cross-fall toward the Stage 1 leach pad. The small catchment area upstream of this road will drain into the Stage 1 leach pad.

This road is designed to convey upstream surface runoff toward the leach pad perimeter near the existing crusher building; however, no ultimate perimeter diversion channel will be constructed in that area. Near the ultimate pad perimeter, along the Stage 2 south perimeter road, a drop inlet will be constructed to divert the flows to the south perimeter diversion culverts discussed in Section 6.11.5.

The required inter-stage road widths were assessed considering up to 1 ft of flow depth adjacent to the 1.5-ft-high geomembrane-lined pad safety berm. As shown on the surface water management plans presented on Figures 3.1 through 3.5, for Stages 1 through 5 of the BCHLF development respectively, most of the upstream surface runoff will be routed around the north side of the facility to minimize the required size of south perimeter diversion culverts. The inter-stage perimeter roads along the south side of Stages 2 through 5 will accommodate the design peak flows within the minimum 20-ft-wide road corridor. However, the inter-stage roads along the north side of the BCHLF will require wider shoulders (i.e. wider overall corridor widths) to pass the larger design peak flows. The required shoulder widths along the north inter-stage perimeter roads will vary from 12 ft to 5 ft, for Stages 1 through 3 respectively, and will be sloped with 20-percent cross-falls away from the perimeter roads.

Because the inter-stage roads will be soil-lined, periodic maintenance will be required following storm events where the roadways experience significant flows. Careful inspections and potential maintenance of all geomembrane-lined berms and adjacent anchor trenches should also be conducted following storm events.

To facilitate maintenance of the inter-stage roads, alternate configurations of the roads may be incorporated to develop constant cross-falls from the toes of the inter-stage perimeter berms to the invert of the “V” ditches created by the steeper cross-falls of the road shoulder widths. Such alternate configurations, as designated on the drawings, shall not be incorporated within 10 ft of any PCMS sump location.

The Stage 1 inter-stage perimeter road is shown in plan on Drawing 400, while its profile and horizontal control data are presented on Drawing 430.

### 6.11.2 Ultimate Perimeter Road and Diversions

The BCHLF ultimate perimeter road will run along portions of each intermediate stage of development, and along the entire exterior perimeter of the ultimate (Stage 6) pad area presented herein. The ultimate perimeter road has been designed with a 16-ft running width to provide construction and maintenance access. A diversion channel will typically run parallel to the ultimate perimeter road to convey runoff from upstream catchments around the facility. This ultimate perimeter diversion channel will receive flows conveyed by the inter-stage roads, but these flow contributions will reduce with each stage of pad construction.

The ultimate perimeter road will progressively be constructed with each stage of the BCHLF pad development. During each stage of development, previously constructed tie-ins of the inter-stage and ultimate perimeter roads will be backfilled and the ultimate perimeter road and diversion channel will be extended to the tie-in with the next inter-stage perimeter road.

From the southwest abutment of the in-heap pond embankment, the ultimate perimeter road will generally be aligned up the southwest side of the valley, then adjacent to FGMI’s existing waste dump haul road, across the face of FGMI’s planned future waste dump to the north side of the valley, down the northeast ridgeline separating the Barnes Creek and Walter Creek valleys (a 100-ft offset from the existing power line has been included in this area), and back down the northeast side of the valley to the in-heap pond embankment crest. The ultimate pad perimeter road design includes grades of up to 25 percent, to better suit the natural topography, and accounts for a 2-percent cross-fall (away from the geomembrane-lined pad perimeter berm) and a 6-inch-thick layer of wearing course. All cut slopes exterior to the ultimate perimeter road and diversion channel have been designed as 1.5H:1V, while all associated fill slopes have been designed as 2H:1V.

A 3-ft-high berm will be constructed along the pad-side edge of the ultimate perimeter road to prevent accidental ingress of vehicles onto the pad and to provide solution and storm water containment within the facility. The pad perimeter berm will generally be constructed of select fine-grained site grading fill, except that the berm face interior to the leach pad will be constructed with a 12-inch-thick layer of compacted prepared subbase material. The pad geomembrane liner will be installed up and over the perimeter berm and be anchored beneath the ultimate perimeter road.

Where the ultimate perimeter road will run adjacent to FGMI's existing waste dump haul road, the pad perimeter diversion channel will be located on the upstream side of the haul road (as the haul road diversion channel) at the toe of the existing and planned future waste dumps. In this area, the ultimate perimeter road will be separated from FGMI's haul road by the construction of a 6-ft-high, 20-ft base-width, rockfill safety berm. The ultimate perimeter road will be located slightly higher than the adjacent haul road such that surface runoff may pass through the rockfill berm and flow across the haul road to the adjacent diversion channel. As such, the haul road in this area should incorporate a minimum 2-percent cross-fall away from the rockfill safety berm. As discussed with FGMI, development of the BCHLF and the proposed ultimate perimeter road alignment will require slight relocation and/or modifications of the existing waste dump haul road. Modifications to the existing waste dump haul road will be completed by FGMI, or others, and are not accounted for herein.

The northern ultimate perimeter channel will convey flows to the emergency spillway outlet channel, as discussed in Section 6.9.3, while the southern ultimate perimeter channel along the existing waste dump haul road will convey flows to the south perimeter diversion culverts discussed in Section 6.11.5.

The ultimate perimeter diversion channel will be riprap-lined and has been sized to pass the peak flow associated with the 100-yr/24-hr winter rain-on-snow storm event, as discussed in Section 3.2.2. The trapezoidal channel has been designed with 2H:1V side-slopes, 0.6- to 2.2-ft flow-depths, 2- to 8-ft bottom widths, and 9- to 18-inch $d_{50}$ riprap. The riprap layer thickness is equal to twice the $d_{50}$ particle size. The maximum design peak flow for the north ultimate perimeter diversion was estimated to be approximately 85 cfs, while the maximum design peak flow for the south diversion channel was estimated to be approximately 88 cfs. These peak flows consider only the surface runoff from upstream catchments external to the final pad footprint (through Stage 6); therefore, 1 ft of freeboard is included in the channel design. The larger maximum peak flows from all upstream catchments (i.e. 254 and 115 cfs for the north and south channels, respectively) will be accommodated within the channels without freeboard. These larger peak flows are the potential flows that could occur during the initial stages of the BCHLF development, but they will be reduced with the progressive construction of the facility.

The approximate stage perimeter boundaries, ultimate perimeter road, and ultimate perimeter diversions are presented in plan on Drawings 140 and 410, while the ultimate perimeter road profile and horizontal control data are presented on Drawings 480 and 485, respectively. Sections and details of the perimeter roads and diversions are presented on Drawings 530, 532, and 534. The surface water management plan for Stages 1 through 6 of the BCHLF development are presented on Figures 3.1 through 3.6, respectively.

### 6.11.3 Stage 1 Haul Road and Diversion

Design of the Stage 1 haul road has been included to provide access for construction and loading of the Stage 1 pad area. The road has been designed with a 100-ft running width, 12-inch-thick layer of wearing course, and a 2-percent cross-fall (toward the adjacent diversion channel). The exterior side of the haul road is designed to include a 6-ft-high, 20-ft base-width safety berm.

The initial portion of the Stage 1 haul road is configured to coincide with the inter-stage perimeter road on the south side of Stage 2. As such the Stage 1 haul road will slope up from the existing crusher platform for approximately 300-ft before sloping down to the upper end of the Stage 1 leach pad in the valley bottom. This initial portion of the Stage 1 haul road is intended to allow priority construction of the southwest corner of the Stage 2 pad area near the beginning of the Stage 2 construction activities. Completion of this Stage 2

pad area is required such that (1) direct heap access can be established from the crusher platform and (2) the Stage 1 haul road can be backfilled as part of the Stage 2 pad construction.

The Stage 1 haul road diversion channel will divert upstream runoff to the inter-stage perimeter road on the north side of the Stage 1 pad area and will mitigate runoff reporting to the Stage 1 south perimeter road and into the Stage 1 pad area. The channel will be riprap-lined and has been sized to pass the peak flow associated with the 100-yr/24-hr winter rain-on-snow storm event discussed in Section 3.2.2. The trapezoidal channel has been designed with 2H:1V side-slopes, 1.0- to 1.5-ft flow-depths, 3- to 4-ft bottom widths, and 9-inch $d_{50}$ riprap. The riprap layer thickness is equal to twice the $d_{50}$ particle size.

The Stage 1 haul road is presented in plan on Drawing 400, with profile and horizontal control data on Drawing 435.

### 6.11.4 In-Heap Storage Pond Embankment Crest Access Road

The ultimate perimeter road will be connected along the southeastern edge of the facility by the in-heap storage pond embankment crest access road.

A 3-ft-high perimeter berm will be constructed along the pad-side edge of the embankment crest access road to prevent accidental ingress of vehicles onto the pad and to provide solution containment within the facility. The pad perimeter berm will generally be constructed of select fine-grained site grading fill, except that the berm face interior to the leach pad will be constructed with a 12-inch-thick layer of compacted prepared subbase material. The pad geomembrane liner will be installed up and over the perimeter berm and be anchored beneath the embankment crest access road.

The embankment crest access road has been designed with a 25-ft running width, a 12-inch-thick layer of wearing course, and a 2-percent cross-fall (away from the geomembrane-lined perimeter berm). On the downstream (southeast) side of the embankment crest access road, a 5-ft corridor has been allotted for access and maintenance of the existing conveyor. A 3-ft-high rockfill safety berm will separate the embankment crest road from the conveyor access corridor. The existing conveyor causeway embankment crest will be slightly widened to provide the approximate 52-ft clear distance for the required berms, road, and access corridor along the northwest side of the conveyor.

The in-heap storage pond embankment crest access road is shown in plan on Drawing 400, while a typical section is presented on Drawing 534.

### 6.11.5 South Perimeter Diversion Culverts

The south perimeter diversion culverts have been included along the south abutment of the facility to pass flows collected along the existing waste dump haul road and ultimately discharge them to the southwest corner of the Fish Creek storm water detention basin (see Section 6.10). These culverts are intended to reduce the current flows diverted by the existing waste dump haul road to the open pit.

The horizontal alignment and vertical profile of the south perimeter diversion culverts are presented in plan on Drawing 540, while typical sections and details associated with the culverts are presented on Drawings 542 and 544. The Fish Creek storm water detention basin is presented in plan on Drawing 560.

The entire alignment of the south perimeter diversion culverts (approximately 1,500 ft), listed as follows, will be constructed during the initial BCHLF development:

- beneath the existing waste dump haul road near the southwest corner of the planned BCHLF,
- down the south abutment of the BCHLF (partially below the ultimate perimeter road and partially on native ground outside the ultimate facility footprint),
- above the crusher conveyor outlet tunnel (supported by a gabion-reinforced fill slope),

- beneath the existing conveyor causeway access road adjacent to the existing crusher building, and
- into the Fish Creek storm water detention basin (see Section 6.10).

Three (3) 36-inch-diameter solid HDPE (SDR17) pipes will be installed along the entire alignment. Where buried, the pipes will be installed within a 15-ft-wide trench on a minimum 6-inch-thick layer of compacted pipe bedding material. Additional pipe bedding material will be placed and compacted around the culverts to their spring-lines. Pipe backfill material will then be placed and compacted from the culvert spring-lines to a minimum of 2 ft above the tops of the culverts. Compacted site grading fill will be placed and compacted above the pipe backfill material such that a minimum 6-ft cover is provided above the culverts under the haul roads and 5-ft of cover elsewhere. Along the culvert corridor above the crusher conveyor outlet tunnel, the pipes will be buried in the same manner noted above to allow access for light duty maintenance vehicles. Refer to Section 6.11.6 for details on the gabion reinforced slope.

During Stages 1 and 2, storm water conveyed along the existing waste dump haul road will flow through only two of the three culverts. The pipe closest to the pad area will be capped at the inlet during this time. The capped pipe will be connected to a sump that will be located along the upstream edge of the Stage 1 haul road. This inlet sump will initially receive flows from the upward sloping portion of the Stage 1 haul road, but eventually it will take the flows conveyed by the inter-stage perimeter road on the south side of Stage 2. As part of the Stage 3 construction activities, the 24-inch-diameter HDPE (SDR 17) inlet pipe to the south perimeter diversion culverts will be decommissioned prior to backfilling the Stage 2 inter-stage road and the remaining portion of the Stage 1 haul road. At that point the end cap will be removed from the inlet of the south perimeter diversion culverts such that all three culverts convey flows.

The upper inlet to the south perimeter diversion culverts has been designed to pass the peak flow runoff associated with the 100-yr/24-hr winter rain-on-snow storm event acting on the entire upstream watershed with 4 ft of head during Stages 1 and 2 (2 barrels in use) and 3.4 ft of head during Stages 3 through 6 (3 barrels in use).

### 6.11.6 South Perimeter Diversion Culverts Reinforced Slope

Between approximately Station 11+00 and Station 12+30 of the south perimeter diversion culverts alignment, the culverts will be installed on a bench that will be constructed across the face of the existing 1.4H:1V fill slope above the crusher conveyor outlet tunnel. Due to the steepness of the existing fill slope, the required bench-supported channel section will be developed by placing a 0.75H:1V gabion-reinforced fill above the conveyor outlet tunnel. A minimum bottom width of 15 ft will be required at the base of the culverts.

The gabion-reinforced slope will be constructed using 3-ft by 3-ft by 6-ft gabion baskets backfilled with gabion rockfill. After each layer of gabions is installed and connected, the void space between the gabions and the existing 1.4H:1V fill slope will be backfilled with 1-ft-thick layers of compacted site grading fill. Once the gabion-reinforced slope is constructed to the intended base level of the south perimeter diversion culverts, as discussed in Section 6.11.5, the three (3) 36-inch-diameter culverts will be installed on a minimum 6-inch-thick layer of compacted pipe bedding. The reinforced slope will be progressively raised with the backfill around the pipes. Additional pipe bedding material will be placed and compacted around the culverts to their spring-lines. Pipe backfill material will then be placed and compacted from the culvert spring-lines to a minimum of 2 ft above the tops of the culverts. Compacted site grading fill will be placed above the pipe backfill layer such that a minimum 5-ft cover is provided above the culverts (to accommodate light-vehicle traffic).

The gabion-reinforced slope, required to support the south perimeter diversion culverts, is presented in plan and section on Drawings 540 and 544, respectively.

## 6.12 Ore Loading, Heap Access, and Culverts

The BCHLF is proposed to be constructed in six stages and will achieve an estimated total storage capacity of 124.4 M cubic yards (cy), or 210.4 million tons (Mt) based on an average in-place ore dry density of 125.3 pcf. Schematic heap configurations were developed based on the design basin grading surface and required set-backs from the upstream face of the in-heap storage pond embankment. Based on the 5-ft freeboard adopted for the in-heap pond, measured from the embankment crest elevation of 1545 fmsl, the first visible full lift of ore above the in-heap storage pond (Lift 3) was started assuming a toe elevation of 1540 fmsl (i.e. the bottom of the in-heap pond freeboard). It is expected that the ore that will be placed below this level, within the in-heap pond, will be loaded in two lifts. Above the 1540 ft elevation, the ore heap was configured with 50-ft lift heights. The exterior slopes of each lift were modeled considering an average angle of repose slope of 1.3H:1V for the ore. Typical bench widths of 85 ft were included between each successive 50-ft lift of ore, such that an overall 3H:1V maximum heap slope results. Localized larger bench widths were included along the front face of the heap to maintain a maximum vertical ore thickness of 500 ft overlying the pad liner system.

Initial construction access and ore haulage to the bottom of the in-heap storage pond will be conducted from the upper end of the valley. The Stage 1 haul road (see Section 6.11.3) originating from the existing crusher platform, has been included in the BCHLF design to meet these needs. This haul road will allow the Stage 1 pad area and the southwestern corner of the Stage 2 pad area to be constructed and loaded. Once the heap is loaded in the southwest corner of Stage 2, below the Stage 1 haul road, loading access to the heap will be established directly from the crusher platform. Loading of the BCHLF heap for Stage 3 and beyond will utilize access points, from the existing waste dump haul road, that will be constructed to every other lift of the ore heap. This concept will allow the heap to be loaded by ramping up, down, or directly across from a minimal number of access point. At each of these heap access points, in-heap culvert crossings will be required to pass storm water runoff from upstream pad areas as they are developed.

Where on-heap culverts are required, they will be placed above the surface of the lead pad overliner layer. The 36-inch-thick overliner layer will be locally overbuilt and compacted to provide a level, benched cross-section to receive the culverts. The benched section will be oriented such that it maintains a relatively constant longitudinal grade (5 percent minimum). Twenty-four- (24-) inch-diameter perforated CPT (Type SP) culverts (the number of pipes varies) will be placed on the benched section and backfilled with additional overliner material. To reduce the potential for damage to the pipes, care will be taken to work the overliner drain aggregate under the pipe haunches and the material will be compacted. The overliner drain aggregate will be placed and compacted to provide a minimum 2 ft of cover over the pipe crowns. Additional road random fill or ore will be placed and compacted to achieve a minimum 6-ft cover above the culverts prior to haul traffic being allowed over the crossing. It has been assumed that four heap access culvert crossings will be required (i.e. one each for the Stages 2, 3, 4, and 5 pad areas). The heap access culvert crossings for Stages 2 and 3 will require 4 barrels each, while the culvert crossings for Stages 4 and 5 require 3 barrels each.

The Stage 2 in-heap culvert crossing, located near the southwest corner of the Stage 2 pad area, will also provide long-term maintenance and operation access to the collar locations of the vertical solution extraction wells. The vertical extraction wells will daylight on the Lift 4 bench (1640 fmsl) along the front face of the heap.

Starting with Lift 15, it is envisioned that loading of the ore heap will be conducted from the northwestern end of the facility, where the waste dump haul road will access the heap at approximately elevation 2170 fmsl. From that point, an on-heap haul road has been included with the schematic heap configuration, and will be utilized for loading the uppermost lifts of ore (Lifts 15 through 18). The on-heap haul road has been configured with a constant grade of 10 percent, a 100-ft running width, and a 6-ft-high by 20-ft base-width, safety berm on the exterior, fill-slope side of the road.

As configured herein, the currently proposed BCHLF grading plan encroaches on the upper portions of the FGMI's existing haul road and proposed future waste dump arrangement. As such, the upper section of

the haul road was modified and included in the schematic heap configurations. This extension to the existing haul road, exterior to the leach pad, was configured with an 8.4-percent grade to match the vertical alignment of the ultimate perimeter road where they run parallel. As discussed in Section 6.11.2, a 6-ft-high, 20-ft base-width, rockfill berm will separate the waste dump haul road from the ultimate pad perimeter road. Further modifications to the existing waste dump haul road may be required by FGMI, but are not included with the work scope presented herein.

Due to the nature of a valley-fill facility, the initial portions of the pad development provide smaller storage capacity than later years. The initial heap loading will utilize a reduced rate and is planned to begin in September of 2019 and be complete by the end of November. During years 2 and 3, heap loading will be limited to July through November but with the full production loading rate of 123,000 tons per day (tpd). After year 3, when additional pad area is available, loading of the ore heap is expected to occur approximately 8 months, or 244 days, out of each year (April through November) at a loading rate of approximately 123,000 tpd, or roughly 30 M tons per annum (tpa). Loading of the BCHLF is estimated to be complete approximately 8 years after the start of loading. Based on a conceptual start date of September 2019, loading of the BCHLF heap would be completed in July 2027. Leaching operations are expected to run year-round for the entire life of the facility.

The schematic heap configurations for Stages 1 through 6 (final), including the approximate locations of the in-heap culvert crossings, are presented in plan on Drawings 600 through 610, while typical sections and details are presented on Drawings 620 and 622. The conceptual lift-by-lift loading schedule for the Barnes Creek ore heap is presented in Table 6.8 on the following page, while the ore heap filling curve is presented on Figure 6.13.

The following bullet points should be considered when reviewing Table 6.8:

- The completion dates shown are estimated for construction completion of each stage of the BCHLF design as presented herein
- The heap storage capacity is based on an average in-place ore dry density of 125.3 pcf
- The first two lifts represent loading within the in-heap storage pond
- Ore loading is typically expected to occur from April 1$^{st}$ to November 30$^{th}$ of each year, for a total of approximately 244 days each year, unless otherwise shown
- Varied loading rates (from 70,000 to 120,000 tpd) are required for Year 1 (2019) based on the estimated pad construction
- It is expected that the start of loading for Years 2 and 3 (2020 and 2021) will be delayed until approximately July 1st of each year, based on the estimated pad construction
- Approximately 30 Mt of ore material is planned to be loaded each year after 2021, for an average loading rate of 123,000 tpd

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**Table 6.8**
**Conceptual Lift-by-Lift Heap Loading Schedule**

| Stage | Completion Date | Lift | Lift Top Elev (ft) | Loading Date | | Avg Loading Rate (tpd) | Volume (cy) | | Storage (tons) | | Days to Load |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Start | End | | Incremental | Cumulative | Incremental | Cumulative | |
| 1 | 31-Aug-19 | 0 | 1450 | base of in-heap pond | | | | | 0 | | 0 |
| | | 1 | 1490 | 1-Sep-19 | 13-Sep-19 | 70,000 | 549,439 | 549,439 | 929,404 | 929,404 | 13.3 |
| | | 2 | 1540 | 14-Sep-19 | 30-Sep-19 | 70,000 | 692,026 | 1,241,465 | 1,170,596 | 2,100,000 | 16.7 |
| | | 2 | 1540 | 1-Oct-19 | 18-Oct-19 | 120,000 | 1,282,653 | 2,524,118 | 2,169,671 | 4,269,671 | 18.1 |
| | | 3 | 1590 | 19-Oct-19 | 30-Nov-19 | 120,000 | 3,044,739 | 5,568,857 | 5,150,329 | 9,420,000 | 42.9 |
| | | 3 | 1590 | 1-Jul-20 | 1-Jul-20 | 120,000 | 71,800 | 5,640,657 | 121,453 | 9,541,453 | 1.0 |
| 2 | 1-Oct-20 | 3 | 1590 | 2-Jul-20 | 7-Jul-20 | 123,000 | 471,499 | 6,112,155 | 797,563 | 10,339,016 | 6.5 |
| | | 4 | 1640 | 8-Jul-20 | 16-Sep-20 | 123,000 | 5,163,652 | 11,275,807 | 8,734,575 | 19,073,591 | 71.0 |
| | | 5 | 1690 | 17-Sep-20 | 30-Nov-20 | 123,000 | 5,416,554 | 16,692,361 | 9,162,373 | 28,235,964 | 74.5 |
| | | 5 | 1690 | 1-Jul-21 | 7-Jul-21 | 123,000 | 581,087 | 17,273,448 | 982,937 | 29,218,901 | 8.0 |
| 3 | 1-Oct-21 | 5 | 1690 | 8-Jul-21 | 17-Jul-21 | 123,000 | 711,773 | 17,985,221 | 1,203,999 | 30,422,900 | 9.8 |
| | | 6 | 1740 | 18-Jul-21 | 7-Nov-21 | 123,000 | 8,184,116 | 26,169,336 | 13,843,841 | 44,266,741 | 112.6 |
| | | 7 | 1790 | 8-Nov-21 | 30-Nov-21 | 123,000 | 1,648,324 | 27,817,661 | 2,788,223 | 47,054,964 | 22.7 |
| | | 7 | 1790 | 1-Apr-22 | 30-Jun-22 | 123,000 | 6,687,333 | 34,504,994 | 11,311,958 | 58,366,922 | 92.0 |
| 4 | 1-Oct-22 | 7 | 1790 | 1-Jul-22 | 17-Jul-22 | 123,000 | 1,203,575 | 35,708,568 | 2,035,907 | 60,402,829 | 16.6 |
| | | 8 | 1840 | 18-Jul-22 | 30-Nov-22 | 123,000 | 9,851,400 | 45,559,968 | 16,664,135 | 77,066,964 | 135.5 |
| | | 8 | 1840 | 1-Apr-23 | 9-Apr-23 | 123,000 | 718,557 | 46,278,525 | 1,215,475 | 78,282,438 | 9.9 |
| | | 9 | 1890 | 10-Apr-23 | 26-Aug-23 | 123,000 | 10,061,970 | 56,340,495 | 17,020,326 | 95,302,764 | 138.4 |
| 5 | 1-Oct-23 | 9 | 1890 | 27-Aug-23 | 18-Sep-23 | 123,000 | 1,706,229 | 58,046,724 | 2,886,171 | 98,188,936 | 23.5 |
| | | 10 | 1940 | 19-Sep-23 | 30-Nov-23 | 123,000 | 5,255,551 | 63,302,275 | 8,890,028 | 107,078,964 | 72.3 |
| | | 10 | 1940 | 1-Apr-24 | 10-Jul-24 | 123,000 | 7,364,718 | 70,666,993 | 12,457,789 | 119,536,752 | 101.3 |
| | | 11 | 1990 | 11-Jul-24 | 30-Nov-24 | 123,000 | 10,377,589 | 81,044,583 | 17,554,211 | 137,090,964 | 142.7 |
| | | 11 | 1990 | 1-Apr-25 | 24-Apr-25 | 123,000 | 1,806,282 | 82,850,865 | 3,055,416 | 140,146,380 | 24.8 |
| 6 | 1-Oct-24 | 11 | 1990 | 25-Apr-25 | 28-Apr-25 | 123,000 | 295,500 | 83,146,364 | 499,852 | 140,646,232 | 4.1 |
| | | 12 | 2040 | 29-Apr-25 | 30-Sep-25 | 123,000 | 11,273,543 | 94,419,907 | 19,069,762 | 159,715,994 | 155.0 |
| | | 13 | 2090 | 1-Oct-25 | 30-Nov-25 | 123,000 | 4,366,983 | 98,786,890 | 7,386,970 | 167,102,964 | 60.1 |
| | | 13 | 2090 | 1-Apr-26 | 9-Jun-26 | 123,000 | 5,124,697 | 103,911,587 | 8,668,681 | 175,771,645 | 70.5 |
| | | 14 | 2140 | 10-Jun-26 | 25-Sep-26 | 123,000 | 7,865,431 | 111,777,018 | 13,304,770 | 189,076,415 | 108.2 |
| | | 15 | 2190 | 26-Sep-26 | 30-Nov-26 | 123,000 | 4,752,179 | 116,529,197 | 8,038,549 | 197,114,964 | 65.4 |
| | | 15 | 2190 | 1-Apr-27 | 18-Apr-27 | 123,000 | 1,367,618 | 117,896,816 | 2,313,395 | 199,428,358 | 18.8 |
| | | 16 | 2240 | 19-Apr-27 | 7-Jun-27 | 123,000 | 3,616,301 | 121,513,117 | 6,117,154 | 205,545,512 | 49.7 |
| | | 17 | 2290 | 8-Jun-27 | 5-Jul-27 | 123,000 | 2,037,337 | 123,550,454 | 3,446,258 | 208,991,770 | 28.0 |
| | | 18 | 2340 | 6-Jul-27 | 17-Jul-27 | 123,000 | 852,921 | 124,403,375 | 1,442,758 | 210,434,528 | 11.7 |

## 6.13 Instrumentation

A series of five (5) survey monuments will be located along the crest of the in-heap pond embankment to allow monitoring of potential movement.

Two (2) underdrain monitoring wells, shown on Drawings 200 and 142, will be installed through the crest of the existing conveyor causeway to allow monitoring of groundwater levels and sampling for assessment of water quality emanating from beneath the facility. The wells will be spaced apart by approximately 150 ft along the pre-conveyor causeway valley bottom to monitor the fluid level within the causeway fill. Pumping of groundwater flows, if needed, will be accomplished via the downstream dewatering wells. The Fish Creek dewatering wells are shown on Drawings 140 and 560; however, additional dewatering wells are located surrounding the existing open pit.

Six (6) pairs of vibrating wire piezometers (piezometers 1 through 12) will be located within the LCRS to monitor the head acting on the secondary liner of the in-heap storage pond. The LCRS piezometer locations are shown in plan on Drawing 300. Two pairs will be located within the bottom of the LCRS sump, while the other four pairs will be located within the LCRS along the base of the in-heap storage pond. Piezometers 1 through 12 shall be Geokon model 4500AL-170kPA (or equivalent). The 4500AL model is specifically designed for lower pressure conditions, and were recommended by Geokon. Barometric readings are required to adjust the piezometric readings from the 4500AL model. The 170kPA variant is rated to withstand maximum pressures two-times this value (i.e. up to approximately 34 meters).

Twenty-three (23) pairs of vibrating wire piezometers will be located within the leach pad overliner layer. Five of these pairs (piezometers 13 through 22) will be located in the base of the in-heap storage pond, within the Stage 1 leach pad, to monitor hydraulic pressures on the primary LLDPE liner and to estimate the water depth in the pond. The remaining pairs will be located in the overliner layer outside the limit of the in-heap storage pond, within Stages 2 through 6, to monitor various potential hydraulic head conditions (i.e. between solution laterals and adjacent to solution headers) acting on the leach pad liner system. Piezometers 13 through 18 shall be Geokon model 4500S-350kPA (or equivalent), while piezometers 19 through 58 (above the confines of the in-heap pond) shall be Geokon model 4500AL-70kPA. The 4500AL model is specifically designed for lower pressure conditions, and were recommended by Geokon. Barometric readings are required to adjust the piezometric readings from the 4500AL model. The 70kPA variant is rated to withstand maximum pressures two-times this value (i.e. up to approximately 14 meters).

The leads from the LCRS and Stage 1 overliner vibrating wire piezometers will extend to a centralized monitoring station located near the middle of the in-heap storage pond embankment crest. The leads from the overliner vibrating wire piezometers for Stages 2 through 6 will be extended upslope to five monitoring stations that will be relocated with each stage of pad development.

The reading and reporting frequency for all piezometers should be conducted such that at least one reading is taken every two weeks, when accessibility allows. Based on the experience with the WCHLF piezometers, it is imperative that frost protection be immediately applied over all piezometers to be installed within the BCHLF.

The planned locations of the survey monuments, underdrain monitoring wells, and the overliner layer vibrating wire piezometers are shown on Drawing 500. Typical sections and details, including setting out data, for the vibrating wire piezometers are presented on Drawing 720.

In addition to the above instrumentation, monitoring will be included to measure and record flows from the LCRS sump and discharge from the PCMS header pipes. Pressure sensors or similar devices will be installed to activate the pumps associated with the LCRS sump. Monitoring for the pumps within the inclined and vertical solution collection wells will include flow measurements and water depths at the pumps.

## 6.14 Drainage at Closure

Closure designs for the BCHLF are not included herein, however, it is anticipated they will follow the assumptions described in the original WCHLF design, which are summarized as follows:

- The designs will include re-contouring of the heap exterior surface to a maximum 3H:1V slope
- The designs will include placing a 1-ft-thick growth medium over the regraded slopes to encourage vegetation growth
- Holes will be drilled through the solution collection wells and their steel base plates, or new drill holes will be completed through the heap in the immediate vicinity of the solution collection wells, then through the leach pad liner system and into the underlying foundation material and the underdrain random fill
- The Fish Creek storm water detention basin, formed in-part by the downstream slope of the existing conveyor causeway embankment and which will be utilized for the operational period of the BCHLF, will not be required for the BCHLF during the post-closure period. As such, it is expected that the basin will be backfilled as part of the BCHLF closure activities.

As a free-draining structure, no differential impoundment of waters will exist upstream of the BCHLF in-heap pond embankment and it will not be considered a jurisdictional dam in the post-closure period. It is understood that FGMI's intended closure strategy will be:

- To create a landform that will be reclaimed under the Reclamation Standard (AS 27.19),
- That the post-closure waste management will be managed per ADEC requirements, and
- That the impoundment function of the dam will no longer be necessary and the dam will be abandoned in place after in-heap rinsing is complete.

# Section 7.0 - Slope Stability and Settlement Analyses

## 7.1 Stability Analyses

Limit-equilibrium slope stability analyses and seismically induced deformation analyses were performed for the design of the BCHLF. The analyses involved using the ultimate heap configuration and underlying foundation materials, selected material properties from previous laboratory testing in support of the BCHLF and the adjacent WCHLF, and estimated phreatic conditions.

Both static and seismic (pseudo-static) loading conditions were used to calculate minimum factors of safety against failure and the pseudo-static results were used together with the method of Bray and Travasarou (2007) to estimate potential seismically induced deformations. In all cases, the analyses were limited to the use of effective stress strength parameters since undrained conditions are not anticipated to occur within the BCHLF during or after operations. Static, limit-equilibrium slope stability analyses were also conducted to evaluate the upstream slope stability of the existing conveyor causeway, which will partially form the BCHLF in-heap storage pond embankment, upon completion of the required upstream basin excavation.

### 7.1.1 Stability Sections Geometry

A total of three cross-sections through the overall BCHLF, which represent the sections believed to be critical with respect to the foundation grades and ore heap configurations, were selected for stability analysis. One additional section through the existing conveyor causeway embankment, upon completion of the planned upstream excavation and prior to construction of the in-heap storage pond embankment, was analyzed. The four cross-sections analyzed are as follows:

- Section A – Middle Section
- Section B – South Abutment Section
- Section C – North Section
- Section D – Existing Conveyor Causeway Section (transient condition of upstream embankment face)

Sections A through C were analyzed to evaluate failures that could potentially occur within the HLF independent of the in-heap pond embankment, and failures that could affect the in-heap pond embankment.

Variations of Sections A, B, and C were analyzed as part of the BCHLF PFS design report (Knight Piésold, 2015b). Section D was added as suggested by the ADNR.

The stability section geometries were developed using the existing ground topography, proposed site grading, foundation preparation, in-heap storage pond embankment configuration, facility liner systems, and the ultimate ore heap configuration.

The stability section locations are shown in plan on Figure 7.1. The locations and geometries, as well as the various material properties, are described further in the following sections.

#### 7.1.1.1 Section A - Middle Section

The Middle Section (Section A) was selected for stability analysis since it includes the in-heap storage pond and embankment, and multiple liner interface conditions present below the in-heap pond.

Section A was also analyzed to evaluate potential failures of the downstream face of the existing conveyor causeway embankment.

#### 7.1.1.2 Section B – South Abutment Section

The South Abutment Section (Section B) was selected as it models a relatively steep and consistent foundation slope over the length of the section, sizeable overall mass, and the steepest front face slope of the ore heap.

#### 7.1.1.3 Section C – North Section

The North Section (Section C) was selected as it models the steepest foundation slope directed outward from the facility while still possessing decent overall mass.

#### 7.1.1.4 Section D – Existing Conveyor Causeway Section

Section D, through the upstream face of the existing conveyor causeway embankment, was selected for stability analysis to assess the transient condition during construction of the in-heap storage pond embankment. The previously deposited low-grade ore that has been stockpiled upstream of the causeway will be excavated to the approximate original construction surface of the conveyor causeway per the excavation plan provided by FGMI. After removal of the low-grade ore, the area will be further excavated to allow construction of the transverse underdrain random fill trench along the upstream toe of the conveyor causeway embankment. As the existing causeway will remain in place to allow continued operation of the conveyor, it is important to evaluate the impact the excavation will have on the conveyor causeway upstream-slope stability.

#### 7.1.1.5 Foundation

Based on the geotechnical site investigations conducted for the facility, and FGMI's planned excavation surface, the foundation beneath the BCHLF was identified as a variable thickness of previously stockpiled low-grade ore and underlying foundation soils. The previously stockpiled material shows evidence of soil intrusion into its voids; however, all material that is planned to be left-in-place is heavily over-consolidated (due to the previous configuration of the low-grade ore stockpile) and is not expected to be significantly lower than the strength of the in-situ low-grade ore. For the purposes of slope stability, the foundation was assumed impenetrable as the materials will provide a higher shear strength than the soil-liner interface which will control the overall heap stability.

### 7.1.2 Material Properties

The material properties for the analyses herein were developed from the results of the geotechnical field investigation and laboratory testing program conducted for the design of the BCHLF. The report on the geotechnical investigations (Knight Piésold, 2017a) details the site investigation and laboratory testing program, and the interpretations made to develop material properties for the analyses. A summary of the material properties adopted for the analyses are provided in Table 7.1.

The overall stability of the BCHLF is expected to be largely governed by the interface shear strength between the leach pad geomembrane and adjacent materials. In various locations within the planned BCHLF, the geomembrane liner will generally reside adjacent to: (1) the underlying prepared subbase material that will form the facility's composite liner system, (2) the overliner material of the solution collection system, and (3) the LCRS geonet within the confines of the in-heap storage pond.

As discussed in Section 4.10.1, the interface shear strength of the prepared subbase against the geomembrane underlying much of the heap foundation was estimated based on the database of historic testing of materials. In evaluating the results of the recently completed liner interface direct shear strength testing as compared to historic testing of materials from the Barnes Creek valley that were utilized in the construction of the WCHLF, the recent testing produced a lower interface strength than the historic testing. It is believed that the recent subbase samples were significantly overworked through numerous iterations of other testing prior to the interface testing. Due to excessive reuse of the subbase material, the results of the associated interface testing are not considered to be representative of the material properties that

will be realized during construction. The historic interface testing includes a larger number of individual test points and includes samples of material that were placed and compacted as part of the full-scale WCHLF construction (i.e. some were recovered from beneath the WCHLF Stage 5 geomembrane before that area was loaded with leach ore). As such, the historic testing was believed to be more representative of the behavior anticipated beneath the BCHLF and was used in analyses of the BCHLF. As with current operating procedure at the WCHLF, on-going testing of the BCHLF Prepared Subbase will be required throughout construction to identify potential variations in interface shear strength based on individual borrow areas and to demonstrate that acceptable interface strengths are achieved. Additional borrow source investigations and testing of the prepared subbase planned for use within the BCHLF will be conducted prior to construction. Updated stability and deformation analyses shall be conducted, as deemed necessary, based on the results of additional testing prior to the BCHLF construction and based on the planned on-going testing program.

Table 7.2 presents the interface shear strength envelope for 80-mil textured LLDPE geomembrane vs BCHLF subbase, while 80-mil textured LLDPE geomembrane versus GSE TenDrain 300 geonet (used for the steeper side-slopes of the BCHLF in-heap storage pond LCRS) is presented in Table 7.3. Figure 1 (Appendix C-1), presents the shear strength envelope estimated as representative of 80-mil textured LLDPE geomembrane vs the prepared subbase material from various BCHLF borrow sources. Figure 2 (Appendix C-1), presents a graphical comparison of the various shear strength failure envelopes.

It should be noted, that at high confining stresses (i.e., above a normal confining stress of approximately 58,000 pounds per square foot (psf) the prepared subbase vs geomembrane interface shear strength will control. As such, the non-linear normal stress versus shear strength function used to represent the geocomposite (geomembrane vs geonet and prepared subbase) has a break in curvature to account for this change in controlling interface, and the higher normal stress conditions are the same as presented in Table 7.2 (for prepared subbase).

**Table 7.1**
**Material Properties used in Stability Analyses**

| Material Type | Moist Unit Weight (pcf) | Saturated Unit Weight (pcf) | Effective Cohesion (psf) | Effective Friction Angle (deg) |
|---|---|---|---|---|
| Leach Ore | 132 | 141.0 | 0 | 40 |
| Random Fill – In-Heap Pond Embankment (Site Grading Fill) | 135 | 153.4 | 0 | 38 |
| Geomembrane Liner vs Prepared Subbase (Subbase Interface) | 100 (Prepared Subbase) | 140.3 (Prepared Subbase) | 0 | Non-Linear Shear Strength Envelope 1 (1) |
| Geomembrane Liner vs LCRS Geonet (Geosynthetic Interface) | 100 (Prepared Subbase) | 140.3 (Prepared Subbase) | 0 | Non-Linear Shear Strength Envelope 2 (1) |
| Foundation Soils (2) | 126.4 | 138.0 | 0 | 32 |
| Foundation Bedrock (2) | Impenetrable | | | |

Notes: 1. Non-linear shear strength envelopes, developed from interface shear strength testing for the various liner systems, are presented in the subsequent tables.

2. Foundation materials (soils and bedrock) were assumed to be impenetrable, where appropriate, to force the potential slip surfaces along the weaker liner interface. For stability evaluations of

the downstream face of the Random Fill In-Heap Pond Embankment, the foundation materials were assigned unit weights and effective shear strength parameters shown in Table 7.1.

**Table 7.2**
**Shear Strength Envelope 1**
**80-mil Textured LLDPE Geomembrane versus BCHLF Subbase**

| Normal Stress (psf) | Shear Stress (psf) |
|---|---|
| 0 | 0 |
| 1,250 | 1,281 |
| 2,500 | 1,982 |
| 5,000 | 3,068 |
| 10,000 | 4,748 |
| 20,000 | 7,348 |
| 30,000 | 9,487 |
| 40,000 | 11,372 |
| 50,000 | 13,089 |
| 60,000 | 14,682 |
| 70,000 | 16,180 |

**Table 7.3**
**Shear Strength Envelope 2**
**80-mil Textured LLDPE Geomembrane versus Geocomposite**
**(prepared subbase and GSE TenDrain 300 geonet)**

| Normal Stress (psf) | Shear Stress (psf) |
|---|---|
| 0 | 0 |
| 1,250 | 310 |
| 2,500 | 620 |
| 5,000 | 1,239 |
| 10,000 | 2,478 |
| 20,000 | 4,956 |
| 30,000 | 7,434 |
| 40,000 | 9,912 |
| 50,000 | 12,390 |
| 57,997 | 14,372 |
| 60,000 | 14,682 |
| 70,000 | 16,180 |

### 7.1.3 Phreatic Conditions

The ore heap will likely be wetted during leaching but is expected to remain fully drained except possibly at the base of the heap and within the in-heap storage pond. The base of the heap above the pad liner surface is expected to be saturated along main drainages during active leaching. Although the maximum hydraulic head acting above the leach pad composite liner system is limited to 1 ft outside the limit of the in-heap storage pond, a 5-ft piezometric head was conservatively considered (outside the limit of the in-heap pond) for the stability analyses herein. Within the in-heap pond, the phreatic surface was modeled assuming the following two conditions:

1. End-of-Operations (EoO) – A phreatic surface at elevation 1540 fmsl, which corresponds to the expected maximum operating pond during and at the end of operations. This pond level corresponds to the bottom of the in-heap pond freeboard. While this is the expected maximum operating level, this

elevated phreatic surface does not represent anticipated normal operating levels within the pond, which will be significantly lower. The phreatic surface within the in-heap pond embankment was set just above the foundation as the material is free draining, the entire pond is constructed with an LCRS, and no phreatic head is anticipated to develop within the embankment during operations.

2. Post-Closure (Post-C) – A phreatic surface at elevation 1560 fmsl, which corresponds to the maximum pond level anticipated post-closure. This is based only on the anticipated ultimate tailing level in the downstream TSF, and assumes the open pit and Fish Creek stockpile area immediately downstream will fill with water to this elevation after dewatering activities are terminated.

The phreatic surface for the analysis of the upstream face of the existing conveyor causeway embankment (Section D) was assumed to be coincident with the foundation. The phreatic surface will be depressed during construction, as the base of the BCHLF in-heap pond and underdrain system shall typically be constructed in dry conditions.

### 7.1.4 Stability Analysis Model

The stability analyses were completed using the computer program SLOPE/W Version 8.15 (GEO-SLOPE, 2012b), which enables the user to conduct limit equilibrium slope stability calculations by a variety of methods. Several methods may be used to search for the critical slip surface, that is, the surface with the lowest factor of safety for a given geometry and material properties. Analyses evaluated wedge (block) failures with sliding along the applicable underlying geomembrane liner interface, which typically control the stability of facilities of this nature. The Spencer (Spencer, E., 1967) method was used to estimate the factor of safety for the various potential slip surfaces in searching for the critical slip surface because that procedure satisfies both force and moment equilibrium, thereby yielding a rigorous solution.

### 7.1.5 Seismic Stability Modeling

In addition to static loading conditions, stability analyses were conducted for seismic loading conditions. The stability of the slope under earthquake loading was evaluated based on the magnitude and potential impact of anticipated permanent slope deformations. Such deformations were estimated using the methodology developed by Bray and Travasarou (2007), which is based on the results of a series of finite element studies calibrated with actual measured movements in dams and structures constructed over geomembrane liner systems (similar to heap leach facilities) and the sliding block concept originally proposed by Newmark (Newmark, N.M., 1965). In analyzing those results, Bray and Travasarou developed a series of equations estimating potential deformations as a function of magnitude, yield accelerations, and applied horizontal accelerations of the sliding mass. The yield acceleration is that horizontal acceleration of the slope, under the influence of a postulated earthquake, at which movement is imminent. It is the horizontal pseudo-static acceleration that produces a computed factor of safety of one. It should be noted that Bray and Travasarou (2007) have estimated deformations based on the assumption that the potential failure moves as a rigid block and/or a semi-rigid block. For the purposes of this study, it is assumed that the potential failure surfaces analyzed for the BCHLF can be reasonably represented by a fully rigid block, as the movement is expected to occur along the liner interface and differential, horizontal movements throughout the potential sliding mass are not expected to be significant.

Bray and Travasarou also provide a simplified equation for estimating displacement assuming a fully rigid "Newmark" style sliding block as follows:

$$LN(D) = -0.22 - 2.83LN(K_y) - 0.333\left(LN(K_y)\right)^2 + .566LN(K_y)LN(PGA) + 3.04LN(PGA) - .244\left(LN(PGA)\right)^2 + 0.278(M-7)$$

Where: $D$ = expected displacement
$k_y$ = yield acceleration
$PGA$ = the peak horizontal ground acceleration expected due to the considered earthquake
$M$ = earthquake magnitude

The free-field PGHAs were evaluated as part of the updated seismic hazard assessment presented in Section 2.5.2. The seismic stability analyses considered the MDE corresponding to dam Hazard Potential Classifications Class III (low) for operating conditions and Class II (significant) for post-closure conditions. Based on the results of the updated seismic hazard assessment, the Class III and Class II MDE (identified as the OBE and MDE herein, respectively), with a magnitude M7.5, have 5-percent and 2-percent probabilities of exceedance in a 50-yr design life, respectively. These probabilities have recurrence intervals of approximately 975 years and 2,475 years, and produce free-field PHGAs of 0.25 g and 0.37 g, respectively.

Pseudo-static slope stability analyses artificially apply the maximum average PHGA of the sliding mass that is expected during a design event. This acceleration is representative of the entire potential sliding mass and can be used to estimate the inertial forces acting on the BCHLF during an earthquake event. If the estimated factor of safety with the applied maximum average PHGA is below one, then deformations may be expected to occur during the earthquake due to inertial forces acting on the heap. The maximum average PGHAs for the sliding mass used for the pseudo-static slope stability analyses were derived from the free-field PGHAs, based on the methods proposed by Makdisi and Seed (1978), to be 0.23 g and 0.26 g, for the OBE and MDE, respectively.

### 7.1.6 Stability and Deformation Results

Results of the slope stability analyses completed for the final (Stage 6) heap configuration associated with the BCHLF design, for the EoO and Post-C conditions of the facility, are summarized in the follow tables:

- Table 7.4 – Stability Analysis Results Section A – Middle Section
- Table 7.5 – Stability Analysis Results Section B – South Abutment Section
- Table 7.6 – Stability Analysis Results Section C – North Section
- Table 7.7 – Stability Analysis Results Section D – Existing Conveyor Causeway Section

The analyses results include the wedge failure surfaces under both static and seismic loadings. The influence of seismic loading was represented by estimates of earthquake-induced deformation ranges under the action of the OBE and MDE seismic design events (Section 2.5.2). The stability analysis results are presented graphically in Appendices C-2, C-3, and C-4, for the static analyses, evaluation of critical yield accelerations (Ky), and pseudo-static analyses, respectively. The figures depict the critical failure surfaces for each cross-section and their calculated minimum factors of safety. Derivation of the maximum average PGHAs for the sliding mass used for the pseudo-static slope stability analyses is provided in Appendix C-5.

**Table 7.4**
**Stability Analysis Results Section A – Middle Section**

| Failure Surface | Minimum Static Factor of Safety | | | Critical Yield Acceleration | | Pseudo-Static Slope Stability Factor of Safety | | |
|---|---|---|---|---|---|---|---|---|
| | Calculated | | Reqd | | | OBE (4) | MDE (5) | |
| | EoO | Post-C | | EoO | Post-C | EoO | EoO | Post-C |
| Failure Through HLF Only (1) | 1.5 | 1.4 | 1.3 | 0.12 g | 0.10 g | 0.7 | 0.6 | 0.6 |
| Failure Through Embankment (2) | 1.7 | 1.7 | 1.5 | 0.21 g | 0.16 g | 0.9 | 0.9 | 0.9 |
| Failure of Embankment Only, Shallow (3) | 1.4 | 1.4 | - | - | - | - | - | - |
| Failure of Embankment Only, Deep | 1.7 | 1.7 | 1.5 | 0.31 | 0.17 | 1.1 | 1.0 | 0.8 |

Notes:
1. Failure occurring along the liner interface below the HLF.
2. Failure occurring along the liner interface below the HLF and impacting the crest of the embankment; therefore, potentially impacting containment of the in-heap pond.
3. Potential shallow and deep failures of the excavated surface of the existing conveyor causeway embankment downstream face were analyzed for completeness; however, these are not considered detrimental to containment within the in-heap storage pond or to the existing conveyor. These should be considered as a potential safety hazard and an area of potential maintenance.
4. The OBE represents conditions during operations.
5. The MDE represents conditions during and post-closure.

**Table 7.5**
**Stability Analysis Results Section B – South Abutment Section**

| Failure Surface | Minimum Static Factor of Safety | | | Critical Yield Acceleration | | Pseudo-Static Slope Stability Factor of Safety | | |
|---|---|---|---|---|---|---|---|---|
| | Calculated | | Reqd | | | OBE (3) | MDE (4) | |
| | EoO | Post-C | | EoO | Post-C | EoO | EoO | Post-C |
| Failure Through HLF Only (1) | 1.7 | 1.6 | 1.3 | 0.16 g | 0.13 g | 0.8 | 0.7 | 0.7 |
| Failure Through Embankment (2) | 2.2 | 2.1 | 1.5 | 0.28 g | 0.28 g | 1.1 | 1.0 | 1.1 |

Notes:
1. Failure occurring along the liner interface below the HLF.
2. Failure occurring along the liner interface below the HLF and impacting the crest of the embankment; therefore, potentially impacting containment of the in-heap pond.
3. The OBE represents conditions during operations.
4. The MDE represents conditions during and post-closure.

**Table 7.6**
**Stability Analysis Results Section C – North Section**

| Failure Surface | Minimum Static Factor of Safety | | | Critical Yield Acceleration | | Pseudo-Static Slope Stability Factor of Safety | | |
|---|---|---|---|---|---|---|---|---|
| | Calculated | | Reqd | | | OBE [3] | MDE [4] | |
| | EoO | Post-C | | EoO | Post-C | EoO | EoO | Post-C |
| Failure Through HLF Only [1] | 1.4 | 1.4 | 1.3 | 0.10 g | 0.09 g | 0.7 | 0.6 | 0.6 |
| Failure Through Embankment [2] | 1.5 | 1.5 | 1.5 | 0.22 g | 0.16 g | 0.9 | 0.9 | 0.8 |

Notes: 1. Failure occurring along the liner interface below the HLF.
2. Failure occurring along the liner interface below the HLF and impacting the crest of the embankment; therefore, potentially impacting containment of the in-heap pond.
3. The OBE represents conditions during operations.
4. The MDE represents conditions during and post-closure.

**Table 7.7**
**Stability Analysis Results Section D – Existing Conveyor Causeway Section**

| Failure Surface | Minimum Static Factor of Safety | | |
|---|---|---|---|
| | Calculated | | Required |
| | End of Operations | Post-Closure | |
| Failure of Embankment Shallow [1] | 1.0 | - | 1.3 |
| Failure of Embankment Deep [2] | 1.9 | - | 1.5 |

Notes: 1. Failure is surficial and seen as a maintenance issue, slope is cut to angle of repose.
2. Failure may impact stability of the conveyor causeway and potentially cause damage to conveyor system or adjacent roadway.

Results of the earthquake induced deformation analyses completed for the ultimate heap configuration associated with the BCHLF design, for the EoO and Post-C conditions of the facility, are summarized in the follow tables:

- Table 7.8 – Earthquake Induced Deformation Results Section A – Middle Section
- Table 7.9 – Earthquake Induced Deformation Results Section B – South Abutment Section
- Table 7.10 – Earthquake Induced Deformation Results Section C – North Section

**Table 7.8**
**Earthquake Induced Deformation Results Section A – Middle Section**

| Failure Surface | Critical Yield Acceleration | | Class III & Class II Hazard Potential Seismic-Induced Deformation (inches) | | |
|---|---|---|---|---|---|
| | | | OBE [4] | MDE [5] | |
| | EoO | Post-C | EoO | EoO | Post-C |
| Failure Through HLF Only [1] | 0.12 g | 0.10 g | 1.0-3.8<br>Avg [3] 1.9 | 2.3-9.3<br>Avg [3] 4.7 | 3.3-13.2<br>Avg [3] 6.6 |
| Failure Through Embankment [2] | 0.21 g | 0.16 g | 0.3-1.0<br>Avg [3] 0.5 | 0.7-2.8<br>Avg [3] 1.4 | 1.3-5.2<br>Avg [3] 2.6 |
| Failure of Embankment | 0.31 g | 0.17 g | 0.1-0.4<br>Avg [3] 0.2 | 0.3-1.1<br>Avg [3] 0.5 | 1.1-4.5<br>Avg [3] 2.3 |

Notes:
1. Failure occurring along the liner interface below the HLF.
2. Failure occurring along the liner interface below the HLF and impacting the crest of the embankment; therefore, potentially impacting containment of the in-heap pond.
3. Average calculated based on Bray and Travasarou (2007) with the range estimated per the recommendation of multiplying the average by 2 for the maximum and dividing by 2 for the minimum.
4. The OBE represents conditions during operations, M7.5 and PHGA = 0.25 g.
5. The MDE represents conditions during and post-closure, M7.5 and PHGA = 0.37 g.

**Table 7.9**
**Earthquake Induced Deformation Results Section B – South Abutment Section**

| Failure Surface | Critical Yield Acceleration | | Class III & Class II Hazard Potential Seismic-Induced Deformation (inches) | | |
|---|---|---|---|---|---|
| | | | OBE [4] | MDE [5] | |
| | EoO | Post-C | EoO | EoO | Post-C |
| Failure Through HLF Only [1] | 0.16 g | 0.13 g | 0.5-2.0<br>Avg [3] 1.0 | 1.3-5.2<br>Avg [3] 2.6 | 2.0-8.0<br>Avg [3] 4.0 |
| Failure Through Embankment [2] | 0.28 g | 0.28 g | 0.1-0.5<br>Avg [3] 0.2 | 0.4-1.4<br>Avg [3] 0.7 | 0.4-1.4<br>Avg [3] 0.7 |

Notes:
1. Failure occurring along the liner interface below the HLF.
2. Failure occurring along the liner interface below the HLF and impacting the crest of the embankment; therefore, potentially impacting containment of the in-heap pond.
3. Average calculated based on Bray and Travasarou (2007) with the range estimated per the recommendation of multiplying the average by 2 for the maximum and dividing by 2 for the minimum.
4. The OBE represents conditions during operations, M7.5 and PHGA = 0.25 g.
5. The MDE represents conditions during and post-closure, M7.5 and PHGA = 0.37 g.

**Table 7.10**
**Earthquake Induced Deformation Results Section C – North Section**

| Failure Surface | Critical Yield Acceleration | | Class III & Class II Hazard Potential Seismic-Induced Deformation (inches) | | |
|---|---|---|---|---|---|
| | | | OBE [4] | MDE [5] | |
| | EoO | Post-C | EoO | EoO | Post-C |
| Failure Through HLF Only [1] | 0.10 g | 0.09 g | 1.4-5.7<br>Avg [3] 2.8 | 3.3-13.2<br>Avg [3] 6.6 | 4.0-15.9<br>Avg [3] 8.0 |
| Failure Through Embankment [2] | 0.22 g | 0.16 g | 0.2-0.9<br>Avg [3] 0.5 | 0.6-2.5<br>Avg [3] 1.3 | 1.3-5.2<br>Avg [3] 2.6 |

Notes:
1. Failure occurring along the liner interface below the HLF.
2. Failure occurring along the liner interface below the HLF and impacting the crest of the embankment; therefore, potentially impacting containment of the in-heap pond.
3. Average calculated based on Bray and Travasarou (2007) with the range estimated per the recommendation of multiplying the average by 2 for the maximum and dividing by 2 for the minimum.
4. The OBE represents conditions during operations, M7.5 and PHGA = 0.25 g.
5. The MDE represents conditions during and post-closure, M7.5 and PHGA = 0.37 g.

### 7.1.7 Stability Conclusions

Based on the results of the stability and deformation analyses conducted for the BCHLF final configuration:

- The heap meets the commonly accepted minimum static factors of safety for both heap leach pads and embankments, where applicable.
- If excavated to angle-of-repose slopes, the upstream face of the existing conveyor causeway embankment will have a potential for surficial sloughing after removal of the adjacent low-grade ore stockpile. While this will not compromise the safety of the overall embankment, it should be viewed as a safety hazard for personnel working along the upstream toe of the causeway during construction and appropriate precautions and monitoring should be incorporated. For potential deep-seated failure, which may impact stability of the conveyor causeway and potentially cause damage to the conveyor system or adjacent roadway, a factor of safety of 1.5 is achieved.
- The downstream face of the existing conveyor causeway may be subject to bench-scale instabilities, but these will not compromise the integrity of the completed in-heap storage pond embankment or the existing conveyor. They should be considered as a potential safety hazard and an area of potential maintenance.
- The estimated potential seismic deformations of the heap during operations (EoO, OBE) range from about 0.1 to 5.7 inches. However, due to the general uncertainty inherent with semi-empirical analyses such as those proposed by Bray and Travasarou (2007), the average deformations are assumed to be representative of potential movements under seismic loading. The maximum average deformation of 2.8 inches is considered acceptable and is not anticipated to compromise the integrity of the liner or LCRS.
- The estimated potential seismic deformation of the heap during closure (EoO, MDE) range from approximately 0.3 to 13.2 inches. As above, the maximum average deformation of 6.6 inches should be considered representative and is considered acceptable. Additionally, during closure, the liner and LCRS systems will be breached to mitigate the build-up of pore fluid within the heap, to the extent possible. As such, closure and post-closure deformations caused by seismic events should only be evaluated based on overall slope movement as damage to the liner will be inconsequential.
- The estimated potential seismic deformation of the heap post-closure (Post-C, MDE) range from approximately 0.4 to 15.9 inches. Again, the maximum average deformation of 8.0 inches should be considered representative and is considered acceptable.

- Some surface raveling and slumping of individual heap lift slopes may occur during heavy storms and/or seismic events; however, movements of this type do not typically affect the overall heap stability but may require periodic maintenance during operations.

## 7.2 Settlement Analyses

### 7.2.1 Introduction and Methodology

Settlement analyses were conducted to evaluate anticipated deformations of the pad foundation that could affect the performance of the proposed BCHLF liner system. Settlements are expected to occur due to stresses induced by loading of the BCHLF and due to the significant variation in compressibility and thickness of the foundation materials expected to underlie the BCHLF. The specific objectives were (1) to confirm that the proposed geomembrane will not be subject to strains larger than those that are allowable under commonly accepted design criteria, and (2) to confirm that the base of the in-heap pond LCRS will maintain positive drainage towards the upstream toe of the in-heap storage pond embankment dam to allow removal of potential leakage through the pond primary liner.

Settlement analyses were performed with the computer program Sigma/W (GEO-SLOPE, 2007), which enables the user to conduct finite element stress-strain deformation analyses on two dimensional cross sections input into the software. A variety of constitutive models are available that describe the stress-strain relationships of individual elements within the finite element mesh. Most of the constitutive models also include a failure criteria beyond which unlimited straining can occur without a further increase in stress.

In performing the settlement analyses for the BCHLF, the initial stress state for each cross section considered was first estimated based on completion of foundation preparation and pad grading. Settlements were then estimated at multiple load steps up to, and including, full build-out of the BCHLF. Due to the largely unsaturated conditions expected in conjunction with relatively slow advancement of the leach ore loading, time dependency due to consolidation of saturated materials was not considered. Settlements were assumed to occur quickly relative to the rate of loading.

### 7.2.2 Settlement Section Geometries and Pore-Water Pressure Conditions

Settlements were estimated at the liner interface along three cross-sections (Sections A, C and D) through the proposed BCHLF. The geometries of the sections analyzed were developed using the 1994 ground topography (prior to waste dump and stockpile development), FGMI's planned excavation surface, the design leach pad grading, and the schematic heap configuration at full build-out (Stage 6). The final BCHLF heap configuration, including the locations of the settlement cross-sections analyzed, is shown in plan on Figure 7.2. Section views of the three sections are provided on Figures 1, 2, and 3 (Appendix D-1).

- Section A was selected as the maximum northwest to southeast section through the facility. Its location was selected because it runs through the proposed embankment dam and along the original Barnes Creek streambed, where the largest depths of compressible native alluvial soils exist.
- Section C was selected as a northeast to southwest section in the middle of the proposed BCHLF because a large depth of site grading fill will be placed along this section on the southwest wall of the valley.
- Section D was selected because a large depth of the loose-dumped compressible mine waste rock will be left in place in this location beneath the proposed BCHLF liner system.

Initial pore water pressure conditions within the foundation soils were conservatively modelled with a piezometric line at the surface of the foundation bedrock except along the downstream portion of Section A where a slightly elevated groundwater table was modelled within the foundation soils based on the results of the BCHLF site investigation program (Knight Piésold, 2017a). Due to limitations within the software, a phreatic surface was not included to model pore pressures developed due to the in-heap pond within the BCHLF; however, a saturated unit weight was utilized in those areas to account for the added total stress acting upon the BCHLF liner system and the underlying foundation. This simplification is not expected to

impact the results significantly because the primary goal of the model is to assess potential deformations of the materials underlying the proposed BCHLF leading to strains in the BCHLF liner system.

### 7.2.3 Material Properties

Material properties for geomechanical finite element settlement analyses of the BCHLF are summarized in Table 7.11 (with additional detail in Table 7.12) on the following pages. These material properties were originally established in the BCHLF geotechnical report (Knight Piésold, 2017a). They were developed using a combination of site investigation data, laboratory testing data and literature values as appropriate. Elastic-perfectly plastic constitutive models were used to model the leach ore, site grading fill, mine waste rock and native soils. Failure criteria were adopted based on previously established angles of internal friction. Stress-strain behavior prior to yield was characterized based on the results of measured compressibility in a rigid wall permeability test apparatus or an oedometer. Young's modulus functions were adopted that allow for increasing stiffness with increasing effective confining stress based on the results of the confined compression testing. The foundation bedrock was modelled with a linear elastic constitutive model with stress-strain characteristics adopted from literature values.

### 7.2.4 Settlement Analysis Results

The results of the settlement analyses for Sections A, C, and D are presented on Figures 1, 2, and 3 (Appendix D-1), respectively. The finite element mesh, along with contours of vertical settlements are provided on those figures. In addition, plots showing the expected settlement along the liner interface for each cross-section are provided on Figures 4, 5, and 6 (Appendix D-1) for Sections A, C, and D, respectively. The results indicate maximum settlements of 16.3, 5.6 and 15.5 ft, respectively. In response to comments from an ADNR reviewer, as second set of settlement analyses was conducted that extended each of the three sections by 400 feet to the right and the left. Relocation of the lateral boundary conditions had no effect on model results. These results are slightly smaller than, but are in general agreement with, the results of the simplified analysis conducted previously for the BCHLF PFS design (Knight Piésold, 2015b). In general, the larger settlements occur in locations where loose-dumped mine waste rock and leach ore are left in place beneath the liner system. Settlements at locations where a thick layer of compacted site grading fill underlies the proposed heap leach pad are generally less. Based on the results of Section A, positive drainage along the base of the facility, towards the upstream toe of the BCHLF in-heap pond embankment, are expected throughout the life of the facility.

#### 7.2.4.1 Uniaxial Strains

Uniaxial strains along each cross section were evaluated using the outputs of expected vertical and horizontal displacement along the pad liner system from Sigma/W. Uniaxial strains were evaluated for each increment between finite element nodes along the liner system by first calculating the initial linear distance between the two end points of each increment using their initial horizontal and vertical coordinates. The calculated vertical and horizontal displacements at each end point were then used to calculate the final distance between the end points of each increment after settlement had taken place. Strains were calculated as the final distance minus the initial distance divided by the initial distance. A detail of this calculation is provided in Appendix D-2. Plots of uniaxial strains along Sections A, C, and D are presented on Figures 7, 8 and 9 (Appendix D-1), respectively. On those figures, elongation strains are denoted as positive, while compressive strains are denoted as negative. Maximum elongation strains of 3.9, 4.1, and 1.3 percent are expected along Sections A, C, and D, respectively. Maximum compressive strains of 4.5, 14.8, and 0.7 percent were calculated for Sections A, C, and D, respectively, although it is noted that the geomembrane will not experience such compressive strains as its lack of stiffness will result in buckling (i.e. wrinkling) of the geomembrane rather than the realization of the calculated compressive strains.

**Table 7.11**
**Geotechnical Material Properties used in Settlement Analyses**

| Material | Unified Soil Classification System | Specific Gravity $G_s$ | Moisture Content | | Unit Weight | | | Effective Stress Friction Angle $\phi'$ (deg) | At-Rest Lateral Earth Pressure Coefficient $K_o$ | Poisson's Ratio $\nu$ | Young's Modulus E (psf) | Constitutive Model |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Natural $w_{nat}$ (% by wt.) | Saturated $w_{sat}$ (% by wt.) | Dry $\gamma_{dry}$ (pcf) | Moist $\gamma_m$ (pcf) | Saturated $\gamma_{sat}$ (pcf) | | | | | |
| Leach Ore (Abandoned) | GP-GC (poorly graded GRAVEL with silty clay and sand) | 2.685[1] | 4.0[5] | 12.6[11] | 125.3[12] | 130.3[17] | 141.0[18] | 38[20] | 0.384[24] | 0.278[25] | Varies[27] | Elastic-Plastic |
| Site Grading Fill (Monte Cristo Creek Weathered Schist - Compacted) | GP-GM (poorly graded GRAVEL with silt and sand) | 2.803[2] | 7.3[6] | 8.4[11] | 141.5[13] | 151.8[17] | 153.4[18] | 43[21] | 0.318[24] | 0.241[25] | Varies[27] | Elastic-Plastic |
| Mine Waste Rock (Monte Cristo Creek Weathered Schist - Loose Dumped) | GP-GM (poorly graded GRAVEL with silt and sand) | 2.803[2] | 7.3[7] | 17.0[11] | 118.5[14] | 127.2[17] | 138.6[18] | 36[22] | 0.412[24] | 0.292[25] | Varies[27] | Elastic-Plastic |
| Foundation Soils (Aeolian Silt, Creek Gravels, Residual Soil from Schist, Colluvial Mixtures) | SM and GM (silty SAND with gravel, silty GRAVEL with sand, etc.) | 2.692[3] | 5.1[8] | 14.7[11] | 120.3[15] | 126.4[17] | 138.0[18] | 32[23] | 0.470[24] | 0.320[25] | Varies[27] | Elastic-Plastic |
| Foundation Bedrock (Fairbanks Schist) | N/A | 2.750[4] | 1.9[9,10] | 1.9[10,11] | 163.1[16] | 166.2[10,17] | 166.2[10,19] | N/A | N/A | 0.200[26] | 4.09E+07[28] | Linear Elastic |

Notes:

1. Per testing summarized in Table 3.1 and presented in Appendix D-2 of Knight Piésold (2017a).
2. Per testing summarized in Table 3.4 and presented in Appendix H-2 of Knight Piésold (2017a).
3. Average of testing summarized in Table 3.5 and presented in Appendix I-2 of Knight Piésold (2017a).
4. Estimated based on data from other schistose materials.
5. Median draindown moisture content value from data illustrated on Figure 3.13 and presented in Appendix D-3 of Knight Piésold (2017a).
6. Assumed that site grading fill will be compacted at natural moisture content per testing illustrated on Figure 3.14 and presented in Appendix H-3 of Knight Piésold (2017a).
7. Median natural moisture content value from data illustrated on Figure 3.14 and presented in Appendix H-3 of Knight Piésold (2017a).
8. Median natural moisture content value for soil above the groundwater table from data summarized in Table 3.6 and presented in Appendix I-3 of Knight Piésold (2017a).
9. $\omega_{nat}=(\gamma_m-\gamma_{dry})/\gamma_{dry}*100$
10. Assumed that bedrock is below groundwater table and will remain saturated.
11. $\omega_{sat}=(G_s*\gamma_w-\gamma_{dry})/(G_s*\gamma_{dry})*100$
12. Average dry unit weight at average heap height of 277.1 feet from data illustrated on Figure 4.3 and presented in Appendix D-5 of Knight Piésold (2017a).
13. Assumed that site grading fill will be compacted to 95 percent of modified Proctor maximum dry density per testing summarized in Table 3.4 and presented in Appendix H-4 of Knight Piésold (2017a).
14. Dry unit weight based on the value (16.88 cubic feet per ton) that FGMI utilizes to estimate required waste rock dump volumes.
15. Median value from data summarized in Table 3.6 and presented in Appendix I-3 of Knight Piésold (2017a).
16. $\gamma_{dry}=(\gamma_{sat}-\gamma_w)/(1-1/G_s)$
17. $\gamma_m=\gamma_{dry}*(1+\omega_{nat}/100)$
18. $\gamma_{sat}=\gamma_w+\gamma_{dry}*(1-1/G_s)$
19. Average value from data on Fairbanks schist from 2015 investigation of tailing storage facility.
20. Per testing illustrated on Figure 3.15 and presented in Appendix D-4 of Knight Piésold (2017a).
21. Per testing illustrated on Figure 3.17 and presented in Appendix H-5 of Knight Piésold (2017a).
22. Friction angle approximated as inclination of angle of repose slopes on Fort Knox mine waste rock dumps.
23. Per testing illustrated on Figure 3.18 and presented in Appendix I-4 of Knight Piésold (2017a).
24. $K_o \approx 1-\sin\phi'$ after Lambe and Whitman (1969).
25. $\nu \approx K_o/(1+K_o)$ for drained loading after GEO-SLOPE (2012).
26. Krynine and Judd (1957) report Poisson's ratio $\nu$=0.08 to 0.20 for schist; 0.20 selected for analyses.
27. Young's modulus varies with confining stress as summarized in Table 7.12.
28. Krynine and Judd (1957) report Young's modulus E=284 to 569 ksi for altered schist; 284 ksi selected for analyses.

**Table 7.12**
**Summary of Young's Modulus Versus Effective Confining Stress Data**

| Vertical Effective Confining Stress $\sigma'_v$ (psf) | Leach Ore Young's Modulus [1] E (psf) | Site Grading Fill Young's Modulus [2] E (psf) | Mine Waste Rock Young's Modulus [3] E (psf) | Foundation Soils Young's Modulus [4] E (psf) |
|---|---|---|---|---|
| 1 | 4.55E+03 | 3.11E+04 | 2.22E+04 | 2.60E+04 |
| 10,000 | 1.58E+05 | 8.30E+05 | 1.96E+05 | 2.37E+05 |
| 20,000 | 3.37E+05 | 1.66E+06 | 3.84E+05 | 4.70E+05 |
| 30,000 | 5.28E+05 | 2.51E+06 | 5.80E+05 | 7.14E+05 |
| 40,000 | 7.29E+05 | 3.37E+06 | 7.81E+05 | 9.65E+05 |
| 50,000 | 9.37E+05 | 4.23E+06 | 9.86E+05 | 1.22E+06 |
| 60,000 | 1.15E+06 | 5.10E+06 | 1.20E+06 | 1.49E+06 |
| 70,000 | 1.37E+06 | 5.98E+06 | 1.41E+06 | 1.75E+06 |
| 80,000 | 1.59E+06 | 6.86E+06 | 1.62E+06 | 2.03E+06 |
| 90,000 | 1.82E+06 | 7.74E+06 | 1.84E+06 | 2.30E+06 |
| 100,000 | 2.05E+06 | 8.63E+06 | 2.06E+06 | 2.58E+06 |

Notes: 1. From compressibility relationship on Figure 3.21 and data in Appendix D-5 of Knight Piésold (2017a).
2. From compressibility relationship on Figure 3.24 and data in Appendix H-6 of Knight Piésold (2017a).
3. From compressibility relationship on Figure 3.25 and data in Appendix H-6 of Knight Piésold (2017a).
4. From compressibility relationship on Figure 3.26 and data in Appendix I-5 of Knight Piésold (2017a).

#### 7.2.4.2 Biaxial Strains

Biaxial elongation strains are two-dimensional strains quantifying aerial enlargement within the liner system. These are typically larger than uniaxial elongation strains because the liner system is being stretched in multiple directions. These strains were estimated by elongating a unit circle into an ellipse by increasing the circle radii in each direction by the calculated maximum elongation strains. Based on the results of the two-dimensional analyses conducted, biaxial strains were calculated based on the locations where Section A intersects Sections C and D. To provide conservative results, the maximum biaxial elongation strains were calculated using the maximum uniaxial elongation strains from Sections A (3.9 percent), C (4.1 percent), and D (1.3 percent) instead of the elongation strains specific to the cross-section intersections. The details of this calculation are provided in Appendix D-3. The results estimate maximum axisymmetric elongation strains of 8.2 and 5.3 percent for the intersections of Sections A and C, and Sections A and D, respectively. These conservatively-derived strains are deemed acceptable in comparison with the nominal 8.0-percent maximum allowable multiaxial strain suggested by Peggs, et al. (2005) for textured LLDPE geomembrane (see Appendix E).

### 7.2.5 Settlement Conclusions

The results of the settlement analyses presented herein indicate that expected settlements at the BCHLF liner system are acceptable and within allowable limits. The results suggest the base of the in-heap pond is expected to maintain positive drainage towards the upstream toe of the in-heap pond embankment to promote the operation of the LCRS. Additionally, the maximum expected uniaxial elongation and biaxial elongation strains of the BCHLF geomembrane liner are approximately 4.1 and 8.2 percent, respectively, which are deemed acceptable in comparison with the nominal 8-percent maximum allowable strain suggested by Peggs, et al. (2005), which is included in Appendix E. The estimated settlements beneath the facility due to loading of the BCHLF ore heap are not expected to adversely affect the integrity of the leach pad liner system. Settlement strains were estimated under the fully loaded heap, through the Stage 6 configuration, because this is when the largest settlements can be expected to occur.

# Section 8.0 - Operational Period Process Water Balance

A process water balance was carried out for the design and operation of the solution management system for the BCHLF. The objectives of the operational period water balance analysis were:

- To identify the monthly fluctuations of water volumes in the BCHLF process circuit for the normal operating life of the facility,
- To provide an estimated range of potential BCHLF water deficits and surpluses that may occur during operations to estimate the amounts of external water required as makeup, and to estimate the amounts to be discharged that may require neutralization treatment, and
- To confirm the adequacy of the volumetric components of the in-heap storage pond and assess required pumping capacities.

The sizing for the in-heap storage pond used in the water balance considered a minimum pond operating level (for pump submergence) and included allocated storage for the 100-yr/24-hr storm event and the 24-hr emergency draindown from the heap.

The operational period water balance contemplates a start date of September 2019 for loading and leaching through to approximately October 2026.

## 8.1 Definitions

The following terminology has been used herein when describing the water balance and the results.

- **Draindown Moisture Content** – The moisture content, based on dry weight, that the ore will reach after free-drainage of solution from the ore following the cessation of leaching.
- **Draindown Volume** – Two types of draindown volumes are considered as follows:
  - **Total Draindown Volume** – The volume of water that would be released from the actively leached ore when the heap leach facility ceases to operate for a lengthy period and the column of solution fully drains. It is the difference between the under leach and draindown moisture contents multiplied by the volume of ore that is being actively leached.
  - **24-hr Draindown Volume** – The portion of the Total Draindown Volume estimated to be released during the first 24 hours (based on full solution flow) of draindown. The in-heap pond capacity accounts for the 24-hr Draindown Volume as a contingency for temporary loss of power or pumps.
- **Evaporation** – The process whereby liquid water gains sufficient energy to enter the gaseous, or vapor, state. Several types of evaporation are used in the water balance such as potential evaporation, evaporation from leach areas, evaporation from unleached areas, and evaporation from ponds (see Section 8.3).
- **In-Heap Pond Operational Levels** – The normal pond operational levels range between a minimum that still provides sufficient submergence for pump operation, and a maximum that still maintains sufficient storage allowance within the pond for the design storm event and emergency draindown volumes throughout the BCHLF development.
- **Leach Cycle Time** – The length of time that solution is applied to a defined heap surface area to remove a portion of the mineral from the ore.
- **Leach Area** – The heap surface area over which solution is applied at any point in time. The leach area is directly related to the solution application rate and the total solution flow rate.
- **Under Leach Moisture Content** – The moisture content, based on the dry weight of the ore, during steady-state solution application.

- **Natural Moisture Content** – The moisture content, based on the dry weight of the ore, as placed in the heap prior to solution application.
- **Pad Contributing Area** – The 2-dimensional area of the heap leach pad that contributes runoff to the in-heap storage pond. In the water balance, the pad contributing area includes all areas within the crest of the geomembrane-lined leach pad perimeter berm (i.e. the active and inactive areas of the heap as well as the surrounding drainage blanket and liner areas). It is the total area assumed to receive precipitation that is either conveyed to the in-heap storage pond, absorbed by the ore, or lost to evaporation or sublimation. All precipitation falling outside of the pad contributing area is captured by diversion ditches and conveyed away from the leach pad. During the operating life of the BCHLF, the pad contributing area is estimated to be expanded in approximately six stages as shown in Table 8.1.

**Table 8.1**
**Estimated Total Contributing Pad Area during BCHLF Development**

| Stage | Estimated Construction Completion Date | Incremental Pad Contributing Area ($ft^2$) | Total Pad Contributing Area ($ft^2$) |
|---|---|---|---|
| 1 | Aug 31, 2019 | 2,083,555 | 2,083,555 |
| 2 | Sep 30, 2020 | 2,104,666 | 4,188,221 |
| 3 | Sep 30, 2021 | 2,015,011 | 6,203,232 |
| 4 | Sep 30, 2022 | 2,000,538 | 8,203,769 |
| 5 | Sep 30, 2023 | 2,470,876 | 10,674,645 |
| 6 | Sep 30, 2024 | 1,906,176 | 12,580,821 |

- **Solution Application Rate** – The average rate at which solution is applied to the heap surface per unit area. This rate may fluctuate during the leach cycle based on flow characteristics of the ore, operator preference, and mineral-specific characteristics.
- **Sublimation** – The direct conversion of snow or ice to vapor without passing through the liquid state.
- **Total Solution Flow Rate** – The rate at which solution is delivered to the heap. The total solution flow rate is directly related to the solution application rate and leach area. Commonly, the leach area is determined as the total solution flow rate divided by the solution application rate.

## 8.2 Input Parameters

The general input criteria for the BCHLF operational period water balance is summarized in Table 8.2 on the following page.

**Table 8.2**
**Water Balance Input Criteria**

| Item | Design Criteria | Source |
|---|---|---|
| Loading and Leaching | | |
| Expected dates of operation | | |
| *start of loading* | *Sep 1, 2019* | FGMI, KP |
| *end of loading* | *Jul 2027* (**see Note 1**) | FGMI, C |
| *end of operational leaching* | *Oct 2026* (**see Note 2**) | KP |
| Ore loading rates (**see Note 1**) | | |
| *typical* | *123,000 tpd (Apr though Nov, 244 days/year, 30 M tpa)* | FGMI |
| *year 2019* | *70,000 tpd (Sep), 120,000 tpd (Oct and Nov)* | KP |
| *years 2020 and 2021* | *123,000 tpd (Jul through Nov)* | KP |
| Leach cycle | 60 days | FGMI |
| Solution application rate | 0.005 gpm/ft$^2$ | FGMI |
| Total solution flow rate | 16,000 gpm (max) | FGMI |
| Area under leach | 3.2 M ft$^2$ (max) | C |
| Method of application | Drip emitters (buried in winter) | FGMI |
| In-Heap Storage Pond | | |
| Base elevation | 1450 fmsl | FGMI, KP |
| Crest elevation | 1545 fmsl | FGMI, KP |
| Spillway invert elevation | 1542 fmsl | KP |
| Key storage components (**see Note 3**) | | |
| • Minimum operating depth | 44 ft | FGMI, KP |
| *pump submergence* | *24 ft* | FGMI |
| *well drawdown contingency* | *20 ft (approximate)* | FGMI, KP |
| • Operational flexibility | The range between the minimum level (to provide sufficient submergence for pump operation) and maximum level (to still maintain sufficient storage for other key components) | FGMI, KP |
| • Design storm event | 100-yr/24-hr recurrence (volume varies based on pad area) | FGMI, KP |
| • Emergency draindown | 24-hr duration (~23 M gal typ based on 16,000 gpm flowrate) | FGMI, KP |
| • Freeboard | 5 ft (to remain consistent with the WCHLF) | FGMI |
| Ore Properties | | |
| Average in-place dry density | 125.3 pcf | KP |
| Permeability | k = 1.8×10$^{-1}$ cm/sec | KP |
| Gravimetric moisture contents | | |
| *natural* | *2.6* | FGMI, KP |
| *under leach* | *5.4* | FGMI, KP |
| *residual (draindown)* | *4.0* | FGMI, KP |
| Porosity | 21.8 percent (avg in in-heap pond based on ultimate heap) | KP, C |

Notes:
1. The end of loading is based on the schematic heap geometry and understood approach to loading of the ore heap. Reduced loading rates and/or periods of loading are required for the first three years of the BCHLF operation based on the estimated availability of pad area.
2. The operational period water balance was modeled through the end of October 2026. While loading and leaching is expected to continue after this date, this is when the available top area diminishes to the point that a reduced solution flow and/or residual leaching of side-slopes should be considered.
3. The design storm event volume and operational flexibility are inversely related, and will fluctuate as the pad area is progressively expanded. The operational flexibility will also be impacted by climatological fluctuations resulting from the operational period process water balance.

In addition to the input criteria listed in Table 8.2, the water balance input data also included site climatological data and the leach pad and ore heap geometries. The site climatological data used for the BCHLF operational period process water balance consisted of 102 years of actual and synthetic precipitation and evaporation data from 1915 to 2016. The analyses of these data are presented in the BCHLF Climatic/Hydrologic Characterization Report (Knight Piésold, 2016b).

Based on the schematic BCHLF final design heap configurations (Drawings 600 through 610), a conceptual loading and construction schedule was developed for the life of the facility (see Table 6.7). Using this information and the solution flow criteria from Table 8.2, the following additional input data was developed and is presented graphically in Appendix F:

- **Figure 1** (Appendix F) presents the relationships of pad and heap footprint areas over time. The difference between the pad and heap footprint areas represents the exposed lined pad area.
- **Figure 2** (Appendix F) presents the total barren solution flows to the heap over time. The total barren solution flows are limited by the top area of the heap available for leaching. After an initial ramp-up period, the target barren solution flow of 16,000 gpm can be achieved in approximately August 2020.
- **Figure 3** (Appendix F) presents the average ore thicknesses beneath the top areas of the heap under leach and takes into consideration the locations of the under-leach areas. The under-leach ore thicknesses affect the time required for applied solution to report to the in-heap pond. In addition, the under-leach ore thicknesses help define the associated solution uptake volume (that will temporarily be stored within the heap during leaching) and solution draindown volume (that will eventually report to the in-heap pond).

## 8.3 Water Balance Model

A GoldSim water balance model developed by Knight Piésold was used to simulate the water transfers and storages for the BCHLF. GoldSim is a simulation software used for dynamic modeling of complex systems. GoldSim supports decision and risk analysis by simulating future performance while quantitatively representing the uncertainty and risks inherent in all complex systems (GoldSim, 2014).

In general, the water balance model estimates the monthly change in the free water content of the heap leach system based on precipitation, evaporation, sublimation, and absorption or release of moisture from the ore. The water balance equation can be written as follows:

$$MC = I - O$$

Where: $MC$ = Net monthly change in the free water content of the heap leach system
$I$ = Monthly volume of inflow to the heap leach system from precipitation, solution flow, and release of moisture from the ore during draindown
$O$ = Monthly volume of outflow from the heap leach system due to evaporation, solution removal, sublimation, or uptake of moisture by the ore in going from its natural to under leach moisture content

The water balance assumes that water may be added to the system as:

1. Leaching solution sourced from the TSF pond,
2. Precipitation in the form of rain or snow, or
3. Natural moisture in the mined ore.

Water may be removed from the system by:

1. Evaporation or sublimation, or
2. Pumping from the in-heap pond to another storage facility, CIC processing, or to neutralization treatment and discharge to the TSF.

Water may be temporarily or permanently stored in the system as:

1. Water used in bringing the ore from its natural moisture content to its under leach moisture content,
2. Water stored in the pore spaces of the heap (including areas under leach, residual moisture in drained areas of the heap, and storage within the in-heap pond), or
3. Water stored as snow on the surface of the heap.

Evaporative losses from the system may be enhanced during the summer months when there is an excess of water in the system by using sprinklers or sprays on the heap surface in-lieu of drip emitters.

Other assumptions used in developing the water balance are:

- Leaching of the ore will be conducted on a year-round basis throughout the operational period,
- Residual leaching, rinsing, and closure of the facility is expected to follow the operational period, but are not modeled within this operational period water balance,

Loading of the BCHLF ore heap has been modeled to commence on September 1, 2019. The ore heap will be loaded in a series of 18 lifts; each with a nominal thickness of 50 ft. Each lift of ore will form a level platform for the subsequent loading of the next lift. While a lift is being loaded, the surface of the active leach area may be comprised entirely of a portion of the previous lift, or it may consist partly of both the previous and new lifts. The active leach area will be limited by the basin geometry at the beginning of the project life, but will gradually increase to 3.2 M $ft^2$ near the completion of Lift 4 (August 2020). The active leach area will then remain constant at 3.2 M $ft^2$ from August 2020 through October 2026. The operational period water balance was modeled through the end of October 2026. While loading and leaching is likely to continue after this date, this is when the available top area diminishes to the point that solution flow will need to reduce and/or residual leaching of side-slopes should be considered.

### 8.3.1 Model Methodology

Knight Piésold's GoldSim water balance model is based on historical climatic and hydrologic data to evaluate seasonal, annual, and decadal patterns that have occurred in the past to more accurately identify extreme (wet and dry) conditions that may occur in the future. The model is run on a monthly time-step and the results are summarized and presented on a monthly-basis. It utilizes the Index Sequential Method (ISM) to simulate possible combinations of historical climatic events and mine operational conditions.

The ISM relies on historical climate data to generate a set of equally likely climate realizations (runs) for given future operational conditions. For example, to simulate a 5-year water balance (January 2016 through December 2020) using the site-specific 102-year climate record (1915 through 2016), climate years 1 through 5 (January 1915 through December 1919) are applied to the first run, climate years 2 through 6 (January 1916 through December 1920) for the second run, and so on. When there are fewer than 5 years of climate data remaining in the record after the selected start year, the climate data loops back to the beginning of the record. For example, if the starting climate year is 2014, the simulation uses data from years 2014 through 2016 followed by data from years 1915 through 1916 to generate the 5 years of climate data. The process continues until each of the 102 years has been used as the starting point. The result from the model is then 102 independent possible outcomes (e.g., volumes, flows, etc.) for each month that the facilities will be in operation based on the varying climate record and operational conditions.

The outcomes from the model are analyzed using deterministic and probabilistic approaches. The deterministic analyses summarize the direct outcomes from the model by calculating the minimum, average, and maximum values for the various monthly water volumes and flows tracked within the model. These analyses are limited to the historic climate record, which may not contain a sufficiently long record to have captured extreme (wet and dry) events that are possible. Because of this, probabilistic analyses are performed to estimate various percent chances of exceedance values for the various monthly water volumes and flows tracked within the model. The probabilistic analyses include necessary statistical

parameters (e.g., average, standard deviation, skewness, etc.) and have the capability of extrapolating beyond the deterministic outcomes using a Gumbel Extreme Type I probability distribution. Thus, the probabilistic analyses are not limited to the deterministic data and can quantify the likelihood of experiencing the predicted outcomes (e.g., volumes, flows, etc.).

### 8.3.2 Storage Facility Volume Capacity Estimation Methods

Storage facility volume capacity requirements for the BCHLF were estimated using the deterministic and probabilistic methods. These analyses estimate the maximum storage requirements for the facility per the established design criteria. The method that produces the largest storage volume (deterministic or probabilistic) is typically selected as the requirement for the design of the storage facility. The probabilistic volume will likely be greater than the deterministic volume if there is a large standard deviation associated with the outcomes from the model (i.e. large storage volume fluctuations over time). The following describes the two methods used for estimating the required storage capacity for the BCHLF in-heap pond:

- *Deterministic Method – Storm Event Volumes*
  The BCHLF deterministic analyses were based on the deterministic outcomes from the model, the design storm event criteria, and the 24-hr draindown. Deterministic analyses typically combine the predicted monthly storage volumes, the water volumes generated from the monthly design storm events, and the 24-hr draindown volume to estimate monthly storage requirements. The draindown and design storm event volumes for the BCHLF are pre-defined by other criteria (i.e. total solution flow, pad footprint, and design storm event precipitation). Therefore, the deterministic average storage volumes were forced to stay below the in-heap pond level required to maintain capacity for both the design storm event and the 24-hr draindown components. For the deterministic approach, any solution exceeding this in-heap pond level was sent to neutralization treatment and discharged to the TSF.
- *Probabilistic Method – Percent-Chances-of-Exceedance Volumes*
  The BCHLF probabilistic analyses were based on the deterministic outcomes from the model and the Gumbel Extreme Type I probability distribution. Frequency analyses were performed on the predicted storage volumes for each month that the storage facility is in operation and thus encompass the wider range of potential climatological conditions. The results of the frequency analyses are monthly percent-chances-of-exceedance storage capacities. For the BCHLF probabilistic approach, the 1-percent chance-of-exceedance plus the 24-hr draindown volumes were selected as the design basis. Therefore, the probabilistic storage volumes were forced to stay below the in-heap pond level required to maintain capacity for the 24-hr draindown component. For the probabilistic approach, any solution exceeding this in-heap pond level was sent to neutralization treatment and discharged to the TSF.

## 8.4 Water Balance Results

The results of the BCHLF operational period water balance are presented in Appendix F (Figures 4 through 11). The information presented is discussed as follows:

- **Figure 4** (Appendix F) presents the required makeup flows from the TSF to the BCHLF. Based on the 50-percent chance-of-exceedance condition, up to 9,000 gpm makeup will be required during the first two years of operation for initial wetting-up of the ore, while the remaining years will require significantly less on a periodic basis.
- **Figure 5** (Appendix F) presents the cumulative makeup requirements as total flow volumes from the TSF (see Figure 4).
- **Figure 6** (Appendix F) presents the pregnant solution pumping rate from the BCHLF in-heap pond to the CIC plant. The flows pumped to CIC processing will be limited during the start of operations with approximately 16,000 gpm being reached around November 2020. Fluctuations in pregnant solution flow may occur over the project life due to wetting-up of fresh ore and/or drier climatic periods, with a lower-bound of approximately 15,000 gpm based on the 90-percent chance-of-exceedance condition.
- **Figure 7** (Appendix F) presents the probabilistic monthly volumes for the BCHLF in-heap pond in relation to the in-heap pond emergency spillway invert (1542 fmsl) volume and the pond volume required to

maintain capacity for the 24-hr draindown. The required volume to accommodate the 24-hr draindown condition will vary at the start of the project based on the barren solution flowrate to the heap. As illustrated, the typical 24-hr draindown volume of 3.08 M ft3, or 23.04 M gal based on the 16,000 gpm application rate, is accommodated throughout the operational period. To achieve the conditions presented, neutralization treatment will be required (see Figure 9) in accordance with trigger levels at which time the treatment must be started (see Figure 11). The minimum pond volume presented (~3.8 M $ft^3$), is the volume associated with the design criteria minimum operating depth (44 ft, 1494 fmsl).

- **Figure 8** (Appendix F) presents the deterministic-average in-heap pond volumes inclusive of the monthly volumes estimated from the 100-yr/24-hr storm event plus the 24-hr draindown. As illustrated, under the average deterministic conditions, the design components are managed to stay within the BCHLF in-heap pond storage capacity requirements throughout the operational period. To achieve the conditions presented, neutralization treatment will be required (see Figure 9) in accordance with trigger levels at which the treatment must be started (see Figure 11).
- **Figure 9** (Appendix F) presents the estimated pumping that will be required for the operational period of the BCHLF from the CIC plant to neutralization treatment and subsequent discharge to the TSF. These pumping rates are required to manage water volumes within the in-heap pond and achieve the conditions shown on Figures 7 and 8. As illustrated, estimated monthly-basis treatment rates of up to 3,800 gpm may be required during the operational period. This does not account for increased pumping rates that may be required due to availability/operational factors. Under normal (50-percent chance of exceedance) conditions, required pumping and neutralization will fluctuate from zero to approximately 800 gpm. Water treatment from the BCHLF is estimated to be required starting in March 2020 at approximately 250 gpm. When neutralization treatment is conducted, additional pumping from the in-heap pond (at the same rate as neutralization) will be required directly to the barren circuit.
- **Figure 10** (Appendix F) presents the cumulative neutralization treatment requirements as total flow volumes from the BCHLF (from the CIC plant) that will be discharged to the TSF (see Figure 9).
- **Figure 11** (Appendix F) presents the BCHLF in-heap pond operational period trigger elevations for pumping from the CIC plant to neutralization treatment. Pumping in accordance with these trigger elevations is required to manage water volumes within the in-heap pond and achieve the conditions shown on Figures 7 and 8.

## 8.5 Water Balance Summary

Key points from the BCHLF operational period process water balance include:

- The operational period water balance reflects a start date of September 2019 and models conceptual loading of the heap through approximately October 2026
- Initial heap loading (first 3 years) is limited by construction of the pad area and the maximum loading rate
- Barren solution flow is limited by the top area of the heap, with the target flowrate of 16,000 gpm achievable by approximately August 2020
- Makeup flow of up to 9,000 gpm (based on the 50-percent chance of exceedance) will be required during the first two years of operation, while the remaining years will require significantly less on a periodic basis
- The operational period water balance considered deterministic and probabilistic analyses to confirm the in-heap pond storage capacity
  - Deterministic – average + 100-yr/24-hr storm event + 24-hr draindown
  - Probabilistic – 1-percent chance of exceedance + 24-hr draindown
- To achieve storage of the required components within the in-heap pond, neutralization treatment will be required in accordance with designated trigger levels
  - 250-gpm capacity starting March 2020
  - 2,800-gpm capacity by 2021 with a gradual increase to 3,800-gpm capacity by 2026
  - The treatment rates presented herein do not account for increased pumping that may be required due to availability/operational factors

# Section 9.0 - Construction Material Take-Off Quantities

Construction MTO quantity estimates were completed for all identified components of the BCHLF specifically within Knight Piésold's work scope. The MTOs herein have been estimated based predominantly on the design elements presented on Knight Piésold's current drawings (as noted in the Table of Contents herein). Notable changes from the Revision 0 MTOs are primarily due to changes to the number and sizes of PCMS channels, PCMS sump sizes, additional piezometers required, one additional underdrain monitoring well and the extension of the well depths to the estimated bedrock contact, and the removal of the south perimeter diversion outlet channel.

As with other recent work associated with the WCHLF, unit rates and cost estimates associated with the BCHLF MTO quantities will be completed by FGMI. The unit rates developed should be based on the most recent technical specifications and design drawings for the BCHLF. The MTO quantities herein generally include for the following:

- Site grading cut and fill (only broken out for Stage 1 with Stages 2 through 6 combined)
- Underdrains, PCMS, and LCRS
- Prepared subbase and liner system (including double lining of the in-heap storage pond)
- Overliner and solution collection pipework
- Solution extraction wells (inclined and vertical)
- In-heap storage pond emergency spillway and outlet channel
- Geotextile and erosion protection for diversion channels
- Wearing course for pad perimeter roads
- Culvert crossings (for perimeter diversions and on-heap access)
- Steepened slope reinforcement (external to leach pad)
- Instrumentation

The construction MTO quantities herein do not account for the following:

- The pregnant and barren pipeline corridor (adjacent to the BCHLF and between the pad and plant site)
- Haul roads (external to the leach pad) and/or haul ramps (on-heap)
- Pumps and accessories (unless otherwise noted)

The following notes should be considered when using the MTO quantities included herein:

1. The general earthwork quantities represent comparison of a compiled FGMI proposed excavation and future waste dump surface versus the KP design surfaces. Significant excess fill will be required based on FGMI's proposed excavation. It is assumed this excess fill will be sourced from mine waste.
2. Underdrain excavation, aggregate, site grading fill, and geotextile quantities were increased by 15 percent to account for field adjustments to intercept unforeseen seeps and springs.
3. CPT Pipework and geosynthetics quantities have been increased by 5 and 10 percent, respectively (for procurement purposes), to account for seam overlaps, waste, and scrap during construction.
4. Earthwork quantities are neat. Bulking and shrinkage factors have not been applied.

Figure 9.1 presents the comparative volume surface (i.e. depths of cut and fill) associated with the pad grading plan versus FGMI's proposed excavation surface.

The MTO quantities for the BCHLF design are presented in Table 9.1 on the following pages. Additional notes should be referenced on the last page of Table 9.1.

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**Table 9.1**
**Staged Construction Material Take-Off Quantities, Rev 1 (Final Design)**

| Item No. | Description | Unit | Staged Material Take-Off Quantities | | | | | | Total Quantity |
|---|---|---|---|---|---|---|---|---|---|
| | | | Stage 1 (EL 1545) | Stage 2 (EL 1640) | Stage 3 (EL 1730) | Stage 4 (EL 1820) | Stage 5 (EL 1950) | Stage 6 (Final) | |
| **EARTHWORKS** | | | | | | | | | |
| | **Earthworks - Leach Pad, Perimeter Roads, and Diversions (includes south perimeter diversion outlet channel)** | | | | | | | | |
| 1 | Site grading cut (**See Note 1**) | cy | **439,420** | 1,674,056 | | | | | **2,113,476** |
| 2 | Site Grading Fill (**See Note 1**) | cy | 770,269 | 4,536,353 | | | | | 5,306,621 |
| 3 | Excess Site Grading Fill required (assumed from mine waste) | cy | **330,848** | 2,862,297 | | | | | **3,193,145** |
| 4 | Load, haul, place, shape and compact Site Grading Fill for ultimate pad perimeter berm | cy | 532 | 379 | 359 | 646 | 1,486 | 2,287 | 5,689 |
| 5 | Process and stockpile Prepared Subbase (includes material for leach pad basin and perimeter berms) | cy | 80,409 | 82,326 | 78,724 | 78,284 | 95,704 | 73,822 | 489,268 |
| 6 | Load, haul, place, and compact Prepared Subbase from stockpile (12-inch-thick layer, includes rough grade surface compaction) | cy | 80,409 | 82,326 | 78,724 | 78,284 | 95,704 | 73,822 | 489,268 |
| 7 | Prepare surface of Prepared Subbase for geomembrane placement (surficial rock removal) | sy | 241,228 | 246,977 | 236,171 | 234,852 | 287,111 | 221,466 | 1,467,805 |
| 8 | Crush, screen, and stockpile mill reject for Overliner (minus 2-inch) | cy | 238,412 | 243,247 | 234,102 | 232,726 | 284,306 | 218,243 | 1,451,036 |
| 9 | Load, haul, and place Overliner from stockpile (36-inch-thick layer, includes local compaction of Overliner around header pipes) | cy | 238,412 | 243,247 | 234,102 | 232,726 | 284,306 | 218,243 | 1,451,036 |
| 10 | Excavate and backfill geomembrane termination trench (approx. 6 cf per ft) | ft | 7,200 | 7,938 | 8,228 | 8,209 | 7,744 | 5,615 | 44,933 |
| 11 | Crush, screen, and stockpile Road Wearing Course (minus 3-inch) | cy | 2,879 | 2,636 | 2,699 | 2,902 | 3,375 | 3,327 | 17,818 |
| 12 | Load, haul, place, and compact Road Wearing Course from stockpile (6-inch-thick layer) | cy | 2,879 | 2,636 | 2,699 | 2,902 | 3,375 | 3,327 | 17,818 |
| 13 | Process and stockpile Riprap, D50=18" | cy | 186 | 1,008 | 892 | 874 | 781 | 3,174 | 6,915 |
| 14 | Process and stockpile Riprap, D50=9" | cy | **72** | 0 | 0 | 0 | 1,769 | 1,639 | **3,481** |
| 15 | Load, haul, and place Riprap in perimeter diversion channels, D50=18" (36-inch-thick layer) | cy | 186 | 1,008 | 892 | 874 | 781 | 3,174 | 6,915 |
| 16 | Load, haul, and place Riprap in perimeter diversion channels, D50=9" (18-inch-thick layer) | cy | **72** | 0 | 0 | 0 | 1,769 | 1,639 | **3,481** |
| | **Earthworks - Underdrain System** | | | | | | | | |
| 17 | Excavate leach pad underdrain trenches (primarily above valley bottom) (**See Note 2**) | cy | 8,452 | 8,656 | 8,392 | 8,512 | 9,944 | 6,071 | 50,026 |
| 18 | Load, haul, and dump overburden soils from underdrain trenches to unsuitable or suitable stockpile (**See Note 2**) | cy | 6,339 | 6,492 | 6,294 | 6,384 | 7,458 | 4,553 | 37,520 |
| 19 | Stockpile uncrushed mill reject for Underdrain Aggregate (main underdrains, finger drains, and URF drainage wells) (**See Note 2**) | cy | **6,367** | 6,492 | 6,294 | 6,384 | 7,458 | 4,553 | **37,548** |
| 20 | Load, haul, place, and compact Underdrain Aggregate into underdrain trenches and URF drainage wells from stockpile (**See Note 2**) | cy | **6,367** | 6,492 | 6,294 | 6,384 | 7,458 | 4,553 | **37,548** |
| 21 | Place and compact Site Grading Fill into underdrain trenches from trench excavation (**See Note 2**) | cy | 2,113 | 2,164 | 2,098 | 2,128 | 2,486 | 1,518 | 12,507 |
| 22 | Excavate Underdrain Random Fill trenches (in valley bottom) | cy | 22,948 | 5,720 | 2,019 | 0 | 0 | 0 | 30,686 |
| 23 | Load, haul, and dump overburden soils from Underdrain Random Fill trenches to unsuitable or suitable stockpile | cy | 22,948 | 5,720 | 2,019 | 0 | 0 | 0 | 30,686 |
| 24 | Drill Underdrain Random Fill (URF) drainage wells in transverse drain (7 wells, 30-ft deep each) | ft | **210** | 0 | 0 | 0 | 0 | 0 | **210** |
| 25 | Stockpile Underdrain Random Fill | cy | 19,042 | 4,746 | 1,675 | | | | 25,463 |
| 26 | Load, haul, place, and compact Underdrain Random Fill into valley-bottom trenches from stockpile | cy | 19,042 | 4,746 | 1,675 | | | | 25,463 |
| 27 | Place and compact Site Grading Fill into Underdrain Random Fill trenches from trench excavation | cy | 3,906 | 974 | 344 | | | | 5,223 |
| | **Earthworks - Process Component Monitoring System (PCMS)** | | | | | | | | |
| 28 | Excavate leach pad PCMS sumps (along Stage 1 perimeter) and channels | cy | **49** | **3,159** | **1,987** | **1,931** | **2,360** | **1,534** | **11,019** |
| 29 | Load, haul, and dump overburden soils from PCMS sump and channel excavations to unsuitable or suitable stockpile | cy | **49** | **3,159** | **1,987** | **1,931** | **2,360** | **1,534** | **11,019** |
| 30 | Process and stockpile Prepared Subbase for PCMS channels, sumps, and outlets | cy | **31** | **1,115** | **701** | **681** | **833** | **541** | **3,903** |
| 31 | Load, haul, place, and compact Prepared Subbase in PCMS channels, sumps, and outlet from stockpile (includes rough grade surface compaction) | cy | **31** | **1,115** | **701** | **681** | **833** | **541** | **3,903** |
| 32 | Crush, screen, and stockpile crushed mill reject for PCMS Drain Aggregate (minus 2-inch) | cy | **24** | **2,044** | **1,285** | **1,249** | **1,527** | **993** | **7,122** |
| 33 | Load, haul, and place PCMS Drain Aggregate in PCMS channels from stockpile | cy | **24** | **2,044** | **1,285** | **1,249** | **1,527** | **993** | **7,122** |
| 34 | Load, haul, and place PCMS Drain Aggregate in PCMS sump from stockpile | cy | **24** | 0 | 0 | 0 | 0 | 0 | **24** |
| 35 | Excavate PCMS header pipe trench under Stage 1 inter-stage perimeter road | cy | **129,879** | 0 | 0 | 0 | 0 | 0 | **129,879** |
| 36 | Supply, place, and compact Pipe Bedding in PCMS header trench | cy | **556** | 0 | 0 | 0 | 0 | 0 | **556** |
| 37 | Supply, place, and compact Pipe Backfill in PCMS header trench | cy | **626** | 0 | 0 | 0 | 0 | 0 | **626** |
| 38 | Replace and compact Site Grading Fill in PCMS header trench | cy | **3,622** | 0 | 0 | 0 | 0 | 0 | **3,622** |
| 39 | Construct PCMS channel termination at inter-stage perimeter road | ea | 0 | **5** | **5** | **5** | **2** | 0 | **17** |

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**Table 9.1**
**Staged Construction Material Take-Off Quantities, Rev 1 (Final Design)**

| Item No. | Description | Unit | Staged Material Take-Off Quantities | | | | | | Total Quantity |
|---|---|---|---|---|---|---|---|---|---|
| | | | Stage 1 (EL 1545) | Stage 2 (EL 1640) | Stage 3 (EL 1730) | Stage 4 (EL 1820) | Stage 5 (EL 1950) | Stage 6 (Final) | |
| | **Earthworks - In-Heap Storage Pond Leachate Collection and Recovery System (LCRS)** | | | | | | | | |
| 40 | Crush, screen, and stockpile mill reject for LCRS Aggregate (Overliner, minus 2-inch) | cy | 43,765 | | | | | | 43,765 |
| 41 | Load, haul, and place LCRS Aggregate (Overliner) from stockpile to pond floor LCRS (36-inch-thick layer and 6-ft-deep sump) | cy | 43,765 | | | | | | 43,765 |
| 42 | Crush, screen, and stockpile mill reject for LCRS Drain Aggregate (minus 0.5-inch) | cy | 149 | | | | | | 149 |
| 43 | Load, haul, and place LCRS Drain Aggregate from stockpile to LCRS pump cans surround zone (24-inch-thick layer) | cy | 149 | | | | | | 149 |
| 44 | Process and stockpile Prepared Subbase for LCRS (beneath LCRS drain aggregate and secondary geomembrane) | cy | 16,671 | | | | | | 16,671 |
| 45 | Load, haul, place, and compact Prepared Subbase from stockpile (12-inch-thick layer, includes rough grade surface compaction) | cy | 16,671 | | | | | | 16,671 |
| 46 | Prepare surface of Prepared Subbase for geomembrane placement (surficial rock removal) | sy | 50,012 | | | | | | 50,012 |
| | **Earthworks - In-Heap Storage Pond Spillway and Outlet Channel** | | | | | | | | |
| 47 | Site grading cut (spillway outlet channel low-level culvert crossing) | cy | 410 | | | | | | 410 |
| 48 | Supply, place, and compact Pipe Bedding (spillway outlet channel low-level culvert crossing) | cy | 57 | | | | | | 57 |
| 49 | Supply, place, and compact Pipe Backfill (spillway outlet channel low-level culvert crossing) | cy | 180 | | | | | | 180 |
| 50 | Replace and compact Site Grading Fill (spillway outlet channel low-level culvert crossing) | cy | 173 | | | | | | 173 |
| 51 | Supply and place 3000 psi Reinforced Concrete for spillway | cy | 40 | | | | | | 40 |
| 52 | Process and stockpile Riprap, 9" nominal size | cy | 213 | 3 | | | | | 216 |
| 53 | Load, haul, and place Grouted Riprap in spillway outlet channel (9" nominal size, 6" grout thickness) | cy | 213 | 3 | | | | | 216 |
| | **Earthworks - South Haul Road Diversion Channel** | | | | | | | | |
| 54 | Process material for Riprap, D50=18" | cy | 9,671 | | | | | | 9,671 |
| 55 | Load, haul, and place Riprap in diversion channel, D50=18" | cy | 9,671 | | | | | | 9,671 |
| 56 | Process material for Riprap, D50=9" | cy | 506 | | | | | | 506 |
| 57 | Load, haul, and place Riprap in diversion channel, D50=9" | cy | 506 | | | | | | 506 |
| | **Earthworks - Stage 1 Haul Road Diversion Channel** | | | | | | | | |
| 58 | Process material for Riprap, D50=18" | cy | 1,963 | | | | | | 1,963 |
| 59 | Load, haul, and place Riprap in diversion channel, D50=18" | cy | 1,963 | | | | | | 1,963 |
| | **Earthworks - South Perimeter Diversion Culvert** | | | | | | | | |
| 60 | Site grading cut | cy | 18,381 | | | | | | 18,381 |
| 61 | Supply, place, and compact Pipe Bedding | cy | 1,304 | | | | | | 1,304 |
| 62 | Supply, place, and compact Pipe Backfill | cy | 3,854 | | | | | | 3,854 |
| 63 | Replace and compact Site Grading Fill | cy | 10,942 | | | | | | 10,942 |
| 64 | Crush, screen, and stockpile Road Wearing Course (minus 3-inch) | cy | 438 | | | | | | 438 |
| 65 | Load, haul, place, and compact Road Wearing Course from stockpile (12-inch-thick layer) | cy | 438 | | | | | | 438 |
| 66 | Supply and install 3-ft by 6-ft Gabions for reinforced slope above crusher/conveyor tunnel outlet (includes rockfill, ties, and anchors) | ea | 121 | | | | | | 121 |
| 67 | Load, haul, place, and compact Site Grading Fill (behind and to form Gabion reinforced slope above conveyor tunnel outlet) | cy | 2,065 | | | | | | 2,065 |
| 68 | Process and stockpile Riprap, D50=9" for Stage 2 inlet (constructed during Stage 1) | cy | 35 | | | | | | 35 |
| 69 | Load, haul, and place Riprap for Stage 2 inlet, D50=9" (18-inch-thick layer) | cy | 35 | | | | | | 35 |
| 70 | Load, haul, place, shape and compact Site Grading Fill for inlet protection berm | cy | 34 | | | | | | 34 |
| | **Earthworks - Heap Access Culvert Crossings (heap access from leach pad perimeter)** | | | | | | | | |
| 71 | Crush, screen, and stockpile mill reject for Overliner (minus 2-inch) | cy | | 700 | 624 | 522 | 528 | | 2,374 |
| 72 | Load, haul, place, and compact Overliner from stockpile (for backfill of in-heap culvert crossings) | cy | | 700 | 624 | 522 | 528 | | 2,374 |

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**Table 9.1**
**Staged Construction Material Take-Off Quantities, Rev 1 (Final Design)**

| Item No. | Description | Unit | Staged Material Take-Off Quantities | | | | | | Total Quantity |
|---|---|---|---|---|---|---|---|---|---|
| | | | Stage 1 (EL 1545) | Stage 2 (EL 1640) | Stage 3 (EL 1730) | Stage 4 (EL 1820) | Stage 5 (EL 1950) | Stage 6 (Final) | |
| | **Earthworks - Inclined Solution Extraction Wells** | | | | | | | | |
| 73 | Crush, screen, and stockpile mill reject for Overliner (minus 2-inch) | cy | 3,645 | | | | | | 3,645 |
| 74 | Load, haul, and place Overliner from stockpile (36-inch-thick layer surrounding inclined extraction wells) | cy | 2,934 | | | | | | 2,934 |
| 75 | Load, haul, and place Overliner from stockpile (36-inch-thick layer surrounding inclined well solution collector pipes) | cy | 711 | | | | | | 711 |
| 76 | Stockpile uncrushed mill reject (Drain Aggregate Surround Zone for inclined extraction wells) | cy | 856 | | | | | | 856 |
| 77 | Load, haul, place, and compact Drain Aggregate Surround Zone around inclined extraction wells | cy | 856 | | | | | | 856 |
| | **Earthworks - Vertical Solution Extraction Wells** | | | | | | | | |
| 78 | Stockpile uncrushed mill reject (Drain Aggregate Surround Zone for vertical extraction wells) | cy | 55,076 | | | | | | 55,076 |
| 79 | Load, haul, place, and compact Drain Aggregate Surround Zone around vertical extraction wells (22.5-ft avg radius, 5-ft-thick lifts) | cy | 55,076 | | | | | | 55,076 |
| **GEOSYNTHETICS** | | | | | | | | | |
| | **Geosynthetics - Leach Pad and Perimeter Diversions** | | | | | | | | |
| 80 | Supply and install 80-mil double-side textured LLDPE geomembrane for in-heap pond secondary liner | sf | 2,455,184 | | | | | | 2,455,184 |
| 81 | Supply and install 80-mil double-side textured LLDPE geomembrane for leach pad primary liner | sf | 2,450,176 | 2,534,960 | 2,434,063 | 2,425,051 | 2,945,450 | 2,278,288 | 15,067,987 |
| 82 | Supply and install 8 oz/sy non-woven geotextile for perimeter diversion channels | sf | **3,279** | 13,305 | 11,913 | 11,816 | 52,990 | 82,576 | **175,879** |
| | **Geosynthetics - Underdrain System** | | | | | | | | |
| 83 | Supply and install 8 oz/sy non-woven geotextile above main underdrains (**See Note 2**) | sf | 70,942 | 72,655 | 70,438 | 71,445 | 83,461 | 50,955 | 419,896 |
| 84 | Supply and install 8 oz/sy non-woven geotextile above valley-bottom random fill underdrain | sf | 116,007 | 28,916 | 10,206 | | | | 155,129 |
| | **Geosynthetics - PCMS** | | | | | | | | |
| 85 | Supply and install 8 oz/sy non-woven geotextile above PCMS Drain Aggregate for PCMS channels | sf | 0 | **66,850** | **42,045** | **40,866** | **49,953** | **32,468** | **232,182** |
| 86 | Supply and install 80 mil double-side textured LLDPE geomembrane within PCMS channels (roll width 22 ft) | sf | 0 | **63,229** | **39,767** | **38,652** | **47,247** | **30,710** | **219,604** |
| 87 | Supply and install 8 oz/sy non-woven geotextile for PCMS sumps | sf | **643** | 0 | 0 | 0 | 0 | 0 | **643** |
| 88 | Supply and install 6' x 4' x 1/2"-thick neoprene sheet on PCMS sump floors | ea | **5** | 0 | 0 | 0 | 0 | 0 | **5** |
| 89 | Supply and install 80 mil double-side textured LLDPE geomembrane for PCMS sumps | sf | **1,613** | 0 | 0 | 0 | 0 | 0 | **1,613** |
| | **Geosynthetics - LCRS** | | | | | | | | |
| 90 | Supply and install LCRS tri-planar geonet between primary and secondary geomembrane on slopes within Stage 1 boundary | sf | 2,039,441 | | | | | | 2,039,441 |
| 91 | Supply and install LCRS tri-planar geonet between non-woven geotextile and LCRS Aggregate (Overliner) on pond floor | sf | 490,110 | | | | | | 490,110 |
| 92 | Supply and install 8 oz/sy non-woven geotextile between Prepared Subbase and geonet on pond floor | sf | 490,110 | | | | | | 490,110 |
| 93 | Supply and install geosynthetic clay liner below LCRS sump secondary liner | sf | 2,588 | | | | | | 2,588 |
| 94 | Supply and install conveyor belt wear sheet above LCRS sump secondary liner and below the LCRS outlet pipes | sf | 2,265 | | | | | | 2,265 |
| | **Geosynthetics - In-Heap Storage Pond Spillway and Outlet Channel** | | | | | | | | |
| 95 | Supply and install 80-mil double-side textured LLDPE geomembrane (downstream of spillway to low-level culvert crossing) | sf | 8,329 | 122 | | | | | 8,451 |
| 96 | Supply and install 8 oz/sy non-woven geotextile (beneath spillway outlet channel riprap erosion protection) | sf | 15,620 | 122 | | | | | 15,742 |
| 97 | Supply and install batten bars for connection of geosynthetics to reinforced concrete spillway (includes accessories) | ft | 112 | | | | | | 112 |
| 98 | Anchor upstream and downstream geosynthetics to spillway batten bars | ft | 112 | | | | | | 112 |
| | **Geosynthetics - South Perimeter Diversion Culvert** | | | | | | | | |
| 99 | Supply and install 8 oz/sy non-woven geotextile (beneath Stage 2 inlet riprap erosion protection, constructed during Stage 1) | sf | 700 | | | | | | 700 |
| | **Geosynthetics - South Haul Road Diversion Channel** | | | | | | | | |
| 100 | Supply and install 8 oz/sy non-woven geotextile below haul road channel riprap erosion protection | sf | 145,465 | | | | | | 145,465 |
| | **Geosynthetics - Stage 1 Haul Road Diversion Channel** | | | | | | | | |
| 101 | Supply and install 8 oz/sy non-woven geotextile below haul road channel riprap erosion protection | sf | 49,475 | | | | | | 49,475 |
| | **Geosynthetics - Inclined Solution Extraction Wells** | | | | | | | | |
| 102 | Supply and install conveyor belt wear sheet above primary liner under steel riser pipes | sf | 275 | | | | | | 275 |

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**Table 9.1**
**Staged Construction Material Take-Off Quantities, Rev 1 (Final Design)**

| Item No. | Description | Unit | Staged Material Take-Off Quantities | | | | | | Total Quantity |
|---|---|---|---|---|---|---|---|---|---|
| | | | Stage 1 (EL 1545) | Stage 2 (EL 1640) | Stage 3 (EL 1730) | Stage 4 (EL 1820) | Stage 5 (EL 1950) | Stage 6 (Final) | |
| | **Geosynthetics - Vertical Solution Extraction Wells** | | | | | | | | |
| 103 | Supply and install 80-mil smooth LLDPE geomembrane for riser pipe wrap | sf | 7,869 | | | | | | 7,869 |
| **PIPEWORK** | | | | | | | | | |
| | **Pipework - Leach Pad** | | | | | | | | |
| 104 | Supply and install 4" dia. perforated CPT (type SP) N-12 solution lateral pipework, includes fittings | ft | 43,737 | 37,148 | 33,660 | 33,544 | 44,514 | 40,232 | 232,834 |
| 105 | Supply and install 12" dia. perforated CPT (type SP) N-12 solution collector pipework, includes fittings | ft | 2,933 | 1,550 | 1,377 | 1,123 | 737 | 88 | 7,810 |
| 106 | Supply and install 24" dia. perforated CPT (type SP) N-12 solution header pipework, includes fittings | ft | 13,950 | 5,986 | 3,877 | 3,532 | 4,269 | 2,940 | 34,555 |
| 110 | Supply and install 12" dia. N-12 45-degree elbow | ea | | 1 | | 1 | | | 2 |
| 113 | Supply and install 24" dia. To 12" dia. N-12 wye | ea | 5 | 1 | | | | | 6 |
| 114 | Supply and install 24" dia. To 12" dia. N-12 tee | ea | 3 | | | | | | 3 |
| 117 | Supply and install 24" dia. N-12 wye | ea | 6 | | | | | | 6 |
| 118 | Supply and install 24" dia. N-12 45-degree elbow | ea | | 1 | | | | | 1 |
| | **Pipework - PCMS** | | | | | | | | |
| 119 | Supply and install 4" dia. solid HDPE (SDR 9) PCMS header pipe with slots (within PCMS sumps) | ft | **26** | **26** | 0 | 0 | 0 | 0 | **53** |
| 120 | Supply and install 4" dia. solid HDPE (SDR 11) PCMS header pipe (includes 1 x 1' length slotted pipe for each sump) | ft | **13,946** | 0 | 0 | 0 | 0 | 0 | **13,946** |
| 121 | Supply and install 4" dia. solid HDPE 45-degree elbow | ea | **5** | 0 | 0 | 0 | 0 | 0 | **5** |
| 122 | Supply and install 4" dia. solid HDPE 90-degree elbow within PCMS sump and outlet | ea | **10** | 0 | 0 | 0 | 0 | 0 | **10** |
| 123 | Supply and install 4" dia. LLDPE pipe boots (5 PCMS header pipes with 3 liner penetrations each) | ea | **15** | 0 | 0 | 0 | 0 | 0 | **15** |
| | **Pipework - LCRS** | | | | | | | | |
| 124 | Supply and install 4" dia. perforated CPT (type SP) LCRS lateral drain | ft | 3,271 | | | | | | 3,271 |
| 125 | Supply and install 8" dia. perforated CPT (type SP) LCRS toe drain | ft | 6,109 | | | | | | 6,109 |
| 126 | Supply and install 18" dia. steel LCRS outlet pipe | ft | 588 | | | | | | 588 |
| | **Pipework - In-Heap Storage Pond Spillway and Outlet Channel** | | | | | | | | |
| 127 | Supply and install 36" dia. solid HDPE (SDR 17) culvert (spillway outlet channel low-level culvert crossing) | ft | 180 | | | | | | 180 |
| | **Pipework - South Perimeter Diversion Culverts** | | | | | | | | |
| 128 | Supply and install 36" dia. solid HDPE (SDR 17) culverts | ft | 4,591 | | | | | | 4,591 |
| 129 | Supply and install 36" dia. solid HDPE (SDR 17) 45-degree elbow | ea | 6 | | | | | | 6 |
| 130 | Supply and install 24" dia. solid HDPE (SDR 17) culvert for Stage 2 inlet (constructed during Stage 1) | ft | 12 | | | | | | 12 |
| 131 | Supply and install 36" x 36" x 24" dia. HDPE (SDR 17) 45-degree reducing tee for Stage 2 inlet (constructed during Stage 1) | ea | 1 | | | | | | 1 |
| 132 | Supply and install 24" dia. solid HDPE (SDR 17) 45-degree elbow for Stage 2 inlet (constructed during Stage 1) | ea | 1 | | | | | | 1 |
| | **Pipework - Heap Access Culvert Crossings (heap access from leach pad perimeter)** | | | | | | | | |
| 133 | Supply and install 24" dia. perforated CPT (type SP) culvert | ft | | 993 | 885 | 693 | 702 | | 3,273 |
| | **Pipework - Inclined Solution Extraction Wells** | | | | | | | | |
| 134 | Supply and install inclined solution extraction wells (includes 36" dia. pump cans with screened sections) (Grade B, ASTM A53, 0.45" wall thickness steel) | ft | 1,686 | | | | | | 1,686 |
| 135 | Supply and install inclined solution extraction well base manifold (includes 3 x 36" dia. 45-degree elbows, 3 x 36" dia. 90 degree tees, and 2 x 36" to 30" dia. Reducers) (Grade B, ASTM A53, 0.45" wall thickness steel) | ea | 1 | | | | | | 1 |
| 136 | Supply and install 24" dia. perforated CPT (type SP) solution collectors | ft | 600 | | | | | | 600 |

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**Table 9.1**
**Staged Construction Material Take-Off Quantities, Rev 1 (Final Design)**

| Item No. | Description | Unit | Staged Material Take-Off Quantities | | | | | | Total Quantity |
|---|---|---|---|---|---|---|---|---|---|
| | | | Stage 1 (EL 1545) | Stage 2 (EL 1640) | Stage 3 (EL 1730) | Stage 4 (EL 1820) | Stage 5 (EL 1950) | Stage 6 (Final) | |
| | **Pipework - Vertical Solution Extraction Wells** | | | | | | | | |
| 137 | Supply and install vertical solution extraction well (includes well base plate, 5' long x 36" dia. riser pipe, 30" dia steel plumbing at well base, and liner protection layers) (Grade B, ASTM A53, 0.45" wall thickness steel) | ea | 5 | | | | | | 5 |
| 138 | Supply and install 36" dia. vertical extraction well casings (126-ft per well) (Grade B, ASTM A53, 0.45" wall thickness steel) | ft | 730 | | | | | | 730 |
| 139 | Supply and install 36" dia. Vertical extraction well screen sections (23-ft per well) (Grade B, ASTM A53, 0.45" wall thickness steel)<br>(screened sections to be fabricated and slotted by pipe manufacturer according to design drawings) | ft | 115 | | | | | | 115 |
| 140 | Supply and install 38" dia. x 2' travel distance single-style compression fittings (for 36" dia. well casings) (4 per well) | ea | 20 | | | | | | 20 |
| 141 | Supply and install 38" dia. x 6' travel distance single-style compression fittings (for 36" dia. well casings) (2 per well) | ea | 10 | | | | | | 10 |
| | **Pipework - Underdrain Monitoring Wells** | | | | | | | | |
| 142 | Supply and install underdrain monitoring well<br>(includes average 245-ft-tall 6-inch-dia. steel well casing with blank and slotted sections, pump, and accessories) | ea | **3** | | | | | | **3** |
| | **Pipework - Piezometers** | | | | | | | | |
| 143 | Supply and install 2" dia. solid PVC (Schd 80) LCRS piezometer cable conduit (assumes 1 conduit per piezometer pair) | ft | **5,929** | 0 | 0 | 0 | 0 | 0 | **5,929** |
| 144 | Supply and install 2" dia. solid PVC (Schd 80) Overliner piezometer cable conduit (assumes 1 conduit per piezometer pair) | ft | **6,088** | **3,495** | **3,689** | **4,111** | **6,335** | **5,138** | **28,857** |
| 145 | Supply and install 12" dia. Perforated CPT backup pipe along piezometer conduit (1 per line, overliner piezometers) | ft | **176** | **2,263** | **1,537** | **1,450** | **1,828** | **677** | **7,931** |
| **INSTRUMENTATION** | | | | | | | | | |
| | **Instrumentation - Piezometers** | | | | | | | | |
| 146 | Supply and install LCRS vibrating wire piezometer (including sand surround and geotextile) (Geokon 4500AL-170kPa, or equivalent) | ea | 12 | 0 | 0 | 0 | 0 | 0 | 12 |
| 147 | Supply and install LCRS piezometer cable (includes 5% increase for slack) | ft | **12,451** | 0 | 0 | 0 | 0 | 0 | **12,451** |
| 148 | Supply and install Overliner vibrating wire piezometer (including sand surround and geotextile) (Geokon 4500AL-70kPa, or equivalent) | ea | **3** | 0 | 0 | 0 | 0 | 0 | **3** |
| 149 | Supply and install Overliner vibrating wire piezometer (including sand surround and geotextile) (Geokon 4500S-350kPa, or equivalent) | ea | **7** | **16** | **8** | **6** | **4** | **2** | **43** |
| 150 | Supply and install Overliner piezometer cable (includes 5% increase for slack) | ft | **12,785** | **7,340** | **7,746** | **8,633** | **13,304** | **10,790** | **60,599** |
| 151 | Supply and install piezometer readout box | ea | **2** | **4** | 0 | 0 | 0 | 0 | **6** |
| 152 | Relocate piezometer readout box | ea | **0** | **1** | **5** | **5** | **4** | **1** | **16** |
| | **Instrumentation - Settlement Monuments** | | | | | | | | |
| 153 | Supply and install settlement monument | ea | 5 | | | | | | 5 |

Notes:
1. The general pad earthworks quantities represent a comparison of a compiled FGMI proposed future excavation and waste dump surface versus the KP design grade surfaces. Significant excess fill will be required according to FGMI's proposed configurations. It is assumed this excess fill will be sourced from mine waste.
2. Underdrain excavation, aggregate, Site Grading Fill, and geotextile quantities were increased by 15 percent to account for field adjustments to intercept unforeseen seeps and springs.
3. CPT Pipework and geosynthetics quantities have been increased by 5 and 10 percent, respectively (for procurement purposes), to account for seam overlaps, waste, and scrap during construction.
4. Earthwork quantities are neat. Bulking and shrinkage factors have not been applied.
5. Bolded entries without shading represent MTO quantities that were revised with the Revision 1 issuance.
6. Entries shaded in Blue represent MTO quantities that were not revised with the Revision 1 issuance.
7. Entries shaded in Grey represent line-item descriptions that were revised with the Revision 1 issuance.
8. Entries shaded in Yellow represent new line-items developed for the Revision 1 issuance.

# Section 10.0 - Considerations for Follow-Up Engineering

The following bullet-points represent items associated with the BCHLF that are not addressed as part of the design herein:

- In-Heap Storage Pond Dam Break Analysis

  Based on the configuration of the BCHLF in-heap storage pond embankment (combined with the existing conveyor causeway embankment), a dam break that would result in the rapid, uncontrolled release of solution from the facility is considered highly unlikely.

  The possibility of a traditional dam break is further reduced because the in-heap storage pond embankment and existing conveyor causeway will be buttressed by the Fish Creek stockpile area (planned to be excavated and backfilled higher than the causeway)

  A dam break analysis for the BCHLF is not considered necessary by Knight Piésold.

- Haul Road and Solution Pipe Corridor Design

  The existing waste dump haul road along the southwest side of the planned BCHLF footprint will be utilized for loading the ore heap. Other than the initial Stage 1 haul road presented herein, no additional haul roads are expected, nor are they accounted for in Knight Piésold's BCHLF design.

  A solution pipe corridor will be required for the pregnant and barren pipelines from the BCHLF to the mill site. Design of this structure was not included in Knight Piésold's work scope and is being prepared by others.

- Closure Design

  While a potential, preliminary closure concept is discussed herein, closure design was not included in Knight Piésold's work scope and is not provided. Closure design has typically been conducted internally within FGMI.

  It is understood that FGMI's intended closure strategy will be to create a landform that will be reclaimed under the Reclamation Standard (AS 27.19), that the post-closure waste management will be managed per ADEC requirements, and that the impoundment function of the dam will no longer be necessary and the dam will be abandoned in place after in-heap rinsing is complete.

The following items are recommended for consideration during the planning of construction activities associated with the BCHLF:

- Excavation of the Barnes Creek and Fish Creek Stockpile Areas

  FGMI's planned removal of the low-grade ore stockpiles suggests the development of steep cut slopes that are likely to be safety hazards during construction.

  As discussed with the State of Alaska, the planned excavation to the downstream slope of the conveyor causeway embankment has already been conducted and resulted in final slopes varying from 1.3H:1.0V to 2.0H:1.0V. Immediately downstream of these excavated slopes, the Fish Creek stockpile area has been backfilled to form the storm water detention basin discussed in Section 6.10.

  It is recommended that cut slopes no steeper than 1.5H:1V be incorporated into the excavation to the upstream face of the existing conveyor causeway embankment. Following excavation to the upstream slope of the existing conveyor causeway, the embankment face will be backfilled according to the design drawings and Earthwork Specifications to form the upstream face of the in-heap storage embankment. Regardless, safety precautions should be incorporated into construction planning for steep-slope excavations adjacent to the existing conveyor causeway excavation embankment.

- Stage 1 Construction Schedule

  The Stage 1 MTO quantities do not consider phases of construction, but will likely be spread over multiple years. Input from FGMI will be required to develop realistic construction goals for the completion of the Stage 1 BCHLF.

- Prepared Subbase

  The geotechnical investigations have quantified the nature and expected performance of the various materials that will comprise the foundation for the BCHLF, and work has been done to identify and confirm the adequacy of construction materials; however, the work completed to date has not yet sufficiently defined the sources of prepared subbase for all stages of the BCHLF. Sufficient material will be available for Stage 1 of the BCHLF construction but as part of the preparation for the subsequent stages, further investigations will be needed to define adequate sources of that material. A key priority should be made to effectively using, and stockpiling for future use, all potential prepared subbase material encountered during development of the BCHLF.

  As with current operating procedure at the WCHLF, on-going testing of the BCHLF Prepared Subbase will be required throughout construction to identify potential variations in interface shear strength based on individual borrow areas and to demonstrate acceptable interface strengths are achieved. Additional borrow source investigations and testing of the prepared subbase planned for use within the BCHLF will be conducted prior to construction.

# Section 11.0 - Summary and Conclusions

This report covers the BCHLF design for the Fort Knox mine operated by FGMI.

The BCHLF has been designed with an arrangement similar to the WCHLF. This is in part due to the success of the construction and operation of the WCHLF, and the gained knowledge of the materials and their behavior in it. The BCHLF will be a valley-fill heap leach pad with an in-heap solution storage pond. The BCHLF will extend up the valley from the existing Barnes Creek conveyor causeway, and will be comprised of six distinct and progressively developed stages (Stages 1 through 6). The facility development will generally include construction procedures and components similar to those approved and used on the WCHLF.

Geotechnical site investigations conducted in 2011 and 2016 confirmed:

- The Fairbanks Schist bedrock underlying the valley is fractured, but comprises interlocked blocks of competent bedrock that are expected to form a sound foundation for the BCHLF
- The existing conveyor causeway embankment is expected to form a stable downstream toe for the BCHLF, and a stable containing embankment for the in-heap pond for the BCHLF
- Interbedded layers of higher hydraulic conductivity colluvial material near the base of the conveyor causeway are expected to adequately convey groundwater flows from beneath the BCHLF to the Fish Creek stockpile area, particularly with the flows distributed along the upstream toe of the causeway by the transverse underdrain random fill trench and the drainage wells drilled into the bottom of the transverse trench to intersect the interbedded colluvial layers
- Remnants of the low-grade ore stockpile, as well as the excavation spoils and native soils that are planned to be left in place, exhibit (1) adequate stiffness such that settlement and deformation of the liner is expected to be well within tolerable limits, and (2) adequate strength such that the stability of the BCHLF will meet appropriate factors of safety

The geotechnical investigations have quantified the nature and expected performance of the various materials that will comprise the foundation for the BCHLF, and work has been done to identify and confirm the adequacy of construction materials; however, the work completed to date has not yet sufficiently defined the sources of prepared subbase for all stages of the BCHLF. The prepared subbase under the WCHLF has been constructed using select and processed weathered Fairbanks Schist, which is prevalent in the area. A similar approach is planned for the BCHLF. However, the site of the proposed BCHLF is predominantly disturbed and the underlying weathered schist that would be used appears to be contaminated in places with the overlying natural topsoil that was not removed prior to development of the Barnes Creek low-grade ore stockpile. While sufficient prepared subbase material will be available for the BCHLF Stage 1 construction, it is uncertain how much suitable material will be available for the subsequent stages. On-going investigations will be needed to identify those sources or alternate materials. A key priority should be made to effectively using, and stockpiling for future use, all potential prepared subbase material encountered during development of the BCHLF.

At its final configuration, through Stage 6 development, the BCHLF ore heap will reach a maximum height of approximately 800 ft from toe to crest, and will provide a storage capacity of approximately 124.4 M cy, or 210.4 Mt based on an average in-place ore dry density of 125.3 pcf. This is the same overall density as in the WCHLF since the ore is expected to be very similar. A maximum 500-ft vertical ore thickness over the pad liner system has been adopted also to remain consistent with the WCHLF. Slope stability analyses of the full build out Stage 6 heap were conducted under both static and seismic loading conditions using parameters developed from the BCHLF geotechnical investigation and from previous analyses of similar materials for the WCHLF development, and the results provided Factors of Safety and estimated deformation values that are within appropriate limits for the facility. Settlement analyses, conducted to assess the effect of constructing the BCHLF over the existing materials that will be left in place, indicate

that flow reversal and localized low-points are not expected to develop along the pad liner system and that the estimated foundation settlements are well within tolerable limits that could affect the integrity of the BCHLF liner system.

The overall contributing catchment area to the BCHLF is approximately 760 acres; however, a significant portion of this is upstream of the current and future waste dump within the upper reaches of the valley and will not produce surface runoff to the BCHLF. Approximately 516 acres of the overall upstream catchment is expected to contribute surface runoff to the BCHLF, and at its final (Stage 6) configuration the BCHLF will occupy about 290 acres of this contributing area.

In part, due to the Fort Knox location where cold weather conditions are experienced for significant periods of each year, and to remain consistent with the operational success of the WCHLF, the BCHLF has been designed with an in-heap pond for storage of process solution and storm water. It will be contained behind an approximate 95-ft-high in-heap pond embankment, with a crest elevation of 1545 fmsl, that will be developed on the upstream side of the existing Barnes Creek conveyor causeway.

FGMI currently plans to deliver a total solution flow of 16,000 gpm to the heap; however, to allow flexibility for future possible increases the BCHLF solution collection system has been designed to accommodate a 50-percent increase to 24,000 gpm. The maximum unit rate of solution applied to the heap will be 0.005 gpm/$ft^2$. The overliner solution collection system has been designed to collect and convey the process fluid from the base of the heap to the in-heap storage pond while limiting the maximum hydraulic head acting above the leach pad composite liner system to 1 ft (outside the confines of the pond).

The leach solution within the in-heap pond will be removed by a series of extraction wells located near the upstream toe of the in-heap storage pond embankment:

- Three inclined solution extraction wells will be installed on top of the overliner layer and will be extended up the northeast corner of the in-heap pond to the crest of the in-heap pond embankment. They will be used for solution collection during initial operations until the permanent vertical wells can be brought into operation (i.e. until before ore placement is completed to near the collar elevations of the vertical wells). Because the inclined wells will be primarily used for the initial operations, and will be installed on top of the overliner layer, they will not be directly connected to the leach pad solution collection system. However, to improve hydraulic performance, the bottoms of the three wells will be interconnected with a manifold assembly and two perforated solution collectors will be extended from the manifold assembly out into the pond.
- For the long-term operation of the facility, five vertical solution extraction wells will be constructed as large diameter pumping sumps within the in-heap storage pond. Each well will include a platform base installed at the bottom of the in-heap pond, at approximate elevation 1450 fmsl, just upstream of the impounding embankment toe. The bottom of each well will be connected directly to the leach pad solution collection header pipes to promote a high level of hydraulic efficiency. The vertical wells will be extended from their bases to daylight through the Lift 4 bench on the front face of the heap. Each well has been designed to accommodate a 4,000 gpm pump such that at 16,000 gpm a total of four wells will be in service, with the fifth well available for backup use.

Based on FGMI's experience with the WCHLF to date, any restriction of solution flow through the voids in the ore and into the wells is expected to be negligible, with likely only a small drawdown of the pond solution level at the wells.

Smaller pumping wells will also be installed for the LCRS beneath the in-heap pond. The LCRS pump cans will consist of two inclined pipes running up the upstream face of the in-heap pond embankment. Two 250 gpm pumps will be operated within the LCRS pump cans, as needed, from the start of leaching operations within the in-heap pond.

To support the BCHLF final design, an updated climate study and operational period process water balance were completed. Based on seasonal ground conditions and associated infiltration, January rain-on-snow

events were found to produce the largest design peak flows from surface runoff (i.e. for channel and culvert hydraulic design); however, the September rain-on-snow events were typically found to produce the largest overall volumes (i.e. the 100-yr/24-hr design storm storage component within the in-heap pond). These data and the staged footprint limits for the BCHLF have been used to size the in-heap pond storage components. Above the maximum normal operating levels, the following three primary components are accounted for within the in-heap pond:

- 100-yr/24-hr design storm event (September rain-on-snow)
- 24-hr emergency draindown due to loss of power or pumps
- Freeboard allowance

The most important findings from the in-heap pond storage components are the maximum normal operating levels that should be maintained. These are the stage-by-stage pond levels that should not be exceeded under normal operating conditions such that the required pond storage capacities for the design storm water, draindown, and freeboard allowance are always available. In comparing the maximum normal operating level for each stage of the BCHLF development with the proposed pond minimum normal operating level, a large degree of operational flexibility will be provided.

In the very unlikely event that the heap surface becomes frozen and a major thawing and runoff event follows, it is possible that the spillway may come into service. The spillway has been designed to safely pass the flow from a completely frozen and impermeable heap surface, at the Stage 6 full build out arrangement, receiving runoff from the winter 100-yr/24-hr design storm event. The spillway will be located on the northeast abutment of the in-heap pond embankment. Solution exiting through the spillway will be directed safely downstream to the Fish Creek storm water detention basin that has been formed between the excavated downstream slope of the over widened conveyor causeway embankment and FGMI's backfilling of the Fish Creek stockpile area. The storm water detention basin will receive any flows discharged through the BCHLF emergency spillway and from external surface runoff diverted around the facility. The design storage requirement for the basin was calculated to be approximately 4.93 $Mft^3$, based on the 100-yr/24-hr January rain-on-snow design storm event. While waters collected in the detention basin are expected to rapidly infiltrate beneath the Fish Creek stockpile area and be picked up by dewatering wells in the area, FGMI has indicated that the volume of water within the detention basin will be controlled via direct pumping from the basin as needed. Provisions for pumping of waters collected in the Fish Creek storm water detention basin will be included in FGMI's O&M Manual.

During the very unlikely condition that the heap surface is frozen, storm water runoff routed off the heap would be required to pass beneath the haul ramps (along the southern heap perimeter) that are planned to access the heap for loading the facility. Four in-heap culvert crossings are included in the BCHLF design and have been sized to pass the peak flow runoff associated with the 100-yr/24-hr design storm event acting on the fully loaded BCHLF Stage 6 heap configuration.

Initial construction access and ore haulage into the bottom of the in-heap storage pond will be conducted from the upper (west) end of the valley. Due to the upward-expanding nature of a valley-fill facility, the initial lifts of ore will provide smaller storage capacities than those developed in later stages. The initial heap loading will utilize a reduced rate between 70,000 and 120,000 tpd, and is planned to begin in September and end in November of 2019. During years 2 and 3, heap loading will be conducted between July and November but at the full production loading rate of 123,000 tpd. After year 3, when additional pad area is available, loading of the ore heap is expected to occur approximately 8 months, or 244 days, out of each year (April through November) at the same loading rate of approximately 123,000 tpd, or roughly 30 M tpa. It is estimated that loading of the BCHLF will be completed in July 2027, approximately 8 years after the start of loading.

Perimeter roads and surface water diversion channels have been included in the design for each stage of the BCHLF. They have been divided into two categories; inter-stage and ultimate. Ultimate perimeter

roads and diversion channels will wrap around the final pad footprint, while the inter-stage perimeter roads/diversions will be located just above each stage of the pad development through Stage 5.

Each inter-stage perimeter road has been sized and graded to convey the January 100-yr/24-hr design storm runoff, from the upstream catchment areas, along the road corridor. The surface runoff diverted by the inter-stage roads will preferentially be conveyed along the north side of the facility to avoid routing additional flows toward the open pit and the crusher building southwest of the facility. The ultimate perimeter channel, located predominantly around the northeast and east sides of the BCHLF, will collect and convey the runoff from the inter-stage perimeter roads (and any runoff from smaller catchment areas above the final pad footprint) and discharge the flows to the Fish Creek stockpile area. The ultimate perimeter diversion channel will be riprap-lined and has been sized to pass the peak flow associated with the January 100-yr/24-hr design storm event.

Perimeter diversion culverts have been included along the south side of the facility to pass flows collected by the existing waste dump haul road (located on the southwest side of the BCHLF) and discharge them to the Fish Creek storm water detention basin. These culverts are intended to reduce the current flows diverted by the existing waste dump haul road to the open pit. The south perimeter diversion culverts will be constructed during the initial BCHLF development, and will be constructed along the following alignment:

- beneath the existing waste dump haul road near the southwest corner of the BCHLF,
- down the south abutment of the BCHLF (partially below the ultimate perimeter road and partially on native ground outside the ultimate facility footprint),
- above the crusher conveyor outlet tunnel (supported by a gabion-reinforced fill slope),
- beneath the existing conveyor causeway access road adjacent to the existing crusher building, and
- into the Fish Creek storm water detention basin.

The south perimeter diversion culverts have been sized to pass the peak flows associated with the January 100-yr/24-hr design storm event. Gabion reinforced slopes will be required to support the culverts crossing above the conveyor outlet tunnel.

Incrementally staged construction material take-off (MTO) quantity estimates were completed for all identified components of the BCHLF specifically within Knight Piésold's work scope and are included in this report. Unit rates and cost estimates associated with the MTOs will be completed by FGMI and are not included in this report.

Although closure design was not included in Knight Piésold's work scope and is not provided, it is understood that FGMI's intended closure strategy will be to create a landform that will be reclaimed under the Reclamation Standard (AS 27.19), that the post-closure waste management will be managed per ADEC requirements, and that the impoundment function of the dam will no longer be necessary and the dam will be abandoned in place after in-heap rinsing is complete.

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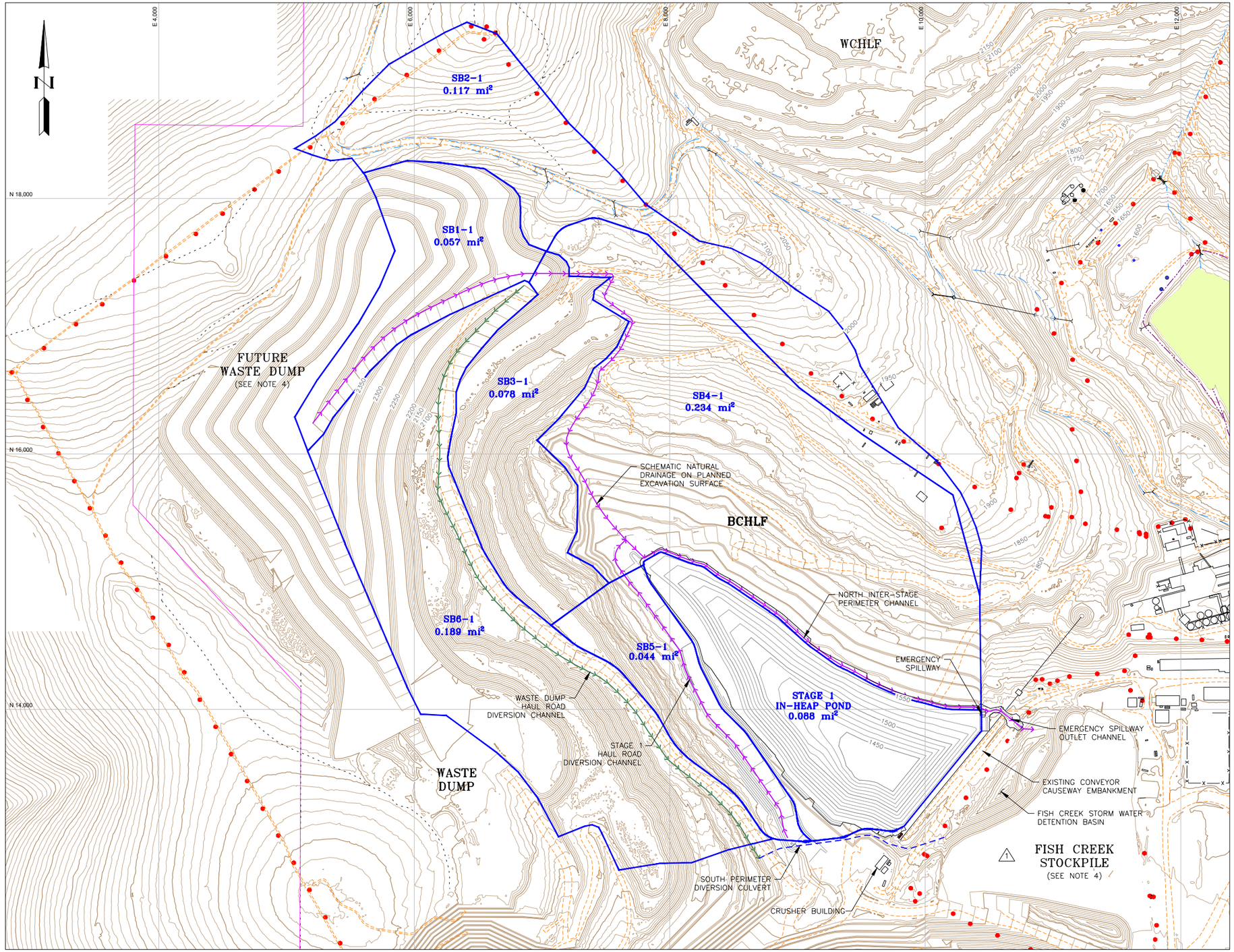
# Section 13.0 - Acronyms and Abbreviations

| Acronym or Abbreviation | Definition |
|---|---|
| ADGGS | Alaska Division of Geological and Geophysical Surveys |
| ADNR | Alaska Department of Natural Resources |
| ASTM | American Society for Testing and Materials |
| cf | cubic-foot (feet) |
| cfs | cubic-foot (feet) per second |
| cm/sec | centimeter(s) per second |
| CN | curve number |
| CPT | corrugated polyethylene tubing |
| CQA | construction quality assurance |
| cy | cubic-yard(s) |
| $^{o}F$ | degree(s) Fahrenheit |
| $D_{50}$ | average mean diameter/particle size |
| $D_n$ | nominal diameter/particle size |
| dia | diameter |
| DST | double-side textured |
| EoO | end of operations |
| FEMA | Federal Emergency Management Agency |
| FGMI | Fairbanks Gold Mining, Inc. |
| fmsl | feet above mean sea level |
| ft | foot (feet) |
| $ft^2$ | square-foot (feet) |
| $ft^3$ | cubic-foot (feet) |
| g | acceleration due to Earth's gravity |
| GCL | geosynthetic clay liner |
| gpm | gallon(s) per minute |
| GPS | Global Positioning System |
| H:V | horizontal to vertical |
| HDPE | high-density polyethylene |
| HEC-HMS | Hydrologic Engineering Center Hydrologic Modeling System |
| hr | hour |
| Knight Piésold | Knight Piésold and Co. |
| LCRS | leachate collection and recovery system |
| LIT | liner integrity test(s), liner integrity testing |
| LLDPE | linear-low density polyethylene |
| m | meter(s) |
| M | million |
| max | maximum |
| MCE | maximum credible earthquake |
| MDD | maximum dry density |
| MDE | maximum design earthquake |
| min | minimum |
| mph | mile(s) per hour |
| Mt | million-ton(s) |
| MTO(s) | material take-off (quantities) |

| Acronym or Abbreviation | Definition |
|---|---|
| MWS | maximum water surface |
| n | porosity |
| N/A | not applicable |
| No(s). | number(s) |
| O&M | Operation and Maintenance |
| OBE | operating basis earthquake |
| OMC | optimum moisture content |
| oz/sy | ounce(s) per square yard |
| pcf | pound(s) per cubic-foot |
| PCMS | process component monitoring system |
| PFS | prefeasibility study |
| PHGA | peak horizontal ground acceleration |
| PLS | process leach solution |
| post-c | post-closure |
| psf | pound(s) per square-foot |
| psi | pound(s) per square-inch |
| QC | quality control |
| $R^2$ | coefficient of determination used in linear regression analyses |
| RQD | rock quality designation |
| sf | square-foot (feet) |
| SoA | State of Alaska |
| SPT | standard penetration test |
| tpa | ton(s) per annum |
| tpd | ton(s) per day |
| TSF | tailing storage facility |
| USACE | U.S. Army Corps of Engineers |
| USBR | United States Bureau of Reclamation |
| USCS | Unified Soil Classification System |
| USGS | United States Geological Survey |
| UV | ultraviolet |
| WCHLF | Walter Creek Heap Leach Facility |
| $yd^3$ | cubic-yard(s) |
| yr | year |

# Figures




## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- GRADING SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- WATERSHED BOUNDARY
- SB1-1 — WATERSHED DESIGNATION
- PROPOSED DIVERSION (CONVEYED TO THE NORTH ABUTMENT)
- PROPOSED DIVERSION (CONVEYED TO THE SOUTH ABUTMENT)
- EXISTING ROAD
- TRAIL
- EXISTING STRUCTURE
- EXISTING FENCE
- EXISTING CULVERT
- EXISTING DRAINAGE/DIVERSION
- EXISTING UTILITY POLE
- TAILING (CONFIGURATION AS OF AUGUST 2015)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE SOUTH PERIMETER DIVERSION CULVERT WILL COLLECT AND CONVEY RUNOFF FROM THE UPSTREAM WASTE DUMPS THAT WILL BE CARRIED ALONG THE SOUTH HAUL ROAD IN ORDER TO REDUCE RUNOFF THAT WOULD OTHERWISE REPORT TO THE PIT. FLOW CONVEYED IN THE CULVERT WILL DISCHARGE INTO THE FISH CREEK STORM WATER DETENTION BASIN. (1)
3. BASINS WERE DELINEATED BASED ON GROUND TYPE (WASTE ROCK VS NATURAL/EXCAVATED GROUND) AS WELL AS WHETHER THE AREA IS INTERNAL OR EXTERNAL TO THE ULTIMATE PAD FOOTPRINT.
4. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.

## REVISION:

(1) REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED NOTE 2 FOR DISCHARGE TO DETENTION BASIN. MODIFIED FISH CREEK TOPOGRAPHY.

400 0 400 800 FEET

1" = 400' AT FULL SIZE (ANSI D)
1" = 800' AT HALF SIZE (ANSI B)

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

PROJECT
FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE
# STAGE 1 WATER MANAGEMENT PLAN

CLIENT
FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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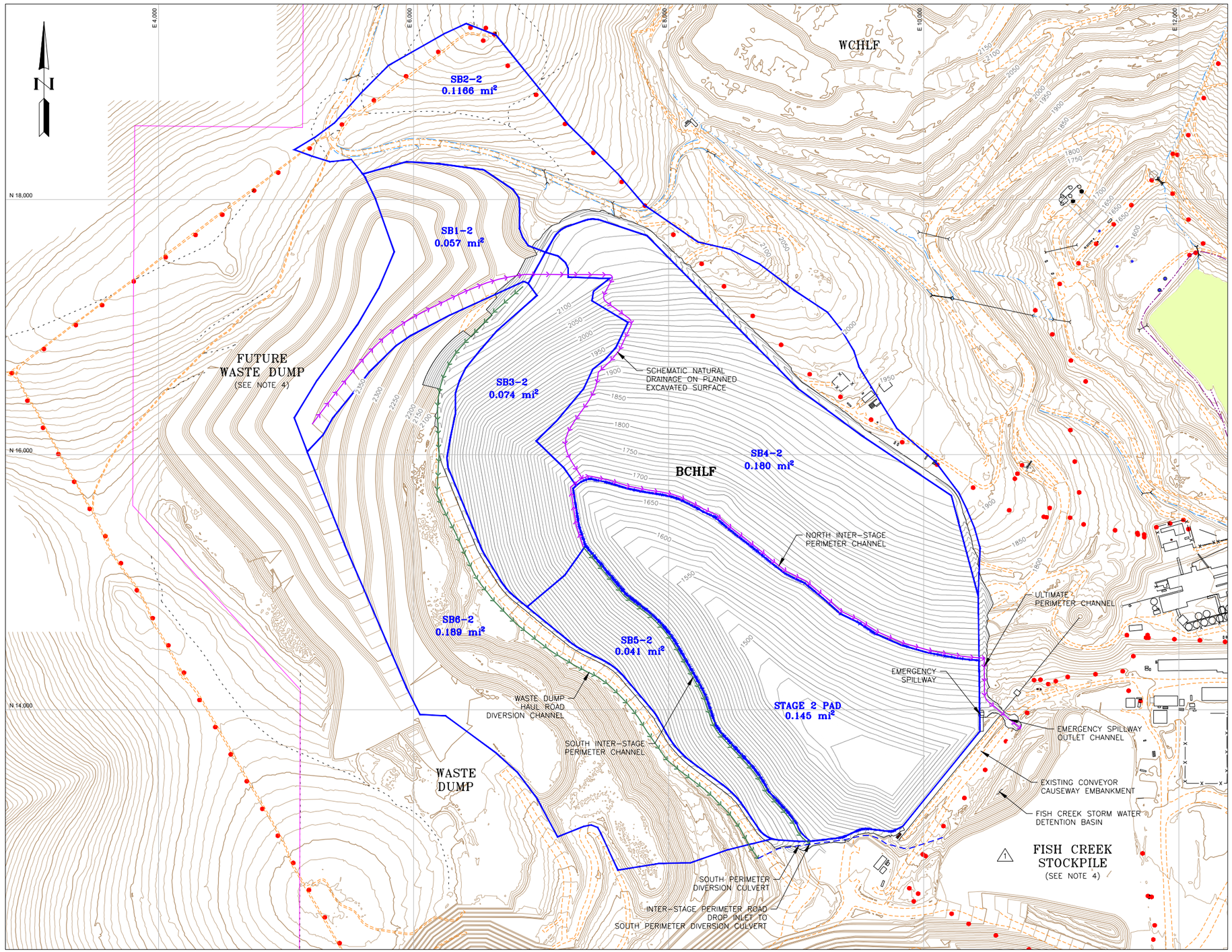


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- GRADING SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- WATERSHED BOUNDARY
- SB1-2 — WATERSHED DESIGNATION
- PROPOSED DIVERSION (CONVEYED TO THE NORTH ABUTMENT)
- PROPOSED DIVERSION (CONVEYED TO THE SOUTH ABUTMENT)
- EXISTING ROAD
- TRAIL
- EXISTING STRUCTURE
- EXISTING FENCE
- EXISTING CULVERT
- EXISTING DRAINAGE/DIVERSION
- EXISTING UTILITY POLE
- TAILING (CONFIGURATION AS OF AUGUST 2015)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. (1) THE SOUTH PERIMETER DIVERSION CULVERT WILL COLLECT AND CONVEY RUNOFF FROM THE UPSTREAM WASTE DUMPS THAT WILL BE CARRIED ALONG THE SOUTH HAUL ROAD IN ORDER TO REDUCE RUNOFF THAT WOULD OTHERWISE REPORT TO THE PIT. FLOW CONVEYED IN THE CULVERT WILL DISCHARGE INTO THE FISH CREEK STORM WATER DETENTION BASIN.
3. BASINS WERE DELINEATED BASED ON GROUND TYPE (WASTE ROCK VS NATURAL/EXCAVATED GROUND) AS WELL AS WHETHER THE AREA IS INTERNAL OR EXTERNAL TO THE ULTIMATE PAD FOOTPRINT.
4. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.

## REVISION:

(1) REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED NOTE 2 FOR DISCHARGE TO DETENTION BASIN. MODIFIED FISH CREEK TOPOGRAPHY.

400 0 400 800 FEET

1"=400' AT FULL SIZE (ANSI D)
1"=800' AT HALF SIZE (ANSI B)

REFERENCE:
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- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

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PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 2 WATER MANAGEMENT PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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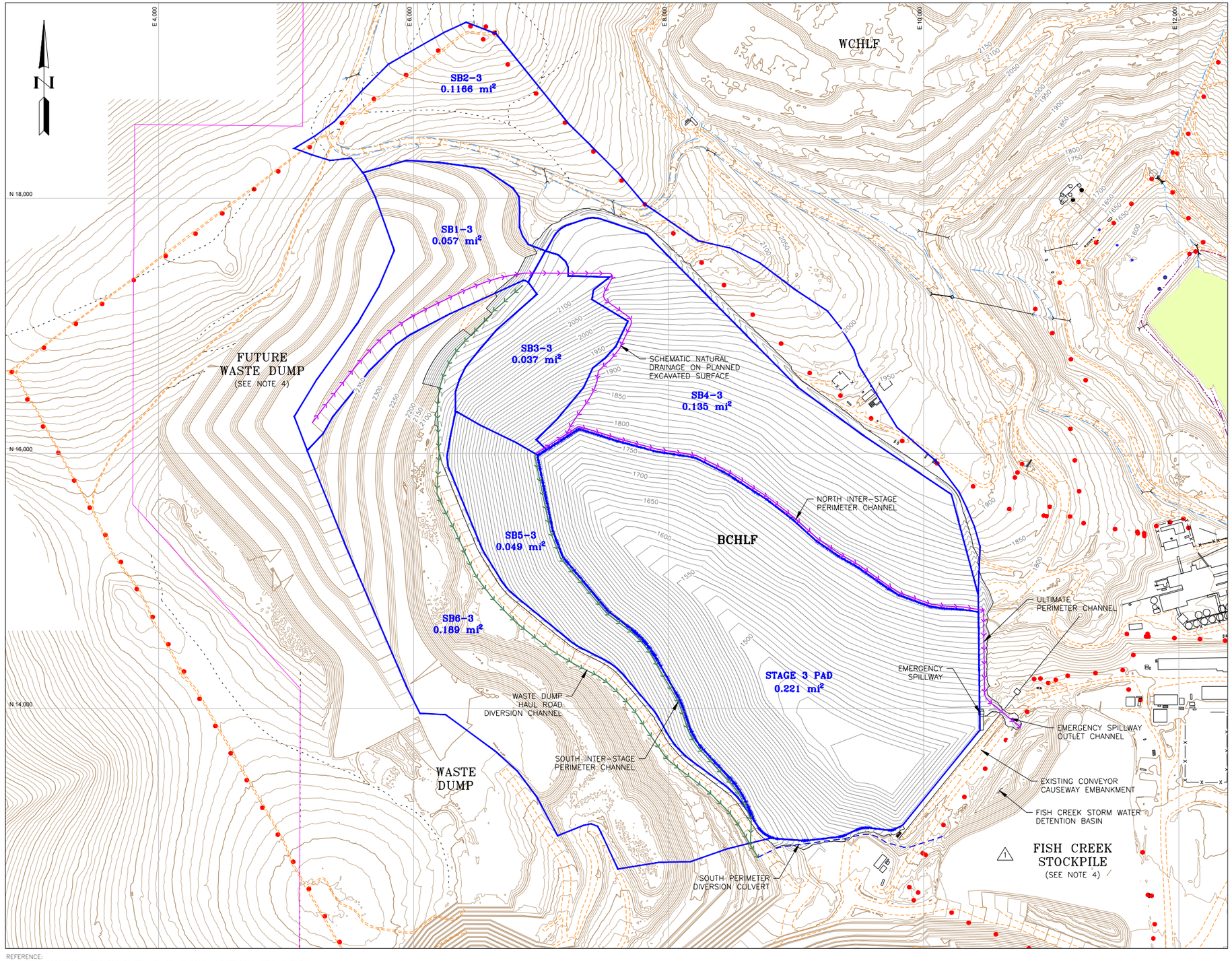


## LEGEND:

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- GRADING SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- WATERSHED BOUNDARY
- SB1-3 — WATERSHED DESIGNATION
- PROPOSED DIVERSION (CONVEYED TO THE NORTH ABUTMENT)
- PROPOSED DIVERSION (CONVEYED TO THE SOUTH ABUTMENT)
- EXISTING ROAD
- TRAIL
- EXISTING STRUCTURE
- EXISTING FENCE
- EXISTING CULVERT
- EXISTING DRAINAGE/DIVERSION
- EXISTING UTILITY POLE
- TAILING (CONFIGURATION AS OF AUGUST 2015)

## NOTES:

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## REVISION:

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- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 3 WATER MANAGEMENT PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

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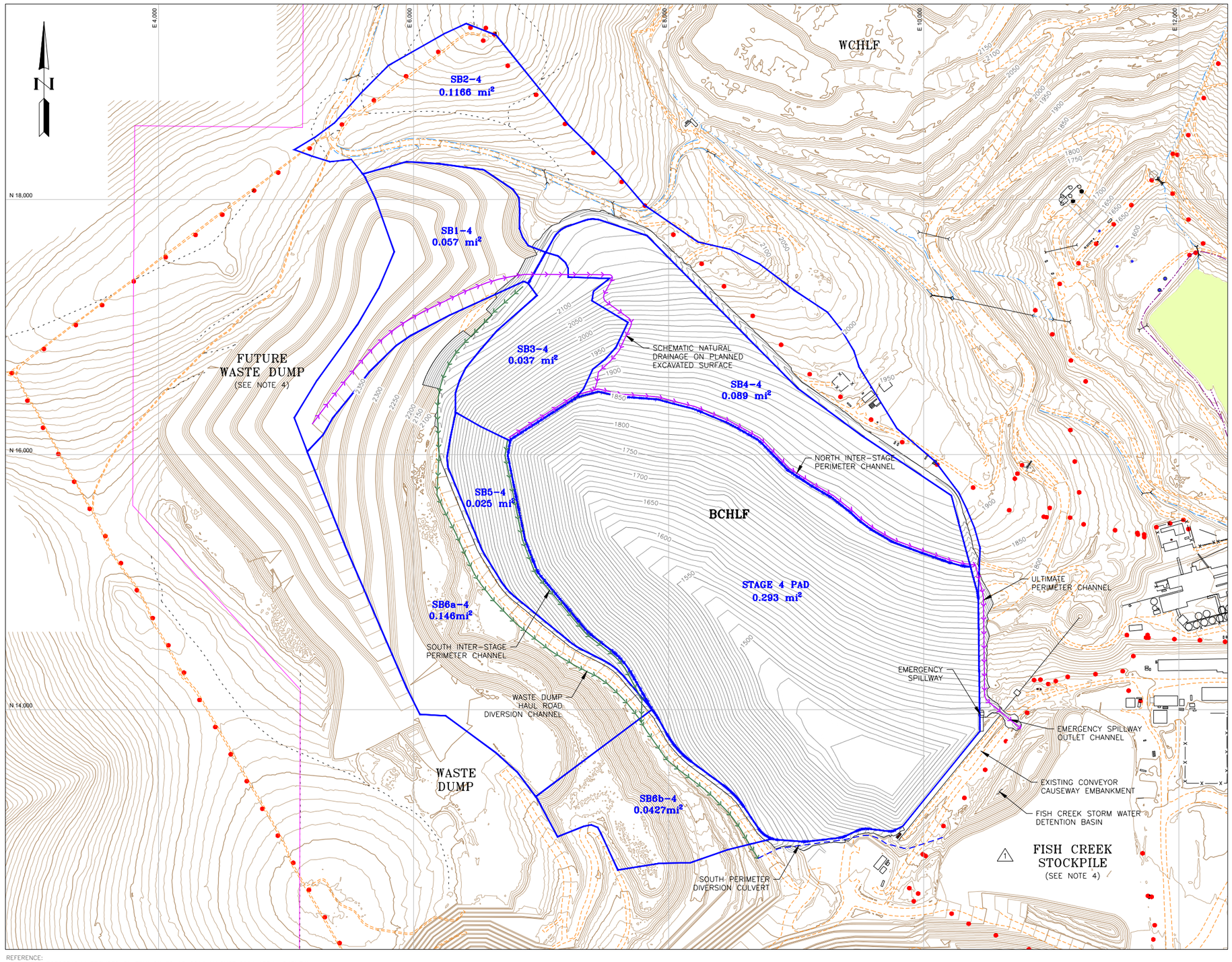


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- GRADING SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- WATERSHED BOUNDARY
- SB1-4 — WATERSHED DESIGNATION
- PROPOSED DIVERSION (CONVEYED TO THE NORTH ABUTMENT)
- PROPOSED DIVERSION (CONVEYED TO THE SOUTH ABUTMENT)
- EXISTING ROAD
- TRAIL
- EXISTING STRUCTURE
- EXISTING FENCE
- EXISTING CULVERT
- EXISTING DRAINAGE/DIVERSION
- EXISTING UTILITY POLE
- TAILING (CONFIGURATION AS OF AUGUST 2015)

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## REVISION:

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- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 4 WATER MANAGEMENT PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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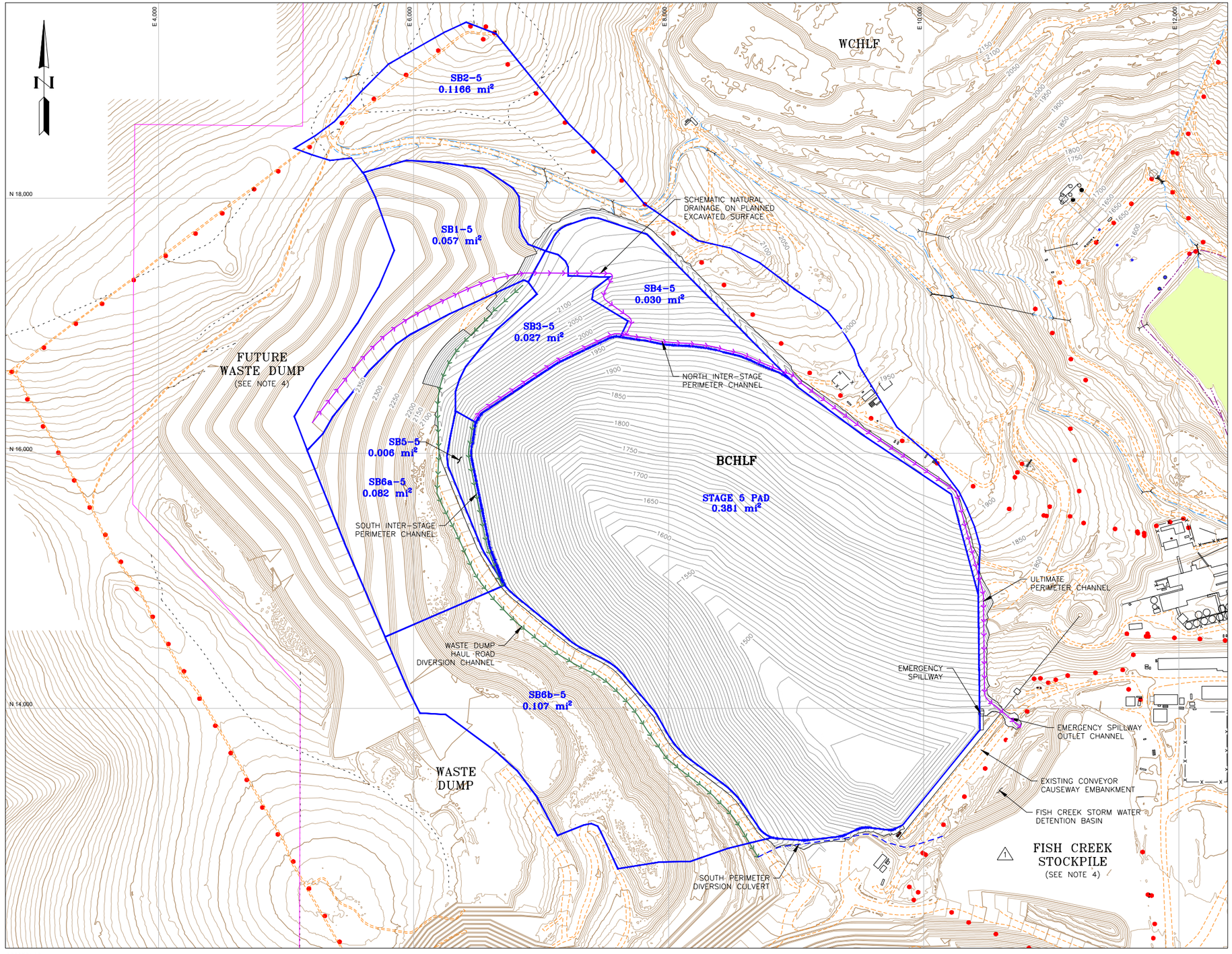


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- GRADING SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- WATERSHED BOUNDARY
- SB1-5 — WATERSHED DESIGNATION
- PROPOSED DIVERSION (CONVEYED TO THE NORTH ABUTMENT)
- PROPOSED DIVERSION (CONVEYED TO THE SOUTH ABUTMENT)
- EXISTING ROAD
- TRAIL
- EXISTING STRUCTURE
- EXISTING FENCE
- EXISTING CULVERT
- EXISTING DRAINAGE/DIVERSION
- EXISTING UTILITY POLE
- TAILING (CONFIGURATION AS OF AUGUST 2015)

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## REVISION:

(1) REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED NOTE 2 FOR DISCHARGE TO DETENTION BASIN. MODIFIED FISH CREEK TOPOGRAPHY.

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PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 5 WATER MANAGEMENT PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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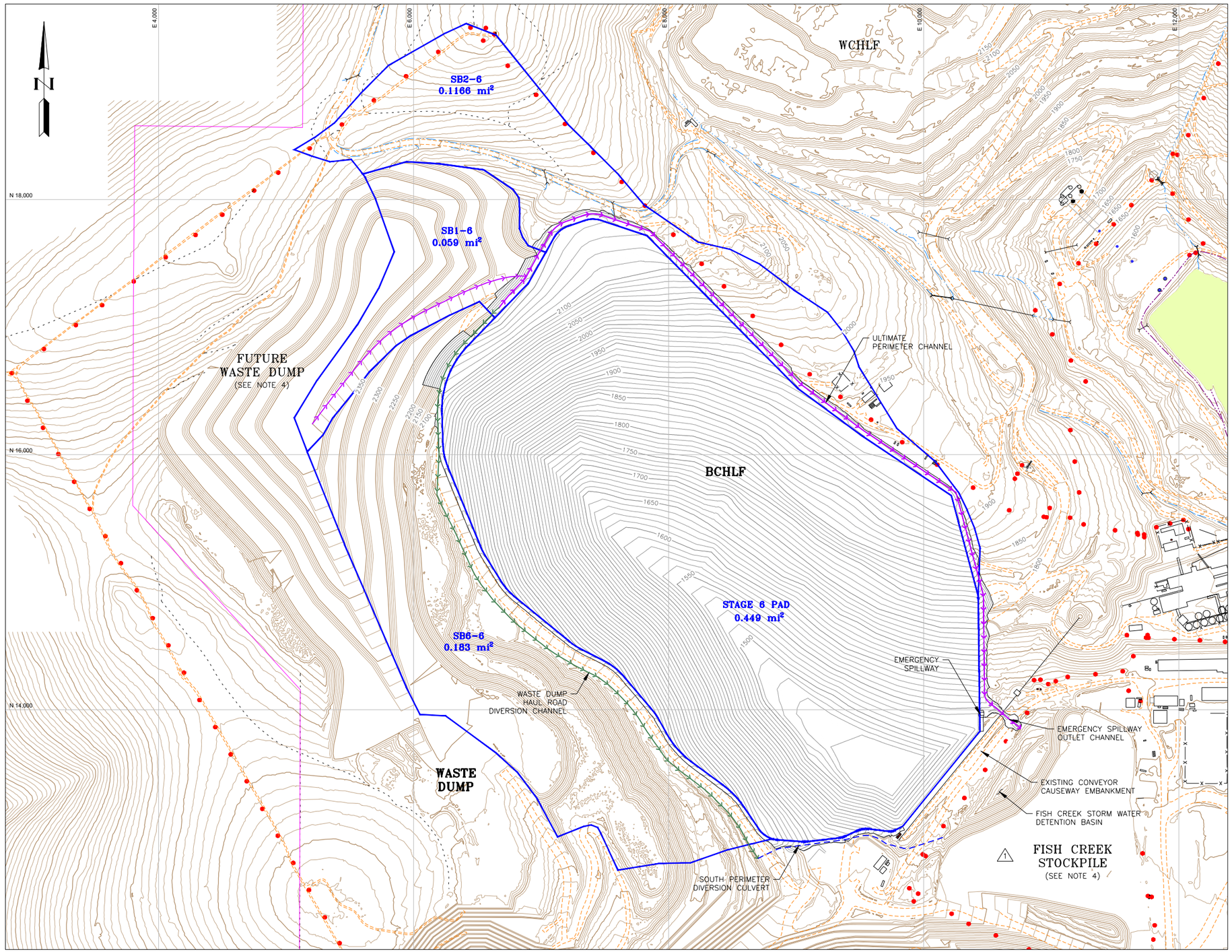


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- GRADING SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- WATERSHED BOUNDARY
- SB1-6 — WATERSHED DESIGNATION
- PROPOSED DIVERSION (CONVEYED TO THE NORTH ABUTMENT)
- PROPOSED DIVERSION (CONVEYED TO THE SOUTH ABUTMENT)
- EXISTING ROAD
- TRAIL
- EXISTING STRUCTURE
- EXISTING FENCE
- EXISTING CULVERT
- EXISTING DRAINAGE/DIVERSION
- EXISTING UTILITY POLE
- TAILING (CONFIGURATION AS OF AUGUST 2015)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. (1) THE SOUTH PERIMETER DIVERSION CULVERT WILL COLLECT AND CONVEY RUNOFF FROM THE UPSTREAM WASTE DUMPS THAT WILL BE CARRIED ALONG THE SOUTH HAUL ROAD IN ORDER TO REDUCE RUNOFF THAT WOULD OTHERWISE REPORT TO THE PIT. FLOW CONVEYED IN THE CULVERT WILL DISCHARGE INTO THE FISH CREEK STORM WATER DETENTION BASIN.
3. BASINS WERE DELINEATED BASED ON GROUND TYPE (WASTE ROCK VS NATURAL/EXCAVATED GROUND) AS WELL AS WHETHER THE AREA IS INTERNAL OR EXTERNAL TO THE ULTIMATE PAD FOOTPRINT.
4. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.

## REVISION:

(1) REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED NOTE 2 FOR DISCHARGE TO DETENTION BASIN. MODIFIED FISH CREEK TOPOGRAPHY.

400 0 400 800 FEET
1"=400' AT FULL SIZE (ANSI D)
1"=800' AT HALF SIZE (ANSI B)

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 6 WATER MANAGEMENT PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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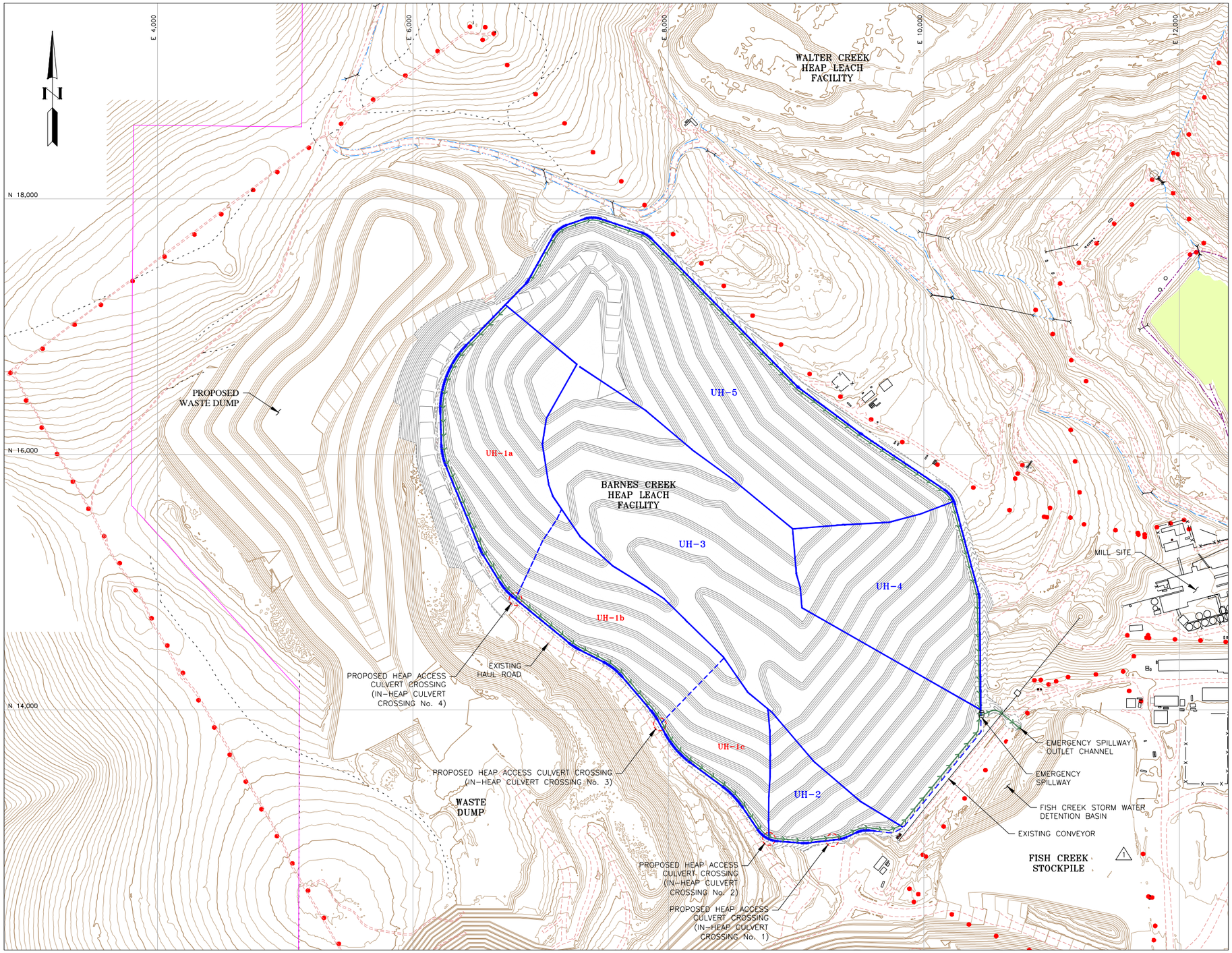


## LEGEND:

- 1500 — EXISTING GROUND SURFACE CONTOUR AND EL. FEET
- 1500 — ULTIMATE HEAP DESIGN SURFACE CONTOUR AND EL. FEET
- PROPERTY BOUNDARY
- WATERSHED BOUNDARY
- WATERSHED SUB-BOUNDARY
- UH-1 — WATERSHED DESIGNATION
- UH-1a — WATERSHED SUB-DESIGNATION
- HEAP SURFACE RUNOFF DIVERTED WITHIN THE LINED LEACH PAD PERIMETER
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- EXISTING STRUCTURES
- TAILING (CONFIGURATION AS OF AUGUST 2014)

**TABLE 1**
**BARNES CREEK HEAP LEACH FACILITY**

| WATERSHED DESIGNATION | AREA (square miles) |
|---|---|
| UH-1a | 0.053 |
| UH-1b | 0.040 |
| UH-1c | 0.023 |
| UH-2 | 0.018 |
| UH-3 | 0.149 |
| UH-4 | 0.054 |
| UH-5 | 0.117 |

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).

## REVISION:

1 MODIFIED FISH CREEK TOPOGRAPHY FOR DETENTION BASIN.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

PLAN

400 0 400 800 FEET

1" = 400' AT FULL SIZE (ANSI D)
1" = 800' AT HALF SIZE (ANSI B)

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: ULTIMATE HEAP CONFIGURATION WATERSHED PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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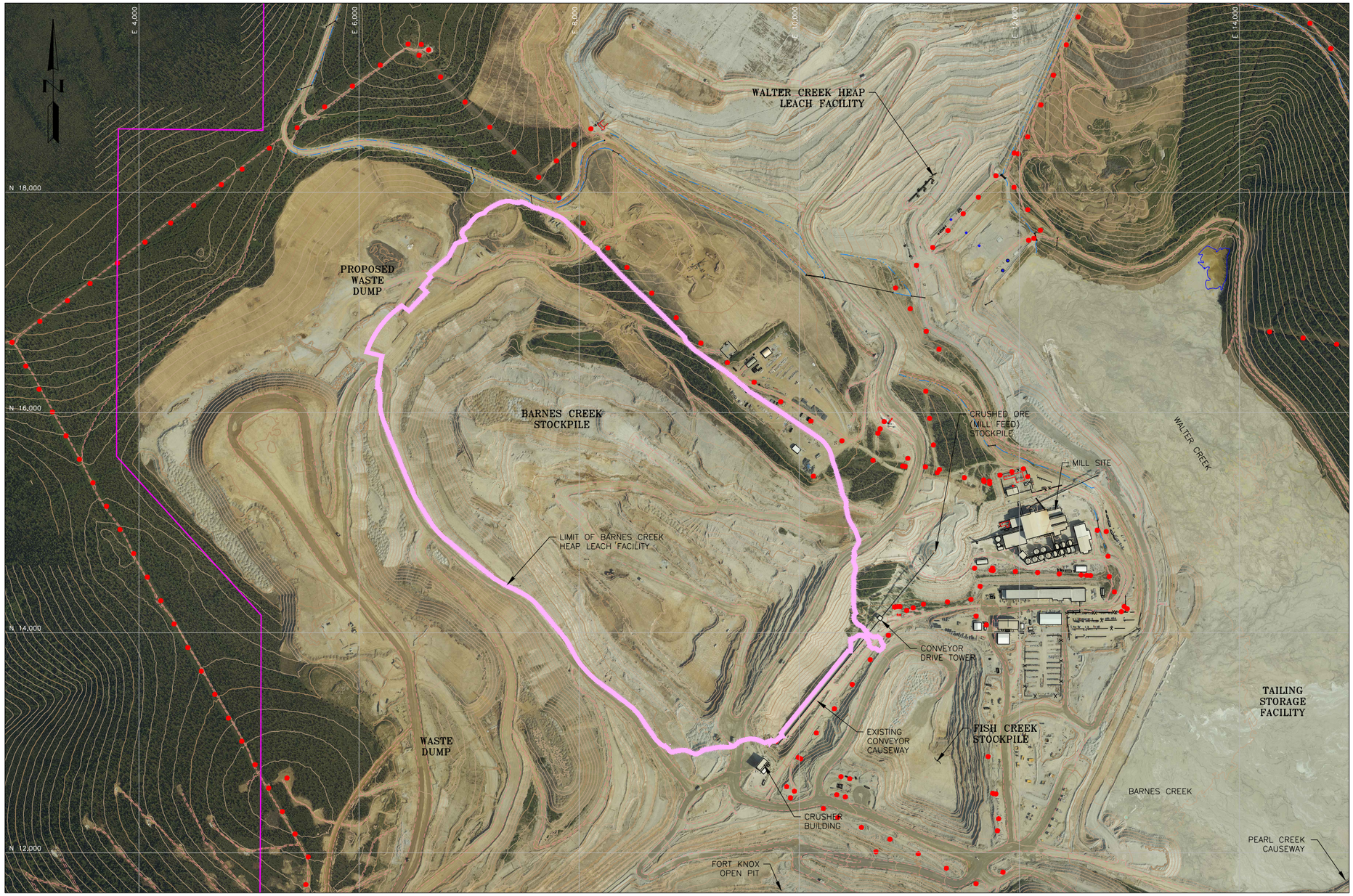


## LEGEND:

- 1500 — EXISTING GROUND SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- PROPERTY BOUNDARY
- EXISTING ROAD
- TRAIL
- EXISTING STRUCTURE
- ULTIMATE FOOTPRINT OF PROPOSED BCHLF

## NOTE:

1. SCALE BAR MEASURES 3.0" ON A FULL SIZE PLOT (ANSI–D) AND 1.5" ON A HALF SIZE PLOT (ANSI–B).

REFERENCE:
– AERIAL TOPOGRAPHY AND PHOTO PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)

## PLAN

500 0 500 1000 FEET

1" = 500' AT FULL SIZE (ANSI D)
1" = 1000' AT HALF SIZE (ANSI B)

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: BARNES CREEK VALLEY GENERAL ARRANGEMENT WITH 2015 AERIAL PHOTOGRAPH

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| | | | | | |
|---|---|---|---|---|---|
| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
| DRAWN BY | CB | DV101 | 00089.99 | 6.01 | 0 |
| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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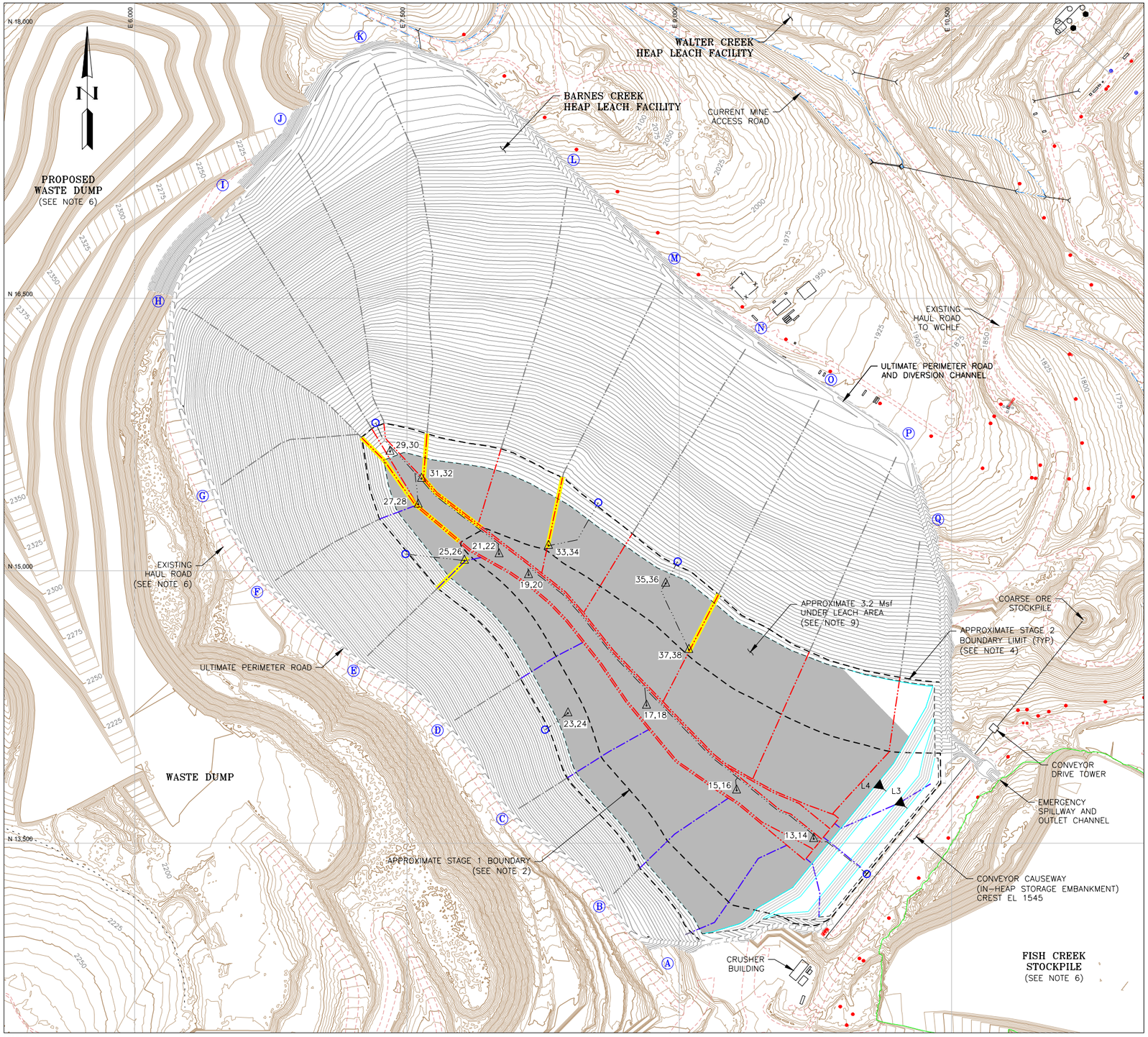


## LEGEND:

- EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (FUTURE STAGES)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (FUTURE STAGES)
- PCMS CHANNEL UNDERLYING PAD LINER SYSTEM BENEATH SOLUTION COLLECTOR OR HEADER (SEE NOTE 10)
- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
3. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH CROSSING OVER SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT FIVE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
8. THE PIEZOMETERS, PIEZOMETER CABLE CONDUITS, AND SOLUTION COLLECTION PIPEWORK WILL BE INSTALLED ON THE LEACH PAD BASIN GRADING SURFACE PRIOR TO HEAP LOADING. THE LEACH PAD BASING CONTOURS BENEATH THE HEAP ARE NOT SHOWN FOR CLARITY.
9. THE 3.2 Msf UNDER LEACH AREA IS BASED ON FGMI'S PLANNED 16,000 gpm TOTAL SOLUTION FLOW TO THE HEAP AND THE TARGET 0.005 gpm/sf UNIT APPLICATION RATE. THE LEACH CONFIGURATIONS SHOWN ASSUME LOADING AND LEACHING WILL PROGRESS FROM THE SOUTHWEST TO NORTHEAST EDGES OF THE VALLEY.
10. THE PCMS CHANNELS HAVE BEEN SITED SUCH THAT AT LEAST TWO CHANNELS ARE LOCATED BENEATH THE ACTIVE UNDER LEACH AREA AT ANY TIME, AND SUCH THAT THE PCMS HEADER PIPES (SEE DRAWING 200) WILL DISCHARGE FLOW INTO THE NORTHEAST CORNER OF THE IN-HEAP STORAGE POND FOR MONITORING.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

ASSESSMENT OF UNDER LEACH AREA
STAGE 2, LIFT 4

984.25 0 984.25 1968.5 FEET
1"=984.25' AT FULL SIZE (ANSI D)
1"=1968.5' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 2, LIFT 4 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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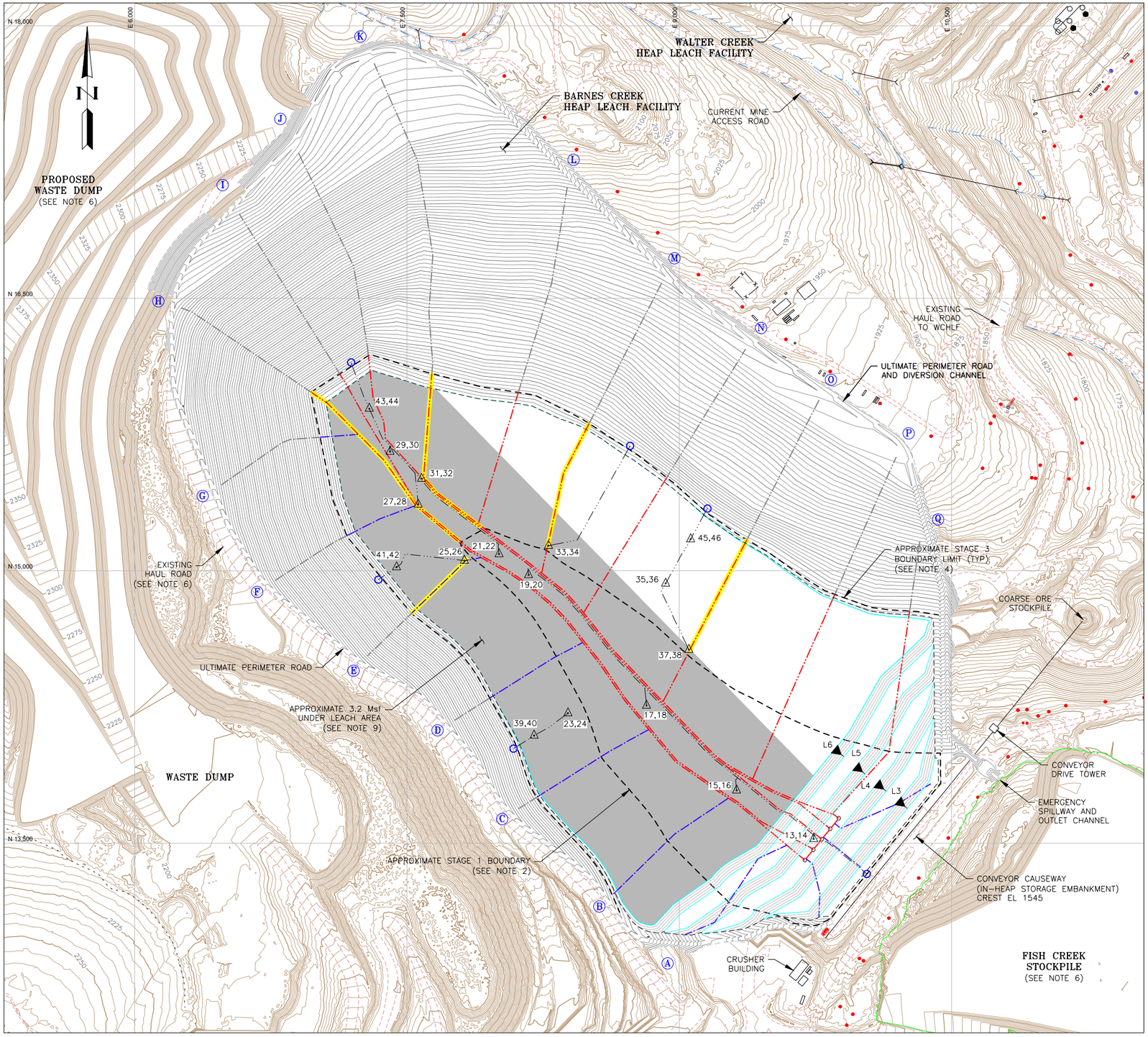


REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

## ASSESSMENT OF UNDER LEACH AREA
## STAGE 3, LIFT 6

300 0 300 600 FEET

1"=300' AT FULL SIZE (ANSI D)
1"=600' AT HALF SIZE (ANSI B)

## LEGEND:

- 2100 — EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- 2100 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- 1590 — HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (FUTURE STAGES)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (FUTURE STAGES)
- PCMS CHANNEL UNDERLYING PAD LINER SYSTEM BENEATH SOLUTION COLLECTOR OR HEADER (SEE NOTE 10)
- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
3. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH CROSSING OVER SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT FIVE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
8. THE PIEZOMETERS, PIEZOMETER CABLE CONDUITS, AND SOLUTION COLLECTION PIPEWORK WILL BE INSTALLED ON THE LEACH PAD BASIN GRADING SURFACE PRIOR TO HEAP LOADING. THE LEACH PAD BASING CONTOURS BENEATH THE HEAP ARE NOT SHOWN FOR CLARITY.
9. THE 3.2 Msf UNDER LEACH AREA IS BASED ON FGMI'S PLANNED 16,000 gpm TOTAL SOLUTION FLOW TO THE HEAP AND THE TARGET 0.005 gpm/sf UNIT APPLICATION RATE. THE LEACH CONFIGURATIONS SHOWN ASSUME LOADING AND LEACHING WILL PROGRESS FROM THE SOUTHWEST TO NORTHEAST EDGES OF THE VALLEY.
10. THE PCMS CHANNELS HAVE BEEN SITED SUCH THAT AT LEAST TWO CHANNELS ARE LOCATED BENEATH THE ACTIVE UNDER LEACH AREA AT ANY TIME, AND SUCH THAT THE PCMS HEADER PIPES (SEE DRAWING 200) WILL DISCHARGE FLOW INTO THE NORTHEAST CORNER OF THE IN-HEAP STORAGE POND FOR MONITORING.

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 3, LIFT 6 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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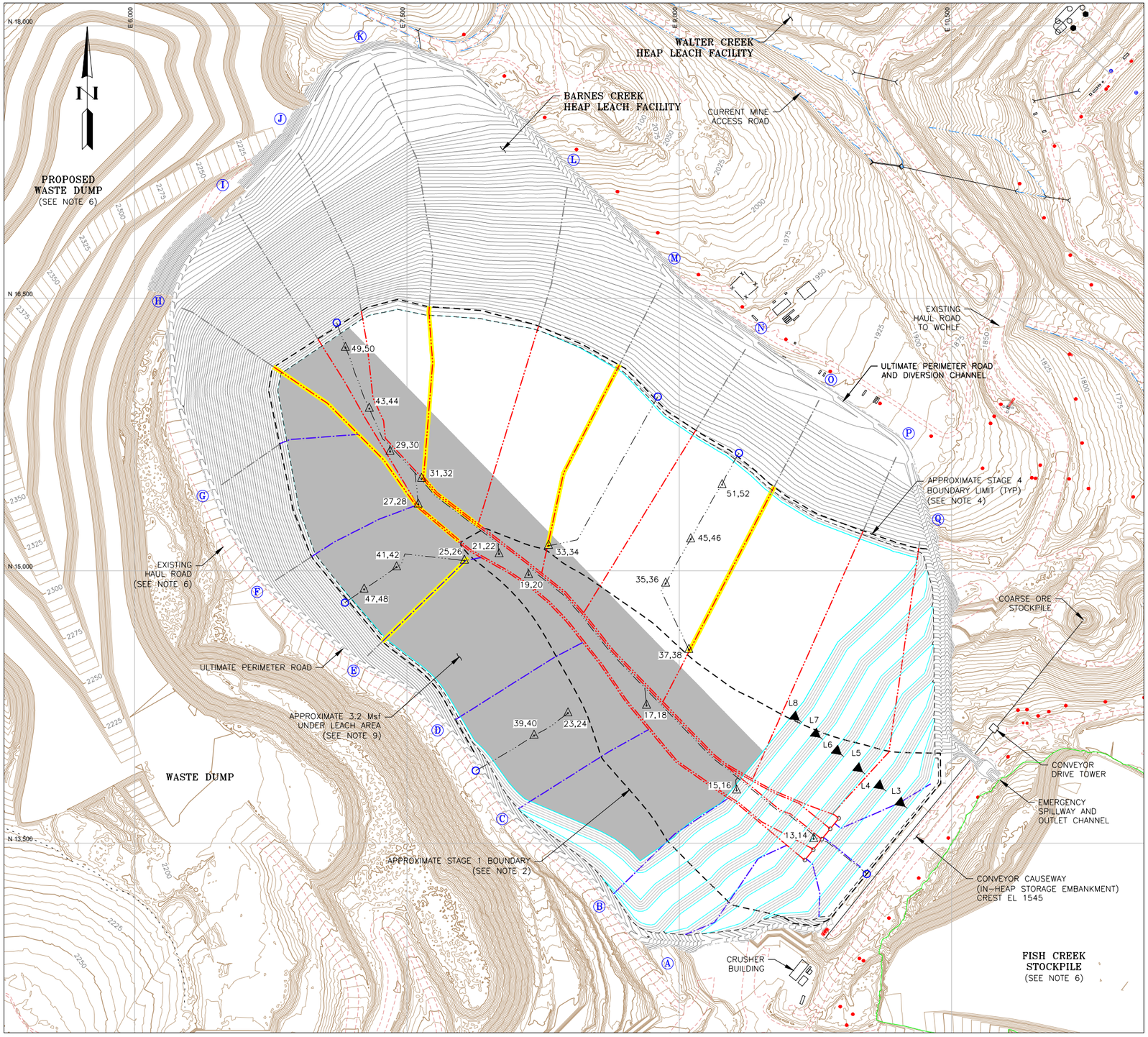


## LEGEND:

- EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (FUTURE STAGES)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (FUTURE STAGES)
- PCMS CHANNEL UNDERLYING PAD LINER SYSTEM BENEATH SOLUTION COLLECTOR OR HEADER (SEE NOTE 10)
- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
3. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH CROSSING OVER SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT FIVE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
8. THE PIEZOMETERS, PIEZOMETER CABLE CONDUITS, AND SOLUTION COLLECTION PIPEWORK WILL BE INSTALLED ON THE LEACH PAD BASIN GRADING SURFACE PRIOR TO HEAP LOADING. THE LEACH PAD BASING CONTOURS BENEATH THE HEAP ARE NOT SHOWN FOR CLARITY.
9. THE 3.2 Msf UNDER LEACH AREA IS BASED ON FGMI'S PLANNED 16,000 gpm TOTAL SOLUTION FLOW TO THE HEAP AND THE TARGET 0.005 gpm/sf UNIT APPLICATION RATE. THE LEACH CONFIGURATIONS SHOWN ASSUME LOADING AND LEACHING WILL PROGRESS FROM THE SOUTHWEST TO NORTHEAST EDGES OF THE VALLEY.
10. THE PCMS CHANNELS HAVE BEEN SITED SUCH THAT AT LEAST TWO CHANNELS ARE LOCATED BENEATH THE ACTIVE UNDER LEACH AREA AT ANY TIME, AND SUCH THAT THE PCMS HEADER PIPES (SEE DRAWING 200) WILL DISCHARGE FLOW INTO THE NORTHEAST CORNER OF THE IN-HEAP STORAGE POND FOR MONITORING.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

ASSESSMENT OF UNDER LEACH AREA
STAGE 4, LIFT 8

300 0 300 600 FEET

1"=300' AT FULL SIZE (ANSI D)
1"=600' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 4, LIFT 8 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 6.04 | 1 |
| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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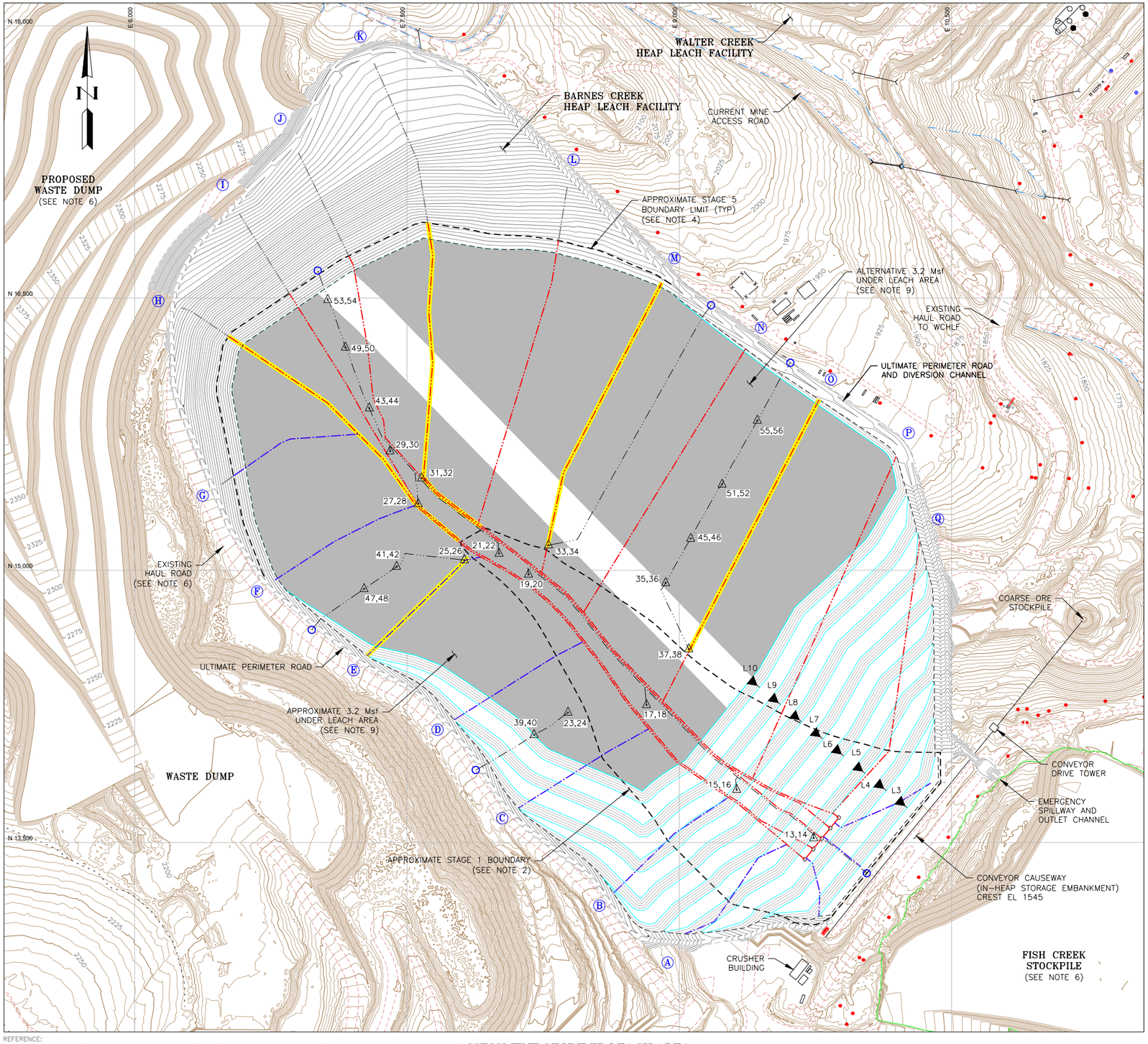


## LEGEND:

- EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (FUTURE STAGES)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (FUTURE STAGES)
- PCMS CHANNEL UNDERLYING PAD LINER SYSTEM BENEATH SOLUTION COLLECTOR OR HEADER (SEE NOTE 10)
- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
3. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH CROSSING OVER SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT FIVE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
8. THE PIEZOMETERS, PIEZOMETER CABLE CONDUITS, AND SOLUTION COLLECTION PIPEWORK WILL BE INSTALLED ON THE LEACH PAD BASIN GRADING SURFACE PRIOR TO HEAP LOADING. THE LEACH PAD BASING CONTOURS BENEATH THE HEAP ARE NOT SHOWN FOR CLARITY.
9. THE 3.2 Msf UNDER LEACH AREA IS BASED ON FGMI'S PLANNED 16,000 gpm TOTAL SOLUTION FLOW TO THE HEAP AND THE TARGET 0.005 gpm/sf UNIT APPLICATION RATE. THE LEACH CONFIGURATIONS SHOWN ASSUME LOADING AND LEACHING WILL PROGRESS FROM THE SOUTHWEST TO NORTHEAST EDGES OF THE VALLEY.
10. THE PCMS CHANNELS HAVE BEEN SITED SUCH THAT AT LEAST TWO CHANNELS ARE LOCATED BENEATH THE ACTIVE UNDER LEACH AREA AT ANY TIME, AND SUCH THAT THE PCMS HEADER PIPES (SEE DRAWING 200) WILL DISCHARGE FLOW INTO THE NORTHEAST CORNER OF THE IN-HEAP STORAGE POND FOR MONITORING.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

ASSESSMENT OF UNDER LEACH AREA
STAGE 5, LIFT 10

300 0 300 600 FEET

1"=300' AT FULL SIZE (ANSI D)
1"=600' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 5, LIFT 10 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 6.05 | 1 |
| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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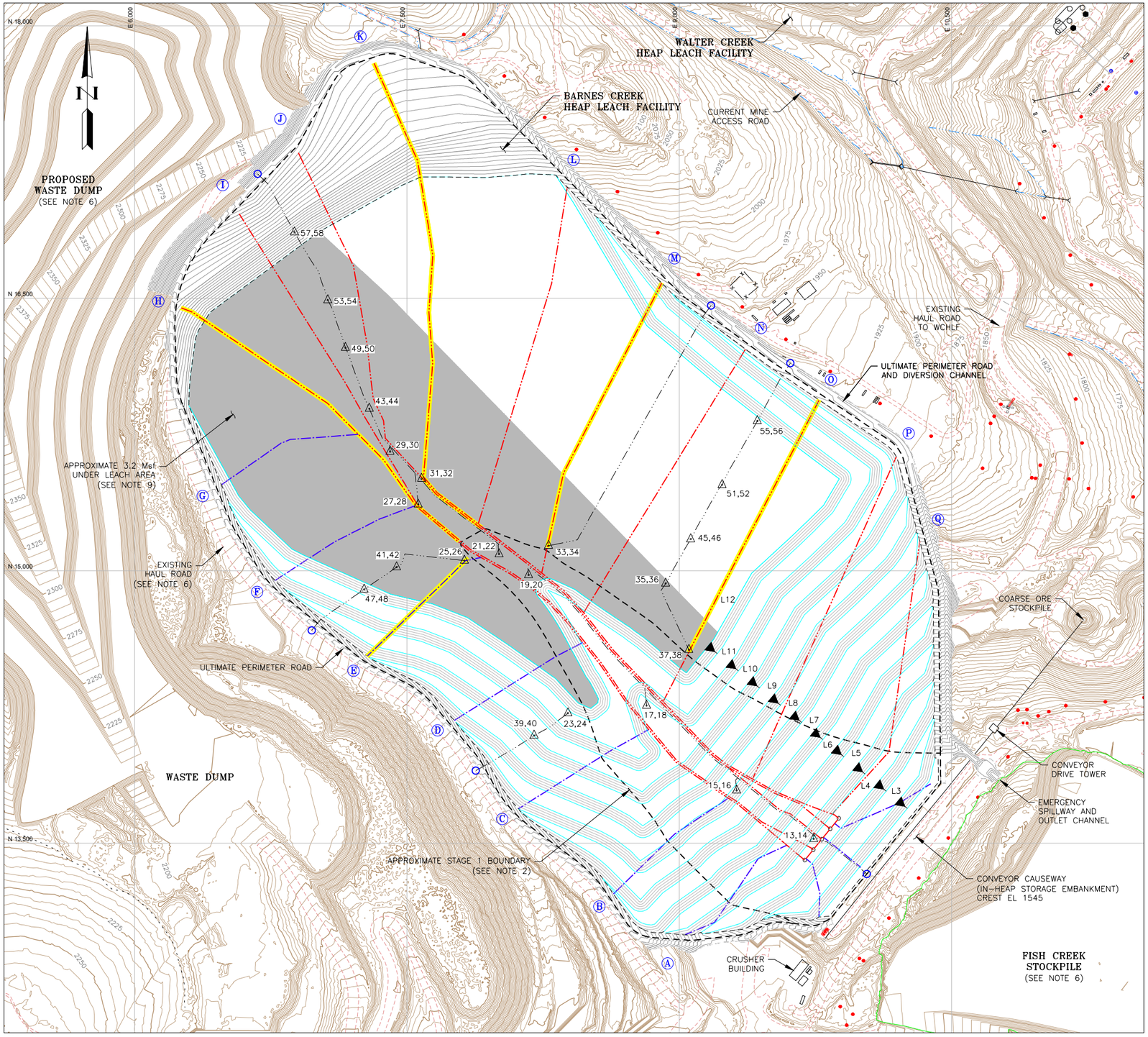


## LEGEND:

- EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
- PCMS CHANNEL UNDERLYING PAD LINER SYSTEM BENEATH SOLUTION COLLECTOR OR HEADER (SEE NOTE 10)
- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
3. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH CROSSING OVER SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT FIVE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
8. THE PIEZOMETERS, PIEZOMETER CABLE CONDUITS, AND SOLUTION COLLECTION PIPEWORK WILL BE INSTALLED ON THE LEACH PAD BASIN GRADING SURFACE PRIOR TO HEAP LOADING. THE LEACH PAD BASING CONTOURS BENEATH THE HEAP ARE NOT SHOWN FOR CLARITY.
9. THE 3.2 Msf UNDER LEACH AREA IS BASED ON FGMI'S PLANNED 16,000 gpm TOTAL SOLUTION FLOW TO THE HEAP AND THE TARGET 0.005 gpm/sf UNIT APPLICATION RATE. THE LEACH CONFIGURATIONS SHOWN ASSUME LOADING AND LEACHING WILL PROGRESS FROM THE SOUTHWEST TO NORTHEAST EDGES OF THE VALLEY.
10. THE PCMS CHANNELS HAVE BEEN SITED SUCH THAT AT LEAST TWO CHANNELS ARE LOCATED BENEATH THE ACTIVE UNDER LEACH AREA AT ANY TIME, AND SUCH THAT THE PCMS HEADER PIPES (SEE DRAWING 200) WILL DISCHARGE FLOW INTO THE NORTHEAST CORNER OF THE IN-HEAP STORAGE POND FOR MONITORING.

REFERENCE:
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- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

ASSESSMENT OF UNDER LEACH AREA
STAGE 6, LIFT 12

300 0 300 600 FEET

1"=300' AT FULL SIZE (ANSI D)
1"=600' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 6, LIFT 12 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 6.06 | 1 |
| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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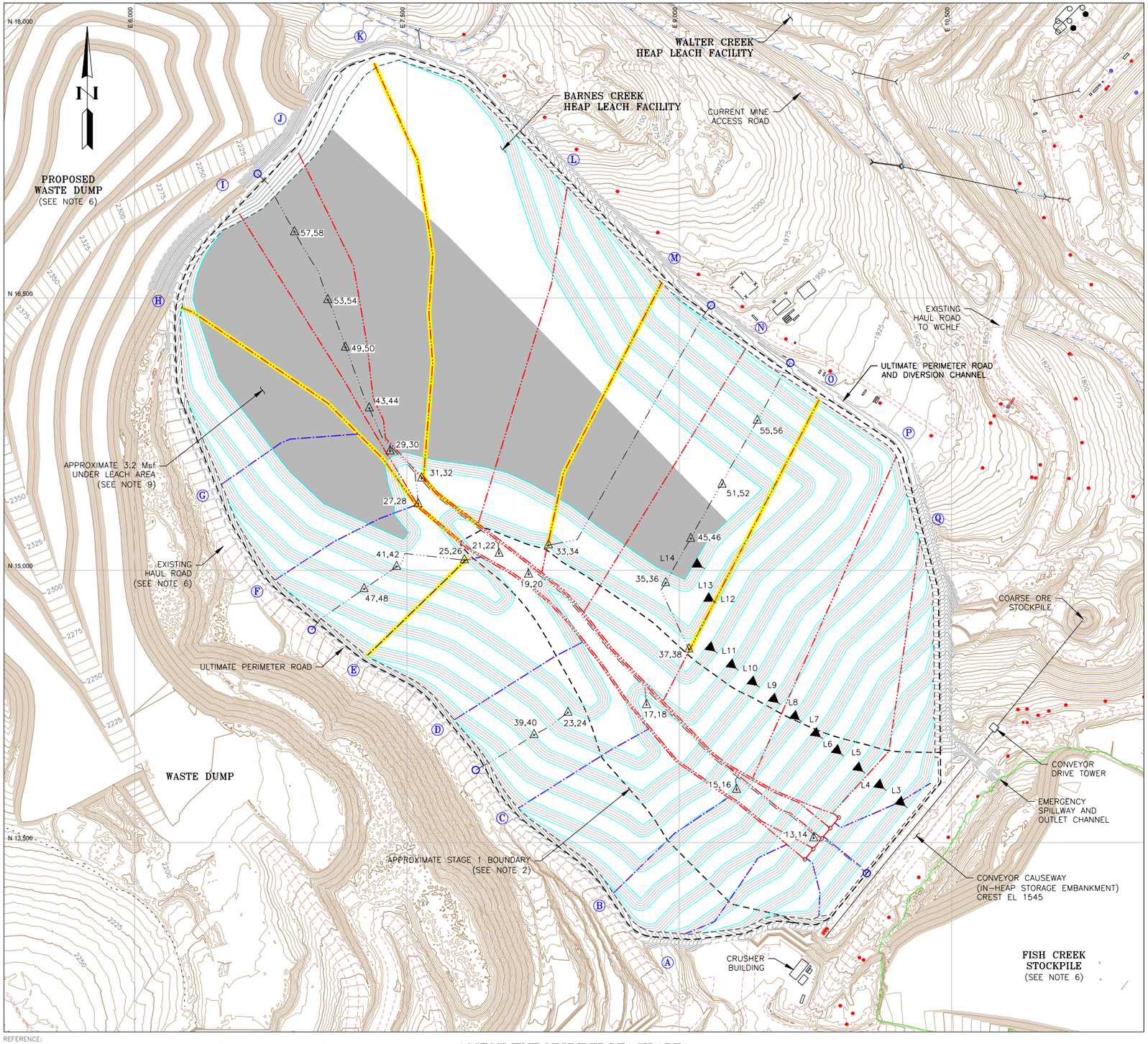


## LEGEND:

- EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
- PCMS CHANNEL UNDERLYING PAD LINER SYSTEM BENEATH SOLUTION COLLECTOR OR HEADER (SEE NOTE 10)
- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
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6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
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REFERENCE:
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ASSESSMENT OF UNDER LEACH AREA
STAGE 6, LIFT 14

300 0 300 600 FEET
1"=300' AT FULL SIZE (ANSI D)
1"=600' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 6, LIFT 14 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 6.07 | 0 |
| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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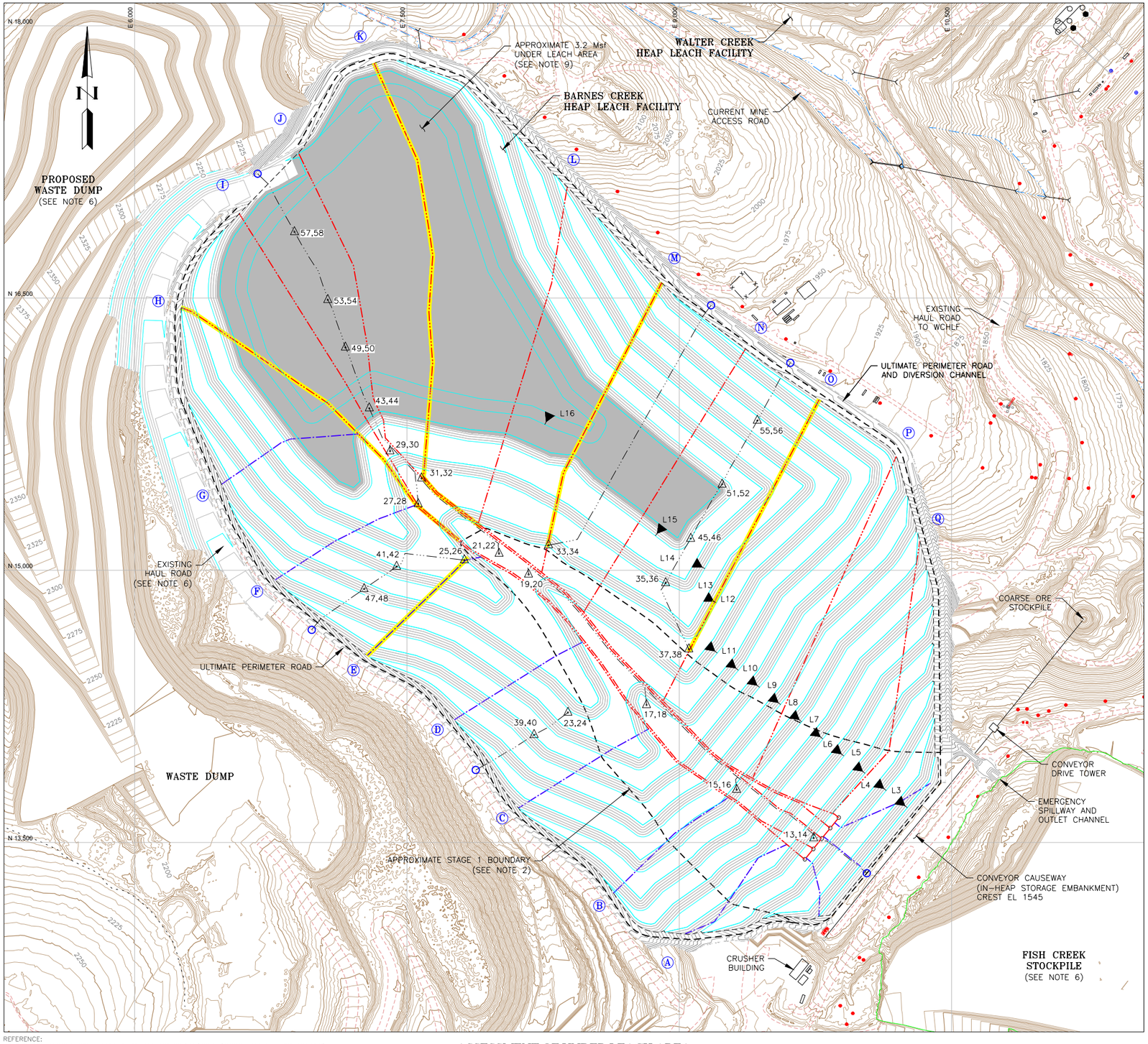


## LEGEND:

- EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
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- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
3. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH CROSSING OVER SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT FIVE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
8. THE PIEZOMETERS, PIEZOMETER CABLE CONDUITS, AND SOLUTION COLLECTION PIPEWORK WILL BE INSTALLED ON THE LEACH PAD BASIN GRADING SURFACE PRIOR TO HEAP LOADING. THE LEACH PAD BASING CONTOURS BENEATH THE HEAP ARE NOT SHOWN FOR CLARITY.
9. THE 3.2 Msf UNDER LEACH AREA IS BASED ON FGMI'S PLANNED 16,000 gpm TOTAL SOLUTION FLOW TO THE HEAP AND THE TARGET 0.005 gpm/sf UNIT APPLICATION RATE. THE LEACH CONFIGURATIONS SHOWN ASSUME LOADING AND LEACHING WILL PROGRESS FROM THE SOUTHWEST TO NORTHEAST EDGES OF THE VALLEY.
10. THE PCMS CHANNELS HAVE BEEN SITED SUCH THAT AT LEAST TWO CHANNELS ARE LOCATED BENEATH THE ACTIVE UNDER LEACH AREA AT ANY TIME, AND SUCH THAT THE PCMS HEADER PIPES (SEE DRAWING 200) WILL DISCHARGE FLOW INTO THE NORTHEAST CORNER OF THE IN-HEAP STORAGE POND FOR MONITORING.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

ASSESSMENT OF UNDER LEACH AREA
STAGE 6, LIFT 16

300 0 300 600 FEET

1"=300' AT FULL SIZE (ANSI D)
1"=600' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 6, LIFT 16 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 6.08 | 0 |
| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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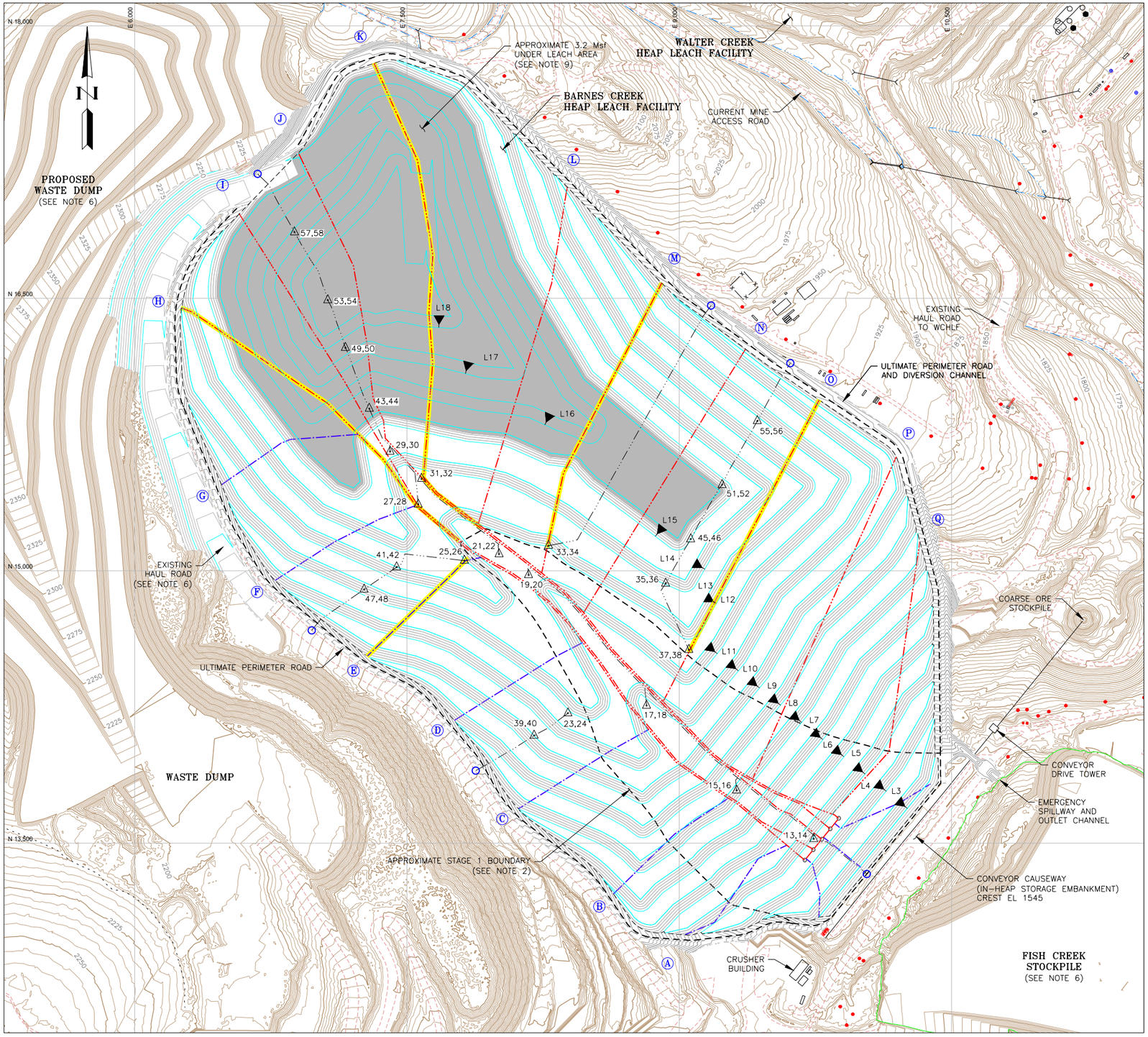


## LEGEND:

- 2100 — EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- 2100 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- 1590 — HEAP SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (THROUGH CURRENT STAGE, SEE NOTE 8)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (THROUGH CURRENT STAGE, SEE NOTE 8)
- PCMS CHANNEL UNDERLYING PAD LINER SYSTEM BENEATH SOLUTION COLLECTOR OR HEADER (SEE NOTE 10)
- VERTICAL SOLUTION EXTRACTION WELL
- △13-14 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 2, 5, 7, AND 8)
- PIEZOMETER CABLE CONDUIT (SEE NOTES 5 AND 8)
- PIEZOMETER READOUT BOX (SEE NOTE 5)
- (A) SOLUTION COLLECTOR/HEADER DESIGNATION

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. PCMS CHANNELS ARE ONLY PLANNED FOR CONSTRUCTION ABOVE THE STAGE 1 BOUNDARY LIMIT WHERE THE LEACH PAD LINER SYSTEM CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
3. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH CROSSING OVER SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT FIVE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. PIEZOMETERS HAVE BEEN SITED BETWEEN SOLUTION LATERALS AND ADJACENT TO SOLUTION HEADERS, NEAR THE LOWER ENDS OF EACH PCMS CHANNEL, TO MONITOR POTENTIALLY DIFFERENT CONDITIONS ABOVE THE PAD LINER SYSTEM. REFER TO DRAWINGS 500 AND 720 FOR MORE INFORMATION.
8. THE PIEZOMETERS, PIEZOMETER CABLE CONDUITS, AND SOLUTION COLLECTION PIPEWORK WILL BE INSTALLED ON THE LEACH PAD BASIN GRADING SURFACE PRIOR TO HEAP LOADING. THE LEACH PAD BASING CONTOURS BENEATH THE HEAP ARE NOT SHOWN FOR CLARITY.
9. THE 3.2 Msf UNDER LEACH AREA IS BASED ON FGMI'S PLANNED 16,000 gpm TOTAL SOLUTION FLOW TO THE HEAP AND THE TARGET 0.005 gpm/sf UNIT APPLICATION RATE. THE LEACH CONFIGURATIONS SHOWN ASSUME LOADING AND LEACHING WILL PROGRESS FROM THE SOUTHWEST TO NORTHEAST EDGES OF THE VALLEY.
10. THE PCMS CHANNELS HAVE BEEN SITED SUCH THAT AT LEAST TWO CHANNELS ARE LOCATED BENEATH THE ACTIVE UNDER LEACH AREA AT ANY TIME, AND SUCH THAT THE PCMS HEADER PIPES (SEE DRAWING 200) WILL DISCHARGE FLOW INTO THE NORTHEAST CORNER OF THE IN-HEAP STORAGE POND FOR MONITORING.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY). DV101-00089-FortKnox -- EG-TOPO-2014-08-05
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-EG-FromClient_04292015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015. DV101-00089-FortKnox -- 79-BCHLF -- 79-BC-FromClient_WasteDump
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017. DV101-00089-FortKnox -- 99-BCHLF_Final_Des -- 99-FishCreekStockpileBackfill-FGMI-022717

ASSESSMENT OF UNDER LEACH AREA
STAGE 6, LIFT 18

300 0 300 600 FEET

1"=300' AT FULL SIZE (ANSI D)
1"=600' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | PCMS CHANNELS AND<br>UNDER LEACH AREA ASSESSMENT<br>STAGE 6, LIFT 18 |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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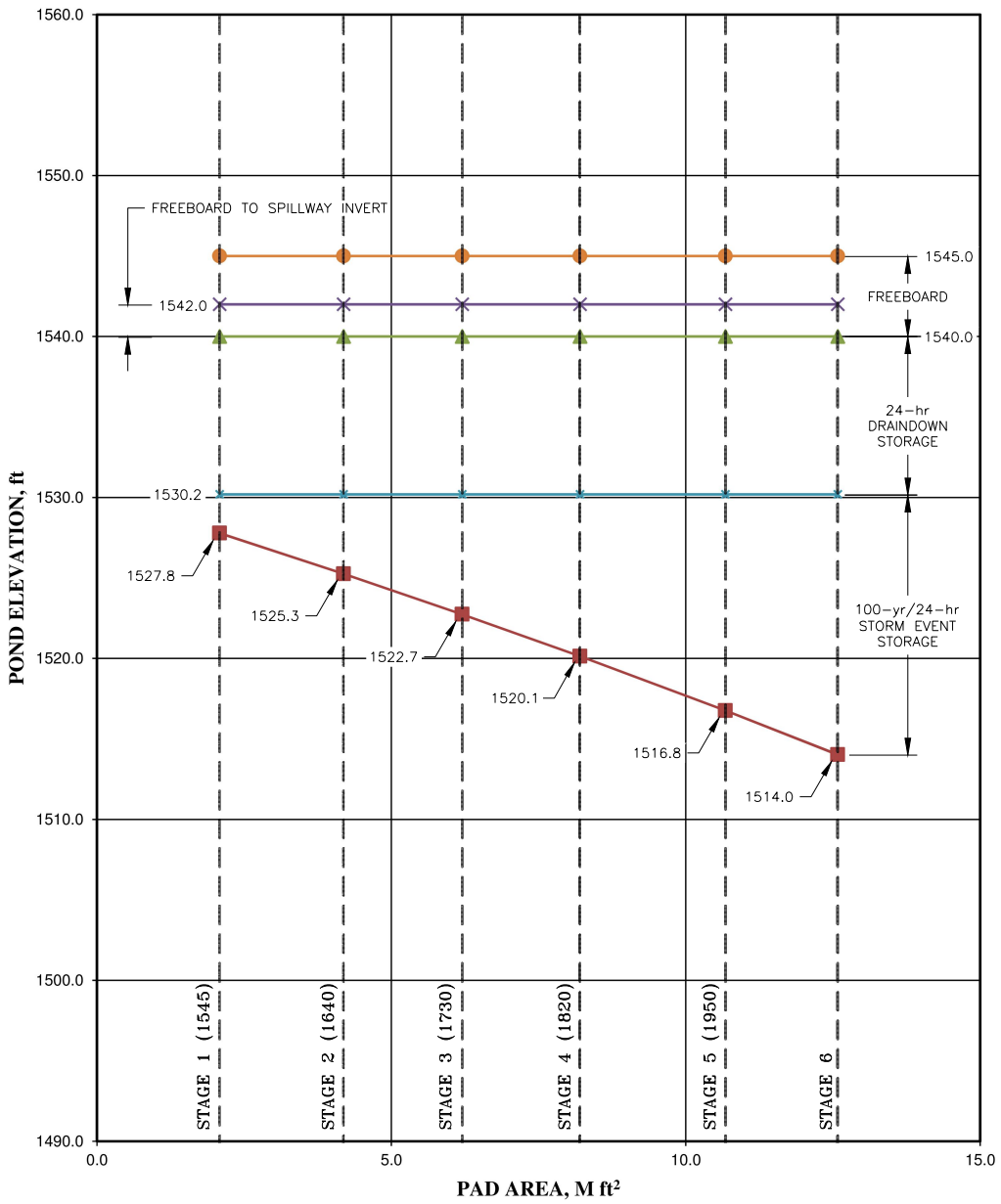


## NOTES:

1. THE POND LEVELS PRESENTED ARE BASED ON THE IN-HEAP STORAGE POND FILLING CURVE DEVELOPED FOR THE BCHLF DESIGN.
2. THE PORE CAPACITY OF THE ORE WITHIN THE BCHLF IN-HEAP STORAGE POND CONSIDERS THE ULTIMATE PROPOSED HEAP CONFIGURATION AND HEIGHT OF MATERIAL ABOVE THE POND FLOOR.
3. THE 100-yr/24-hr STORM EVENT COMPONENT WAS BASED ON A MONTHLY MAXIMUM RAIN ON SNOW EVENT OCCURRING IN SEPTEMBER WITH A RESULTING PRECIPITATION DEPTH OF 4.0 inches.
4. THE BCHLF 24-hr DRAINDOWN COMPONENT CONSIDERS THE FULL 16,000 gpm SOLUTION FLOW RATE OVER THE 24-hr PERIOD.
5. THE LOWER BOUNDS OF THE POND LEVEL OPERATING RANGE FOR THE BCHLF IS RECOMMENDED TO BE INITIALLY SET AT EL 1494' (APPROXIMATELY 40' TO 44' ABOVE THE BASE OF THE SOLUTION EXTRACTION WELLS); HOWEVER, THE FINAL LOWER BOUND WILL BE ESTABLISHED BY FGMI THROUGH OPERATIONAL EXPERIENCE.
6. THE STAGE ELEVATIONS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE STAGE PERIMETER ROAD.

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGED PAD AREAS VERSUS IN-HEAP STORAGE POND ELEVATIONS

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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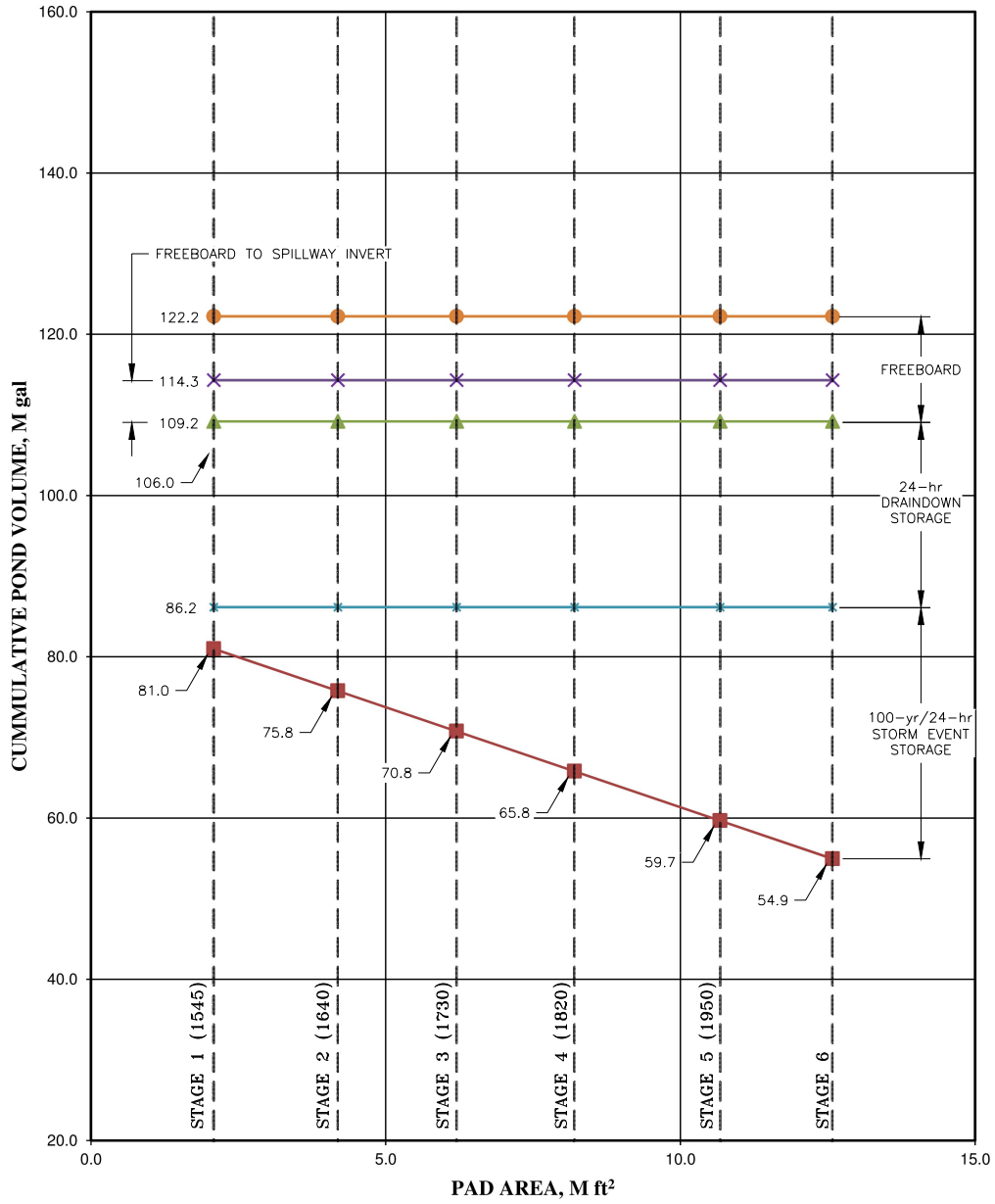


## NOTES:

1. THE POND LEVELS PRESENTED ARE BASED ON THE IN-HEAP STORAGE POND FILLING CURVE DEVELOPED FOR THE BCHLF DESIGN.
2. THE PORE CAPACITY OF THE ORE WITHIN THE BCHLF IN-HEAP STORAGE POND CONSIDERS THE ULTIMATE PROPOSED HEAP CONFIGURATION AND HEIGHT OF MATERIAL ABOVE THE POND FLOOR.
3. THE 100-yr/24-hr STORM EVENT COMPONENT WAS BASED ON A MONTHLY MAXIMUM RAIN ON SNOW EVENT OCCURRING IN SEPTEMBER WITH A RESULTING PRECIPITATION DEPTH OF 4.0 inches.
4. THE BCHLF 24-hr DRAINDOWN COMPONENT CONSIDERS THE FULL 16,000 gpm SOLUTION FLOW RATE OVER THE 24-hr PERIOD.
5. THE LOWER BOUNDS OF THE POND LEVEL OPERATING RANGE FOR THE BCHLF IS RECOMMENDED TO BE INITIALLY SET AT EL 1494' (APPROXIMATELY 40' TO 44' ABOVE THE BASE OF THE SOLUTION EXTRACTION WELLS); HOWEVER, THE FINAL LOWER BOUND WILL BE ESTABLISHED BY FGMI THROUGH OPERATIONAL EXPERIENCE.
6. THE STAGE ELEVATIONS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE STAGE PERIMETER ROAD.

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGED PAD AREAS VERSUS IN-HEAP STORAGE POND VOLUMES

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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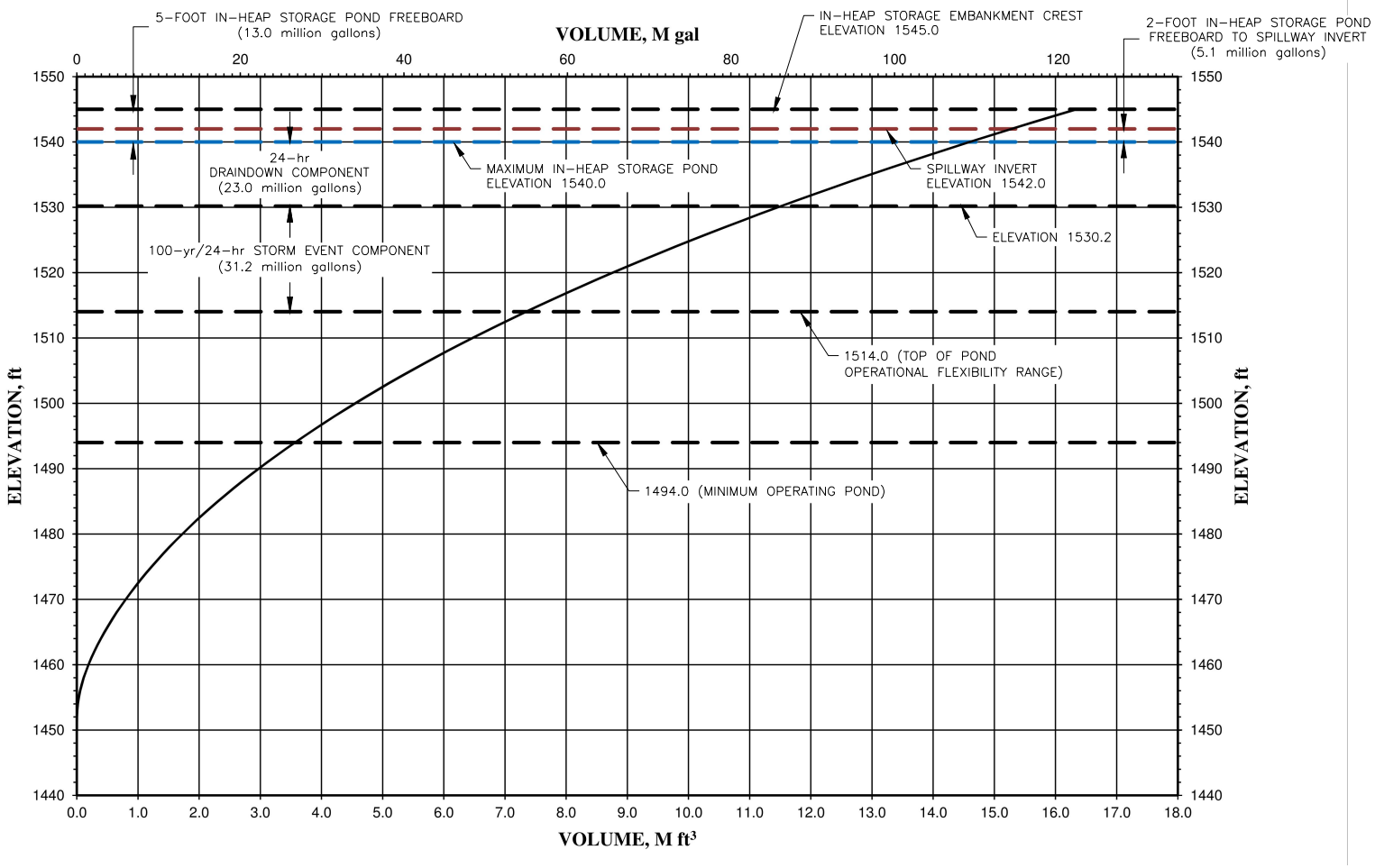


## NOTES:

1. THE POND LEVELS PRESENTED ARE BASED ON THE IN-HEAP STORAGE POND FILLING CURVE DEVELOPED FOR THE BCHLF DESIGN.
2. THE PORE CAPACITY OF THE ORE WITHIN THE BCHLF IN-HEAP STORAGE POND CONSIDERS THE ULTIMATE PROPOSED HEAP CONFIGURATION AND HEIGHT OF MATERIAL ABOVE THE POND FLOOR.
3. THE 100-yr/24-hr STORM EVENT COMPONENT WAS BASED ON A MONTHLY MAXIMUM RAIN ON SNOW EVENT OCCURRING IN SEPTEMBER WITH A RESULTING PRECIPITATION DEPTH OF 4.0 inches.
4. THE BCHLF 24-hr DRAINDOWN COMPONENT CONSIDERS THE FULL 16,000 gpm SOLUTION FLOW RATE OVER THE 24-hr PERIOD.
5. THE LOWER BOUNDS OF THE POND LEVEL OPERATING RANGE FOR THE BCHLF IS RECOMMENDED TO BE INITIALLY SET AT EL 1494' (APPROXIMATELY 40' TO 44' ABOVE THE BASE OF THE SOLUTION EXTRACTION WELLS); HOWEVER, THE FINAL LOWER BOUND WILL BE ESTABLISHED BY FGMI THROUGH OPERATIONAL EXPERIENCE.
6. THE STAGE ELEVATIONS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE STAGE PERIMETER ROAD.

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | IN-HEAP STORAGE POND FILLING CURVE AND STAGE 6 (FINAL) STORAGE COMPONENTS |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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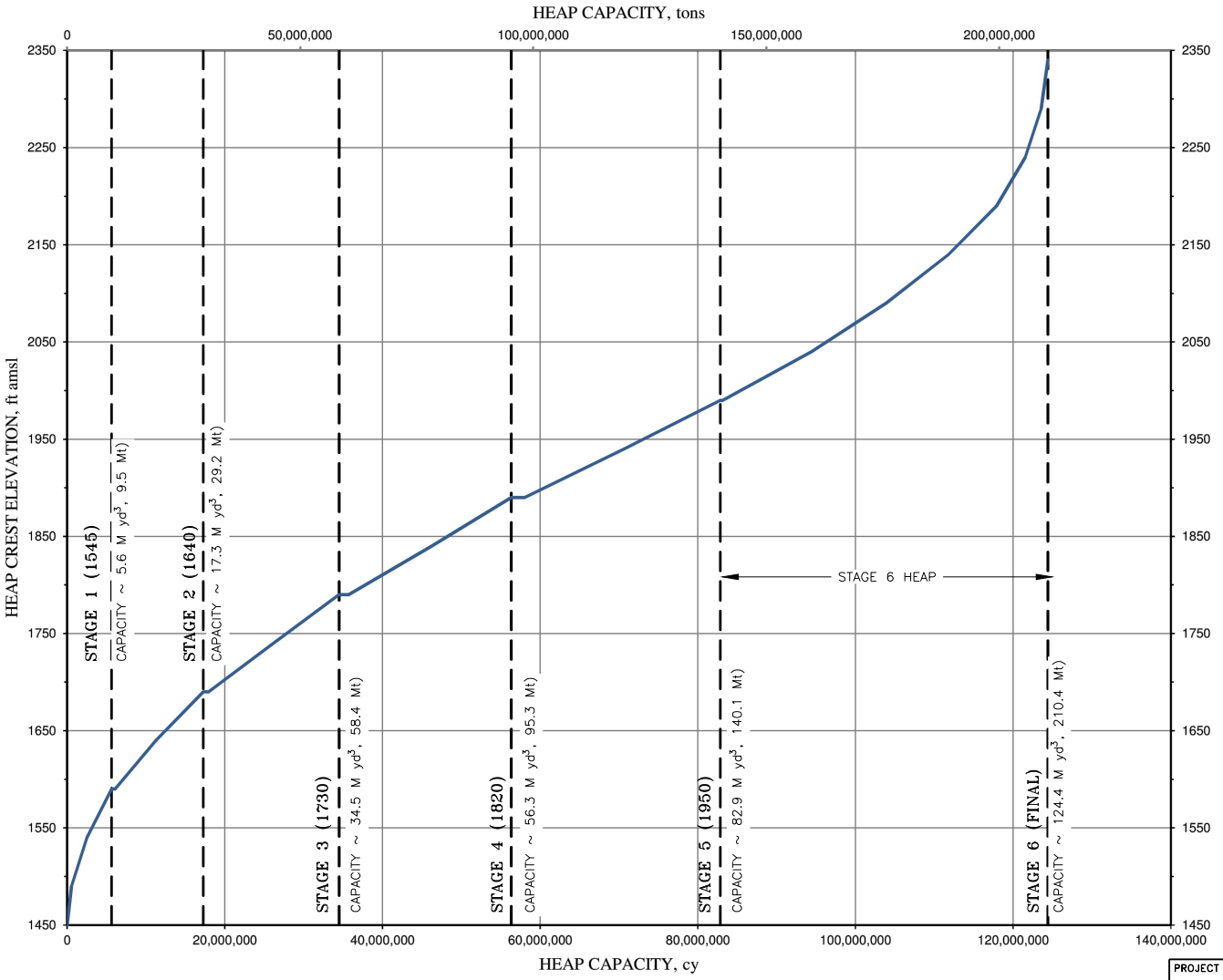


## NOTES:

1. THE APPROXIMATE HEAP CAPACITY IS BASED ON AN ORE DENSITY OF 125.3 lb/ft$^3$.
2. THE STAGE ELEVATIONS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE STAGE PERIMETER ROAD.

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: ORE HEAP FILLING CURVE

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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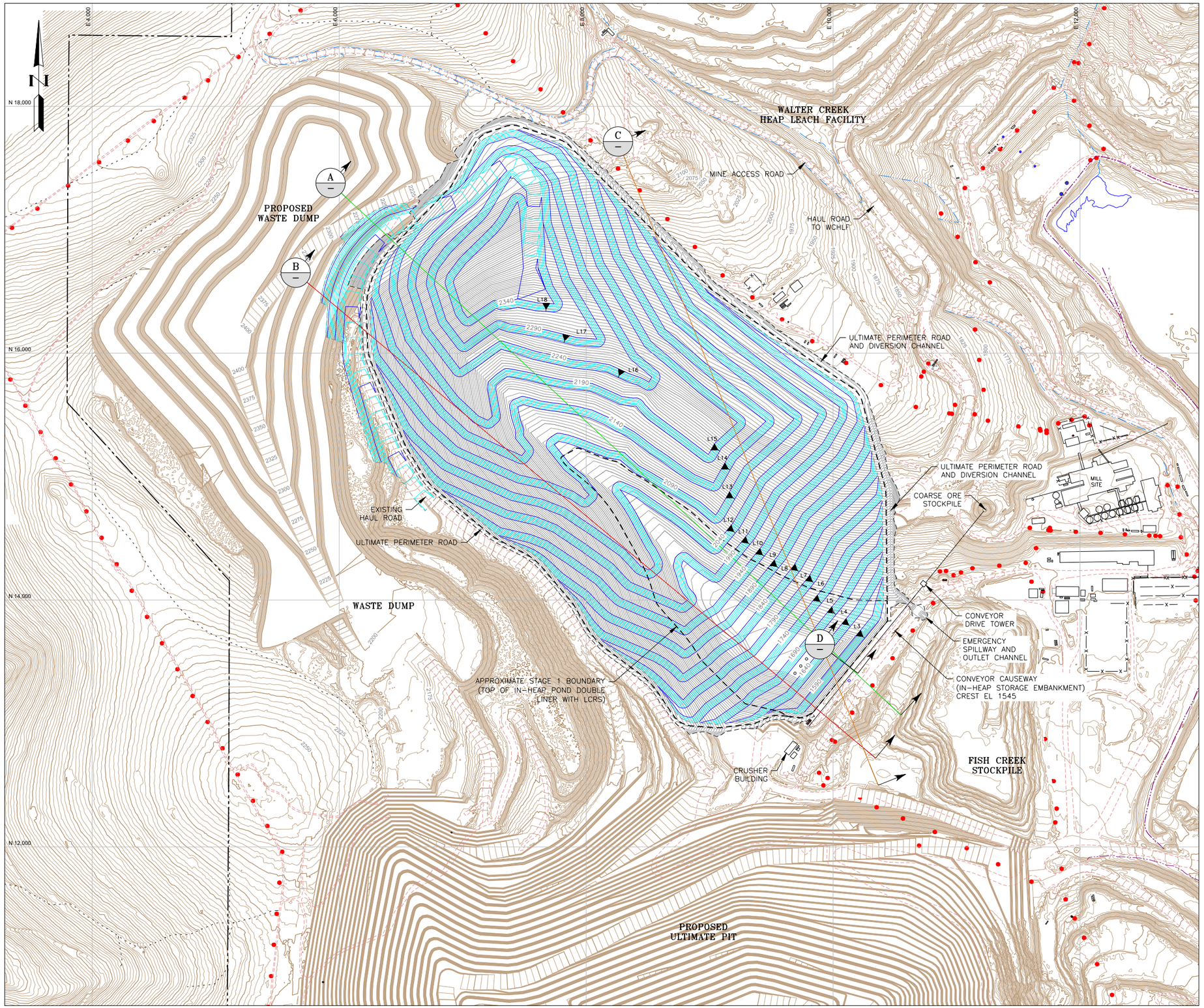


## LEGEND:

- 2100 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 1550 — HEAP SURFACE CONTOUR AND EL, FEET
- 2100 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- VERTICAL SOLUTION EXTRACTION WELL
- L3 — HEAP LIFT DESIGNATION
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).

## REVISION:

1 REMOVED NON-ESSENTIAL INFORMATION (EXTERNAL CHANNELS AND CULVERTS). REMOVED SOUTH PERIMETER OUTLET CHANNEL.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SLOPE STABILITY SECTIONS LOCATION PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| | | | | | |
|---|---|---|---|---|---|
| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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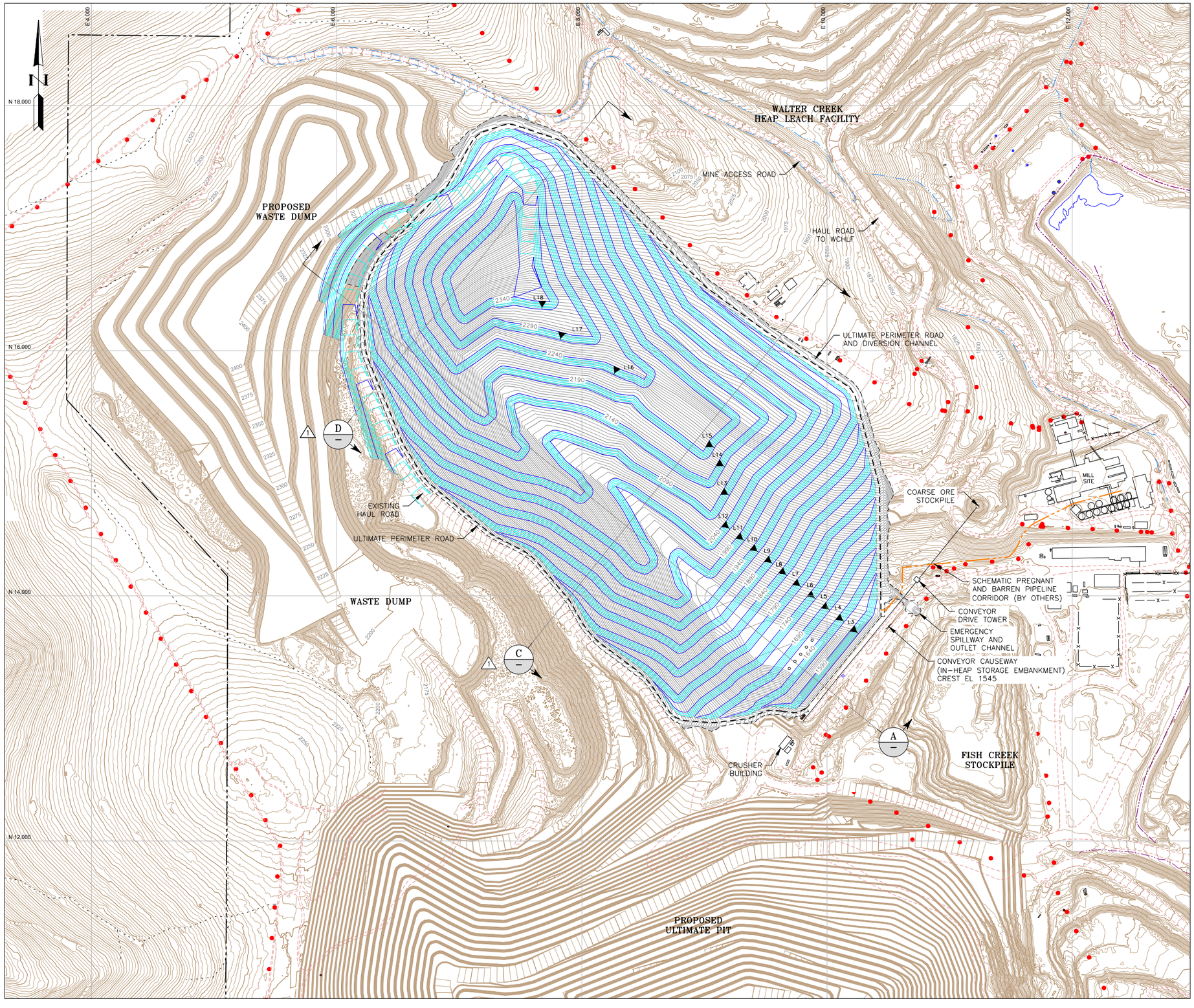


## LEGEND:

- 2100 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 1550 — HEAP SURFACE CONTOUR AND EL, FEET
- 2100 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- VERTICAL SOLUTION EXTRACTION WELL
- L3 — HEAP LIFT DESIGNATION
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. (1) SECTION B, FROM THE GEOTECHNICAL SECTIONS AND GEOLOGIC CROSS-SECTIONS (SEE DRAWINGS 110, 120, AND 132), IS CUT ALONG THE CREST OF THE IN-HEAP STORAGE POND EMBANKMENT AND WAS NOT USED IN THE SETTLEMENT ANALYSES.

## REVISION:

(1) RENAMED PREVIOUS SECTIONS B AND C TO SECTIONS C AND D, RESPECTIVELY, TO MATCH SETTLEMENT CALCULATION PACKAGE. ADDED NOTE 2 DISCUSSING THE OMISSION OF SECTION B FROM THE SETTLEMENT ANALYSES.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SETTLEMENT SECTIONS LOCATION PLAN

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 7.02 | 1 |
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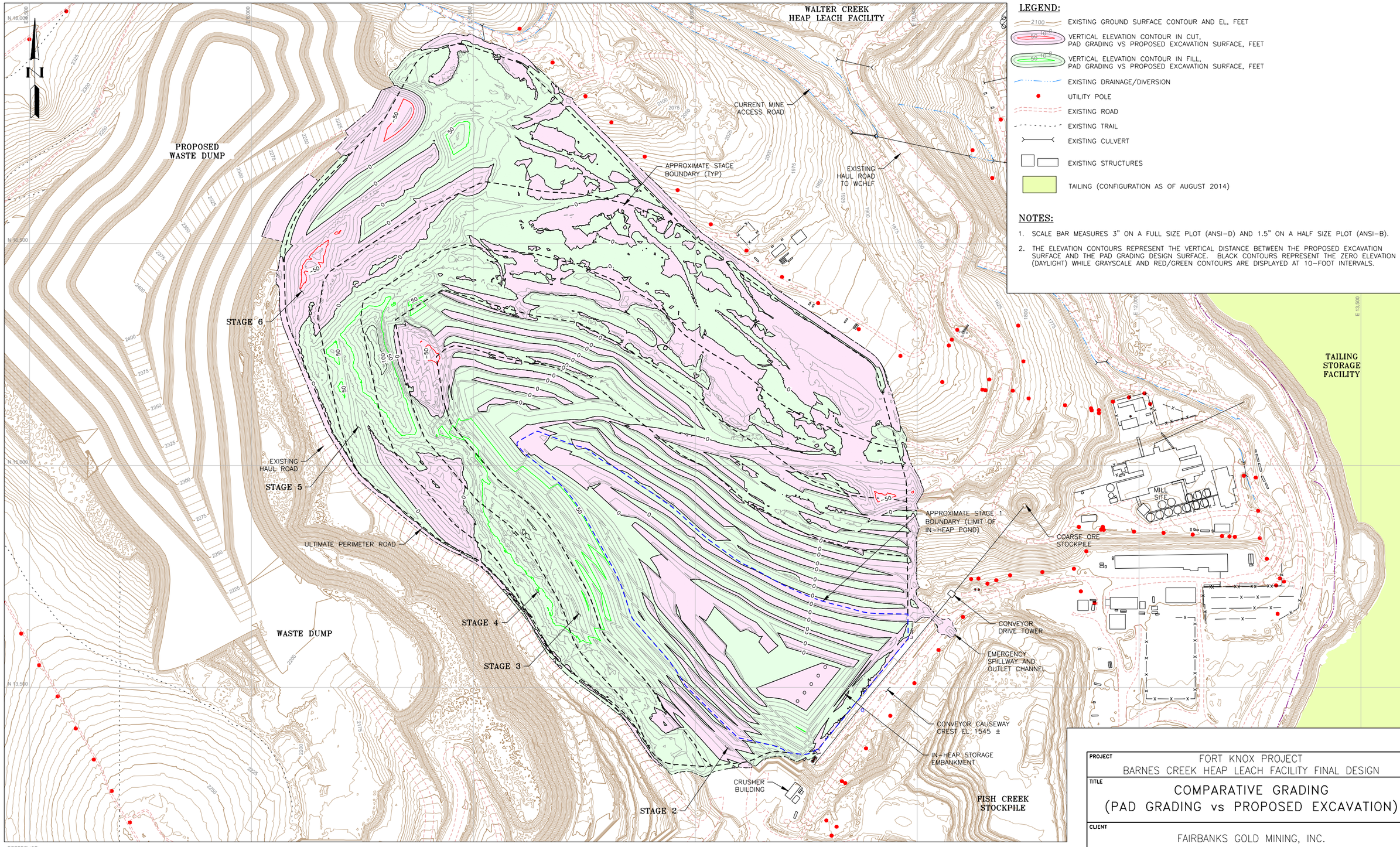


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- VERTICAL ELEVATION CONTOUR IN CUT, PAD GRADING VS PROPOSED EXCAVATION SURFACE, FEET
- VERTICAL ELEVATION CONTOUR IN FILL, PAD GRADING VS PROPOSED EXCAVATION SURFACE, FEET
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- EXISTING STRUCTURES
- TAILING (CONFIGURATION AS OF AUGUST 2014)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE ELEVATION CONTOURS REPRESENT THE VERTICAL DISTANCE BETWEEN THE PROPOSED EXCAVATION SURFACE AND THE PAD GRADING DESIGN SURFACE. BLACK CONTOURS REPRESENT THE ZERO ELEVATION (DAYLIGHT) WHILE GRAYSCALE AND RED/GREEN CONTOURS ARE DISPLAYED AT 10-FOOT INTERVALS.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

| | |
|---|---|
| PROJECT | FORT KNOX PROJECT<br>BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN |
| TITLE | COMPARATIVE GRADING<br>(PAD GRADING vs PROPOSED EXCAVATION) |
| CLIENT | FAIRBANKS GOLD MINING, INC. |

Knight Piésold CONSULTING

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 9.01 | 0 |
| ACTIVITY CODE | N/A | XREF NUMBER | N/A | | |

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Knight Piésold
CONSULTING

# Drawings

# FAIRBANKS GOLD MINING, INC.
# FORT KNOX PROJECT

# BARNES CREEK HEAP LEACH FACILITY
# FINAL DESIGN

## REISSUED FOR CONSTRUCTION
## MARCH 24, 2017

Mar. 24/17

DRAWING INDEX

| DRAWING NUMBER | DRAWING TITLE | REVISION | REV DATE |
|---|---|---|---|
| 000 | COVER SHEET AND INDEX | 2 | 03/24/17 |
| 100 | PROJECT LOCATION AND OVERALL SITE PLAN | 0 | 01/27/17 |
| 110 | GEOTECHNICAL INVESTIGATION PLAN | 0 | 01/27/17 |
| 120 | GEOLOGIC MAP WITH PRE-DISTURBANCE TOPOGRAPHY | 0 | 01/27/17 |
| 130 | SITE GEOLOGIC SECTIONS, SHEET 1 OF 2 | 1 | 03/24/17 |
| 132 | SITE GEOLOGIC SECTIONS, SHEET 2 OF 2 | 0 | 01/27/17 |
| 140 | GENERAL ARRANGEMENT PLAN (TOPOGRAPHIC MAP OF DAM SITE) | 2 | 03/24/17 |
| 142 | IN-HEAP STORAGE POND EMBANKMENT PROFILE AND CROSS-SECTIONS | 2 | 03/24/17 |
| 144 | IN-HEAP STORAGE POND EMBANKMENT GENERAL DETAILS | 2 | 03/17/17 |
| 200 | UNDERDRAIN SYSTEM AND PCMS PLAN | 2 | 03/24/17 |
| 202 | UNDERDRAIN RANDOM FILL PROFILES AND HORIZONTAL CONTROL DATA | 1 | 03/24/17 |
| 204 | PCMS CHANNEL PROFILES AND HORIZONTAL CONTROL DATA, SHEET 1 OF 2 | 1 | 03/17/17 |
| 205 | PCMS CHANNEL PROFILES AND HORIZONTAL CONTROL DATA, SHEET 2 OF 2 | 0 | 03/17/17 |
| 206 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 1 OF 8 | 2 | 03/24/17 |
| 210 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 2 OF 8 | 1 | 03/17/17 |
| 215 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 3 OF 8 | 2 | 03/17/17 |
| 220 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 4 OF 8 | 2 | 03/17/17 |
| 222 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 5 OF 8 | 0 | 03/17/17 |
| 225 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 6 OF 8 | 1 | 03/17/17 |
| 230 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 7 OF 8 | 1 | 03/17/17 |
| 235 | UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 8 OF 8 | 1 | 03/17/17 |
| 300 | IN-HEAP STORAGE POND LCRS PLAN | 2 | 03/17/17 |
| 310 | IN-HEAP STORAGE POND LCRS SECTIONS AND DETAILS, SHEET 1 OF 2 | 2 | 03/24/17 |
| 320 | IN-HEAP STORAGE POND LCRS SECTIONS AND DETAILS, SHEET 2 OF 2 | 2 | 03/24/17 |
| 400 | STAGE 1 GRADING PLAN | 2 | 03/17/17 |
| 401 | IN-HEAP STORAGE POND GRADING PLAN | 2 | 03/17/17 |
| 410 | STAGE GRADING PLAN THRU STAGE 6 | 2 | 03/17/17 |
| 420 | TYPICAL GRADING SECTIONS | 0 | 01/27/17 |
| 430 | STAGE 1 PERIMETER ROAD PROFILE AND HORIZONTAL CONTROL DATA | 1 | 03/17/17 |
| 435 | STAGE 1 HAUL ROAD PROFILE AND HORIZONTAL CONTROL DATA | 0 | 01/27/17 |
| 480 | ULTIMATE PERIMETER ROAD PROFILE | 0 | 01/27/17 |

DRAWING INDEX

| DRAWING NUMBER | DRAWING TITLE | REVISION | REV DATE |
|---|---|---|---|
| 485 | ULTIMATE PERIMETER ROAD HORIZONTAL CONTROL DATA | 0 | 01/27/17 |
| 500 | SOLUTION COLLECTION PIPEWORK AND INSTRUMENTATION PLAN | 2 | 03/17/17 |
| 510 | VERTICAL SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 1 OF 3 | 0 | 01/27/17 |
| 512 | VERTICAL SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 2 OF 3 | 0 | 01/27/17 |
| 514 | VERTICAL SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 3 OF 3 | 0 | 01/27/17 |
| 516 | INCLINED SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 1 OF 3 | 2 | 03/17/17 |
| 517 | INCLINED SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 2 OF 3 | 1 | 02/07/17 |
| 518 | INCLINED SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 3 OF 3 | 1 | 03/17/17 |
| 520 | SOLUTION PIPEWORK AND LINER SYSTEM SECTIONS AND DETAILS | 1 | 03/17/17 |
| 530 | PAD PERIMETER SECTIONS AND DETAILS, SHEET 1 OF 3 | 1 | 03/17/17 |
| 532 | PAD PERIMETER SECTIONS AND DETAILS, SHEET 2 OF 3 | 1 | 03/17/17 |
| 534 | PAD PERIMETER SECTIONS AND DETAILS, SHEET 3 OF 3 | 2 | 03/17/17 |
| 540 | SOUTH PERIMETER DIVERSION CULVERTS PLAN AND PROFILE | 1 | 03/17/17 |
| 542 | SOUTH PERIMETER DIVERSION CULVERTS SECTIONS AND DETAILS, SHEET 1 OF 2 | 1 | 03/17/17 |
| 544 | SOUTH PERIMETER DIVERSION CULVERTS SECTIONS AND DETAILS, SHEET 2 OF 2 | 1 | 02/07/17 |
| 550 | IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL PLAN AND PROFILE, STAGE 1 | 2 | 03/17/17 |
| 551 | IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL PLAN, STAGE 2 | 2 | 03/17/17 |
| 552 | IN-HEAP STORAGE POND SPILLWAY SECTIONS AND DETAILS, SHEET 1 OF 3 | 2 | 03/17/17 |
| 554 | IN-HEAP STORAGE POND SPILLWAY SECTIONS AND DETAILS, SHEET 2 OF 3 | 2 | 03/17/17 |
| 556 | IN-HEAP STORAGE POND SPILLWAY SECTIONS AND DETAILS, SHEET 3 OF 3 | 2 | 03/17/17 |
| 560 | FISH CREEK STORM WATER DETENTION BASIN PLAN | 0 | 03/17/17 |
| 562 | FISH CREEK STORM WATER DETENTION BASIN TYPICAL SECTIONS | 0 | 03/24/17 |
| 600 | STAGE 1 SCHEMATIC HEAP CONFIGURATION | 2 | 03/17/17 |
| 602 | STAGE 2 (EL 1640) SCHEMATIC HEAP CONFIGURATION | 2 | 03/17/17 |
| 604 | STAGE 3 (EL 1730) SCHEMATIC HEAP CONFIGURATION | 2 | 03/17/17 |
| 606 | STAGE 4 (EL 1820) SCHEMATIC HEAP CONFIGURATION | 2 | 03/17/17 |
| 608 | STAGE 5 (EL 1950) SCHEMATIC HEAP CONFIGURATION | 2 | 03/17/17 |
| 610 | STAGE 6 (FINAL) SCHEMATIC HEAP CONFIGURATION | 2 | 03/17/17 |
| 620 | SCHEMATIC HEAP SECTIONS AND DETAILS, SHEET 1 OF 2 | 0 | 01/27/17 |
| 622 | SCHEMATIC HEAP SECTIONS AND DETAILS, SHEET 2 OF 2 | 1 | 02/07/17 |
| 720 | INSTRUMENTATION SECTIONS AND DETAILS | 2 | 03/17/17 |

Prepared for:

FAIRBANKS GOLD MINING, INC.

#1 FORT KNOX ROAD
P.O. BOX 73726
FAIRBANKS, AK 99707-3726

Prepared by:


Knight Piésold CONSULTING

1999 Broadway, Suite 600
Denver, Colorado 80202

460 West Silver Street, Suite 106
Elko, Nevada 89801

3275 W. Ina Road, Suite 109
Tucson, Arizona 85741



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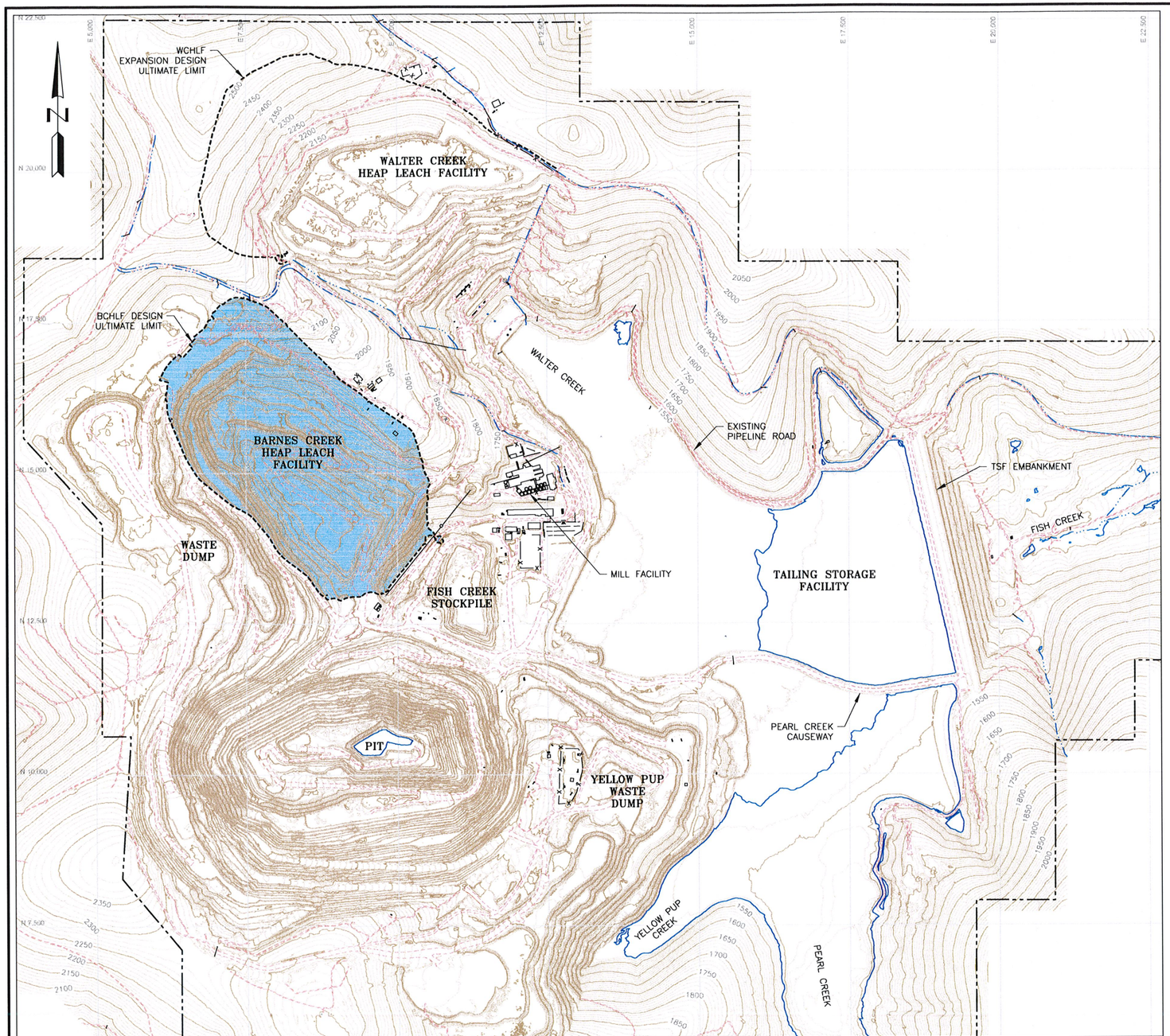



REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- 2015 AS-BUILT INFORMATION PROVIDED BY AAP AND FGMI, NOVEMBER 2015.

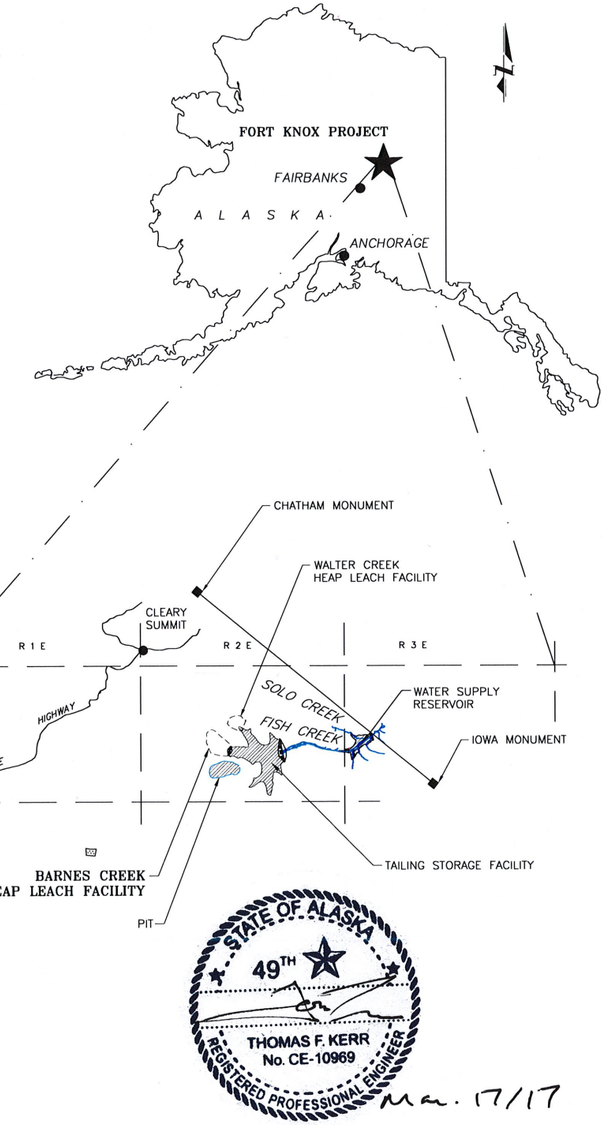
PLAN

1000 0 1000 2000 FEET

1" = 1000' AT FULL SIZE (ANSI D)
1" = 2000' AT HALF SIZE (ANSI B)

**LEGEND:**

- 2200 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING DRAINAGE/DIVERSION
- EXISTING ROAD
- FENCING
- TSF SUPERNATANT POND LIMITS
- EXISTING STRUCTURES




STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).

| REV | DATE | DESCRIPTION | APP'D |
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| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE: N/A | XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: PROJECT LOCATION AND OVERALL SITE PLAN

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 100 | 0 |


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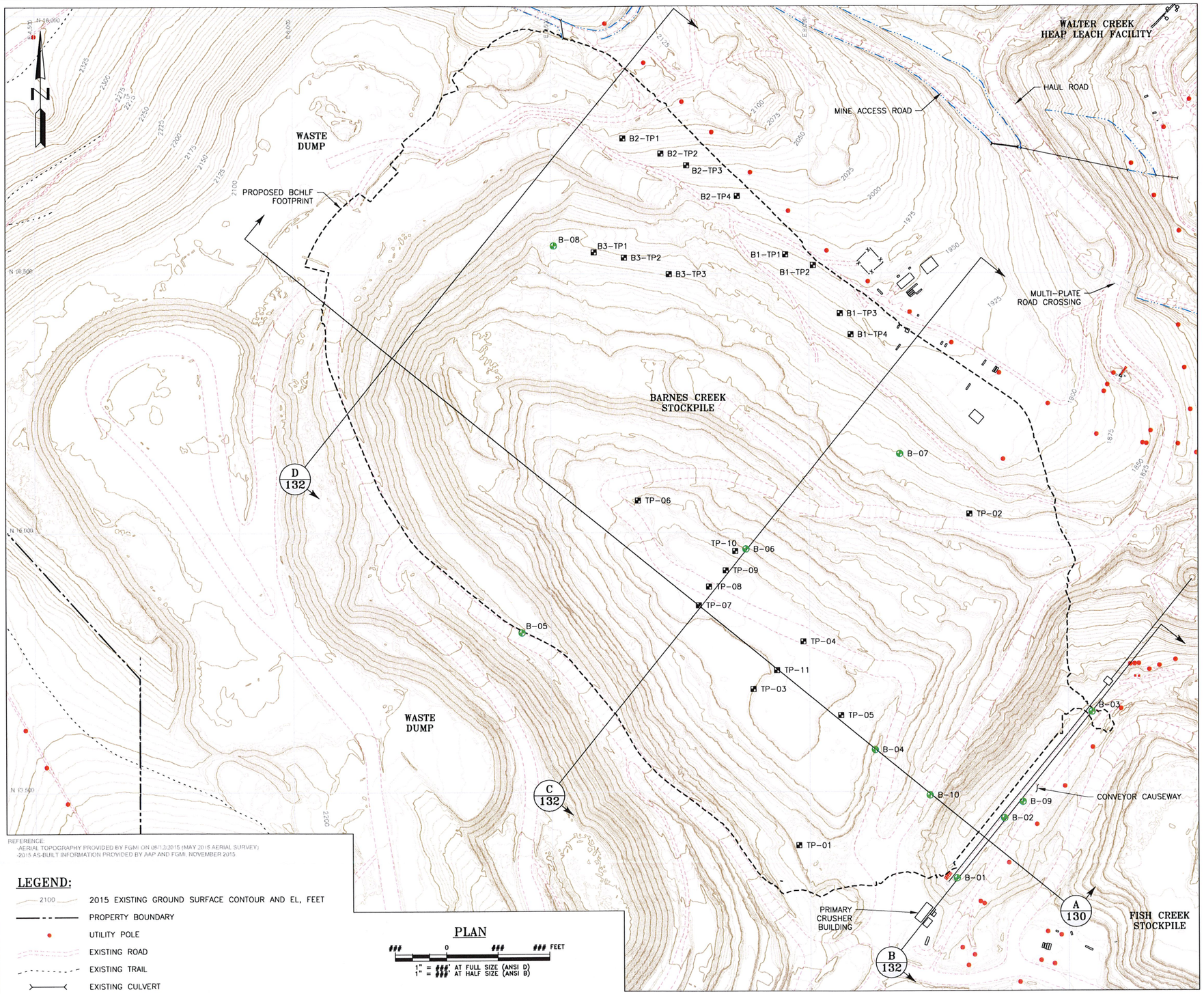


REFERENCE:
-AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
-2015 AS-BUILT INFORMATION PROVIDED BY AAP AND FGMI NOVEMBER 2015

## LEGEND:

- 2100 — 2015 EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- EXISTING STRUCTURES
- B-01 — BORING LOCATION AND DESIGNATION (KNIGHT PIESOLD, 2011 AND 2016)
- TP-01 — TEST PIT LOCATION AND DESIGNATION (KNIGHT PIESOLD, 2016)

PLAN

### 0 ### ### FEET

1" = ### AT FULL SIZE (ANSI D)
1" = ### AT HALF SIZE (ANSI B)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. SUMMARY LOGS OF KNIGHT PIESOLD'S BOREHOLES ARE INCLUDED IN THE BARNES CREEK HEAP LEACH FACILITY GEOTECHNICAL REPORT.
3. BORROW AREAS B1, B2, AND B3 WERE INVESTIGATED AS PART OF THE 2016 FIELD PROGRAM FOR POTENTIAL PREPARED SUBBASE MATERIAL.

**TABLE 1 – 2011 AND 2016 BORINGS**

| BORING No. | NORTHING (ft) | EASTING (ft) | COLLAR ELEVATION (fmsl) | TOTAL DEPTH (ft) |
|---|---|---|---|---|
| B-01 | 13,000.6 | 9,854.1 | 1549.4 | 64.0 SOIL<br>140.8 ROCK |
| B-02 | 13,345.9 | 10,129.8 | 1544.6 | 218.0 SOIL<br>293.8 ROCK |
| B-03 | 13,958.4 | 10,638.8 | 1548.8 | 77.0 SOIL<br>156.2 ROCK |
| B-04 | 13,741.6 | 9,380.8 | 1699.2 | 281.4 SOIL<br>330.6 ROCK |
| B-05 | 14,421.7 | 7,330.2 | 1899.4 | 96.0 SOIL<br>NO CORING |
| B-06 | 14,902.9 | 8,630.3 | 1703.6 | 158.3 SOIL<br>208.8 ROCK |
| B-07 | 15,453.9 | 9,528.6 | 1821.1 | 6.4 SOIL<br>NO CORING |
| B-08 | 16,661.1 | 7,517.8 | 1884.7 | 7.1 SOIL<br>NO CORING |
| B-09 | 13,439.8 | 10,237.0 | 1542.9 | 110.0 SOIL<br>NO CORING |
| B-10 | 13,479.0 | 9,699.5 | 1656.3 | 245.0 SOIL<br>255.0 ROCK |

**TABLE 2 – 2016 TEST PITS**

| TEST PIT No. | NORTHING (ft) | EASTING (ft) | GROUND SURFACE ELEVATION (fmsl) | TOTAL DEPTH (ft) |
|---|---|---|---|---|
| TP-01 | 13,190.5 | 8,934.4 | 1640.9 | 12.0 |
| TP-02 | 15,105.2 | 9,931.5 | 1783.7 | 5.5 |
| TP-03 | 14,092.8 | 8,671.6 | 1506.1 | 15.0 |
| TP-04 | 14,367.9 | 8,963.7 | 1505.5 | 15.0 |
| TP-05 | 13,940.8 | 9,182.5 | 1476.6 | 12.0 |
| TP-06 | 15,185.5 | 8,003.9 | 1579.8 | 13.0 |
| TP-07 | 14,577.8 | 8,354.8 | 1506.6 | 14.0 |
| TP-08 | 14,685.9 | 8,414.1 | 1510.9 | 17.0 |
| TP-09 | 14,779.9 | 8,511.2 | 1528.4 | 13.0 |
| TP-10 | 14,891.8 | 8,567.9 | 1558.2 | 14.0 |
| TP-11 | 14,202.1 | 8,809.3 | 1484.6 | 8.0 |
| B1-TP1 | 16,609.1 | 8,861.3 | 1945.9 | 6.0 |
| B1-TP2 | 16,546.7 | 9,023.8 | 1946.3 | 13.0 |
| B1-TP3 | 16,268.0 | 9,180.4 | 1917.0 | 8.0 |
| B1-TP4 | 16,145.6 | 9,243.1 | 1906.2 | 8.0 |
| B2-TP1 | 17,282.6 | 7,918.0 | 2065.4 | 9.0 |
| B2-TP2 | 17,195.2 | 8,137.7 | 2043.7 | 13.0 |
| B2-TP3 | 17,126.9 | 8,286.5 | 2022.9 | 9.0 |
| B2-TP4 | 16,947.7 | 8,580.9 | 1989.7 | 9.0 |
| B3-TP1 | 16,624.0 | 7,750.8 | 1891.8 | 13.0 |
| B3-TP2 | 16,592.4 | 7,925.3 | 1894.2 | 8.0 |
| B3-TP3 | 16,496.9 | 8,185.5 | 1876.5 | 8.0 |

STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
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| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR |
| | | | CB |
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ACTIVITY CODE N/A
XREF N/A

CLIENT
NOTES: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT
FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE
GEOTECHNICAL INVESTIGATION PLAN

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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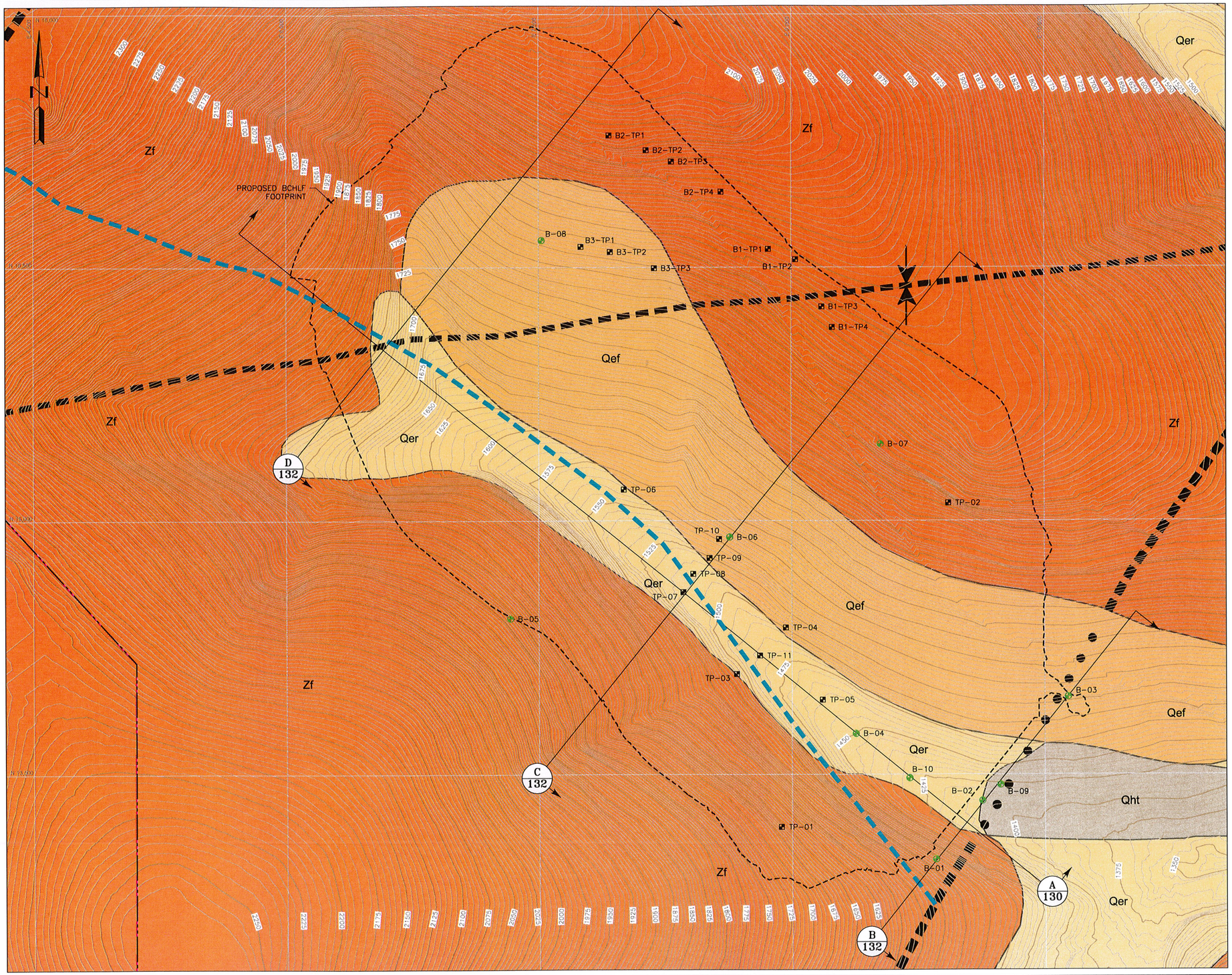


## LEGEND:

- 2100 — 1994 ORIGINAL GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- B-01 — BORING LOCATION AND DESIGNATION (KNIGHT PIESOLD, 2011 AND 2016)
- TP-01 — TEST PIT LOCATION AND DESIGNATION (KNIGHT PIESOLD, 2016)
- CONTACT (APPROXIMATELY LOCATED)
- HIGH ANGLE FAULT (DASHED WHERE INFERRED, DOTTED WHERE CONCEALED)
- SYNCLINE (SHOWING TRACE OF AXIAL PLANE AND DIRECTION OF PLUNGE, DASHED WHERE INFERRED OR CONCEALED, DOTTED WHERE QUESTIONABLE)
- INFERRED FAULT (2011 LOCAL MAPPING)

| | |
|---|---|
| Qht | REWORKED CREEK GRAVEL TAILINGS. PLACER-MINE DREDGE TAILINGS. TAN TO BROWN COLOR GRAVEL WITH COBBLES. |
| Qef | FAIRBANKS LOESS. MASSIVE, HOMOGENEOUS, UNCONSOLIDATED EOLIAN SILT. MAY INCLUDE RETRANSPORTED DEPOSIT ON SLOPES. WELL SORTED CONTAINING LESS THAN 10 PERCENT CLAY. LOCALLY CEMENTED BY IRON OXIDE AND LOCALLY CALCAREOUS. |
| Qer | RETRANSPORTED SILT. MASSIVE, HOMOGENEOUS, UNCONSOLIDATED EOLIAN SILT RETRANSPORTED FROM HILLS TO LOWER SLOPES AND VALLEY BOTTOMS. MAY CONTAIN ORGANIC MATERIAL AND OVERLIE TERTIARY GRAVEL IN VALLEY BOTTOMS. |
| Zf | QUARTZ MUSCOVITE SCHIST, QUARTZITE, AND CHLORITE QUARTZOSE SCHIST. LIGHT TO MEDIUM GRAY, FINE GRAINED, QUARTZITE, QUARTZ MUSCOVITE SCHIST, AND GARNET BIOTITE QUARTZ MUSCOVITE SCHIST. MODERATELY RESISTANT DUE TO ITS RELATIVELY HIGH QUARTZ CONTENT. |

## PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. SUMMARY LOGS OF KNIGHT PIESOLD'S BOREHOLES ARE INCLUDED IN THE BARNES CREEK HEAP LEACH FACILITY GEOTECHNICAL REPORT.
3. BORROW AREAS B1, B2, AND B3 WERE INVESTIGATED AS PART OF THE 2016 FIELD PROGRAM FOR POTENTIAL PREPARED SUBBASE MATERIAL.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

REFERENCE:
- 1994 TOPOGRAPHY PROVIDED BY FGMI.
- GEOLOGIC MAPPING SITE WIDE PROVIDED BY ALASKA DIVISION OF GEOLOGICAL AND GEOPHYSICAL SURVEYS (1996 PRELIMINARY GEOLOGIC MAP OF THE FAIRBANKS MINING DISTRICT, ALASKA)
DATA FILE NAMED: TREND USGS GEOLOGY.PDF
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| REV | DATE | DESCRIPTION | APP'D |
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| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE: N/A XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: GEOLOGIC MAP WITH PRE-DISTURBANCE TOPOGRAPHY

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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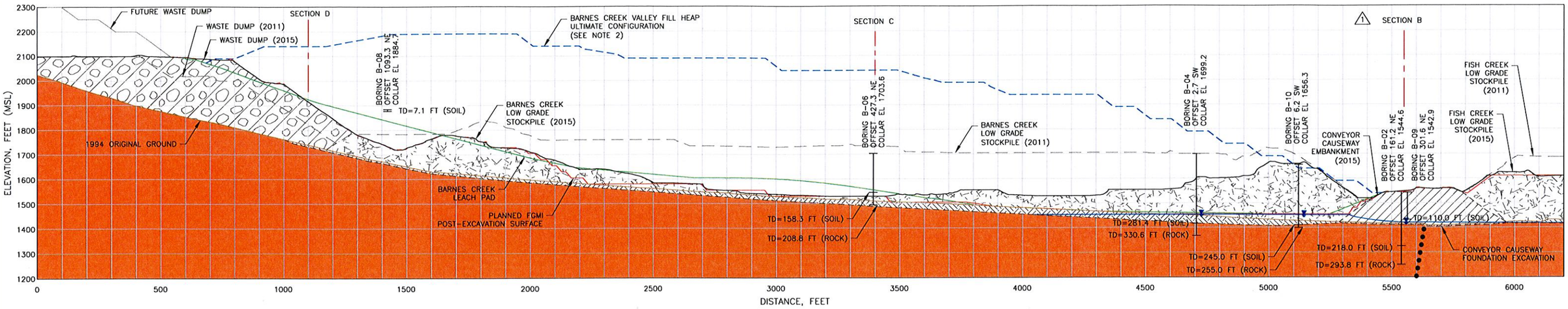


A 110 A 120 LONGITUDINAL GEOLOGIC SECTION ALONG BARNES CREEK VALLEY

200 0 200 400 FEET
1" = 200' AT FULL SIZE (ANSI D)
1" = 400' AT HALF SIZE (ANSI B)

## LEGEND:

CONTACT (APPROXIMATELY LOCATED, DASHED WHERE INFERRED)

HIGH ANGLE FAULT (DOTTED WHERE CONCEALED)

CONVEYOR CAUSEWAY FILL. ENGINEERED FILL COMPRISING MINE WASTE ROCK PLACED AND COMPACTED IN LIFTS.

LOW GRADE ORE. RUN-OF-MINE, LOOSE-DUMPED MINERALIZED MATERIAL OF GRANITIC ORIGIN.

MINE WASTE ROCK. RUN-OF-MINE, LOOSE-DUMPED NON-MINERALIZED MATERIAL OF GRANITIC AND/OR SCHISTOSE ORIGIN.

NATIVE SOIL. FAIRBANKS LOESS (Qef), RETRANSPORTED SILT (Qer), HIGHLY WEATHERED SCHIST AND TERTIARY CREEK GRAVEL INCLUDING COLLUVIAL MIXTURES.

Zf — QUARTZ MUSCOVITE SCHIST, QUARTZITE, AND CHLORITE QUARTZOSE SCHIST. LIGHT TO MEDIUM GRAY, FINE GRAINED, QUARTZITE, QUARTZ MUSCOVITE SCHIST, AND GARNET BIOTITE QUARTZ MUSCOVITE SCHIST. MODERATELY RESISTANT DUE TO ITS RELATIVELY HIGH QUARTZ CONTENT.

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE HEAP CONFIGURATION SHOWN REPRESENTS A MAXIMUM VERTICAL HEAP HEIGHT OF 500 FEET ABOVE THE PAD LINER.
3. THE BOREHOLE LOCATIONS HAVE BEEN PROJECTED TO THE SECTION AS DENOTED BY THE BOREHOLE OFFSETS.
4. SUMMARY LOGS OF KNIGHT PIESOLD'S BOREHOLES ARE INCLUDED IN THE BARNES CREEK HEAP LEACH FACILITY GEOTECHNICAL REPORT.

## REVISION:

1 CORRECTED LOCATION OF SECTION B CALLOUT IN SECTION A.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 24/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 03/24/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SITE GEOLOGIC SECTIONS, SHEET 1 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 130 | 1 |

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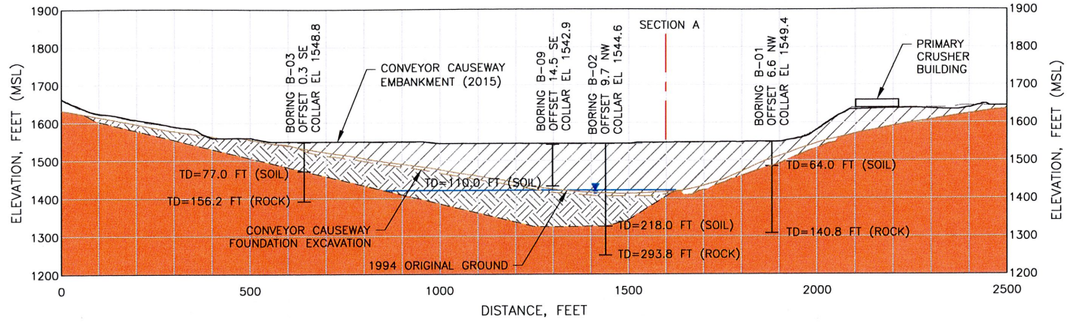
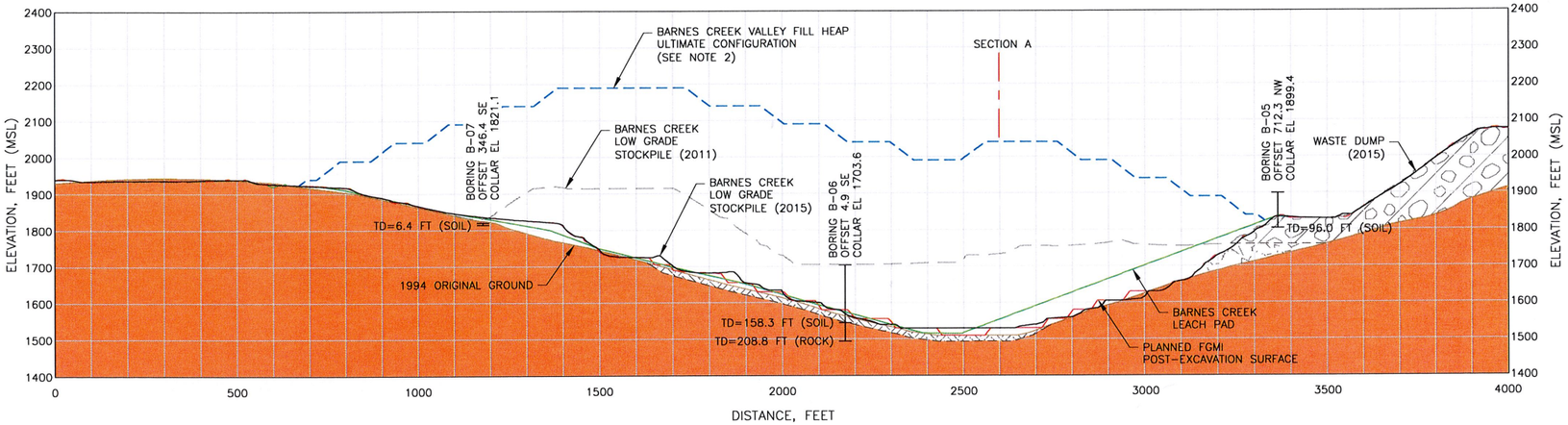
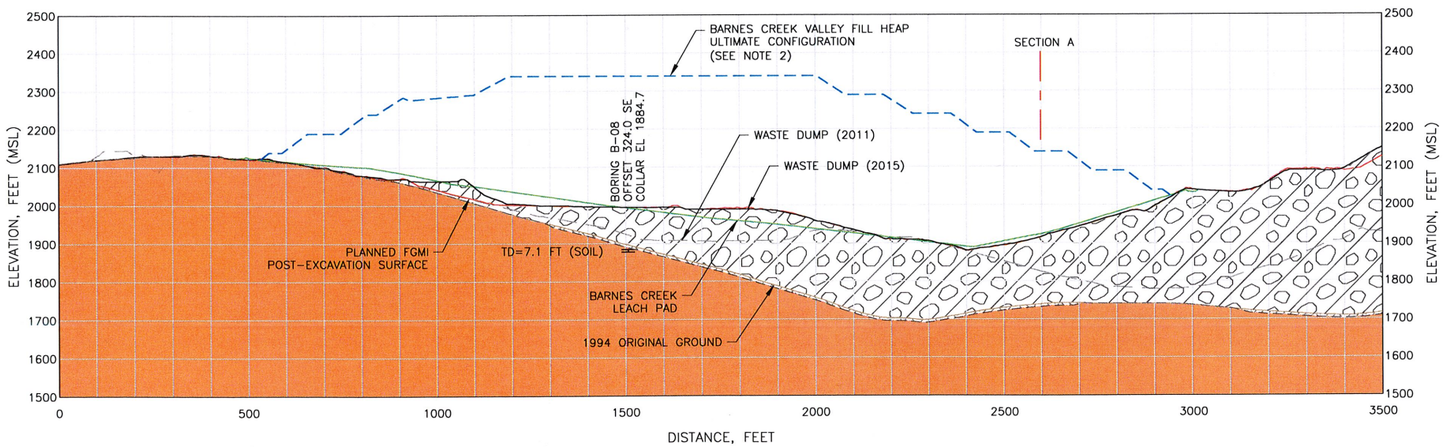



B 110 / B 120 TRANSVERSE GEOLOGIC SECTION ACROSS BARNES CREEK VALLEY
(AT EXISTING CONVEYOR CAUSEWAY)




C 110 / C 120 TRANSVERSE GEOLOGIC SECTION ACROSS BARNES CREEK VALLEY
(MIDDLE SECTION)




D 110 / D 120 TRANSVERSE GEOLOGIC SECTION ACROSS BARNES CREEK VALLEY
(UPPER SECTION)

## LEGEND:

- CONTACT (APPROXIMATELY LOCATED, DASHED WHERE INFERRED)
- CONVEYOR CAUSEWAY FILL. ENGINEERED FILL COMPRISING MINE WASTE ROCK PLACED AND COMPACTED IN LIFTS.
- LOW GRADE ORE. RUN-OF-MINE, LOOSE-DUMPED MINERALIZED MATERIAL OF GRANITIC ORIGIN.
- MINE WASTE ROCK. RUN-OF-MINE, LOOSE-DUMPED NON-MINERALIZED MATERIAL OF GRANITIC AND/OR SCHISTOSE ORIGIN.
- NATIVE SOIL. FAIRBANKS LOESS (Qef), RETRANSPORTED SILT (Qer), HIGHLY WEATHERED SCHIST AND TERTIARY CREEK GRAVEL INCLUDING COLLUVIAL MIXTURES.
- Zf — QUARTZ MUSCOVITE SCHIST, QUARTZITE, AND CHLORITE QUARTZOSE SCHIST. LIGHT TO MEDIUM GRAY, FINE GRAINED, QUARTZITE, QUARTZ MUSCOVITE SCHIST, AND GARNET BIOTITE QUARTZ MUSCOVITE SCHIST. MODERATELY RESISTANT DUE TO ITS RELATIVELY HIGH QUARTZ CONTENT.

200 0 200 400 FEET
1" = 200' AT FULL SIZE (ANSI D)
1" = 400' AT HALF SIZE (ANSI B)
(FOR ALL SECTIONS SHOWN)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE HEAP CONFIGURATION SHOWN REPRESENTS A MAXIMUM VERTICAL HEAP HEIGHT OF 500 FEET ABOVE THE PAD LINER.
3. THE BOREHOLE LOCATIONS HAVE BEEN PROJECTED TO THE SECTION AS DENOTED BY THE BOREHOLE OFFSETS.
4. SUMMARY LOGS OF KNIGHT PIESOLD'S BOREHOLES ARE INCLUDED IN THE BARNES CREEK HEAP LEACH FACILITY GEOTECHNICAL REPORT.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | CADD JTR CB |

DISCLAIMER
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SITE GEOLOGIC SECTIONS, SHEET 2 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 132 | 0 |

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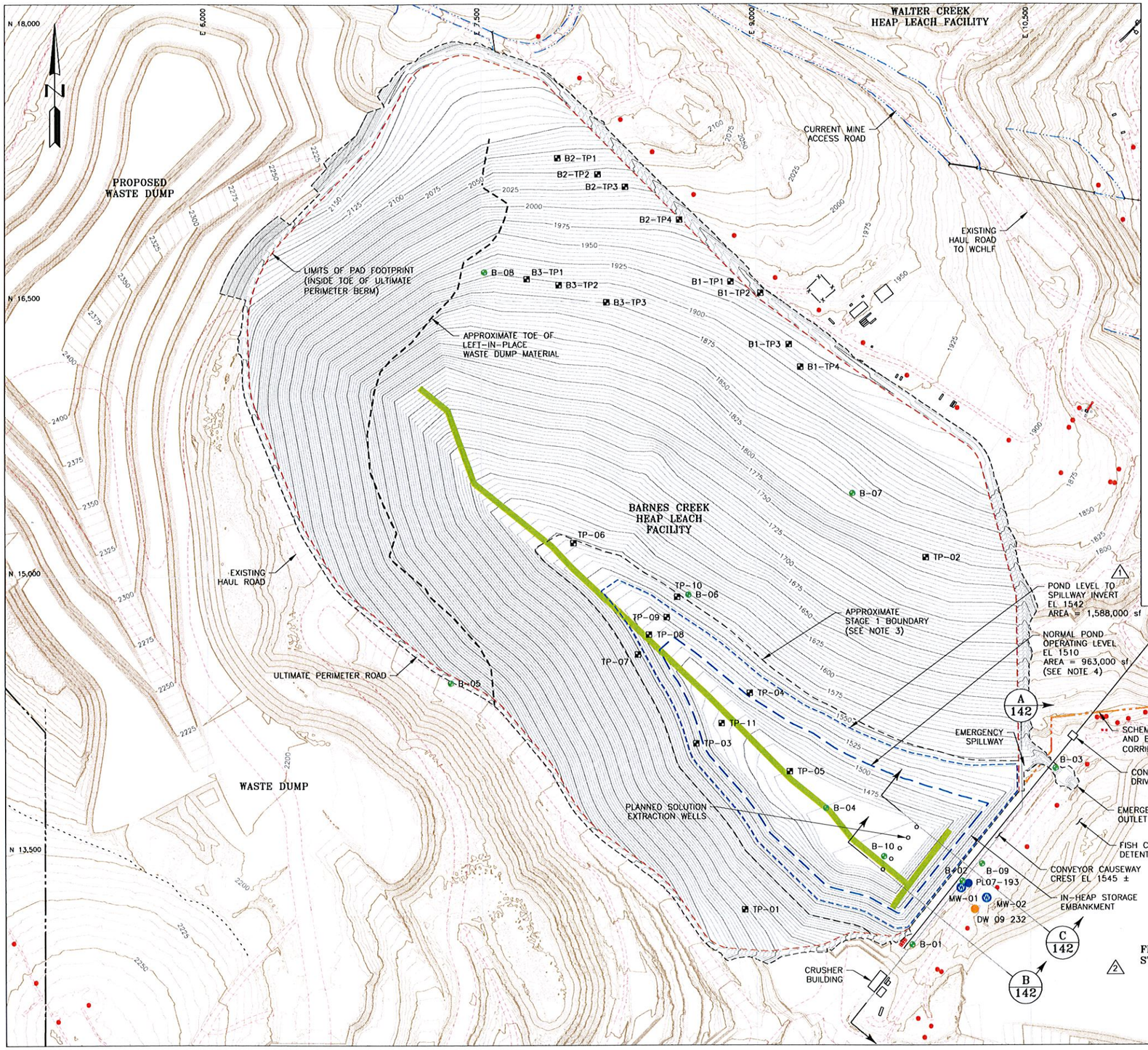


## LEGEND:

- 2100 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 2100 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET (SEE NOTE 2)
- B-01 — BORING LOCATION AND DESIGNATION (KNIGHT PIESOLD, 2011 AND 2016)
- TP-01 — TEST PIT LOCATION AND DESIGNATION (KNIGHT PIESOLD, 2016)
- MW-01 — UNDERDRAIN MONITORING WELL (SEE DRAWING 200, SEE NOTE 8) 1 2
- PL07-173 — EXISTING MONITORING WELL (FGMI) 2
- DW 09 228 — EXISTING DEWATERING WELL (FGMI) 2
- UNDERDRAIN RANDOM FILL (SEE DRAWING 200)
- PROPERTY BOUNDARY
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- EXISTING STRUCTURES
- TAILING (CONFIGURATION AS OF AUGUST 2014)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, AND BOTTOM OF EROSION PROTECTION WITHIN ALL CHANNELS.
3. ABOVE THE STAGE 1 BOUNDARY, THE LEACH PAD WILL BE CONSTRUCTED WITH A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND SHALL BE CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM.
4. 1 THE NORMAL OPERATING LEVEL (EL 1510) IS AN ESTIMATE BASED ON ENGINEERING JUDGEMENT. AREAS AT THE POND STORAGE LEVELS ARE PROVIDED PER ADNR GUIDELINES, HOWEVER, THE POND SURFACE WILL GENERALLY NOT BE EXPOSED AS IT WILL BE CONFINED WITHIN THE HEAP.
5. BORROW AREAS B1, B2, AND B3 WERE INVESTIGATED AS PART OF THE 2016 FIELD PROGRAM FOR POTENTIAL PREPARED SUBBASE MATERIAL.
6. REFER TO DRAWINGS 110 THRU 132 FOR MORE INFORMATION ON THE SITE INVESTIGATION PROGRAM.
7. SUMMARY LOGS OF KNIGHT PIESOLD'S BOREHOLES ARE INCLUDED IN THE BARNES CREEK HEAP LEACH FACILITY GEOTECHNICAL REPORT.
8. AS DISCUSSED WITH ADEC, A THIRD UNDERDRAIN MONITORING WELL WILL BE INSTALLED IN THE VICINITY OF THE TWO IDENTIFIED WELLS. THE EXACT LOCATION OF THE THIRD WELL WILL BE DETERMINED PRIOR TO CONSTRUCTION. REFER TO DRAWING 200 FOR MORE INFORMATION. 2
9. SEVEN (7) DRAINAGE WELLS WILL BE CONSTRUCTED WITHIN THE BASE OF THE TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH TO PROMOTE DRAINAGE FROM THE TRENCH AND INTO THE INTERBEDDED HIGHER CONDUCTIVITY COLLUVIAL MATERIALS BENEATH THE PRE-CONVEYOR CAUSEWAY SURFACE. THE DRAINAGE WELLS WILL BE DRILLED-IN APPROXIMATELY 30' BENEATH THE INVERT OF UNDERDRAIN RANDOM FILL TRENCH AS SHOWN ON DRAWINGS 202 AND 206. 2

## REVISION:

1 MODIFIED NOTE 4 AND WATER LEVEL CALLOUT. ADDED UNDERDRAIN MONITORING/PUMPING WELL LOCATIONS IN PLAN AND CALLOUT TO LEGEND.

2 MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY. ADDED NOTES 8 AND 9. ADDED EXISTING FGMI WELLS IN THE VICINITY OF THE FISH CREEK STOCKPILE AREA. ADJUSTED END OF LONGITUDINAL UNDERDRAIN RANDOM FILL TRENCH.

**PLAN**

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 29/17

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| B | 10/10/16 | REISSUED FOR CLIENT REVIEW | JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| 2 | 03/24/17 | REISSUED FOR CONSTRUCTION | JTR CB |

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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: GENERAL ARRANGEMENT PLAN (TOPOGRAPHIC MAP OF DAM SITE)

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 140 | 2 |

LAST SAVED BY: JREVA
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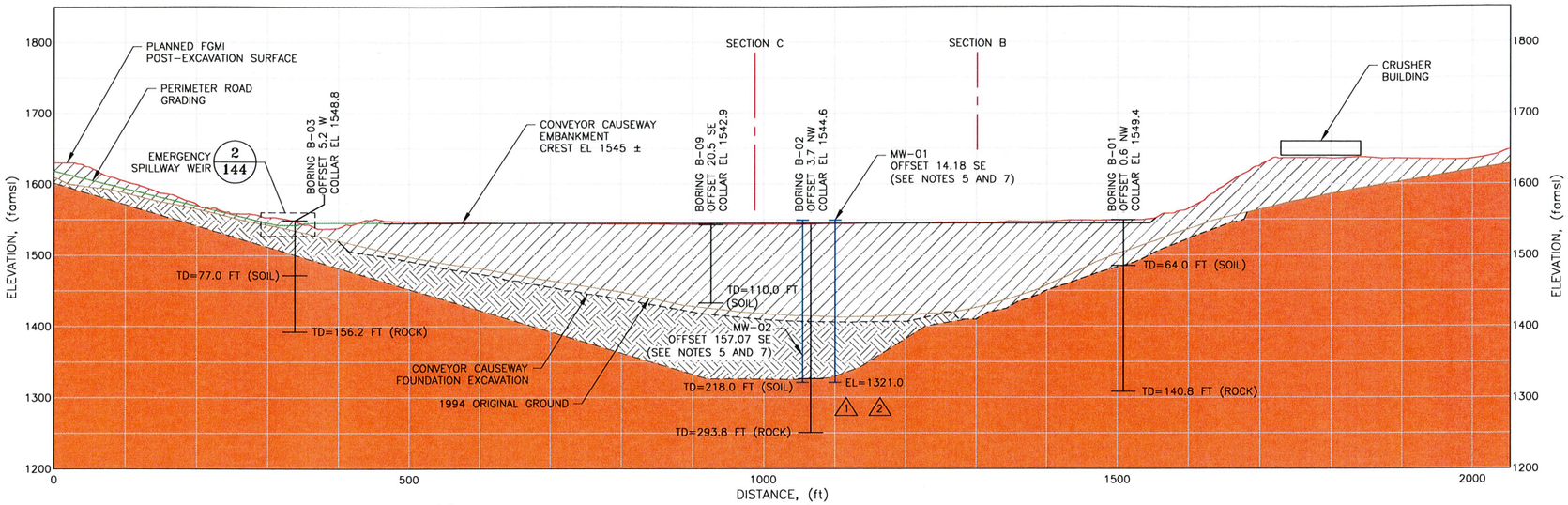
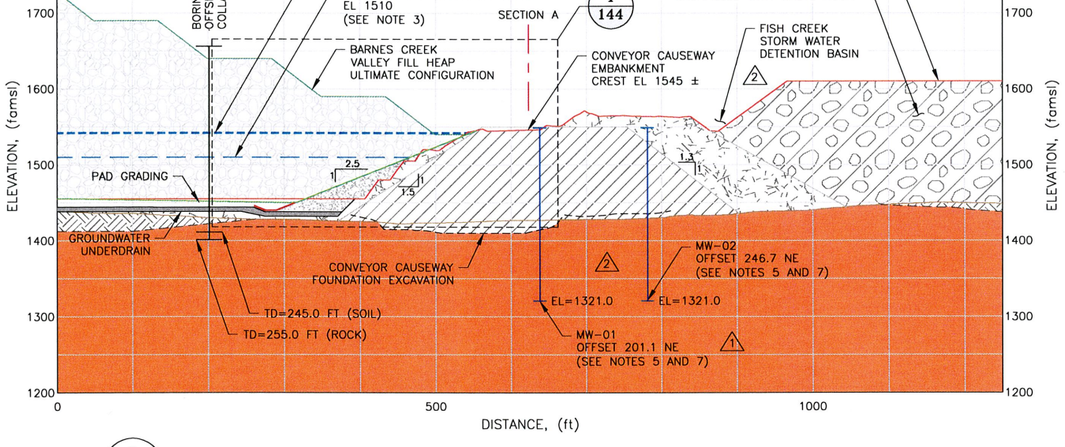



A/140 PROFILE VIEW OF IN-HEAP STORAGE POND EMBANKMENT (DAM) ALONG DAM AXIS

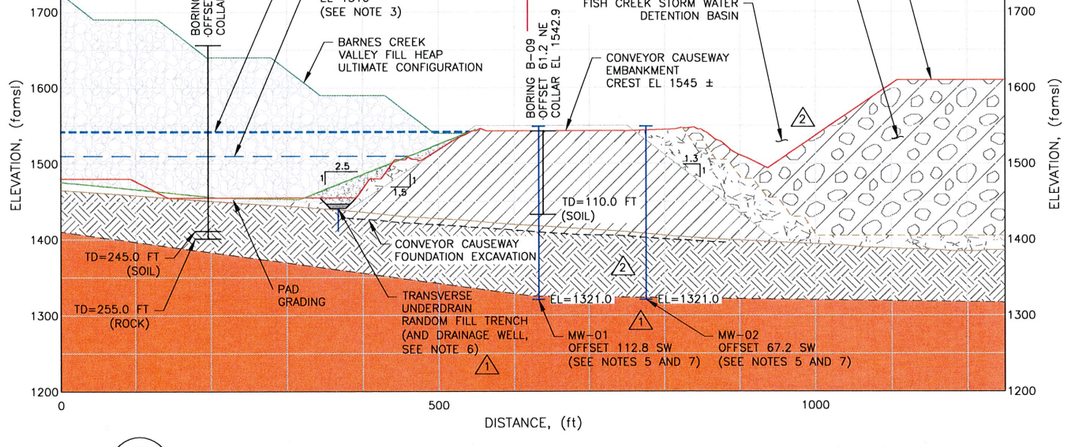
## LEGEND:

- CONTACT (APPROXIMATELY LOCATED, DASHED WHERE INFERRED)
- CONVEYOR CAUSEWAY AND CRUSHER PLATFORM FILL. ENGINEERED RANDOM FILL COMPRISING MINE WASTE ROCK PLACED AND COMPACTED IN LIFTS.
- LOW GRADE ORE. RUN-OF-MINE, LOOSE-DUMPED MINERALIZED MATERIAL OF GRANITIC ORIGIN.
- SITE GRADING FILL
- MINE WASTE ROCK. RUN-OF-MINE, LOOSE-DUMPED NON-MINERALIZED MATERIAL OF GRANITIC AND/OR SCHISTOSE ORIGIN.
- NATIVE SOIL. FAIRBANKS LOESS (Qef), RETRANSPORTED SILT (Qer), HIGHLY WEATHERED SCHIST AND TERTIARY CREEK GRAVEL INCLUDING COLLUVIAL MIXTURES.
- Zf — QUARTZ MUSCOVITE SCHIST, QUARTZITE, AND CHLORITE QUARTZOSE SCHIST. LIGHT TO MEDIUM GRAY, FINE GRAINED, QUARTZITE, QUARTZ MUSCOVITE SCHIST, AND GARNET BIOTITE QUARTZ MUSCOVITE SCHIST. MODERATELY RESISTANT DUE TO ITS RELATIVELY HIGH QUARTZ CONTENT.
- MW-01 — UNDERDRAIN MONITORING WELL DESIGNATION (SEE NOTES 5 AND 7)

100 0 100 200 FEET
1" = 100' AT FULL SIZE (ANSI D)
1" = 200' AT HALF SIZE (ANSI B)




B/140 CROSS-SECTION VIEW OF IN-HEAP STORAGE POND EMBANKMENT (DAM) ADJACENT TO LCRS




C/140 CROSS SECTION VIEW OF IN-HEAP STORAGE POND EMBANKMENT (DAM) WITH UNDERDRAIN RANDOM FILL TOE DRAIN

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE BOREHOLE LOCATIONS HAVE BEEN PROJECTED TO THE SECTION AS DENOTED BY THE BOREHOLE OFFSETS.
3. THE NORMAL OPERATING LEVEL (EL 1510) IS AN ESTIMATE BASED ON ENGINEERING JUDGEMENT. AREAS AT THE POND STORAGE LEVELS ARE PROVIDED PER ADNR GUIDELINES, HOWEVER, THE POND SURFACE WILL NOT BE EXPOSED AS IT WILL GENERALLY BE CONFINED WITHIN THE HEAP.
4. SUMMARY LOGS OF KNIGHT PIESOLD'S BOREHOLES ARE INCLUDED IN THE BARNES CREEK HEAP LEACH FACILITY GEOTECHNICAL REPORT.
5. THE UNDERDRAIN MONITORING WELLS WILL BE INSTALLED THROUGH THE APPROXIMATE LOW-POINT OF THE VALLEY THAT EXISTED PRIOR TO CONSTRUCTION OF THE CONVEYOR CAUSEWAY. THE WELLS SHALL BE DRILLED AND INSTALLED TO BETWEEN 5 AND 10' INTO THE UNDERLYING BEDROCK SURFACE. IF THE BEDROCK SURFACE IS ENCOUNTERED PRIOR TO THE ESTIMATED DEPTHS SHOWN, THE BOREHOLES MAY BE STOPPED. REFER TO DRAWING 200 FOR MORE INFORMATION.
6. SEVEN (7) DRAINAGE WELLS WILL BE CONSTRUCTED WITHIN THE BASE OF THE TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH TO PROMOTE DRAINAGE FROM THE TRENCH AND INTO THE INTERBEDDED HIGHER CONDUCTIVITY COLLUVIAL MATERIALS BENEATH THE PRE-CONVEYOR CAUSEWAY SURFACE. THE DRAINAGE WELLS WILL BE DRILLED-IN APPROXIMATELY 30' BENEATH THE INVERT OF UNDERDRAIN RANDOM FILL TRENCH AS SHOWN ON DRAWING 206. IF CONTACT WITH THE BEDROCK SURFACE IS ENCOUNTERED PRIOR TO COMPLETING THE 30' WELL DEPTH, THE BOREHOLE SHALL BE STOPPED. THE 30' WELL DEPTH SHOULD BE CONSIDERED AN APPROXIMATE MAXIMUM DEPTH.
7. AS DISCUSSED WITH ADEC, A THIRD UNDERDRAIN MONITORING WELL WILL BE INSTALLED IN THE VICINITY OF THE TWO IDENTIFIED WELLS. THE EXACT LOCATION OF THE THIRD WELL WILL BE DETERMINED PRIOR TO CONSTRUCTION. REFER TO DRAWING 200 FOR MORE INFORMATION.

## REVISION:

1. MODIFIED NOTE 3 AND WATER LEVEL CALLOUT. ADDED NOTES 5 AND 6. ADDED UNDERDRAIN MONITORING/PUMPING WELLS IN SECTIONS. ADDED UNDERDRAIN RANDOM FILL DRAINAGE WELL IN SECTION C.
2. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY. MODIFIED NOTES 5 AND 6 REGARDING WELL DEPTHS AND ADDED TWO ADDITIONAL DRAINAGE WELLS FOR THE TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH. ADDED NOTE 7 REGARDING THIRD UNDERDRAIN MONITORING WELL. CORRECTED LOCATIONS OF PLANNED UNDERDRAIN MONITORING WELLS IN SECTION A. EXTENDED DEPTHS OF UNDERDRAIN MONITORING WELLS.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 24/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| B | 10/10/16 | REISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
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| 2 | 03/24/17 | REISSUED FOR CONSTRUCTION | JTR / CB |

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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND EMBANKMENT PROFILE AND CROSS-SECTIONS

| DESIGNED BY | DRAWN BY | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| JTR | CB | DV101 | 00089.99 | 142 | 2 |

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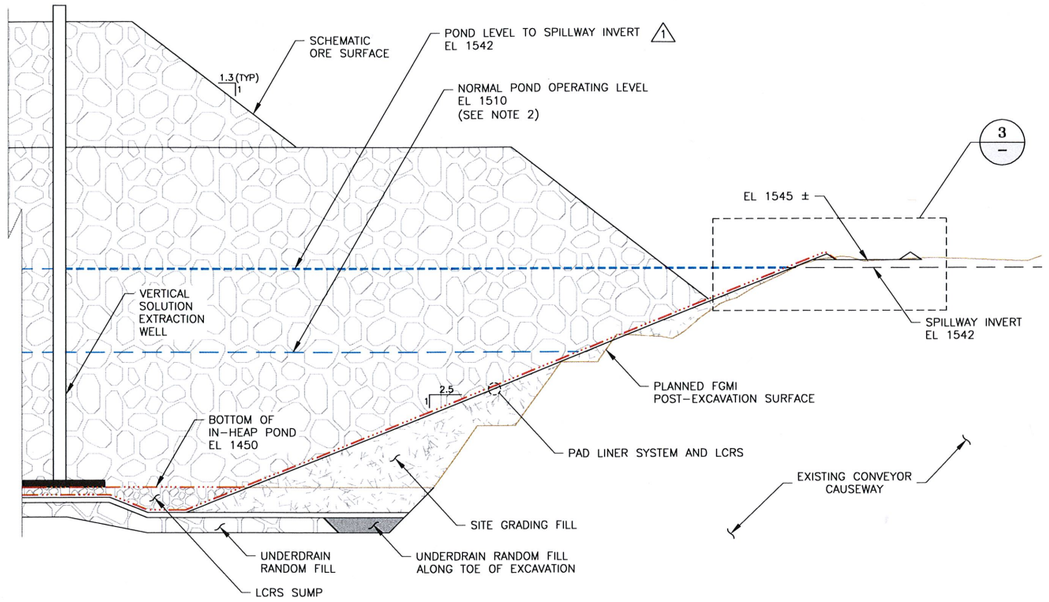
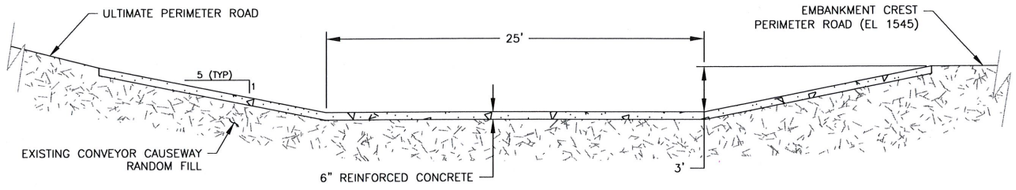


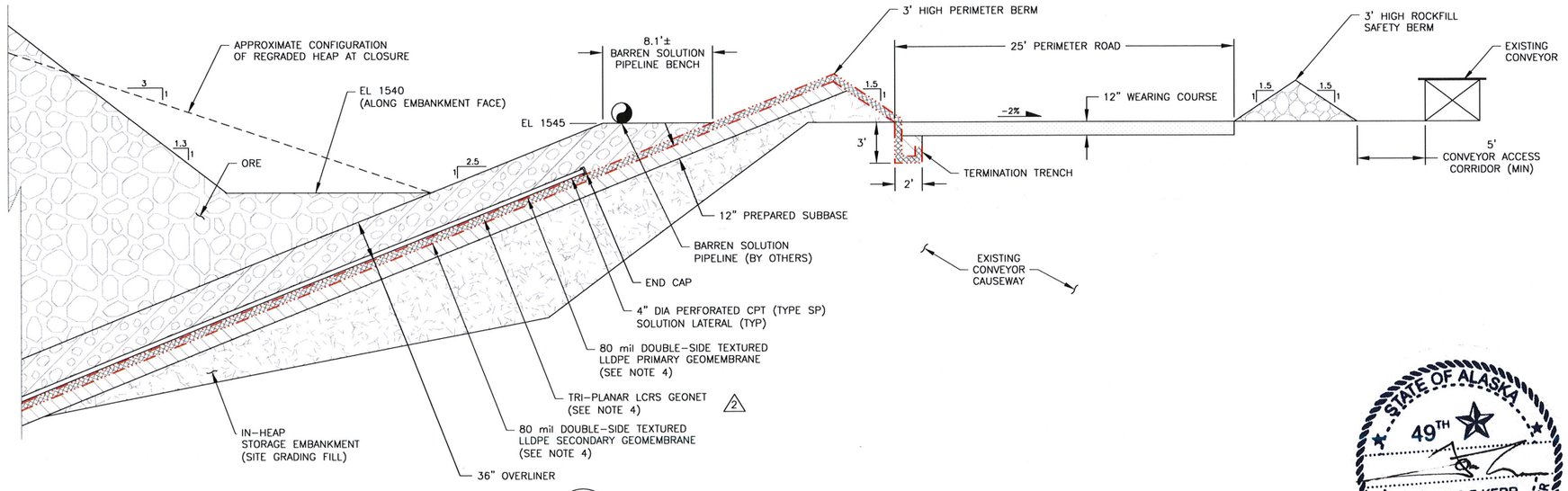

1/142 IN-HEAP STORAGE POND EMBANKMENT (DAM)
ADJACENT TO LCRS SUMP
(SEE NOTE 3)

30 0 30 60 FEET
1" = 30' AT FULL SIZE (ANSI D)
1" = 60' AT HALF SIZE (ANSI B)




2/142 IN-HEAP STORAGE POND EMBANKMENT (DAM)
EMERGENCY SPILLWAY WEIR SECTION
(SEE NOTE 3)

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)




3/– IN-HEAP STORAGE POND EMBANKMENT (DAM)
TYPICAL LINER SYSTEM AND CREST ARRANGEMENT
(SEE NOTE 3)

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

## REVISION:

1 MODIFIED NOTE 2 AND WATER LEVEL CALLOUT.

2 ADDED NOTE 4 SPECIFYING GEOMEMBRANE AND GEONET.

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
1. THE NORMAL OPERATING LEVEL (EL 1510) IS AN ESTIMATE BASED ON ENGINEERING JUDGEMENT. AREAS AT THE POND STORAGE LEVELS ARE PROVIDED PER ADNR GUIDELINES, HOWEVER, THE POND SURFACE WILL NOT BE EXPOSED AS IT WILL BE CONFINED WITHIN THE HEAP.
3. THE DETAILS PROVIDED ON THIS SHEET ARE INTENDED FOR GENERAL INFORMATION TO MEET REGULATORY REQUIREMENTS. REFER TO THE COVER SHEET AND SPECIFIED DRAWINGS FOR MORE DETAILED INFORMATION ON THESE CONCEPTS.
2. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| B | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR CB |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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND EMBANKMENT GENERAL DETAILS

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 144 | 2 |


LAST SAVED BY: CBEANLAND
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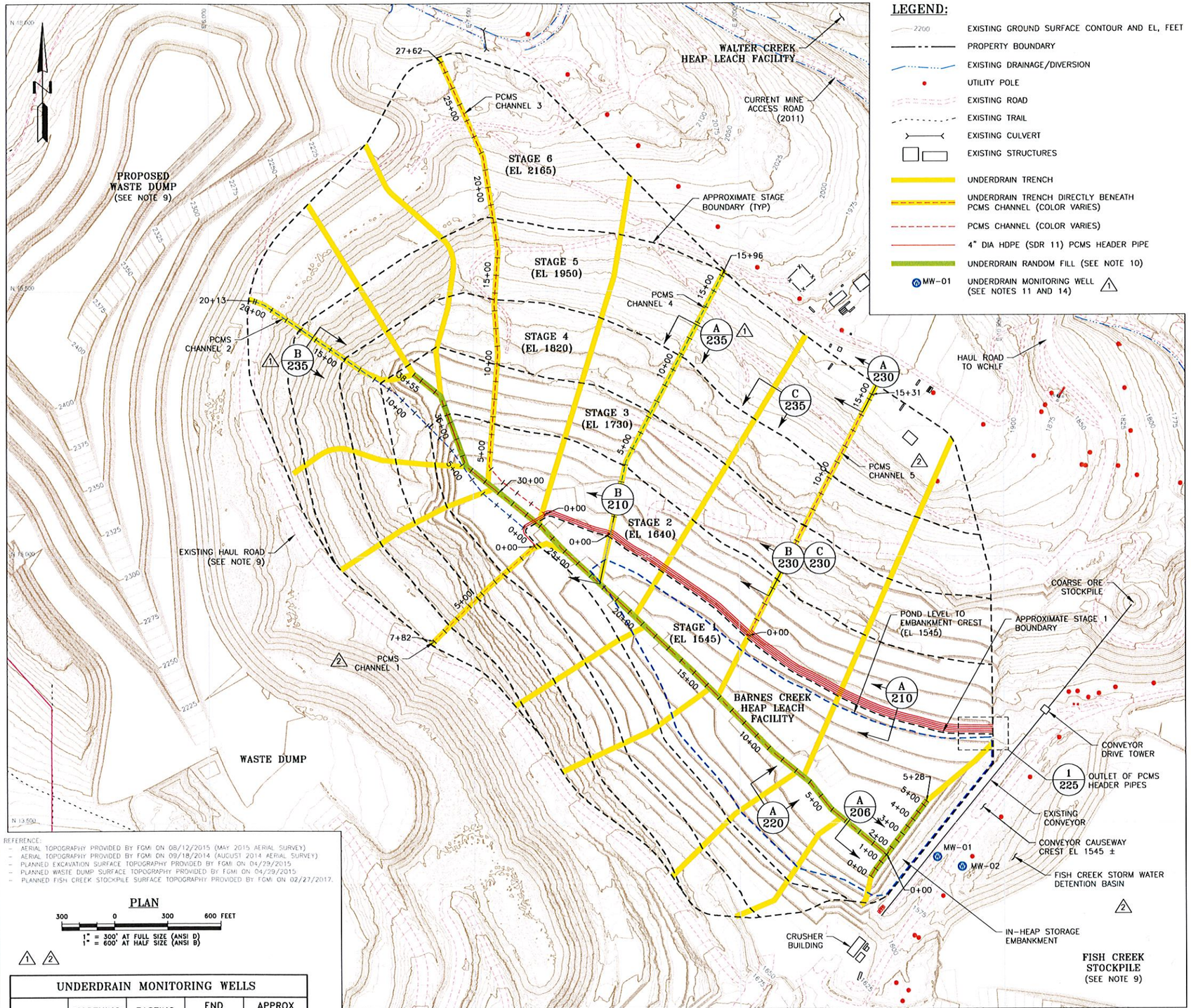
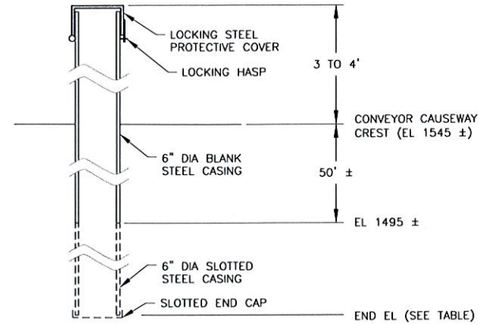



## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET (2200)
- PROPERTY BOUNDARY
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- EXISTING STRUCTURES
- UNDERDRAIN TRENCH
- UNDERDRAIN TRENCH DIRECTLY BENEATH PCMS CHANNEL (COLOR VARIES)
- PCMS CHANNEL (COLOR VARIES)
- 4" DIA HDPE (SDR 11) PCMS HEADER PIPE
- UNDERDRAIN RANDOM FILL (SEE NOTE 10)
- MW-01 UNDERDRAIN MONITORING WELL (SEE NOTES 11 AND 14) ⚠1




⚠1 1/– UNDERDRAIN MONITORING WELL

NTS

(SEE NOTE 11)

## NOTES: ⚠1 ⚠2

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B SHEET).
2. PCMS – THE PROCESS COMPONENT MONITORING SYSTEM IS LOCATED BENEATH THOSE PORTIONS OF THE LINER WHICH HAVE GREATER POTENTIAL FOR LEAKAGE – MAINLY BENEATH THE MAIN SOLUTION COLLECTION HEADERS. THE PCMS CHANNELS ARE LIMITED TO THE AREA OUTSIDE THE IN-HEAP STORAGE POND.
3. ALL UNDERDRAIN TRENCHES TERMINATE AT THE EDGE OF THE UNDERDRAIN RANDOM FILL.
4. UNDERDRAIN TRENCH LOCATIONS SHALL BE ADJUSTED IN THE FIELD DURING THE LEACH PAD GRADING OPERATIONS TO INTERCEPT GROUNDWATER SEEPS AND SPRINGS. IN SOME CASES THIS MAY INVOLVE ADDITIONAL UNDERDRAINS TO REACH SEEPS AND SPRINGS IDENTIFIED DURING FUTURE STAGES OF CONSTRUCTION. ADDITIONAL UNDERDRAINS SHALL BE CONSTRUCTED AS NEEDED BASED ON SECTION D ON DRAWING 235.
5. ALL EXISTING CULVERTS WITHIN THE LEACH PAD BASIN AREA SHALL BE REMOVED PRIOR TO THE START OF LEACH PAD REGRADING.
6. EACH UNDERDRAIN MONITORING WELL WILL HAVE 2 HEAT TRACES AND RTD'S INSTALLED (DESIGN BY OTHERS). THE FIRST LOOP IS A PRIMARY LOOP AND THE SECOND LOOP A BACKUP.
7. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
8. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE STAGE PERIMETER ROAD.
9. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
10. THE UNDERDRAIN RANDOM FILL IS PRIMARILY ENVISIONED AS A LARGE TRENCH DRAIN EXCAVATED DOWN THE CENTER OF THE BARNES CREEK VALLEY. THE UNDERDRAIN RANDOM FILL WILL DROP BELOW THE IN-HEAP STORAGE POND LCRS SUMP AND TERMINATE AT THE UPSTREAM TOE OF THE EXISTING CONVEYOR CAUSEWAY EMBANKMENT. A SECOND UNDERDRAIN RANDOM FILL TRENCH, 20' WIDE BY 6' DEEP, WILL BE EXCAVATED LATERALLY ACROSS THE VALLEY ALONG THE UPSTREAM TOE OF THE CONVEYOR CAUSEWAY.
11. THE UNDERDRAIN MONITORING WELLS WILL BE INSTALLED THROUGH THE APPROXIMATE LOW-POINT OF THE VALLEY THAT EXISTED PRIOR TO CONSTRUCTION OF THE CONVEYOR CAUSEWAY. THE WELLS SHALL BE DRILLED AND INSTALLED TO BETWEEN 5 AND 10' INTO THE UNDERLYING BEDROCK SURFACE, AS SHOWN ON DRAWING 142. IF THE BEDROCK SURFACE IS ENCOUNTERED PRIOR TO THE ESTIMATED DEPTHS SHOWN, THE BOREHOLES MAY BE STOPPED. WELL CASING, SLOTTING, AND PUMPS ARE BY FGMI.
12. SEE DRAWING 202 FOR THE UNDERDRAIN RANDOM FILL PROFILES AND HORIZONTAL CONTROL DATA.
13. SEE DRAWINGS 204 AND 205 FOR THE PCMS CHANNEL PROFILES AND HORIZONTAL CONTROL DATA.
14. AS DISCUSSED WITH ADEC, A THIRD UNDERDRAIN MONITORING WELL WILL BE INSTALLED IN THE VICINITY OF THE TWO IDENTIFIED WELLS. THE EXACT LOCATION OF THE THIRD WELL WILL BE DETERMINED PRIOR TO CONSTRUCTION, HOWEVER, IT IS EXPECTED IT WILL BE COMPLETED TO AN APPROXIMATE DEPTH BETWEEN 225 TO 275' BELOW GROUND SURFACE.
15. SEE DRAWINGS 140 AND 560 FOR THE LOCATIONS OF FGMI'S EXISTING MONITORING AND DEWATERING WELLS IN THE VICINITY OF THE FISH CREEK STOCKPILE AREA.
16. SEVEN (7) DRAINAGE WELLS WILL BE CONSTRUCTED WITHIN THE BASE OF THE TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH TO PROMOTE DRAINAGE FROM THE TRENCH AND INTO THE INTERBEDDED HIGHER CONDUCTIVITY COLLUVIAL MATERIALS BENEATH THE PRE-CONVEYOR CAUSEWAY SURFACE. THE DRAINAGE WELLS WILL BE DRILLED-IN APPROXIMATELY 30' BENEATH THE INVERT OF UNDERDRAIN RANDOM FILL TRENCH AS SHOWN ON DRAWING 206. IF CONTACT WITH THE BEDROCK SURFACE IS ENCOUNTERED PRIOR TO COMPLETING THE 30' WELL DEPTH, THE BOREHOLE SHALL BE STOPPED. THE 30' WELL DEPTH SHOULD BE CONSIDERED AN APPROXIMATE MAXIMUM DEPTH.

## REVISION:

⚠1 MOVED SECTION CALLOUTS. MODIFIED NOTES 10 AND 11. ADDED UNDERDRAIN WELL LOCATIONS IN PLAN, LEGEND, DETAIL, AND SETTING OUT TABLE.

⚠2 ADDED TWO PCMS CHANNELS AND MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY. MODIFIED NOTE 11 AND SETTING OUT DATA REGARDING WELL DEPTHS. ADDED NOTES 14 THROUGH 16.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

⚠1 ⚠2

### UNDERDRAIN MONITORING WELLS

| WELL ID | NORTHING (ft) | EASTING (ft) | END ELEV (fmsl) | APPROX DEPTH (ft) |
|---|---|---|---|---|
| MW-01 | 13,308.5 | 10,122.5 | 1382.0 | 224.0 |
| MW-02 | 13,254.0 | 10,262.3 | 1373.5 | 224.0 |

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| 2 | 03/24/17 | REISSUED FOR CONSTRUCTION | JTR CB |

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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS PLAN

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 200 | 2 |

STATE OF ALASKA 49TH
THOMAS F. KERR No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 24/17

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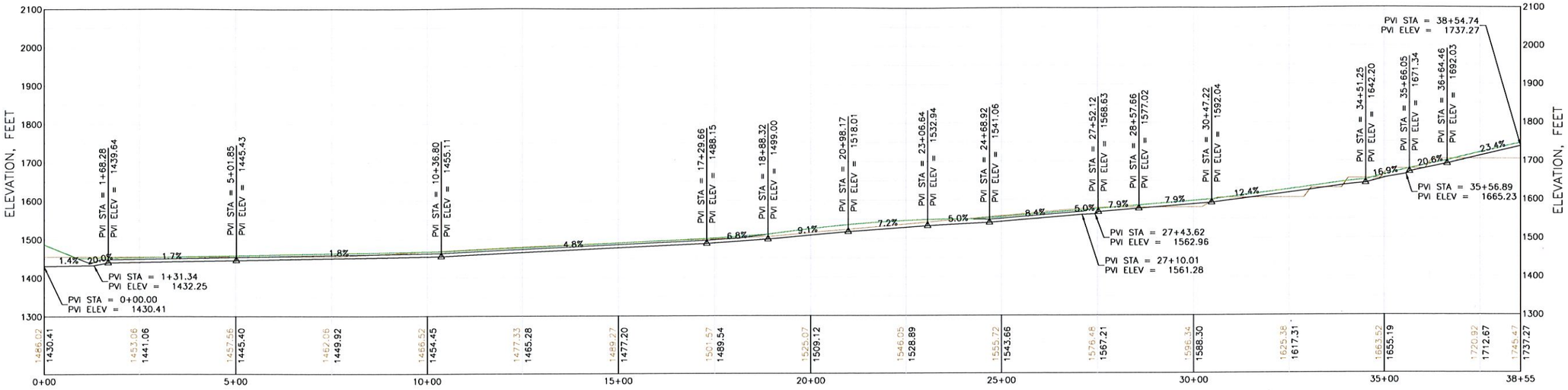
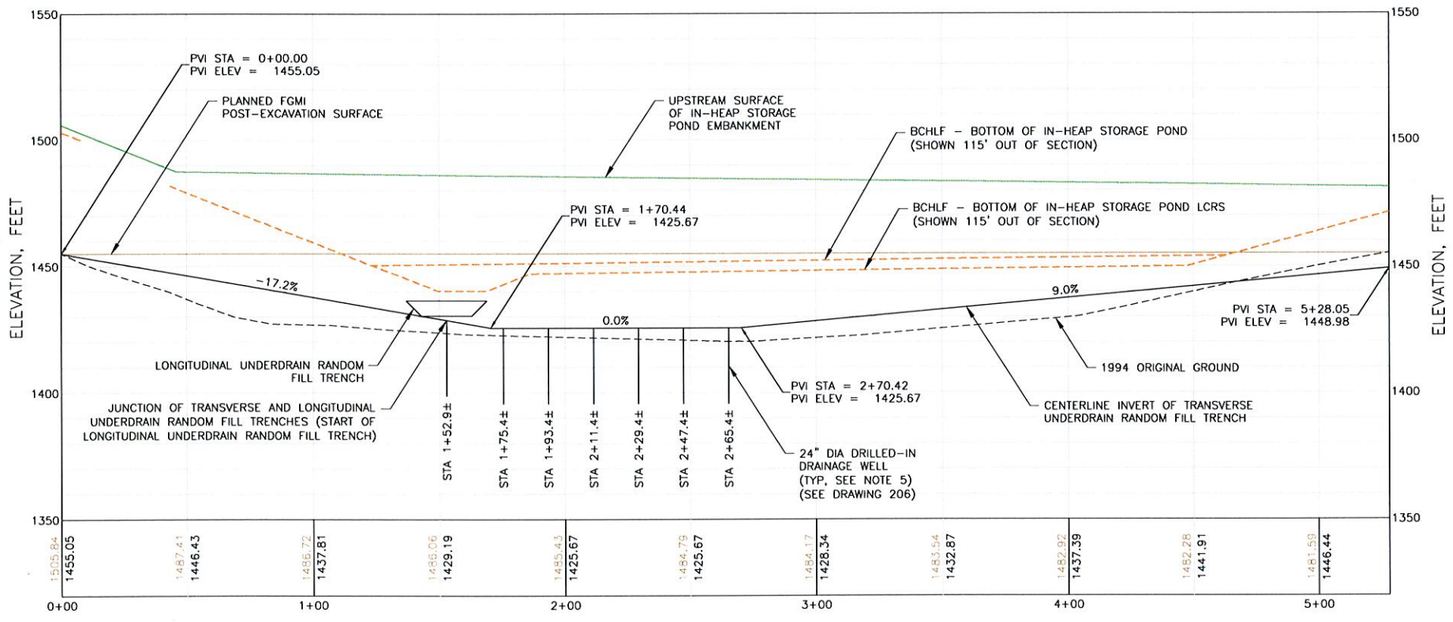



UNDERDRAIN RANDOM FILL LONGITUDINAL TRENCH
PROFILE

150 0 150 300 FEET
1"=150' AT FULL SIZE (ANSI D)
1"=300' AT HALF SIZE (ANSI B)




UNDERDRAIN RANDOM FILL TRAVERSE TRENCH
PROFILE ⚠1

30 0 30 60 Feet
1"=30' AT FULL SIZE (ANSI D)
1"=60' AT HALF SIZE (ANSI B)

| TABLE 1 | | |
|---|---|---|
| UNDERDRAIN RANDOM FILL LONGITUDINAL TRENCH HORIZONTAL CONTROL DATA | | |
| PI STATION | NORTHING | EASTING |
| 0+00.0 | 13,321.73 | 9,834.50 |
| 4+05.7 | 13,576.89 | 9,519.07 |
| 5+96.8 | 13,725.99 | 9,399.54 |
| 8+68.4 | 13,896.69 | 9,188.36 |
| 14+96.1 | 14,342.16 | 8,746.00 |
| 17+56.7 | 14,521.30 | 8,556.81 |
| 20+27.7 | 14,708.40 | 8,360.77 |
| 21+78.9 | 14,811.71 | 8,250.39 |
| 25+20.9 | 15,045.71 | 8,000.87 |
| 26+87.8 | 15,168.07 | 7,887.44 |
| 32+34.0 | 15,510.51 | 7,461.85 |
| 36+56.8 | 15,907.29 | 7,315.85 |
| 38+54.7 | 16,031.63 | 7,161.88 |

| TABLE 2 | | |
|---|---|---|
| UNDERDRAIN RANDOM FILL TRANSVERSE TRENCH HORIZONTAL CONTROL DATA | | |
| PI STATION | NORTHING | EASTING |
| 0+00.0 | 13,197.98 | 9,744.87 |
| 0+40.6 | 13,232.34 | 9,766.57 |
| 5+28.1 | 13,620.41 | 10,061.48 |

LEGEND:

- PLANNED FGMI POST-EXCAVATION SURFACE
- PAD GRADING SURFACE
- UNDERDRAIN RANDOM FILL TRENCH INVERT SURFACE
- △ POINT OF VERTICAL INTERSECTION (PVI, PROFILE ONLY)
- 1625.38 LEACH PAD REGRADED ELEVATION, FEET (PROFILE ONLY)
- 1617.31 ALIGNMENT CENTERLINE ELEVATION, FEET (PROFILE ONLY)

NOTES: ⚠

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL SETTING OUT DATA IS TO BE APPROVED BY THE ENGINEER IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
3. SURVEY CONTROL POINTS SHALL BE PROVIDED BY FGMI.
4. THE UNDERDRAIN RANDOM FILL IS PRIMARILY ENVISIONED AS A LARGE TRENCH DRAIN EXCAVATED DOWN THE CENTER OF THE BARNES CREEK VALLEY. THE UNDERDRAIN RANDOM FILL WILL DROP BELOW THE IN-HEAP STORAGE POND LCRS SUMP AND TERMINATE AT THE UPSTREAM TOE OF THE EXISTING CONVEYOR CAUSEWAY EMBANKMENT. A SECOND UNDERDRAIN RANDOM FILL TRENCH, 20' WIDE BY 6' DEEP, WILL BE EXCAVATED LATERALLY ACROSS THE VALLEY ALONG THE UPSTREAM TOE OF THE CONVEYOR CAUSEWAY.
5. SEVEN (7) DRAINAGE WELLS WILL BE CONSTRUCTED WITHIN THE BASE OF THE TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH TO PROMOTE DRAINAGE FROM THE TRENCH AND INTO THE INTERBEDDED HIGHER CONDUCTIVITY COLLUVIAL MATERIALS BENEATH THE PRE-CONVEYOR CAUSEWAY SURFACE. THE DRAINAGE WELLS WILL BE DRILLED-IN APPROXIMATELY 30' BENEATH THE INVERT OF UNDERDRAIN RANDOM FILL TRENCH AS SHOWN ON DRAWING 206. IF CONTACT WITH THE BEDROCK SURFACE IS ENCOUNTERED PRIOR TO COMPLETING THE 30' WELL DEPTH, THE BOREHOLE SHALL BE STOPPED. THE 30' WELL DEPTH SHOULD BE CONSIDERED AN APPROXIMATE MAXIMUM DEPTH.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 24/17

REVISION:

⚠1 MODIFIED, AND INCREASED SIZE OF, PROFILE FOR TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH. ADDED TWO ADDITIONAL DRAINAGE WELLS. ADDED NOTES 4 AND 5.

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | CADD JTR CB |
| 1 | 03/24/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| | | | |
| | | | |

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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN RANDOM FILL
PROFILES AND HORIZONTAL CONTROL DATA

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 202 | 1 |

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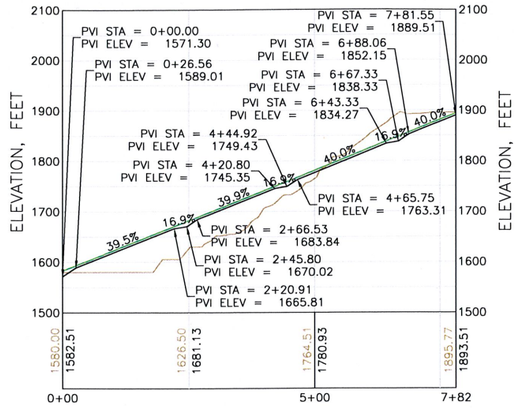
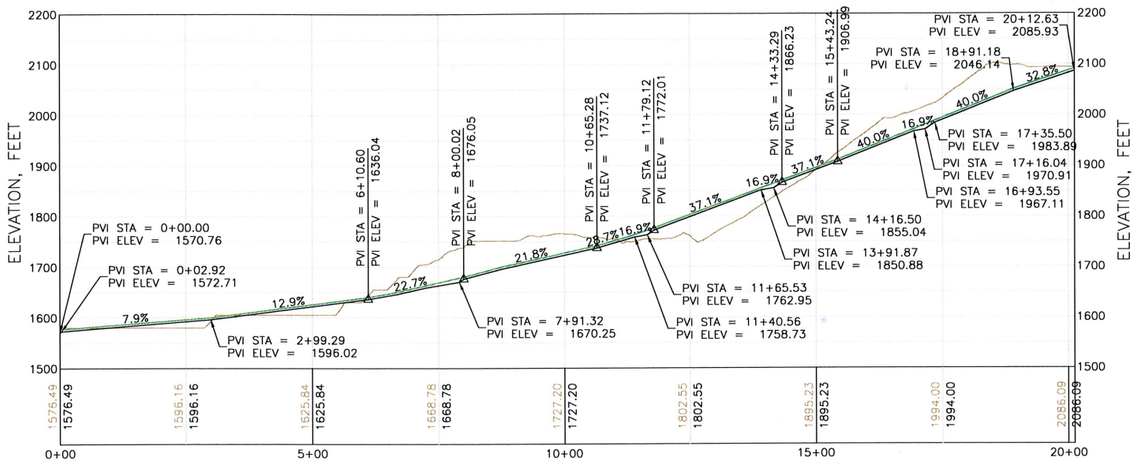



PCMS CHANNEL 1 PROFILE

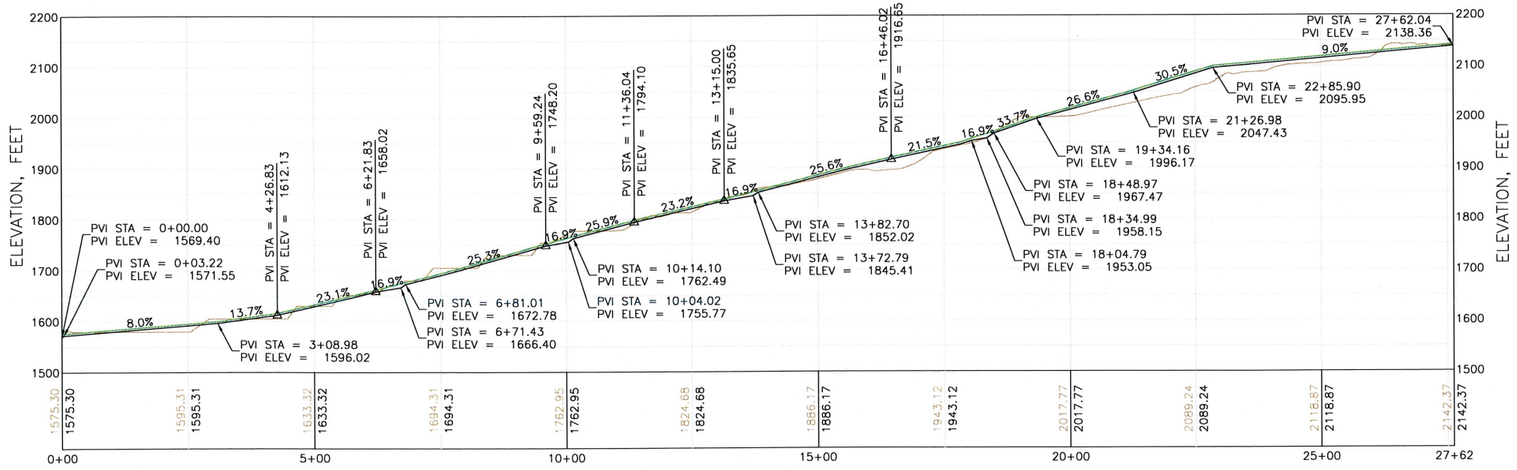
1

| TABLE 1 | | |
|---|---|---|
| PCMS CHANNEL 1 HORIZONTAL CONTROL DATA | | |
| PI STATION | NORTHING | EASTING |
| 0+00.0 | 15,058.10 | 7,847.32 |
| 3+78.8 | 14,807.78 | 7,562.98 |
| 7+81.6 | 14,525.09 | 7,276.15 |




PCMS CHANNEL 2 PROFILE

| TABLE 2 | | |
|---|---|---|
| PCMS CHANNEL 2 HORIZONTAL CONTROL DATA | | |
| PI STATION | NORTHING | EASTING |
| 0+00.0 | 15,165.42 | 7,783.20 |
| 2+99.7 | 15,358.44 | 7,553.91 |
| 3+76.9 | 15,418.69 | 7,505.63 |
| 6+10.0 | 15,609.62 | 7,371.95 |
| 10+51.5 | 15,922.79 | 7,060.82 |
| 19+50.8 | 16,425.08 | 6,314.77 |
| 20+12.6 | 16,447.19 | 6,257.06 |




PCMS CHANNEL 3 PROFILE

| TABLE 3 | | |
|---|---|---|
| PCMS CHANNEL 3 HORIZONTAL CONTROL DATA | | |
| PI STATION | NORTHING | EASTING |
| 0+00.0 | 15,240.30 | 7,906.25 |
| 0+21.6 | 15,253.32 | 7,889.01 |
| 3+07.9 | 15,426.96 | 7,661.40 |
| 4+16.8 | 15,509.71 | 7,590.55 |
| 10+52.7 | 16,143.53 | 7,641.02 |
| 13+33.0 | 16,423.12 | 7,620.32 |
| 16+45.2 | 16,734.53 | 7,641.70 |
| 17+84.6 | 16,872.50 | 7,621.38 |
| 21+76.1 | 17,258.80 | 7,557.83 |
| 27+62.0 | 17,793.65 | 7,318.56 |

## LEGEND:

- PLANNED FGMI POST-EXCAVATION SURFACE
- PAD GRADING SURFACE
- PCMS CHANNEL INVERT SURFACE
- △ POINT OF VERTICAL INTERSECTION (PVI, PROFILE ONLY)
- 1625.38 LEACH PAD REGRADED ELEVATION, FEET (PROFILE ONLY)
- 1617.31 ALIGNMENT CENTERLINE ELEVATION, FEET (PROFILE ONLY)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL SETTING OUT DATA IS TO BE APPROVED BY THE ENGINEER IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
3. SURVEY CONTROL POINTS SHALL BE PROVIDED BY FGMI.

## REVISION:

1 ADDED NEW PCMS CHANNEL 1 AND RENUMBERED OTHERS. MOVED PREVIOUS PCMS CHANNEL 3 TO NEW DRAWING 205.

(TYPICAL SCALE FOR ALL PROFILES)

Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| | | | |
| | | | |

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ACTIVITY CODE: N/A | XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: PCMS CHANNEL PROFILES AND HORIZONTAL CONTROL DATA, SHEET 1 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 204 | 1 |

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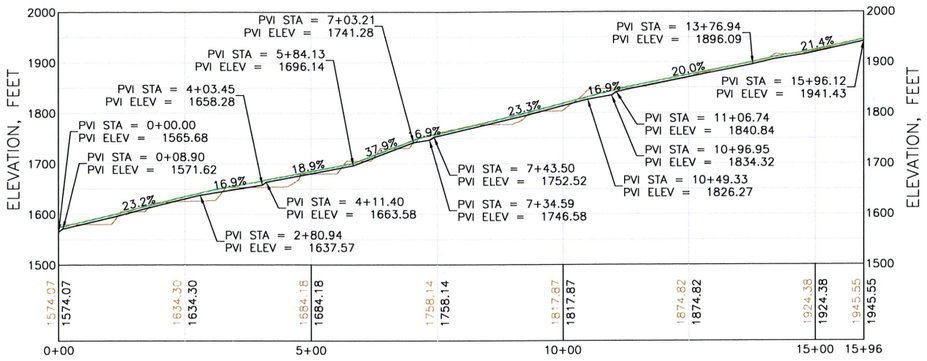
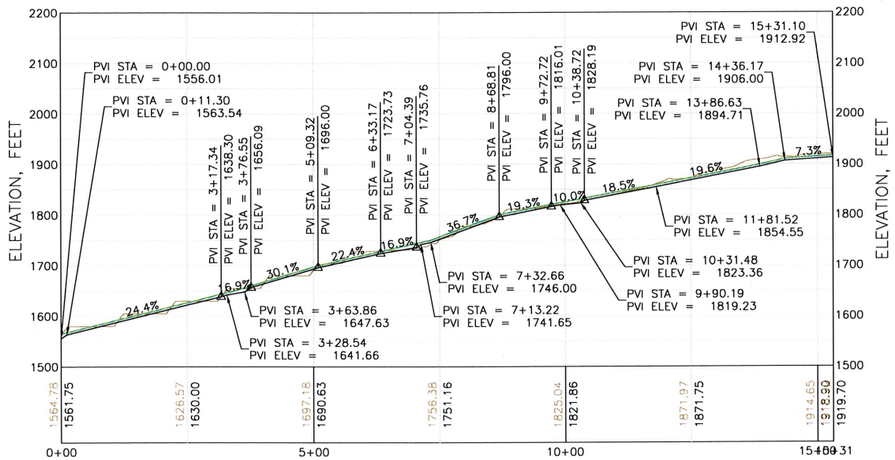



PCMS CHANNEL 4 PROFILE

| TABLE 1 | | |
|---|---|---|
| PCMS CHANNEL 4 HORIZONTAL CONTROL DATA | | |
| PI STATION | NORTHING | EASTING |
| 0+00.0 | 15,128.91 | 8,272.47 |
| 4+17.5 | 15,536.83 | 8,361.52 |
| 15+96.1 | 15,583.88 | 8,902.61 |




PCMS CHANNEL 5 PROFILE

| TABLE 2 | | |
|---|---|---|
| PCMS CHANNEL 5 HORIZONTAL CONTROL DATA | | |
| PI STATION | NORTHING | EASTING |
| 0+00.0 | 14,553.40 | 9,052.18 |
| 15+31.1 | 15,913.19 | 9,755.91 |

## LEGEND:

- PLANNED FGMI POST-EXCAVATION SURFACE
- PAD GRADING SURFACE
- PCMS CHANNEL INVERT SURFACE
- △ POINT OF VERTICAL INTERSECTION (PVI, PROFILE ONLY)
- 1625.38 LEACH PAD REGRADED ELEVATION, FEET (PROFILE ONLY)
- 1617.31 ALIGNMENT CENTERLINE ELEVATION, FEET (PROFILE ONLY)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL SETTING OUT DATA IS TO BE APPROVED BY THE ENGINEER IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
3. SURVEY CONTROL POINTS SHALL BE PROVIDED BY FGMI.

150 0 150 300 FEET

1" = 150' AT FULL SIZE (ANSI D)
1" = 300' AT HALF SIZE (ANSI B)

(TYPICAL SCALE FOR ALL PROFILES)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Ma. 17/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| 0 | 03/17/17 | ISSUED FOR CONSTRUCTION | CADD / JTR / CB |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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: PCMS CHANNEL PROFILES AND HORIZONTAL CONTROL DATA, SHEET 2 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 205 | 0 |

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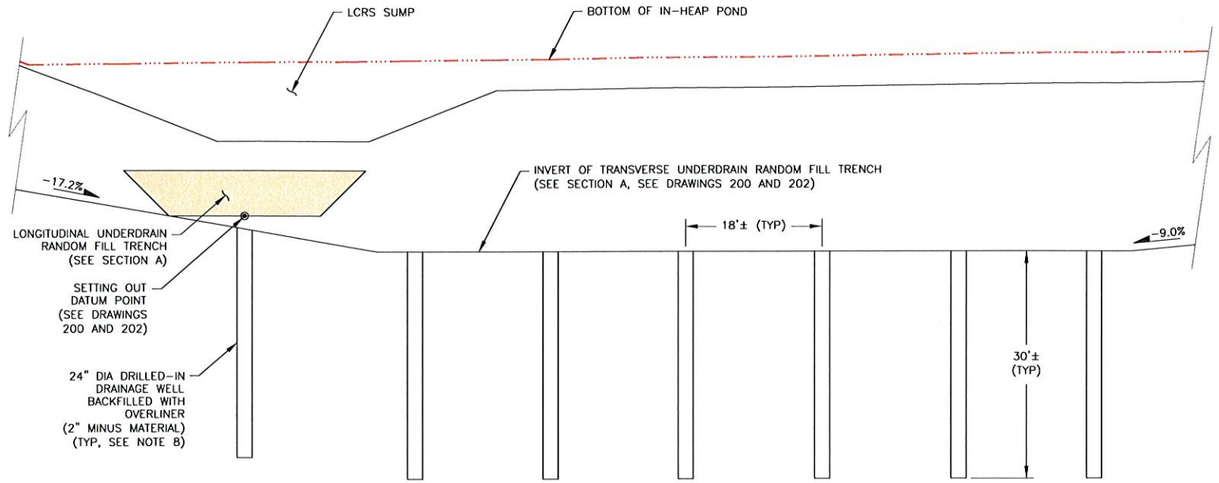
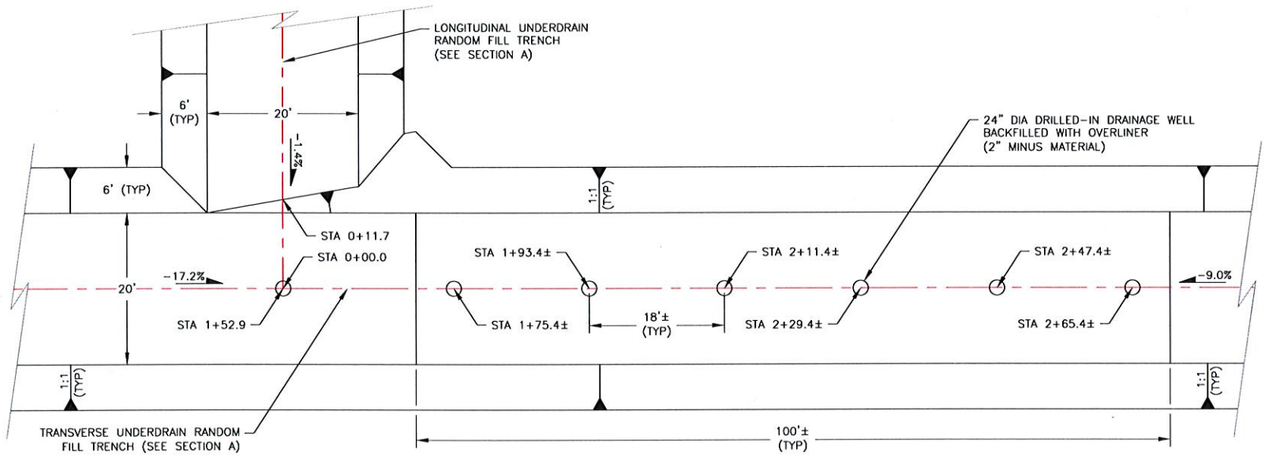


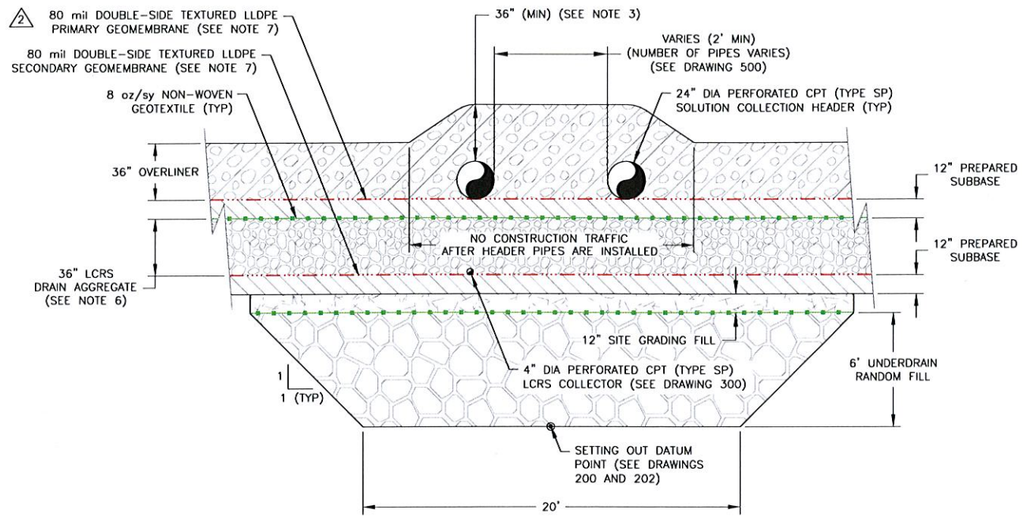

△1 △2 1/– BOTTOM OF IN-HEAP STORAGE POND WITH UNDERDRAIN RANDOM FILL TRENCHES AND DRILLED-IN DRAINAGE WELLS

10 0 10 20 FEET
1"=10' AT FULL SIZE (ANSI D)
1"=20' AT HALF SIZE (ANSI B)



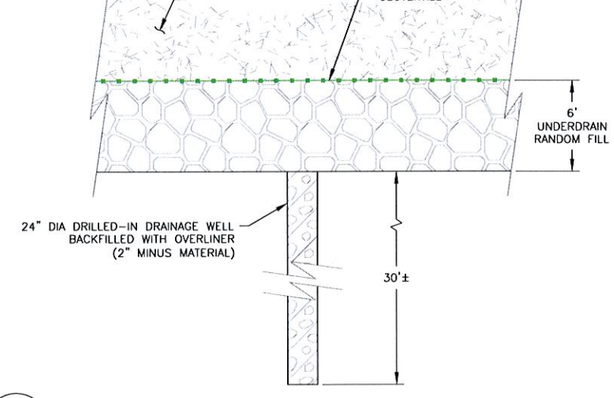

△1 △2 2/– JUNCTION OF UNDERDRAIN RANDOM FILL TRENCHES WITH DRILLED-IN DRAINAGE WELLS

10 0 10 20 FEET
1"=10' AT FULL SIZE (ANSI D)
1"=20' AT HALF SIZE (ANSI B)




△2 A/200 A/500 TYPICAL UNDERDRAIN RANDOM FILL AND OVERLYING COMPONENTS BENEATH FLOOR OF IN-HEAP STORAGE POND

4 0 4 8 FEET
1"=4' AT FULL SIZE (ANSI D)
1"=8' AT HALF SIZE (ANSI B)




△1 3/– TYPICAL DRILLED-IN DRAINAGE WELL BENEATH LATERAL UNDERDRAIN RANDOM FILL TRENCH

5 0 5 10 FEET
1"=5' AT FULL SIZE (ANSI D)
1"=10' AT HALF SIZE (ANSI B)

## REVISION:

△1 ADDED DETAIL CALLOUTS TO TITLES. MODIFIED DETAIL 3 MORE TO SCALE.

△2 ADDED SECTION A FROM DRAWING 220 AND REDUCED SCALE. ADDED NOTES 2 THROUGH 9 (NOTE 7 SPECIFIES GEOMEMBRANE). CHANGED TO SHEET 1 OF 8. MODIFIED DETAILS 1 AND 2 FOR REVISED PROFILE FOR TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH, AND ADDED TWO ADDITIONAL DRAINAGE WELLS. ADDED STATIONING FOR DRAINAGE WELLS.

## NOTES: △2

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
3. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER 12" DIA OR GREATER CPT SOLUTION PIPES. NO CONSTRUCTION TRAFFIC SHALL BE ALLOWED TO PASS OVER THESE AREAS AFTER THE HEADER PIPES ARE INSTALLED. CPT SOLUTION PIPES OF LESS THAN 12" DIA SHALL BE ENCAPSULATED WITHIN A 36"-THICK LAYER OF OVERLINER.
4. WITHIN THE AREA OF THE LEACH PAD COVERED WITH AN LCRS, WHERE THE PAD SLOPES ARE LESS THAN 10%, A 36"-THICK LAYER OF LCRS DRAIN AGGREGATE, OR ALTERNATE CONFIGURATION IF PREFERRED BY FGMI (SEE NOTE 6), WILL BE INCLUDED. A SERIES OF LCRS COLLECTOR PIPES WILL BE ENCAPSULATED WITHIN THE DRAIN AGGREGATE LAYER AND WILL REPORT TO THE LCRS SUMP. SEE DRAWING 300.
5. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED TEES, WYES AND COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
6. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL, IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.
7. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT.
8. THE UNDERDRAIN RANDOM FILL IS PRIMARILY ENVISIONED AS A LARGE TRENCH DRAIN EXCAVATED DOWN THE CENTER OF THE BARNES CREEK VALLEY. THE UNDERDRAIN RANDOM FILL WILL DROP BELOW THE IN-HEAP STORAGE POND LCRS SUMP AND TERMINATE AT THE UPSTREAM TOE OF THE EXISTING CONVEYOR CAUSEWAY EMBANKMENT. A SECOND UNDERDRAIN RANDOM FILL TRENCH, 20' WIDE BY 6' DEEP, WILL BE EXCAVATED LATERALLY ACROSS THE VALLEY ALONG THE UPSTREAM TOE OF THE CONVEYOR CAUSEWAY.
9. SEVEN (7) DRAINAGE WELLS WILL BE CONSTRUCTED WITHIN THE BASE OF THE TRANSVERSE UNDERDRAIN RANDOM FILL TRENCH TO PROMOTE DRAINAGE FROM THE TRENCH AND INTO THE INTERBEDDED HIGHER CONDUCTIVITY COLLUVIAL MATERIALS BENEATH THE PRE-CONVEYOR CAUSEWAY SURFACE. THE DRAINAGE WELLS WILL BE DRILLED-IN APPROXIMATELY 30' BENEATH THE INVERT OF UNDERDRAIN RANDOM FILL TRENCH. IF CONTACT WITH THE BEDROCK SURFACE IS ENCOUNTERED PRIOR TO COMPLETING THE 30' WELL DEPTH, THE BOREHOLE SHALL BE STOPPED. THE 30' WELL DEPTH SHOULD BE CONSIDERED AN APPROXIMATE MAXIMUM DEPTH.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 28/17

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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 1 OF 8 △2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 206 | 2 |

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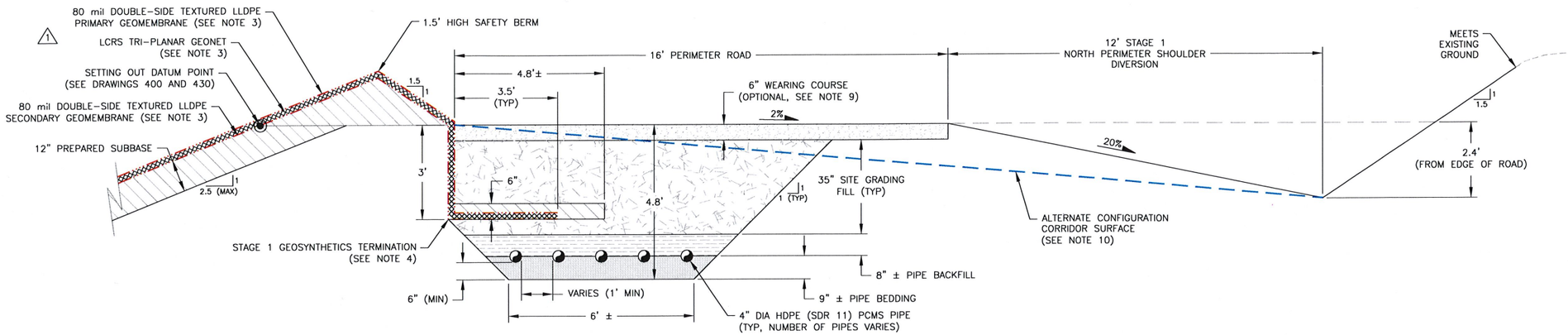
PRINTED BY: JASON REIVA PRINT TIME: 3/24/2017 3:59 PM

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE PCMS CHANNELS SHALL TERMINATE AT SUMPS TO BE LOCATED BENEATH THE STAGE 1 PERIMETER ROAD (SEE DRAWING 430). ANY FLOWS WITHIN THE PCMS CHANNELS WILL BE TRAINED INTO THE PCMS HEADER PIPES AND CONVEYED TO THE DISCHARGE POINTS WITHIN THE NORTHEAST CORNER OF THE IN-HEAP POND.
3. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT. 1
4. THE STAGE 1 GEOSYNTHETICS TERMINATION TRENCH ALONG THE NORTH PERIMETER ROAD SHALL TYPICALLY BE INCORPORATED AND ANCHORED INTO THE BACKFILL OF THE PCMS HEADER PIPES. A MINIMUM 6"-THICK LAYER OF COMPACTED PREPARED SUBBASE SHALL BE CONSTRUCTED OVER THE GEOSYNTHETICS TERMINATION TO MITIGATE EXTERNAL WATERS FROM ENTERING THE LCRS.
5. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
6. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER 12" DIA OR GREATER CPT SOLUTION PIPES. NO CONSTRUCTION TRAFFIC SHALL BE ALLOWED TO PASS OVER THESE AREAS AFTER THE HEADER PIPES ARE INSTALLED. CPT SOLUTION PIPES OF LESS THAN 12" DIA SHALL BE ENCAPSULATED WITHIN A 36"-THICK-LAYER OF OVERLINER.
7. WITHIN THE AREA OF THE LEACH PAD COVERED WITH AN LCRS, WHERE THE PAD SLOPES ARE LESS THAN 10%, A 36"-THICK LAYER OF LCRS DRAIN AGGREGATE, OR ALTERNATE CONFIGURATION IF PREFERRED BY FGMI (SEE NOTE 11), WILL BE INCLUDED. A SERIES OF LCRS COLLECTOR PIPES WILL BE ENCAPSULATED WITHIN THE DRAIN AGGREGATE LAYER AND WILL REPORT TO THE LCRS SUMP. SEE DRAWING 300.
8. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED TEES, WYES AND COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
9. WEARING COURSE IS INTENDED TO IMPROVE THE TRAFFICABILITY OF THE INTER-STAGE ROAD SURFACE; HOWEVER, PLACEMENT OF WEARING COURSE ALONG THE INTER-STAGE PERIMETER ROADS IS OPTIONAL AND MAY BE REMOVED IF PREFERRED BY FGMI. WEARING COURSE MATERIAL SHALL NOT BE PLACED ALONG THE INTER-STAGE PERIMETER ROADS WITHOUT CONFIRMATION FROM FGMI.
10. THE ALTERNATE CONFIGURATION OF THE INTER-STAGE PERIMETER CORRIDOR MAY BE ADOPTED IF PREFFERED BY FGMI. THE CONSTANT CROSS-FALL OF THE ALTERNATE CONFIGURATION SURFACE IS INTENDED TO FACILITATE MAINTENANCE OF THE PERIMETER CORRIDORS, BUT WILL AFFECT TRAFFICABILITY ALONG THE CORRIDORS. THE ALTERNATE CONFIGURATIONS FOR THE STAGES 1 THRU 3 NORTH PERIMETER CORRIDORS WILL RESULT IN CROSS-FALLS OF APPROXIMATELY 9% TO 10%. ALONG THE STAGE 1 NORTH PERIMETER CORRIDOR, THE ALTERNATE CONFIGURATION SHALL NOT BE INCORPORATED WITHIN 10' OF THE PCMS SUMP LOCATIONS.
11. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL, IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.

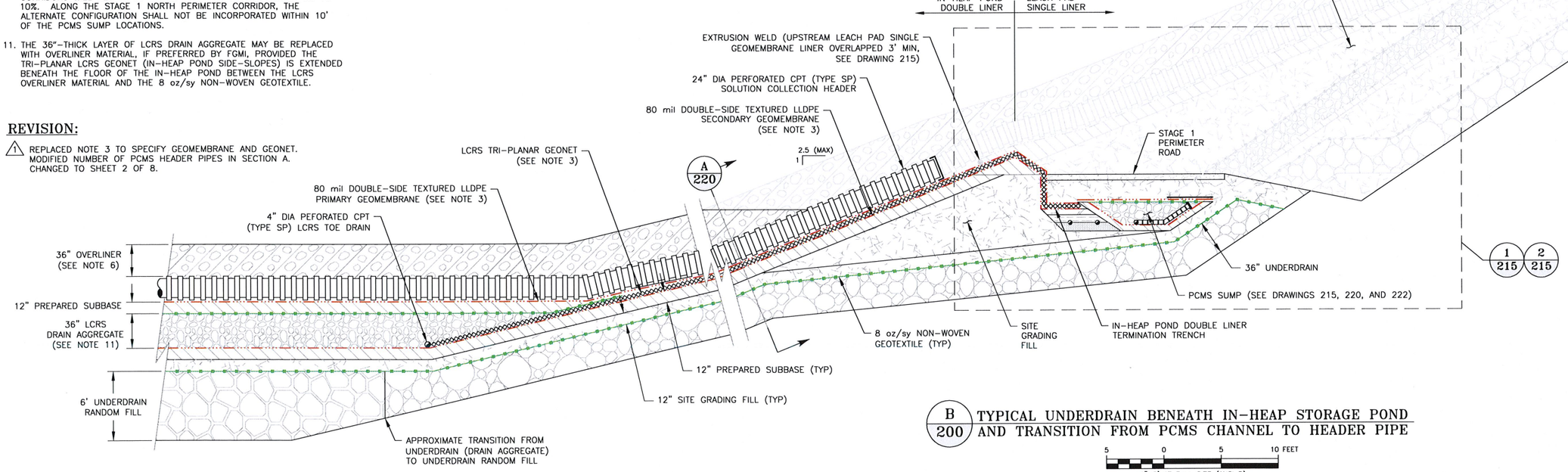
## REVISION:

1 REPLACED NOTE 3 TO SPECIFY GEOMEMBRANE AND GEONET. MODIFIED NUMBER OF PCMS HEADER PIPES IN SECTION A. CHANGED TO SHEET 2 OF 8.




1 A/200 TYPICAL STAGE 1 NORTH PERIMETER ROAD WITH BURIED PCMS HEADER PIPES

2 0 2 4 FEET
1"=2' AT FULL SIZE (ANSI D)
1"=4' AT HALF SIZE (ANSI B)




2 B/200 TYPICAL UNDERDRAIN BENEATH IN-HEAP STORAGE POND AND TRANSITION FROM PCMS CHANNEL TO HEADER PIPE

5 0 5 10 FEET
1"=5' AT FULL SIZE (ANSI D)
1"=10' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 2 OF 8 1

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 210 | 1 |

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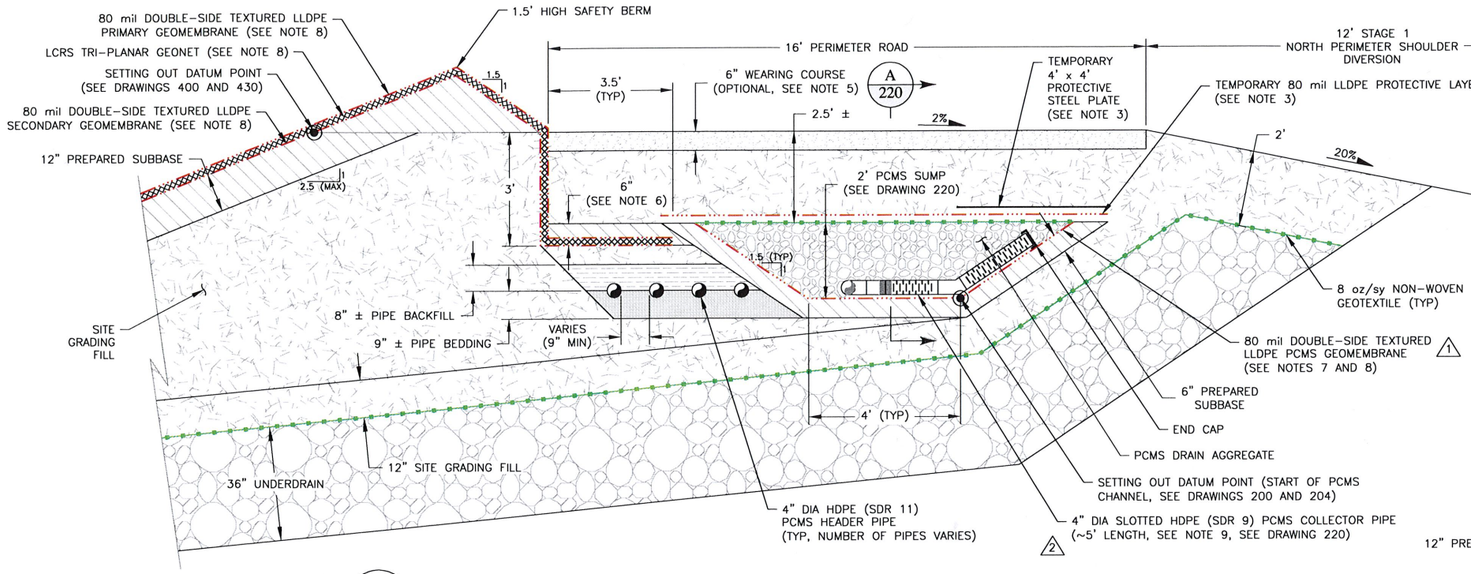
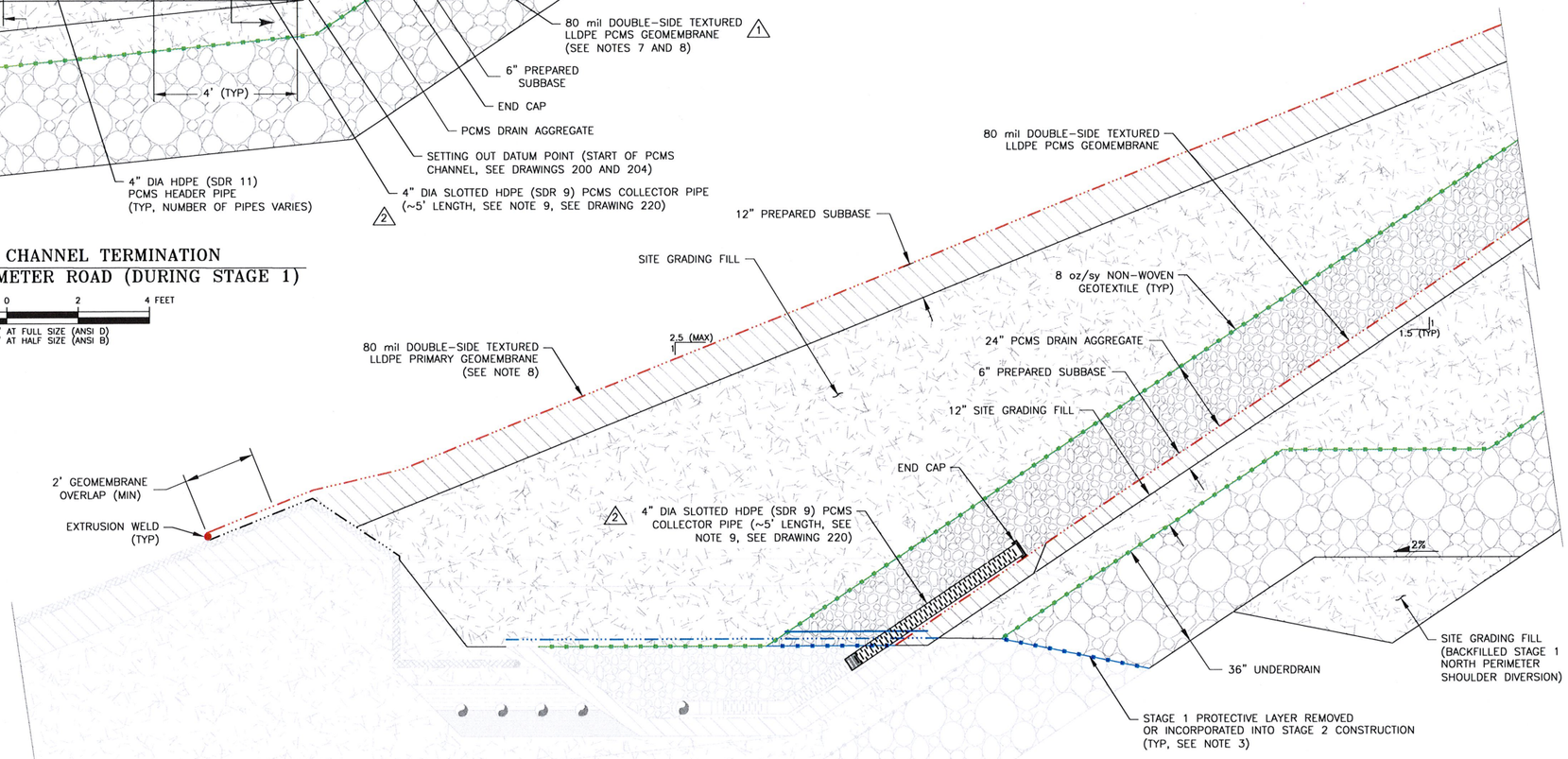



2 1/210 TYPICAL PCMS CHANNEL TERMINATION AT INTER-STAGE PERIMETER ROAD (DURING STAGE 1)

2 0 2 4 FEET
1"=2' AT FULL SIZE (ANSI D)
1"=4' AT HALF SIZE (ANSI B)




2 2/210 TYPICAL PCMS CHANNEL AND UNDER DRAIN TRENCH EXTENSION AT STAGE 1 INTER-STAGE PERIMETER ROAD (DURING STAGE 2)

2 0 2 4 FEET
1"=2' AT FULL SIZE (ANSI D)
1"=4' AT HALF SIZE (ANSI B)

## REVISION:

1 ADDED PCMS GEOMEMBRANE CALLOUT TO DETAIL 1. ADDED NOTE 7.

2 ADDED NOTE 8 SPECIFYING GEOMEMBRANE AND GEONET. ADDED NOTE 9. MODIFIED NUMBER OF PCMS HEADER PIPES IN DETAILS 1 AND 2. CHANGED TO SHEET 3 OF 8.

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE PCMS CHANNELS SHALL TERMINATE AT SUMPS TO BE LOCATED BENEATH THE STAGE 1 PERIMETER ROAD (SEE DRAWING 430). ANY FLOWS WITHIN THE PCMS CHANNELS WILL BE TRAINED INTO THE PCMS HEADER PIPES AND CONVEYED TO THE DISCHARGE POINTS WITHIN THE NORTHEAST CORNER OF THE IN-HEAP POND.
3. THE STAGE 1 PCMS SUMP AND UNDERDRAIN CONSTRUCTION SHALL BE PROTECTED WITH THE PLACEMENT OF OVERLYING LAYERS OF GEOTEXTILE, GEOMEMBRANE, AND STEEL PLATE AS SHOWN. PORTIONS OF THESE LAYERS SHALL BE REMOVED DURING STAGE 2 CONSTRUCTION, AS SHOWN, TO ALLOW PROPER TIE-IN AND EXTENSION OF THE DRAINAGE AND LINER COMPONENTS.
4. THE STAGE 1 GEOSYNTHETICS TERMINATION ALONG THE NORTH PERIMETER ROAD SHALL TYPICALLY BE INCORPORATED AND ANCHORED INTO THE BACKFILL OF THE PCMS HEADER PIPES.
5. WEARING COURSE IS INTENDED TO IMPROVE THE TRAFFICABILITY OF THE INTER-STAGE ROAD SURFACE; HOWEVER, PLACEMENT OF WEARING COURSE ALONG THE INTER-STAGE PERIMETER ROADS IS OPTIONAL AND MAY BE REMOVED IF PREFERRED BY FGMI. WEARING COURSE MATERIAL SHALL NOT BE PLACED ALONG THE INTER-STAGE PERIMETER ROADS WITHOUT CONFIRMATION FROM FGMI.
6. THE STAGE 1 GEOSYNTHETICS TERMINATION TRENCH ALONG THE NORTH PERIMETER ROAD SHALL TYPICALLY BE INCORPORATED AND ANCHORED INTO THE BACKFILL OF THE PCMS HEADER PIPES. A MINIMUM 6"-THICK LAYER OF COMPACTED PREPARED SUBBASE SHALL BE CONSTRUCTED OVER THE GEOSYNTHETICS TERMINATION TO MITIGATE EXTERNAL WATERS FROM ENTERING THE LCRS.
7. THE PCMS HEADER PIPE PENETRATION THROUGH THE PCMS SUMP GEOMEMBRANE LINER, AND THE ASSOCIATED PIPE BOOT, SHALL BE PREFABRICATED PRIOR TO THE ACTUAL FIELD INSTALLATION SUCH THAT, WHEN INSTALLED, THE PIPE PENETRATION (AND SECTION OF SLOTTED HDPE PIPE) WILL BE LOCATED APPROXIMATELY FLUSH WITH THE LOW-POINT OF THE BOTTOM OF THE PCMS SUMP.
8. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.
9. REFER TO DETAIL 2 ON DRAWING 220 FOR THE PCMS COLLECTOR PIPE SLOT ARRANGEMENT.

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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 3 OF 8 2

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| DRAWN BY | CB | DV101 | 00089.99 | 215 | 2 |

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
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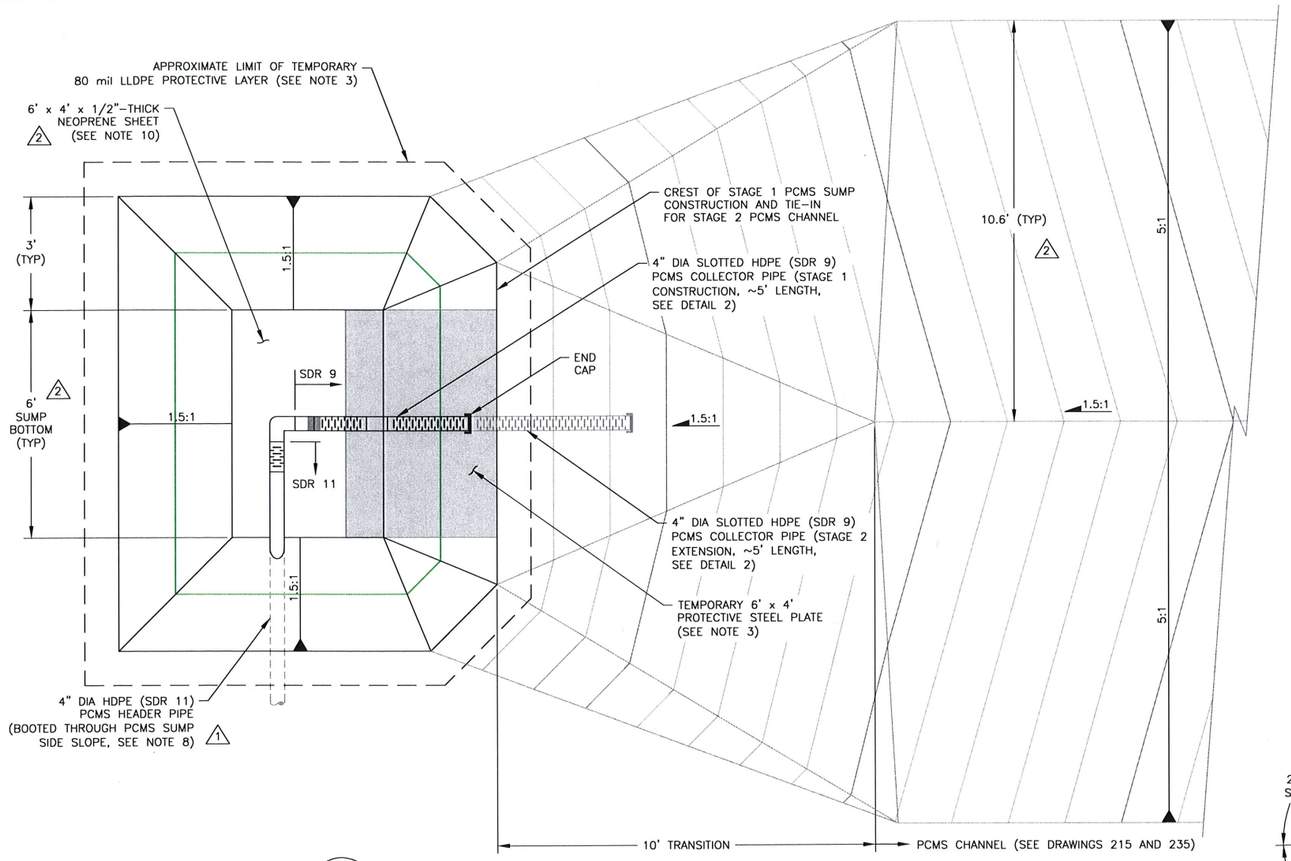
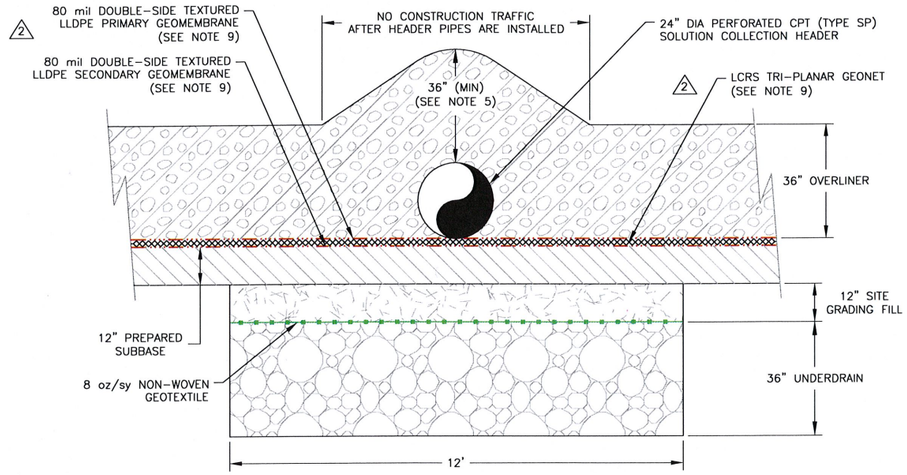
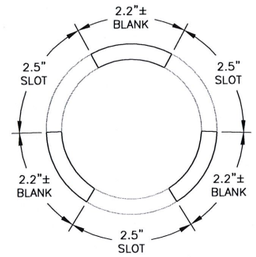


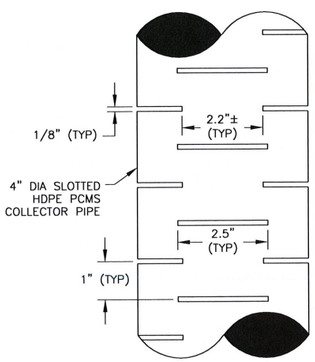

1 — TYPICAL PCMS SUMP TO PCMS CHANNEL TRANSITION 2

(SEE NOTES 8 AND 10) 1



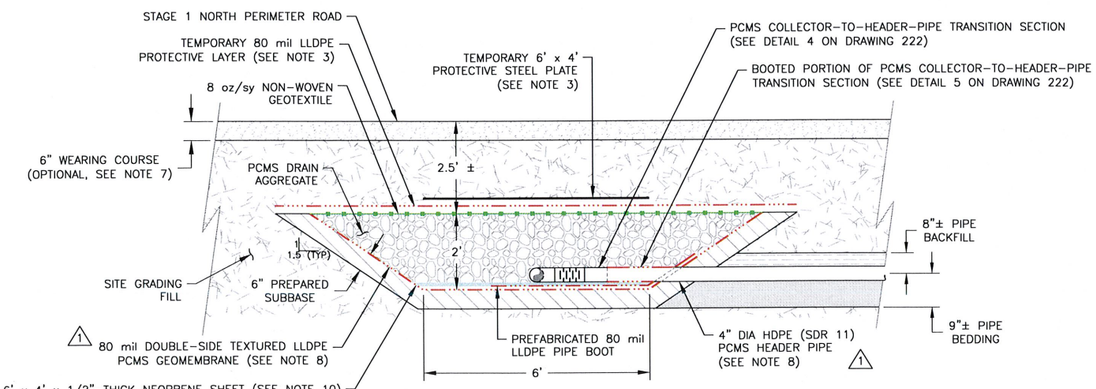

A 200 / A 210 — TYPICAL UNDERDRAIN TRENCH ALONG VALLEY SIDE-SLOPES BENEATH IN-HEAP STORAGE POND




2 — PCMS COLLECTOR PIPE SLOT ARRANGEMENT (PLAN AND PROFILE)

NTS




A 215 — TYPICAL PCMS HEADER PIPE TRANSITION FROM PCMS SUMP 2

(SEE NOTES 8 AND 10) 1

## NOTES: 1 2

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE PCMS CHANNELS SHALL TERMINATE AT SUMPS TO BE LOCATED BENEATH THE STAGE 1 PERIMETER ROAD (SEE DRAWING 430). ANY FLOWS WITHIN THE PCMS CHANNELS WILL BE TRAINED INTO THE PCMS HEADER PIPES AND CONVEYED TO THE DISCHARGE POINTS WITHIN THE NORTHEAST CORNER OF THE IN-HEAP POND.
3. THE STAGE 1 PCMS SUMP AND UNDERDRAIN CONSTRUCTION SHALL BE PROTECTED WITH THE PLACEMENT OF OVERLYING LAYERS OF GEOTEXTILE, GEOMEMBRANE, AND STEEL PLATE AS SHOWN. PORTIONS OF THESE LAYERS SHALL BE REMOVED DURING STAGE 2 CONSTRUCTION, AS SHOWN ON DRAWING 215, TO ALLOW PROPER TIE-IN AND EXTENSION OF THE DRAINAGE AND LINER COMPONENTS.
4. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
5. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER 12" DIA OR GREATER CPT SOLUTION PIPES. NO CONSTRUCTION TRAFFIC SHALL BE ALLOWED TO PASS OVER THESE AREAS AFTER THE HEADER PIPES ARE INSTALLED. CPT SOLUTION PIPES OF LESS THAN 12" DIA SHALL BE ENCAPSULATED WITHIN A 36"-THICK LAYER OF OVERLINER.
6. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED TEES, WYES AND COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
7. WEARING COURSE IS INTENDED TO IMPROVE THE TRAFFICABILITY OF THE INTER-STAGE ROAD SURFACE; HOWEVER, PLACEMENT OF WEARING COURSE ALONG THE INTER-STAGE PERIMETER ROADS IS OPTIONAL AND MAY BE REMOVED IF PREFERRED BY FGMI. WEARING COURSE MATERIAL SHALL NOT BE PLACED ALONG THE INTER-STAGE PERIMETER ROADS WITHOUT CONFIRMATION FROM FGMI.
8. THE PCMS HEADER PIPE PENETRATION THROUGH THE PCMS SUMP GEOMEMBRANE LINER, AND THE ASSOCIATED PIPE BOOT, SHALL BE PREFABRICATED PRIOR TO THE ACTUAL FIELD INSTALLATION SUCH THAT, WHEN INSTALLED, THE PIPE PENETRATION (AND SECTIONS OF SLOTTED HDPE PIPE) WILL BE LOCATED APPROXIMATELY FLUSH WITH THE BOTTOM OF THE PCMS SUMP. SEE DRAWING 222 FOR MORE INFORMATION.
9. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.
10. A NEOPRENE SHEET SHALL BE PLACED TO COVER THE BOTTOM OF THE PCMS SUMP AFTER ALL GEOMEMBRANE, PIPE BOOT, AND PIPEWORK HAVE BEEN INSTALLED. THE NEOPRENE SHEET SHALL BE PLACED BETWEEN THE PCMS PIPEWORK AND THE UNDERLYING SUMP GEOMEMBRANE/PIPE BOOT APRON.
11. REFER TO DRAWING 222 FOR MORE INFORMATION ON THE PCMS SUMP, COLLECTOR PIPE, AND PIPE BOOT.

## REVISION:

1 ADDED NOTE 10.

2 MOVED UNDERDRAIN RANDOM FILL TRENCH SECTION TO DRAWING 206. REMOVED PREVIOUS NOTES 6 AND 8. ADDED NOTE 9 SPECIFYING GEOMEMBRANE AND GEONET. WIDENED PCMS CHANNEL AND SUMP. ADDED NEOPRENE SHEET ON PCMS SUMP BOTTOM. ADDED NOTES 10 AND 11. ADDED REFERENCES TO NEW DRAWING 222. ADDED DETAIL 2. CHANGED TO SHEET 4 OF 8.

2 0 2 4 Feet

1"=2' AT FULL SIZE (ANSI D)
1"=4' AT HALF SIZE (ANSI B)

(TYPICAL SCALE FOR ALL SECTIONS)

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| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |

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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 4 OF 8 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 220 | 2 |

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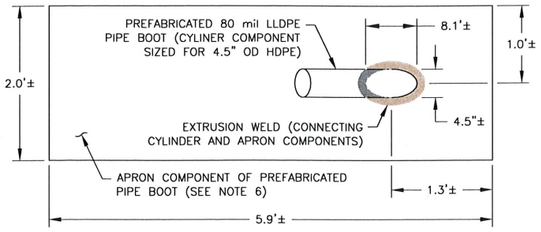
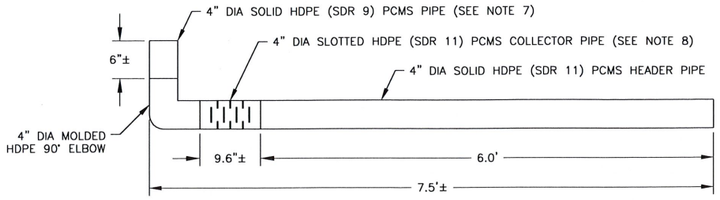
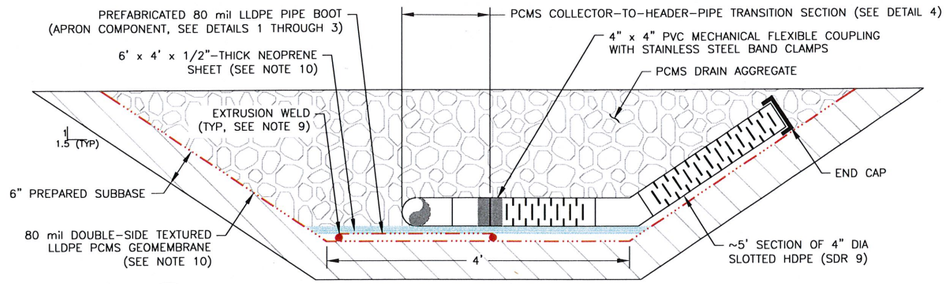


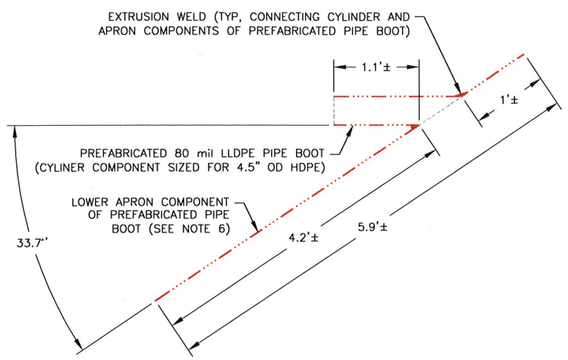

1 — PREFABRICATED PIPE BOOT FOR PCMS HEADER PIPE TRANSITION FROM PCMS SUMP (TOP, FLAT VIEW)
(SEE NOTE 6)



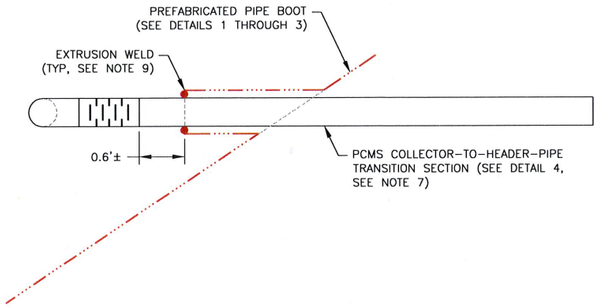

4 — PCMS COLLECTOR–TO–HEADER–PIPE TRANSITION SECTION FOR TRANSITION FROM PCMS SUMP (TOP, FLAT VIEW)
(SEE NOTE 7)



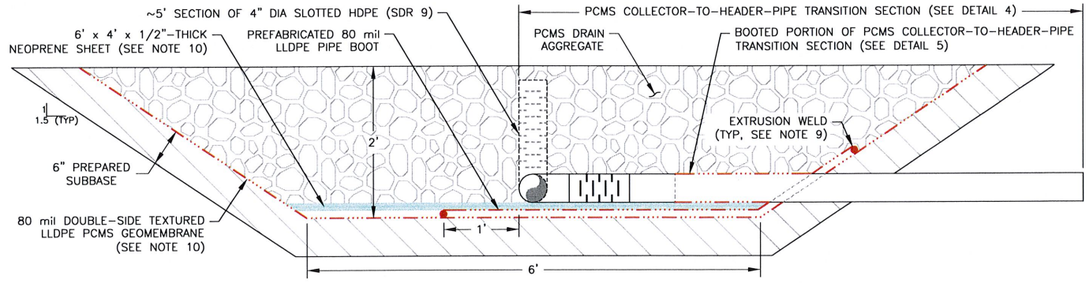

7 — PCMS SUMP WITH COLLECTOR PIPE AND COLLECTOR–TO–HEADER–PIPE TRANSITION (SIDE VIEW)



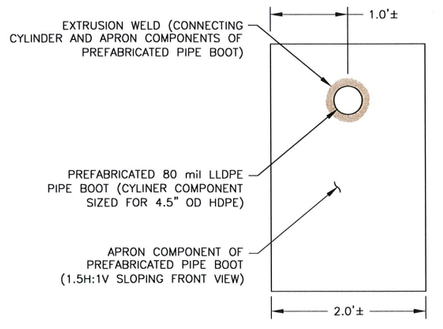

2 — PREFABRICATED PIPE BOOT FOR PCMS HEADER PIPE TRANSITION FROM PCMS SUMP (SIDE VIEW)
(SEE NOTE 6)



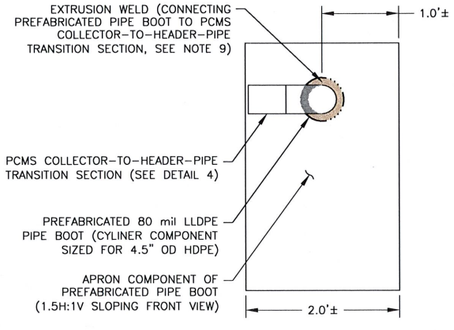

5 — PREFABRICATED PIPE BOOT CONNECTION TO PCMS HEADER PIPE TRANSITION SECTION (SIDE VIEW)
(SEE NOTES 6 AND 7)




8 — PCMS SUMP WITH COLLECTOR PIPE AND COLLECTOR–TO–HEADER–PIPE TRANSITION (FRONT VIEW)



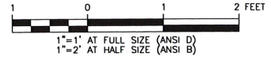

3 — PREFABRICATED PIPE BOOT FOR PCMS HEADER PIPE TRANSITION FROM PCMS SUMP (FRONT, SLOPED VIEW)
(SEE NOTE 6)




6 — PREFABRICATED PIPE BOOT CONNECTION TO PCMS HEADER PIPE TRANSITION SECTION (FRONT, SLOPED VIEW)
(SEE NOTES 6 AND 7)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI–D) AND 1.5" ON A HALF SIZE PLOT (ANSI–B).
2. THE PCMS SUMPS SHALL BE LOCATED BENEATH THE STAGE 1 PERIMETER ROAD (SEE DRAWING 430). ANY FLOWS WITHIN THE PCMS CHANNELS WILL BE TRAINED INTO THE PCMS HEADER PIPES AND CONVEYED TO THE DISCHARGE POINTS WITHIN THE NORTHEAST CORNER OF THE IN–HEAP POND.
3. THE STAGE 1 PCMS SUMP AND UNDERDRAIN CONSTRUCTION SHALL BE PROTECTED WITH THE PLACEMENT OF OVERLYING LAYERS OF GEOTEXTILE, GEOMEMBRANE, AND STEEL PLATE AS SHOWN ON DRAWING 215. PORTIONS OF THESE LAYERS SHALL BE REMOVED DURING STAGE 2 CONSTRUCTION, AS SHOWN ON DRAWING 215, TO ALLOW PROPER TIE–IN AND EXTENSION OF THE DRAINAGE AND LINER COMPONENTS.
4. ALL PIPE TERMINATIONS SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
5. THE PCMS HEADER PIPE PENETRATION THROUGH THE PCMS SUMP GEOMEMBRANE LINER, AND THE ASSOCIATED PIPE BOOT, SHALL BE PREFABRICATED PRIOR TO THE ACTUAL FIELD INSTALLATION SUCH THAT, WHEN INSTALLED, THE PIPE PENETRATION (AND SECTIONS OF SLOTTED HDPE PIPE) WILL BE LOCATED APPROXIMATELY FLUSH WITH THE BOTTOM OF THE PCMS SUMP.
6. THE 80 mil LLDPE PIPE BOOT SHALL BE PREFABRICATED WITH THE CYLINDER AND APRON COMPONENTS SHOWN. THE COMPONENTS SHALL BE CONNECTED SUCH THAT THE LOWER APRON COMPONENT (SEE DETAIL 2) CAN BE BENT FLUSH WITH THE UNDERSIDE OF THE CYLINDER COMPONENT.
7. THE PCMS COLLECTOR–TO–HEADER–PIPE TRANSITION SECTION SHALL BE FABRICATED PRIOR TO CONNECTING TO THE PREFABRICATED PIPE BOOT AND INSTALLATION WITHIN THE PCMS SUMP.
8. REFER TO DETAIL 2 ON DRAWING 220 FOR THE PCMS COLLECTOR PIPE SLOT ARRANGEMENT.
9. PIPES, PIPE BOOTS, AND GEOMEMBRANE LAYERS ARE TYPICALLY SHOWN WITH SMALL OFFSETS TO PROVIDE BETTER CLARITY OF THE DIFFERENT MATERIALS. GAPS ARE NOT INTENDED TO BE DEVELOPED, NOR WILL THEY BE ACCEPTED. JOINED SURFACES AND OVERLYING SURFACES SHALL LAY FLUSH WITH EACH OTHER TO THE EXTENTS POSSIBLE.
10. A NEOPRENE SHEET SHALL BE PLACED TO COVER THE BOTTOM OF THE PCMS SUMP AFTER ALL GEOMEMBRANE, PIPE BOOT, AND PIPEWORK HAVE BEEN INSTALLED. THE NEOPRENE SHEET SHALL BE PLACED BETWEEN THE PCMS PIPEWORK AND THE UNDERLYING SUMP GEOMEMBRANE/PIPE BOOT APRON.




(TYPICAL SCALE FOR ALL SECTIONS)

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| 0 | 03/17/17 | ISSUED FOR CONSTRUCTION | CADD / JTR / CB |
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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 5 OF 8

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 222 | 0 |

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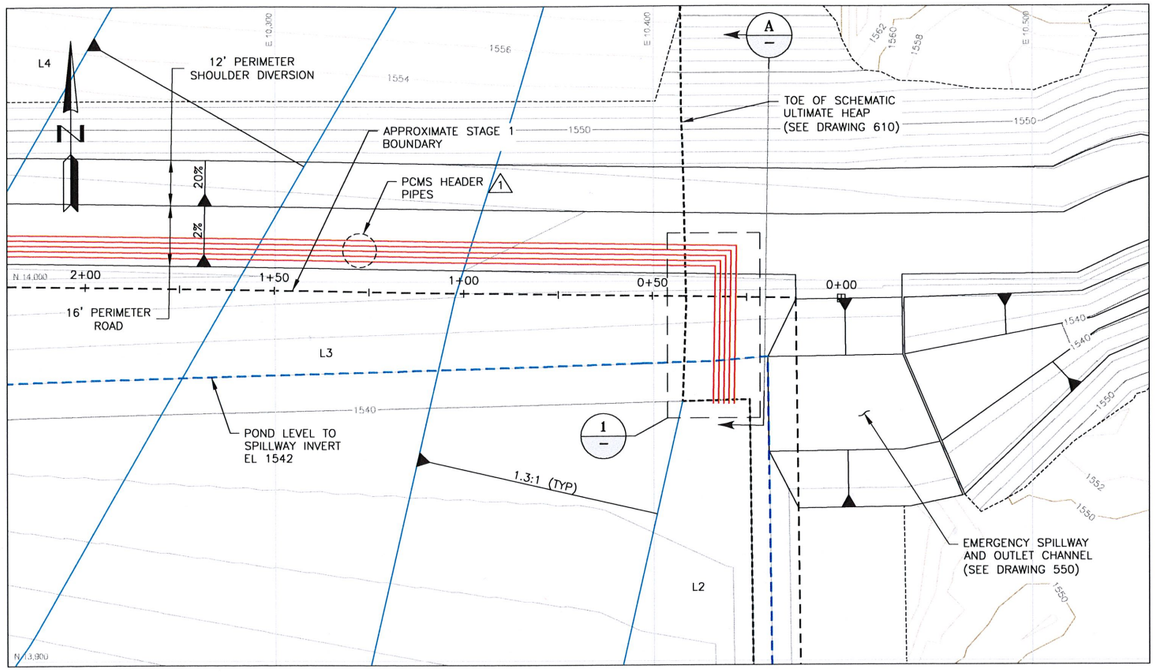



1/200 OUTLET OF PCMS HEADER PIPES

20 0 20 40 Feet

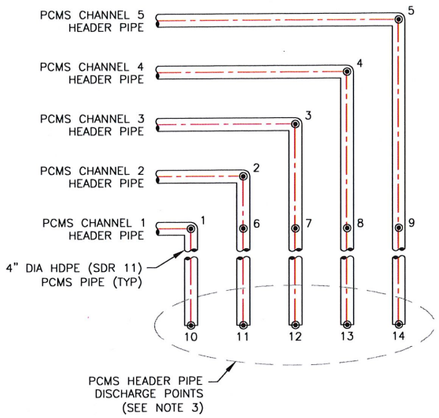
1"=20' AT FULL SIZE (ANSI D)
1"=40' AT HALF SIZE (ANSI B)

## LEGEND:

- 2200 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 2200 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- 4" DIA HDPE (SDR 11) PCMS HEADER PIPE
- L3 — LIFT DESIGNATION
- ⊙1 — SETTING OUT DATA POINT (SEE DETAIL 1, SEE NOTE 10)

## NOTES:

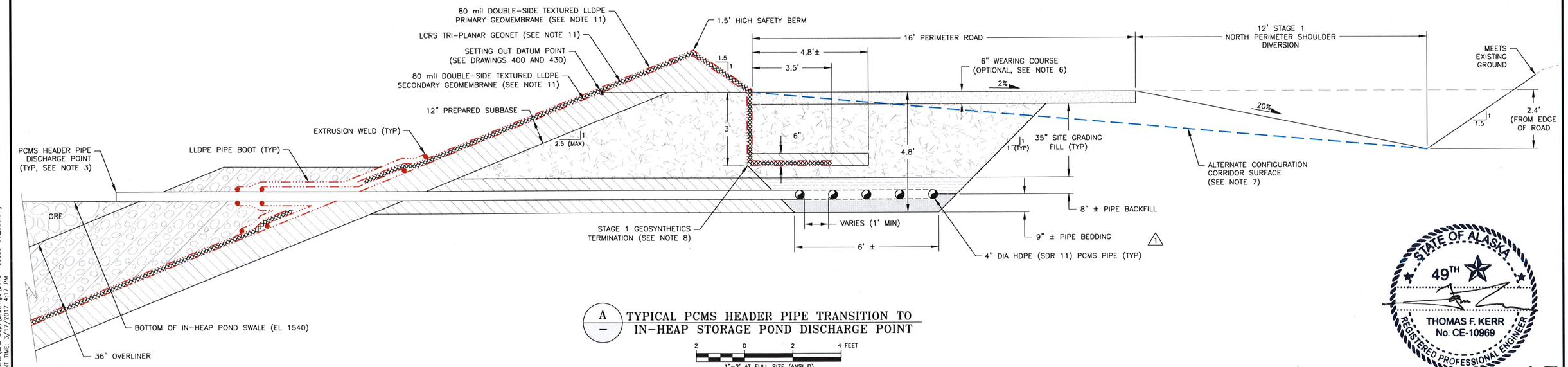
1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE PCMS CHANNELS SHALL TERMINATE AT SUMPS TO BE LOCATED BENEATH THE STAGE 1 PERIMETER ROAD (SEE DRAWING 430). ANY FLOWS WITHIN THE PCMS CHANNELS WILL BE TRAINED INTO THE PCMS HEADER PIPES AND CONVEYED TO THE DISCHARGE POINTS WITHIN THE NORTHEAST CORNER OF THE IN-HEAP POND.
3. THE PCMS HEADER PIPES SHALL EXTEND 1' BEYOND THE SURFACE OF THE OVERLINER DRAINAGE AGGREGATE AND DISCHARGE TO THE SURFACE OF THE IN-HEAP POND SWALE.
4. THE PCMS HEADER PIPE SETTING OUT DATA REPRESENT THE CENTERLINE BOTTOM OF PIPE.
5. THE 4" DIA PCMS HEADERS EACH WILL HAVE 2 HEAT TRACES AND TRD'S INSTALLED. THE FIRST LOOP IS A PRIMARY AND SECOND LOOP A BACKUP. THE RTD AND HEAT TRACE LOOPS WILL BE INSTALLED FOR THE FIRST 70' OF THE 4" DIA PCMS HEADERS.
6. WEARING COURSE IS INTENDED TO IMPROVE THE TRAFFICABILITY OF THE INTER-STAGE ROAD SURFACE; HOWEVER, PLACEMENT OF WEARING COURSE ALONG THE INTER-STAGE PERIMETER ROADS IS OPTIONAL AND MAY BE REMOVED IF PREFERRED BY FGMI. WEARING COURSE MATERIAL SHALL NOT BE PLACED ALONG THE INTER-STAGE PERIMETER ROADS WITHOUT CONFIRMATION FROM FGMI.
7. THE ALTERNATE CONFIGURATION OF THE INTER-STAGE PERIMETER CORRIDOR MAY BE ADOPTED IF PREFFERED BY FGMI. THE CONSTANT CROSS-FALL OF THE ALTERNATE CONFIGURATION SURFACE IS INTENDED TO FACILITATE MAINTENANCE OF THE PERIMETER CORRIDORS, BUT WILL AFFECT TRAFFICABILITY ALONG THE CORRIDORS. THE ALTERNATE CONFIGURATIONS FOR THE STAGES 1 THRU 3 NORTH PERIMETER CORRIDORS WILL RESULT IN CROSS-FALLS OF APPROXIMATELY 9% TO 10%. ALONG THE STAGE 1 NORTH PERIMETER CORRIDOR, THE ALTERNATE CONFIGURATION SHALL NOT BE INCORPORATED WITHIN 10' OF THE PCMS SUMP LOCATIONS.
8. THE STAGE 1 GEOSYNTHETICS TERMINATION TRENCH ALONG THE NORTH PERIMETER ROAD SHALL TYPICALLY BE INCORPORATED AND ANCHORED INTO THE BACKFILL OF THE PCMS HEADER PIPES. A MINIMUM 6"-THICK LAYER OF COMPACTED PREPARED SUBBASE SHALL BE CONSTRUCTED OVER THE GEOSYNTHETICS TERMINATION TO MITIGATE EXTERNAL WATERS FROM ENTERING THE LCRS.
9. THE SETTING OUT DATA REPRESENT THE INVERTS OF THE PCMS HEADER PIPES.
10. ALL SETTING OUT DATA SHALL BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
11. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT. △1




1/— △1 PCMS DISCHARGE POINT SETTING OUT PLAN

### PCMS DISCHARGE POINT SETTING OUT DATA △1

| POINT No. | NORTHING (ft) | EASTING (ft) | ELEVATION (ft) | POINT No. | NORTHING (ft) | EASTING (ft) | ELEVATION (ft) |
|---|---|---|---|---|---|---|---|
| 1 | 14,004 | 10,417 | 1541 | 8 | 14,004 | 10,421 | 1541 |
| 2 | 14,005 | 10,418 | 1541 | 9 | 14,004 | 10,422 | 1541 |
| 3 | 14,007 | 10,419 | 1541 | 10 | 13,968 | 10,416 | 1540 |
| 4 | 14,008 | 10,421 | 1541 | 11 | 13,968 | 10,417 | 1540 |
| 5 | 14,009 | 10,422 | 1541 | 12 | 13,968 | 10,419 | 1540 |
| 6 | 14,004 | 10,418 | 1541 | 13 | 13,968 | 10,420 | 1540 |
| 7 | 14,004 | 10,419 | 1541 | 14 | 13,968 | 10,422 | 1540 |




A/— TYPICAL PCMS HEADER PIPE TRANSITION TO IN-HEAP STORAGE POND DISCHARGE POINT

2 0 2 4 FEET

1"=2' AT FULL SIZE (ANSI D)
1"=4' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

DATE: 7/17

## REVISION:

△1 ADDED TWO PCMS HEADER PIPES AND MODIFIED SPACING AND SETTING OUT DATA. ADDED NOTE 11 SPECIFYING GEOMEMBRANE AND GEONET. CHANGED TO SHEET 6 OF 8

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR |
| | | | CB |
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ACTIVITY CODE: N/A
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Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 6 OF 8 △1

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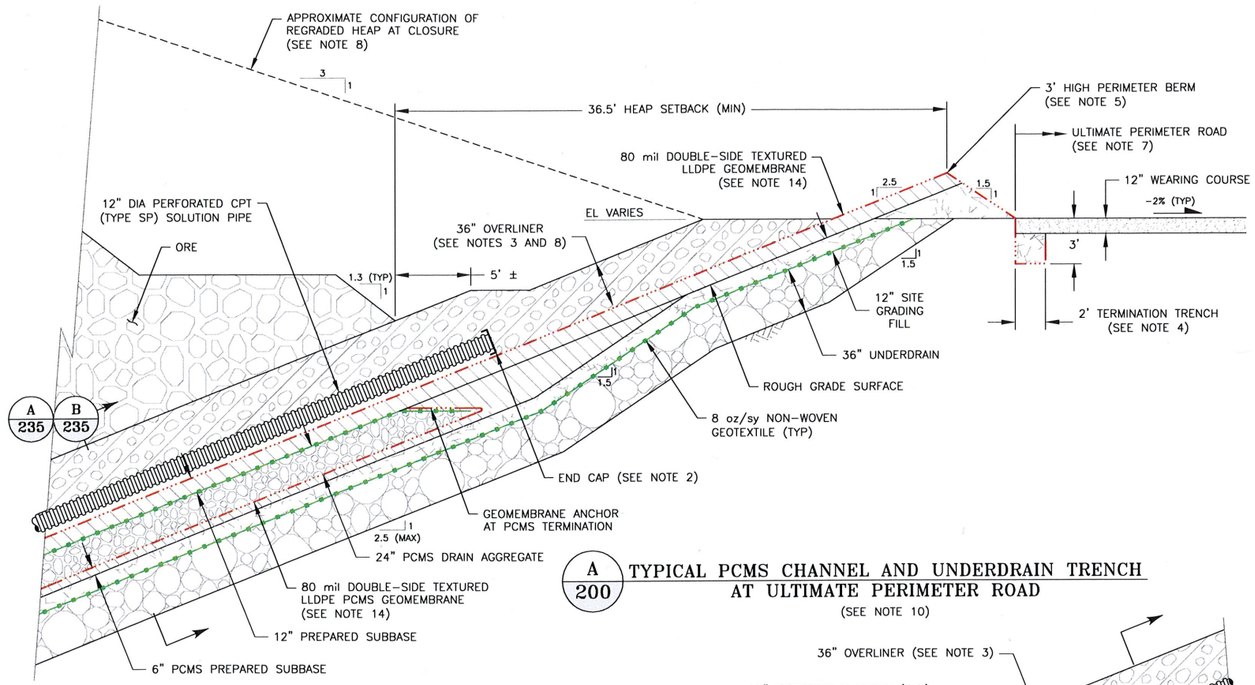
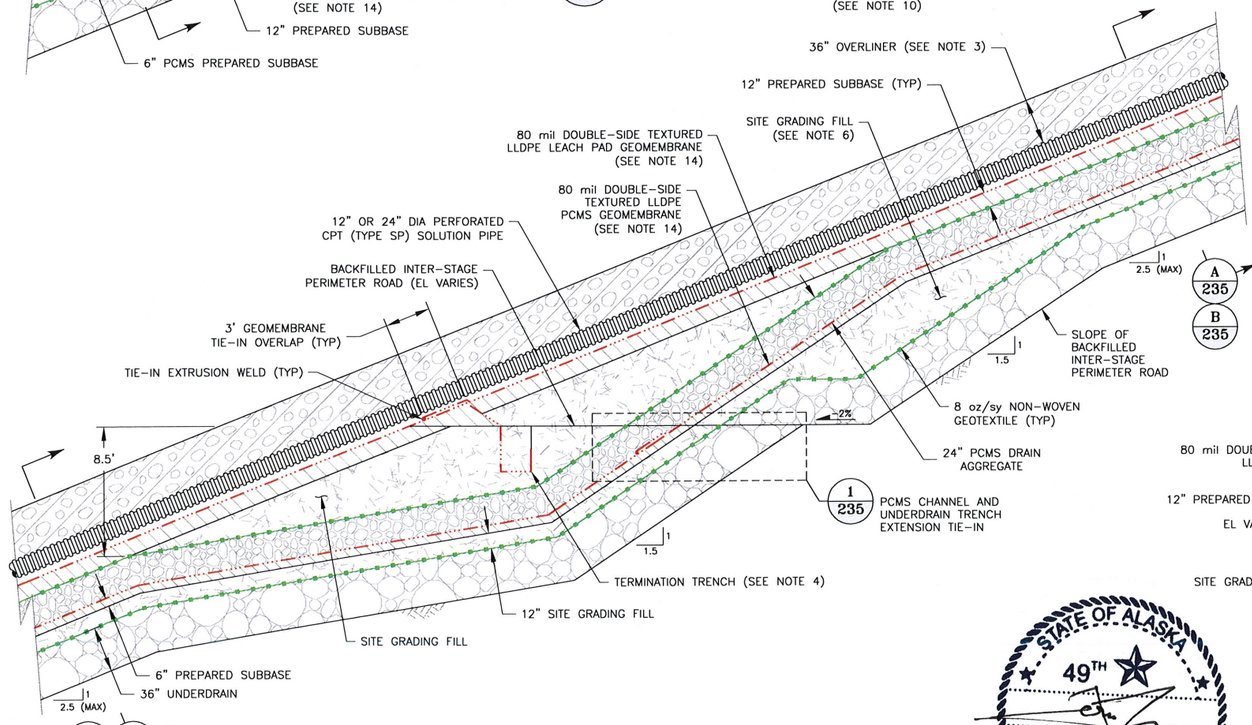
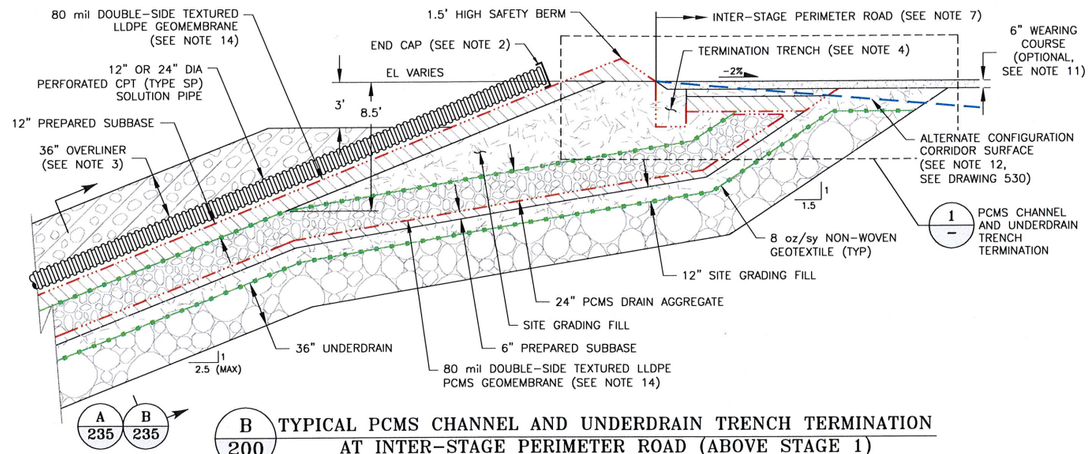



A 200 TYPICAL PCMS CHANNEL AND UNDERDRAIN TRENCH AT ULTIMATE PERIMETER ROAD
(SEE NOTE 10)




C 200 TYPICAL PCMS CHANNEL AND UNDERDRAIN TRENCH EXTENSION AT INTER-STAGE PERIMETER ROAD (ABOVE STAGE 1)
(SEE NOTE 10)



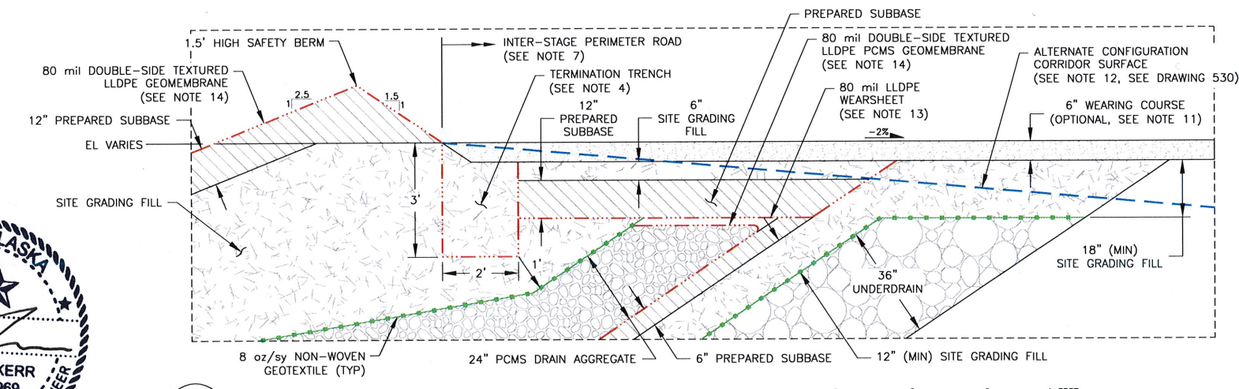

B 200 TYPICAL PCMS CHANNEL AND UNDERDRAIN TRENCH TERMINATION AT INTER-STAGE PERIMETER ROAD (ABOVE STAGE 1)
(SEE NOTE 10)

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
3. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER 12" DIA OR GREATER CPT SOLUTION PIPES. NO CONSTRUCTION TRAFFIC SHALL BE ALLOWED TO PASS OVER THESE AREAS AFTER THE HEADER PIPES ARE INSTALLED. CPT SOLUTION PIPES OF LESS THAN 12" DIA SHALL BE ENCAPSULATED WITHIN A 36"-THICK-LAYER OF OVERLINER.
4. ABOVE STAGE 1, TERMINATION TRENCH BACKFILL SHALL BE CONSTRUCTED OF COMPACTED SITE GRADING FILL.
5. THE PAD PERIMETER BERM SHALL SHALL BE CONSTRUCTED WITH A 12" THICK LAYER OF COMPACTED PREPARED SUBBASE MATERIAL ALONG THE SLOPING FACE OF THE BERM INTERIOR TO THE LEACH PAD. THE UNDERLYING PORTION OF THE PAD PERIMETER BERM SHALL BE CONSTRUCTED OF COMPACTED SITE GRADING FILL UNLESS OTHERWISE AUTHORIZED BY THE ENGINEER AND/OR FGMI.
6. TO EXTEND THE VALLEY FILL LEACH PAD LINING SYSTEM, SITE GRADING FILL SHALL BE PLACED AND COMPACTED TO FORM THE LEACH PAD SURFACE.
7. REFER TO DRAWINGS 530, 532, AND 534 FOR THE INTER-STAGE AND ULTIMATE PAD PERIMETER ROAD SECTIONS AND DETAILS.
8. THE EXTENT OF THE LEACH PAD OVERLINER LAYER, BEYOND THE FOOTPRINT OF THE OPERATIONAL PERIOD HEAP CONFIGURATION, IS INTENDED TO PROTECT THE LEACH PAD GEOMEMBRANE LINER AND ACCOMMODATE THE REGRADED HEAP AT CLOSURE (BASED ON ASSUMED HEAP CLOSURE REGRADING TO A CONSISTENT 3H:1V OVERALL SLOPE, BY OTHERS).
9. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED TEES, WYES AND COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
10. WHERE THE PCMS EXISTS, IT SHALL ALWAYS BE CONSTRUCTED IMMEDIATELY BENEATH THE LEACH PAD LINER SYSTEM. WHERE THE PCMS CHANNEL DOES NOT EXIST, THE GROUNDWATER UNDERDRAIN WILL BE CONSTRUCTED BENEATH THE LEACH PAD LINER SYSTEM ACCORDINGLY. THE GROUNDWATER UNDERDRAINS SHALL BE EXCAVATED INTO EXISTING GROUND, OR LOCATED AT THE INTERFACE OF SITE GRADING FILL WITH EXISTING GROUND.
11. WEARING COURSE IS INTENDED TO IMPROVE THE TRAFFICABILITY OF THE INTER-STAGE ROAD SURFACE; HOWEVER, PLACEMENT OF WEARING COURSE ALONG THE INTER-STAGE PERIMETER ROADS IS OPTIONAL AND MAY BE REMOVED IF PREFERRED BY FGMI. WEARING COURSE MATERIAL SHALL NOT BE PLACED ALONG THE INTER-STAGE PERIMETER ROADS WITHOUT CONFIRMATION FROM FGMI.
12. THE ALTERNATE CONFIGURATION OF THE INTER-STAGE PERIMETER CORRIDOR MAY BE ADOPTED IF PREFFERED BY FGMI. THE CONSTANT CROSS-FALL OF THE ALTERNATE CONFIGURATION SURFACE IS INTENDED TO FACILITATE MAINTENANCE OF THE PERIMETER CORRIDORS, BUT WILL AFFECT TRAFFICABILITY ALONG THE CORRIDORS. THE ALTERNATE CONFIGURATIONS FOR THE STAGES 1 THRU 3 NORTH PERIMETER CORRIDORS WILL RESULT IN CROSS-FALLS OF APPROXIMATELY 9% TO 10%. ALONG THE STAGE 1 NORTH PERIMETER CORRIDOR, THE ALTERNATE CONFIGURATION SHALL NOT BE INCORPORATED WITHIN 10' OF THE PCMS SUMP LOCATIONS (SEE DRAWING 430).
13. WHEN EXTENDING THE PCMS CHANNEL, THE GEOMEMBRANE WEARSHEET SHALL BE REMOVED AND THE PCMS GEOMEMBRANE LINER FOLDED BACK TO ALLOW PROPER CONTINUOUS TIE-IN OF THE PCMS DRAIN AGGREGATE.
14. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. 1

## REVISION:

1 ADDED NOTE 14 SPECIFYING GEOMEMBRANE. CHANGED TO SHEET 7 OF 8.




1 — TYPICAL PCMS CHANNEL AND UNDERDRAIN TRENCH TERMINATION AT INTER-STAGE PERIMETER ROAD

2 0 2 4 FEET
1" = 2' AT FULL SIZE (ANSI D)
1" = 4' AT HALF SIZE (ANSI B)

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)
(TYPICAL SCALE UNLESS OTHERWISE NOTED)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
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| | | | CADD |
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TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 7 OF 8 1

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 230 | 1 |

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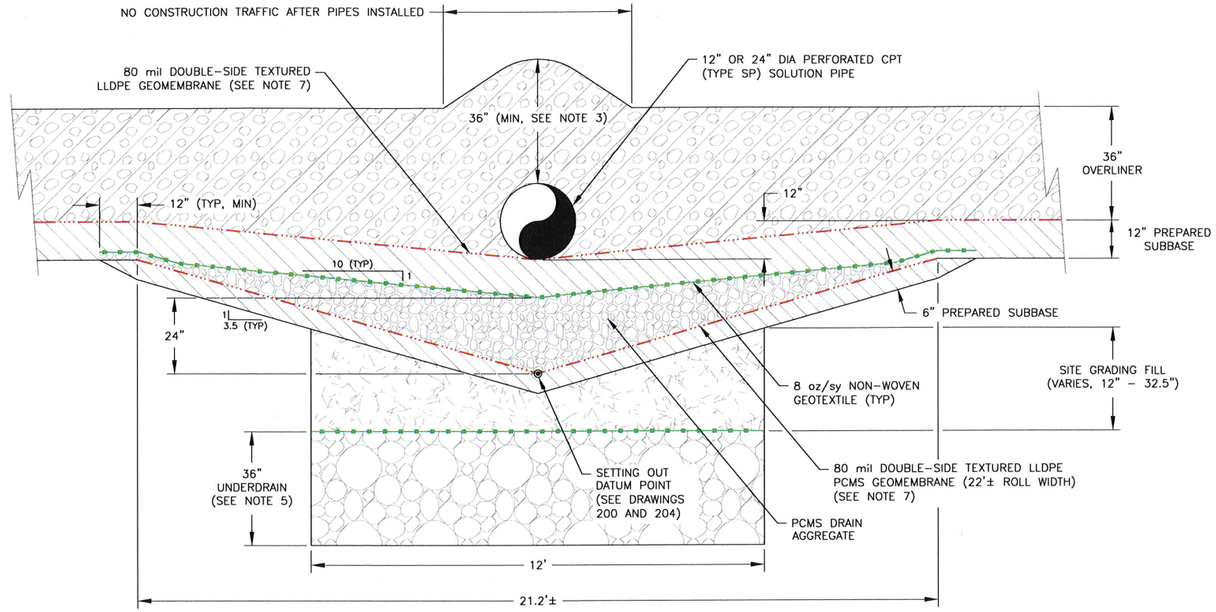
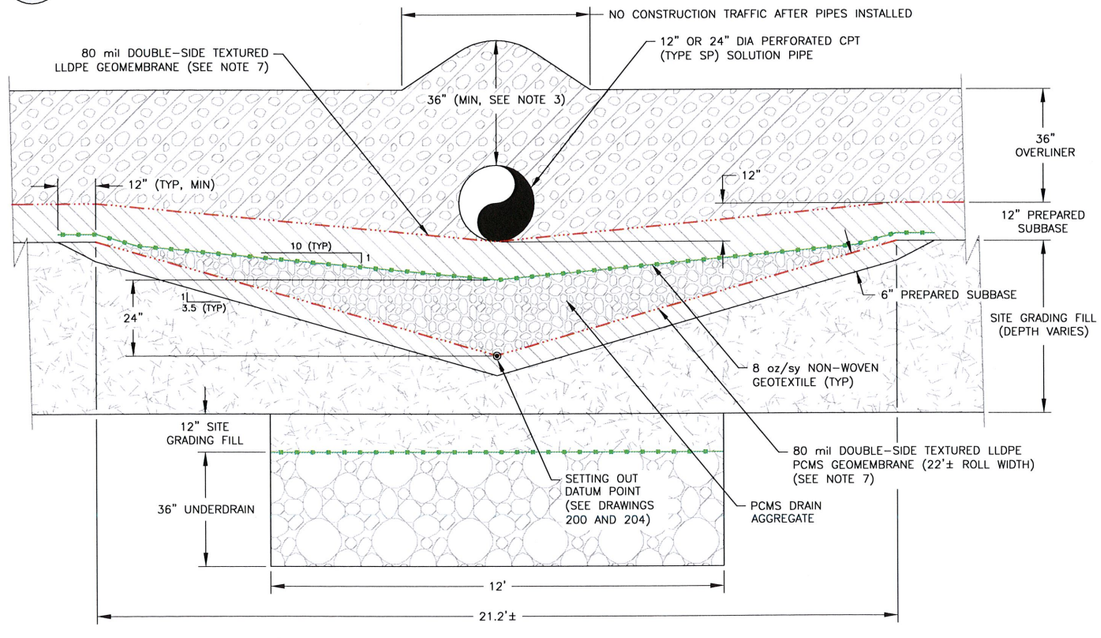
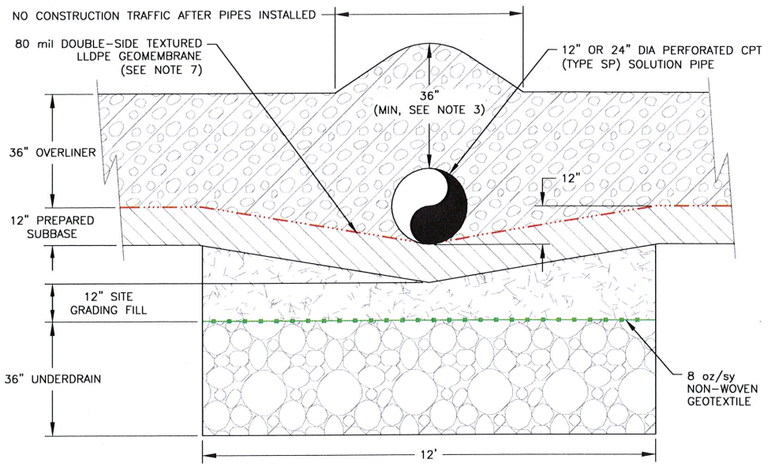


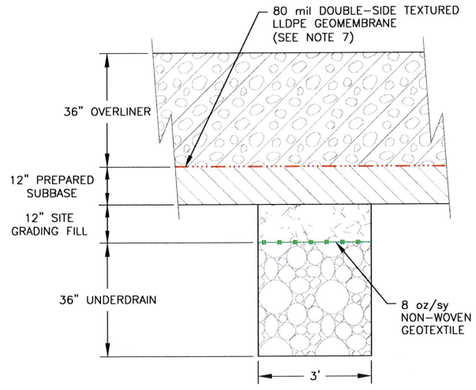

A/200 A/210 A/230 A/500 — TYPICAL PCMS CHANNEL AND OVERLYING SOLUTION HEADER WITH MAIN UNDERDRAIN TRENCH IN ZONES OF SITE GRADING EXCAVATION (SEE NOTE 4) ⚠1



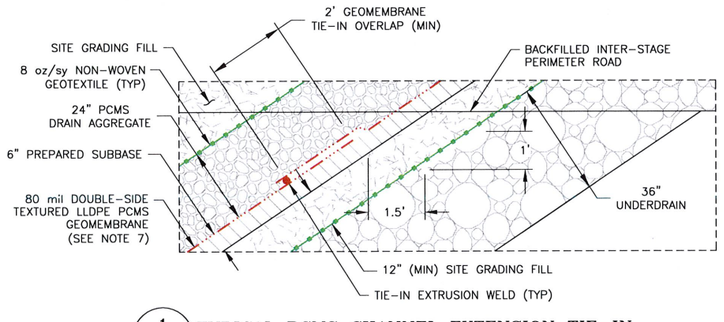

B/200 B/210 B/230 B/500 — TYPICAL PCMS CHANNEL AND OVERLYING SOLUTION HEADER WITH MAIN UNDERDRAIN TRENCH IN ZONES OF SITE GRADING FILL ⚠1




C/200 C/500 — TYPICAL UNDERDRAIN TRENCH OUTSIDE IN-HEAP STORAGE POND (SEE NOTE 4)




D/— TYPICAL UNDERDRAIN FINGER TRENCH (SEE NOTE 5)




1/230 — TYPICAL PCMS CHANNEL EXTENSION TIE-IN (SEE DETAIL 1 ON DRAWING 230 AND NOTE 6)

## REVISION:

⚠1 MODIFIED PCMS CHANNEL TO USE FULL GEOMEMBRANE ROLL WIDTH. ADDED NOTE 7 SPECIFYING GEOMEMBRANE. CHANGED TO SHEET 8 OF 8.

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
3. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER 12" DIA OR GREATER CPT SOLUTION PIPES. NO CONSTRUCTION TRAFFIC SHALL BE ALLOWED TO PASS OVER THESE AREAS AFTER THE HEADER PIPES ARE INSTALLED. CPT SOLUTION PIPES OF LESS THAN 12" DIA SHALL BE ENCAPSULATED WITHIN A 36"-THICK-LAYER OF OVERLINER.
4. WHERE VALLEY DRAINAGES ARE WELL DEFINED, THE MAIN UNDERDRAINS MAY NOT REQUIRE OVER-EXCAVATION TO FORM THE UNDERDRAIN TRENCH. MINIMUM MAIN UNDERDRAIN THICKNESS OF 36" AND CROSS-SECTIONAL AREA OF 36 SQUARE FEET SHALL BE MAINTAINED.
5. UNDERDRAIN TRENCH LOCATIONS SHALL BE ADJUSTED IN THE FIELD BY THE ENGINEER AND/OR FGMI DURING THE LEACH PAD GRADING OPERATIONS TO INTERCEPT GROUNDWATER SEEPS AND SPRINGS. IN SOME CASES THIS MAY INVOLVE ADDITIONAL UNDERDRAINS TO REACH IDENTIFIED SEEPS AND SPRINGS IN FUTURE STAGES OF THE LEACH PAD EXPANSION. ADDITIONAL UNDERDRAIN FINGER TRENCHES WILL BE CONSTRUCTED AS REQUIRED BASED ON SECTION D.
6. WHEN EXTENDING THE PCMS CHANNEL, THE GEOMEMBRANE WEARSHEET SHALL BE REMOVED AND THE PCMS GEOMEMBRANE LINER FOLDED BACK TO ALLOW PROPER CONTINUOUS TIE-IN OF THE PCMS DRAIN AGGREGATE.
7. ⚠1 ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT.

2 0 2 4 FEET
1" = 2' AT FULL SIZE (ANSI D)
1" = 4' AT HALF SIZE (ANSI B)
(TYPICAL SCALE FOR ALL SECTIONS)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
17/17

| REV | DATE | DESCRIPTION | APP'D CADD |
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| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: UNDERDRAIN SYSTEM AND PCMS SECTIONS AND DETAILS, SHEET 8 OF 8 ⚠1

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 235 | 1 |

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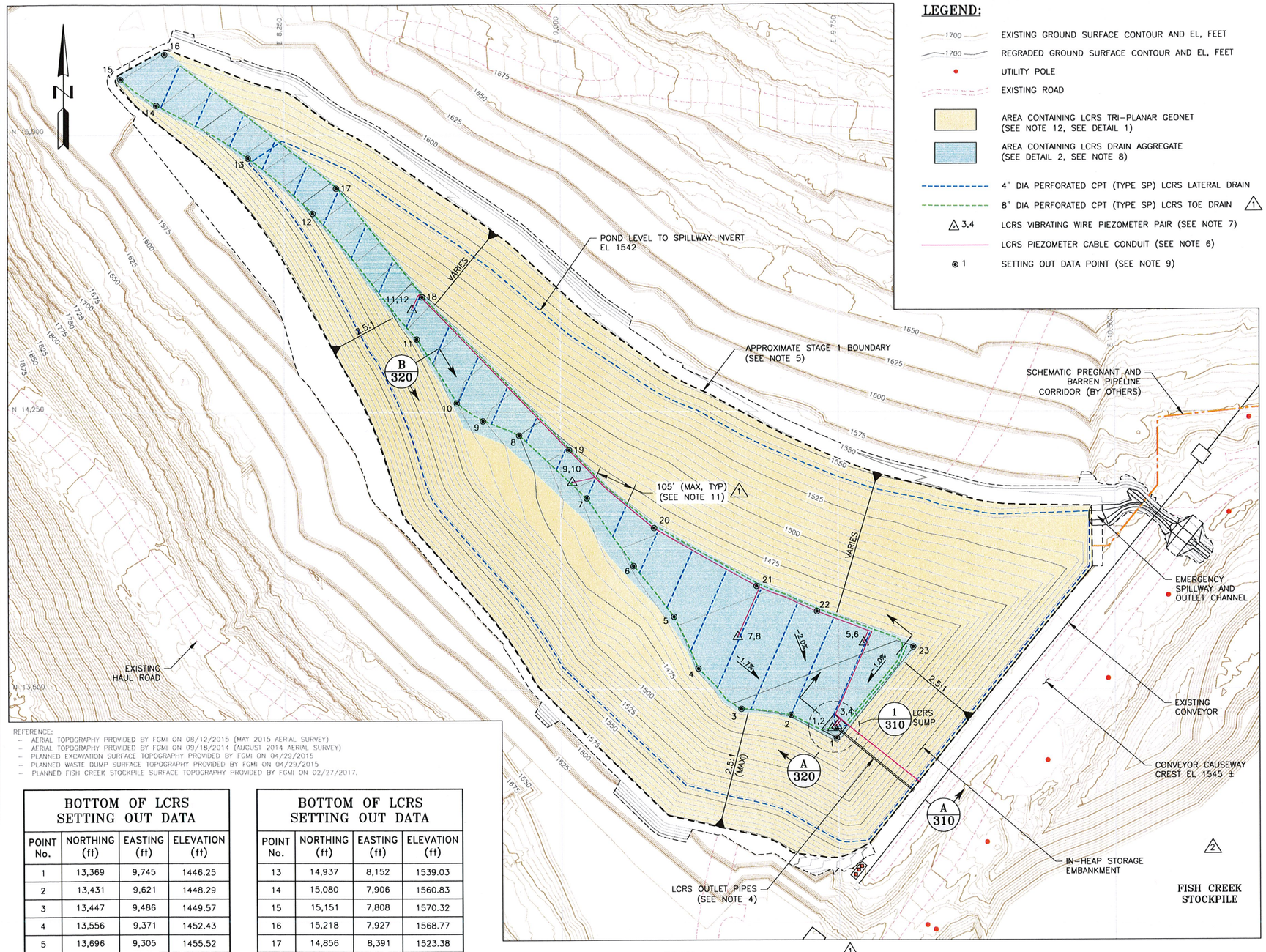



## LEGEND:

- 1700 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 1700 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- UTILITY POLE
- EXISTING ROAD
- AREA CONTAINING LCRS TRI-PLANAR GEONET (SEE NOTE 12, SEE DETAIL 1)
- AREA CONTAINING LCRS DRAIN AGGREGATE (SEE DETAIL 2, SEE NOTE 8)
- 4" DIA PERFORATED CPT (TYPE SP) LCRS LATERAL DRAIN
- 8" DIA PERFORATED CPT (TYPE SP) LCRS TOE DRAIN 1
- △ 3,4 LCRS VIBRATING WIRE PIEZOMETER PAIR (SEE NOTE 7)
- LCRS PIEZOMETER CABLE CONDUIT (SEE NOTE 6)
- ◉ 1 SETTING OUT DATA POINT (SEE NOTE 9)

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

### BOTTOM OF LCRS SETTING OUT DATA

| POINT No. | NORTHING (ft) | EASTING (ft) | ELEVATION (ft) |
|---|---|---|---|
| 1 | 13,369 | 9,745 | 1446.25 |
| 2 | 13,431 | 9,621 | 1448.29 |
| 3 | 13,447 | 9,486 | 1449.57 |
| 4 | 13,556 | 9,371 | 1452.43 |
| 5 | 13,696 | 9,305 | 1455.52 |
| 6 | 13,834 | 9,196 | 1458.87 |
| 7 | 14,017 | 9,070 | 1463.20 |
| 8 | 14,187 | 8,888 | 1475.07 |
| 9 | 14,226 | 8,790 | 1480.36 |
| 10 | 14,275 | 8,718 | 1485.08 |
| 11 | 14,449 | 8,610 | 1493.31 |
| 12 | 14,789 | 8,327 | 1522.20 |

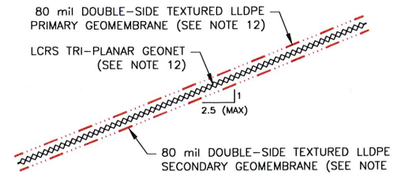
### BOTTOM OF LCRS SETTING OUT DATA

| POINT No. | NORTHING (ft) | EASTING (ft) | ELEVATION (ft) |
|---|---|---|---|
| 13 | 14,937 | 8,152 | 1539.03 |
| 14 | 15,080 | 7,906 | 1560.83 |
| 15 | 15,151 | 7,808 | 1570.32 |
| 16 | 15,218 | 7,927 | 1568.77 |
| 17 | 14,856 | 8,391 | 1523.38 |
| 18 | 14,563 | 8,624 | 1496.70 |
| 19 | 14,147 | 9,023 | 1469.92 |
| 20 | 13,936 | 9,252 | 1460.39 |
| 21 | 13,779 | 9,528 | 1455.49 |
| 22 | 13,711 | 9,691 | 1453.04 |
| 23 | 13,615 | 9,953 | 1449.37 |

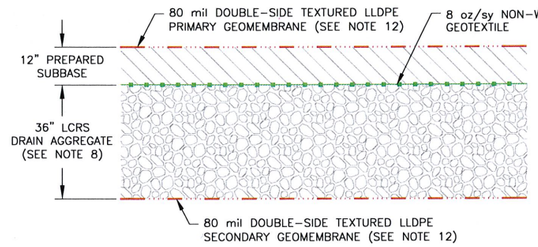
PLAN

150 0 150 300 FEET

1"=150' AT FULL SIZE (ANSI D)
1"=300' AT HALF SIZE (ANSI B)




1/– TYPICAL LCRS AND ADJACENT LINER ON SLOPES 10% OR STEEPER
NTS




2/– TYPICAL LCRS AND ADJACENT LINER ON SLOPES FLATTER THAN 10%
NTS

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B SHEET).
2. LCRS – THE LEACHATE COLLECTION AND RECOVERY SYSTEM (LCRS) PROVIDES DETECTION AND RECOVERY OF POTENTIAL LEAKAGE THROUGH THE PRIMARY GEOMEMBRANE WITHIN THE IN-HEAP STORAGE POND.
3. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURED SUPPLIED TEES, WYES AND COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
4. TWO 18" DIA STEEL LCRS OUTLET PIPES WILL SERVE AS CONDUITS FOR SUBMERSIBLE LCRS PUMPS. THE SUBMERSIBLE PUMPS FOR THE LCRS SHALL BE EQUIPPED WITH DISCHARGE HOSES AND ELECTRICS (BY OTHERS).
5. ABOVE THE STAGE 1 BOUNDARY, THE LEACH PAD WILL BE CONSTRUCTED WITH A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND SHALL BE CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
6. EACH LCRS PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LCRS DRAINS TO AVOID INTERFERENCE TO THE EXTENT POSSIBLE. THE LCRS PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO A CENTRAL READOUT BOX TO BE LOCATED ON THE IN-HEAP STORAGE EMBANKMENT CREST.
7. REFER TO DRAWING 720 FOR INSTRUMENTATION SECTIONS, DETAILS, AND SETTING OUT DATA.
8. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL, IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.
9. THE SETTING OUT DATA REPRESENT THE BOTTOM OF THE IN-HEAP POND SECONDARY GEOMEMBRANE LINER (i.e. THE TOP OF THE UNDERLYING PREPARED SUBBASE) AT THE TOE OF THE SIDE-SLOPE WITHIN THE LCRS.
10. ALL SETTING OUT DATA SHALL BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINNER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
11. THE LCRS LATERAL DRAIN SPACING SHOWN ASSUMES OVERLINER DRAIN AGGREGATE WILL BE USE FOR THE LCRS AGGREGATE MATERIAL ALONG THE POND FLOOR (SEE NOTE 8 AND DETAIL 2). IF LCRS DRAIN 1 AGGREGATE IS USED, THE LCRS LATERAL DRAIN SPACING SHALL BE REDUCED TO 38' PERPENDICULAR BETWEEN THE PIPES.
12. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED 2 EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.

## REVISION:

1 DISTINGUISHED BETWEEN LCRS LATERAL AND TOE DRAINS. CHANGED TOE DRAINS TO 8" DIAMETER. ADDED NOTE 11.

2 ADDED NOTE 12 SPECIFYING GEOMEMBRANE AND GEONET. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND LCRS PLAN

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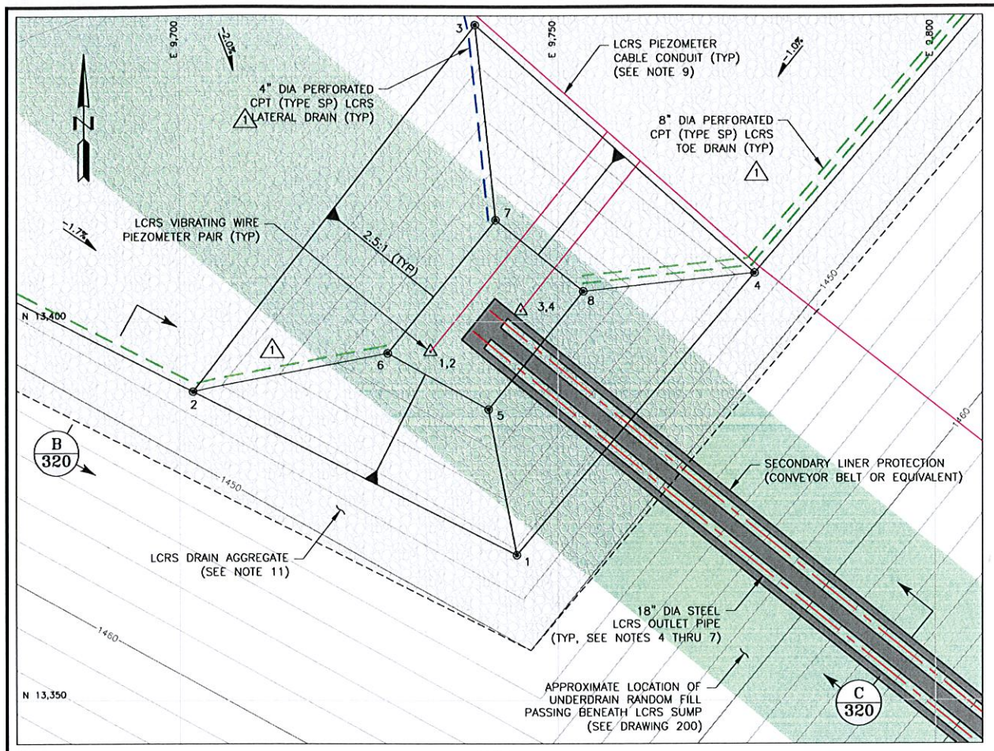



1 / 300 IN-HEAP STORAGE POND LCRS SUMP 2

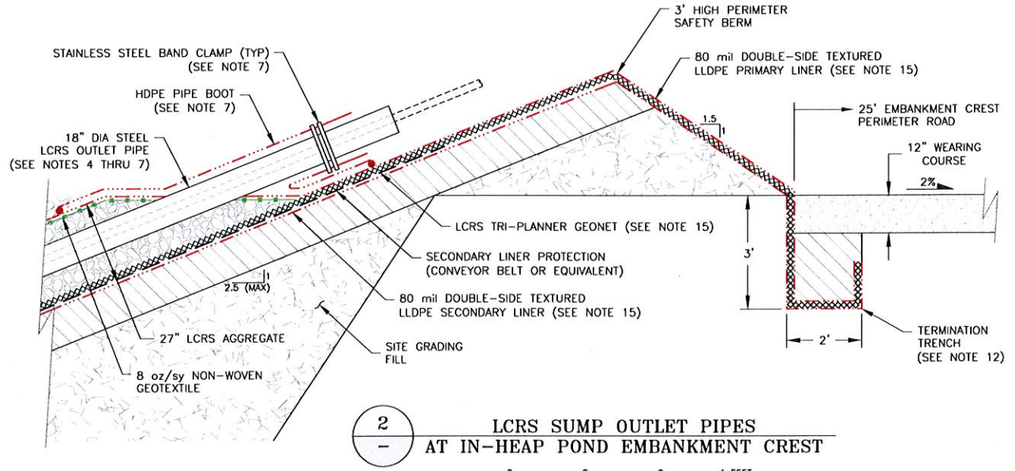
10 0 10 20 FEET
1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)

| LCRS SUMP SETTING OUT DATA 2 | | | |
|---|---|---|---|
| POINT No. | NORTHING (ft) | EASTING (ft) | ELEVATION (ft) |
| 1 | 13,369 | 9,745 | 1446.25 |
| 2 | 13,391 | 9,702 | 1446.96 |
| 3 | 13,439 | 9,739 | 1447.61 |
| 4 | 13,406 | 9,776 | 1446.72 |
| 5 | 13,388 | 9,741 | 1440.25 |
| 6 | 13,396 | 9,727 | 1440.25 |
| 7 | 13,413 | 9,742 | 1440.25 |
| 8 | 13,404 | 9,753 | 1440.25 |

**REVISION:**

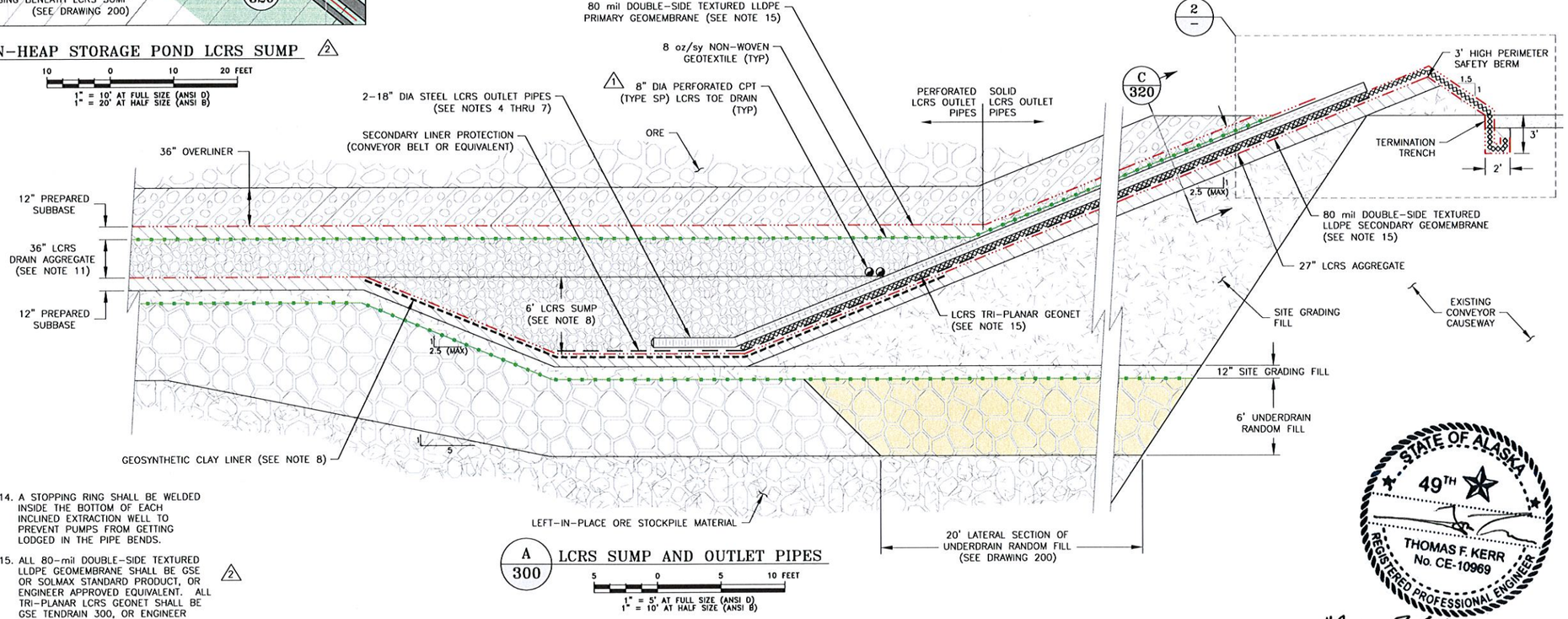
1 DISTINGUISHED BETWEEN LCRS LATERAL AND TOE DRAINS. CHANGED TOE DRAINS TO 8" DIAMETER.

2 ADDED NOTE 15 SPECIFYING GEOMEMBRANE AND GEONET. ADJUSTED LCRS SUMP AND SETTING OUT DATA.




2 / – LCRS SUMP OUTLET PIPES AT IN-HEAP POND EMBANKMENT CREST

2 0 2 4 FEET
1" = 2' AT FULL SIZE (ANSI D)
1" = 4' AT HALF SIZE (ANSI B)




A / 300 LCRS SUMP AND OUTLET PIPES

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

**NOTES:**

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. LCRS – THE LEACHATE COLLECTION AND RECOVERY SYSTEM (LCRS) PROVIDES DETECTION AND RECOVERY OF POTENTIAL LEAKAGE THROUGH THE PRIMARY GEOMEMBRANE WITHIN THE IN-HEAP STORAGE POND.
3. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED TEES, WYES AND COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL LCRS PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
4. TWO 18" DIA STEEL LCRS OUTLET PIPES (GRADE BASTM A53 STEEL PIPE 0.375" WALL THICKNESS) WILL SERVE AS CONDUITS FOR SUBMERSIBLE LCRS PUMPS. THE SUBMERSIBLE PUMPS FOR THE LCRS SHALL BE EQUIPPED WITH DISCHARGE HOSES AND ELECTRICS (BY OTHERS).
5. THREE ROWS OF SLOTS SHALL BE EQUALLY SPACED AROUND THE FIRST 15' OF THE LCRS OUTLET PIPES (WITHIN THE LCRS SUMP). SLOTS SHALL BE 3" x 1/4" ON 2" CENTERS AND THE ENDS SHALL BE FITTED WITH SLOTTED END CAPS.
6. THE LCRS OUTLET PIPES SHALL BE BOOTED THROUGH THE LEACH PAD PRIMARY LINER NEAR THE EMBANKMENT CREST TO ALLOW ACCESS FOR PUMPING AND SAMPLING. THE LCRS OUTLET PIPES SHALL BE CUT SUCH THAT THE LOW POINT OF THE PIPE OPENING DRAINS BACK INTO THE LINED AREA OF THE LEACH PAD.
7. HDPE PIPE BOOTS TO STEEL PIPES SHALL BE SECURED WITH STAINLESS STEEL BAND CLAMPS AND CAULKED WITH SEALANT OR INCLUDE NEOPRENE SEALANT TAPE BETWEEN THE PIPE AND GEOMEMBRANE WHERE THE CLAMPS ARE SITUATED.
8. THE ENTIRE AREA OF THE 6' DEEP LCRS SUMP SHALL BE UNDERLAIN BY GEOSYNTHETIC CLAY LINER (GCL).
9. EACH LCRS PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LCRS DRAINS TO AVOID INTERFERENCE TO THE EXTENT POSSIBLE. THE LCRS PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO A CENTRAL READOUT BOX TO BE LOCATED ON THE IN-HEAP STORAGE EMBANKMENT CREST.
10. REFER TO DRAWING 720 FOR INSTRUMENTATION SECTIONS, DETAILS, AND SETTING OUT DATA.
11. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL, IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.
12. THE STAGE 1 TERMINATION TRENCH SHALL BE BACKFILLED WITH COMPACTED PREPARED SUBBASE TO MITIGATE EXTERNAL WATERS FROM ENTERING THE LCRS.
13. THE SETTING OUT DATA REPRESENT THE BOTTOM OF THE IN-HEAP POND SECONDARY GEOMEMBRANE LINER (i.e. THE TOP OF THE UNDERLYING PREPARED SUBBASE) AT THE CREST AND TOE CORNERS OF THE LCRS SUMP.
14. A STOPPING RING SHALL BE WELDED INSIDE THE BOTTOM OF EACH INCLINED EXTRACTION WELL TO PREVENT PUMPS FROM GETTING LODGED IN THE PIPE BENDS.
15. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT. 2

| REV | DATE | DESCRIPTION | APP'D CADD |
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| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND LCRS SECTIONS AND DETAILS, SHEET 1 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 310 | 2 |

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 24/17

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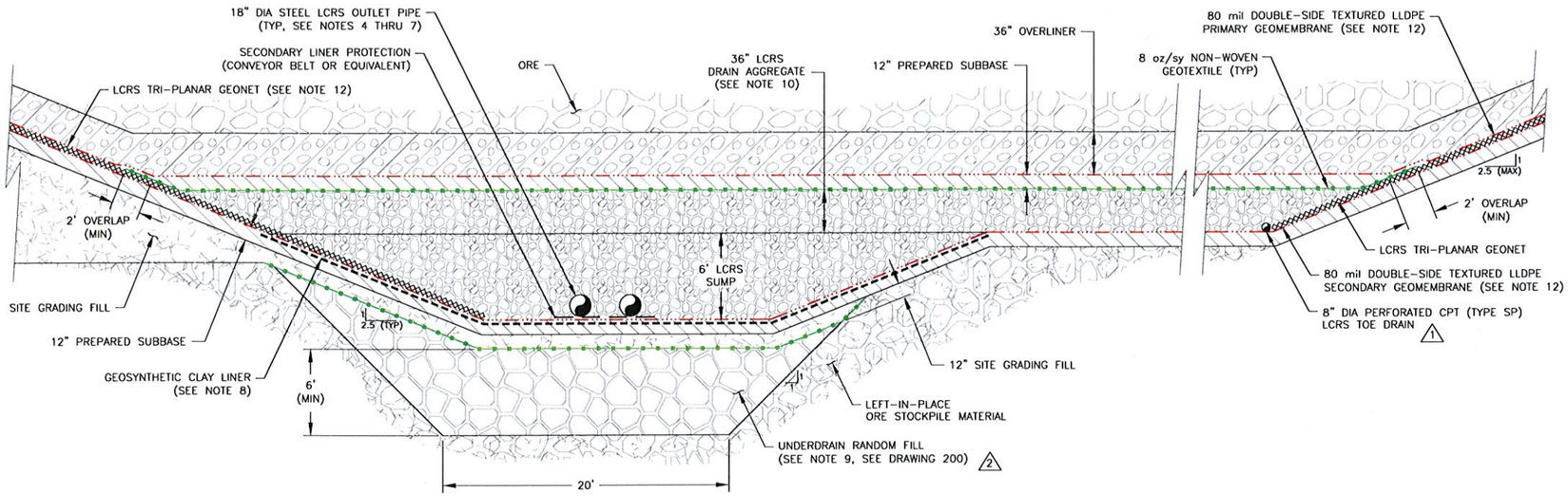


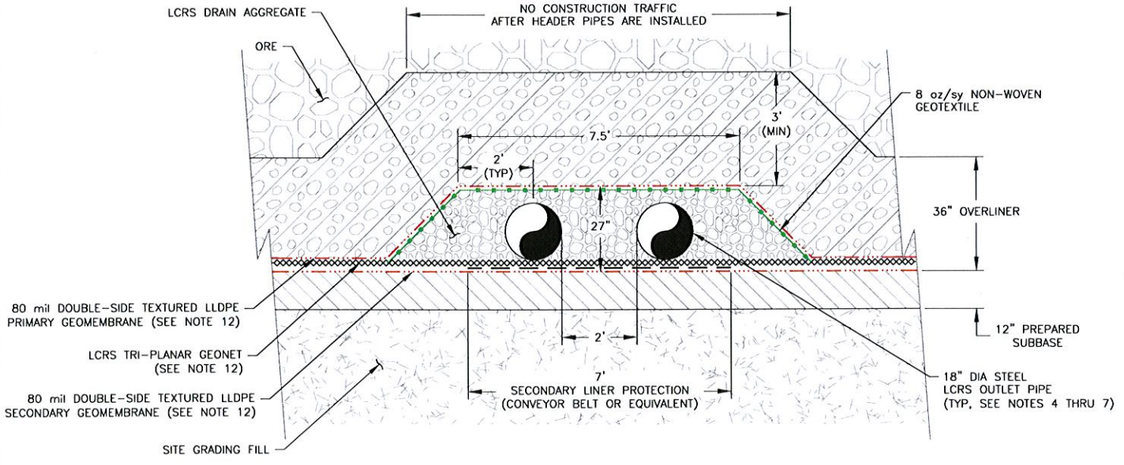

A/300 TYPICAL LCRS SUMP

5 0 5 10 FEET

1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

## NOTES:

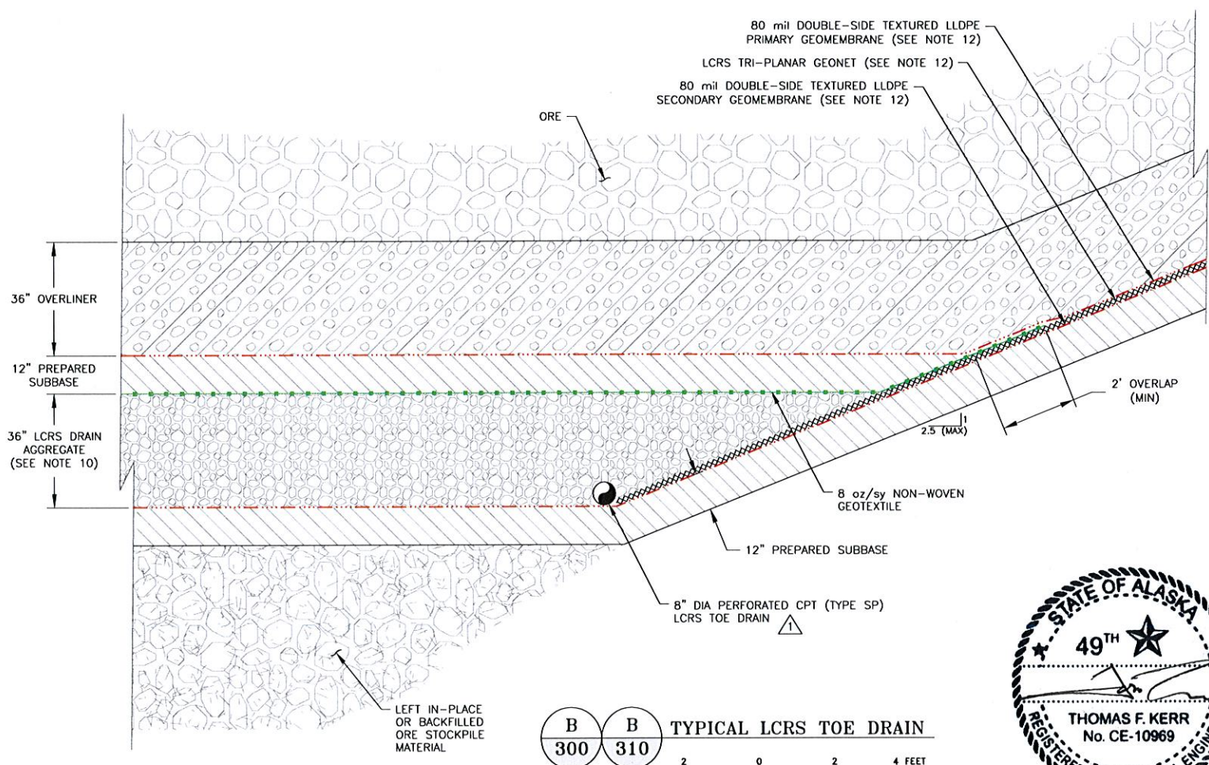
1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. LCRS - THE LEACHATE COLLECTION AND RECOVERY SYSTEM (LCRS) PROVIDES DETECTION AND RECOVERY OF POTENTIAL LEAKAGE THROUGH THE PRIMARY GEOMEMBRANE WITHIN THE IN-HEAP STORAGE POND AREA.
3. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED TEES, WYES AND COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL LCRS PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
4. TWO 18" DIA STEEL LCRS OUTLET PIPES (GRADE BASTM A53 STEEL PIPE 0.375" WALL THICKNESS) WILL SERVE AS CONDUITS FOR SUBMERSIBLE LCRS PUMPS. THE SUBMERSIBLE PUMPS FOR THE LCRS SHALL BE EQUIPPED WITH DISCHARGE HOSES AND ELECTRICS (BY OTHERS).
5. THREE ROWS OF SLOTS SHALL BE EQUALLY SPACED AROUND THE FIRST 15' OF THE LCRS OUTLET PIPES (WITHIN THE LCRS SUMP). SLOTS SHALL BE 3" X 1/4" ON 2" CENTERS AND THE ENDS SHALL BE FITTED WITH SLOTTED END CAPS.
6. THE LCRS OUTLET PIPES SHALL BE BOOTED THROUGH THE LEACH PAD PRIMARY LINER NEAR THE EMBANKMENT CREST TO ALLOW ACCESS FOR PUMPING AND SAMPLING. THE LCRS OUTLET PIPES SHALL BE CUT SUCH THAT THE LOW POINT OF THE PIPE OPENING DRAINS BACK INTO THE LINED AREA OF THE LEACH PAD.
7. HDPE PIPE BOOTS TO STEEL PIPES SHALL BE SECURED WITH STAINLESS STEEL BAND CLAMPS AND CAULKED WITH SEALANT OR INCLUDE NEOPRENE SEALANT TAPE BETWEEN THE PIPE AND GEOMEMBRANE WHERE THE CLAMPS ARE SITUATED.
8. THE ENTIRE AREA OF THE 6' DEEP LCRS SUMP SHALL BE UNDERLAIN BY GEOSYNTHETIC CLAY LINER (GCL).
9. THE UNDERDRAIN RANDOM FILL IS PRIMARILY ENVISIONED AS A LARGE TRENCH DRAIN EXCAVATED DOWN THE CENTER OF THE BARNES CREEK VALLEY. THE UNDERDRAIN RANDOM FILL WILL DROP BELOW THE IN-HEAP STORAGE POND LCRS SUMP AND TERMINATE AT THE UPSTREAM TOE OF THE EXISTING CONVEYOR CAUSEWAY EMBANKMENT. A SECOND UNDERDRAIN RANDOM FILL TRENCH, 20' WIDE BY 6' DEEP, WILL BE EXCAVATED LATERALLY ACROSS THE VALLEY ALONG THE UPSTREAM TOE OF THE CONVEYOR CAUSEWAY.
10. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL, IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.
11. A STOPPING RING SHALL BE WELDED INSIDE THE BOTTOM OF EACH INCLINED LCRS PUMP CAN TO PREVENT PUMPS FROM GETTING LODGED IN THE PIPE BENDS.
12. 2 ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.




C/310 TYPICAL INSTALLATION OF LCRS OUTLET PIPES ALONG UPSTREAM SLOPE OF IN-HEAP STORAGE EMBANKMENT

2 0 2 4 FEET

1" = 2' AT FULL SIZE (ANSI D)
1" = 4' AT HALF SIZE (ANSI B)




B/300 B/310 TYPICAL LCRS TOE DRAIN

2 0 2 4 FEET

1" = 2' AT FULL SIZE (ANSI D)
1" = 4' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH
THOMAS F. KERR No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 24, 17

## REVISION:

1 ADDED TOE DRAIN TO SECTION A. CHANGED TOE DRAINS TO 8" DIA.

2 ADDED NOTE 12 SPECIFYING GEOMEMBRANE AND GEONET. ADJUSTED UNDERDRAIN RANDOM FILL TRENCH BENEATH LCRS SUMP.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| 2 | 03/24/17 | REISSUED FOR CONSTRUCTION | JTR CB |

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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND LCRS SECTIONS AND DETAILS, SHEET 2 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 320 | 2 |

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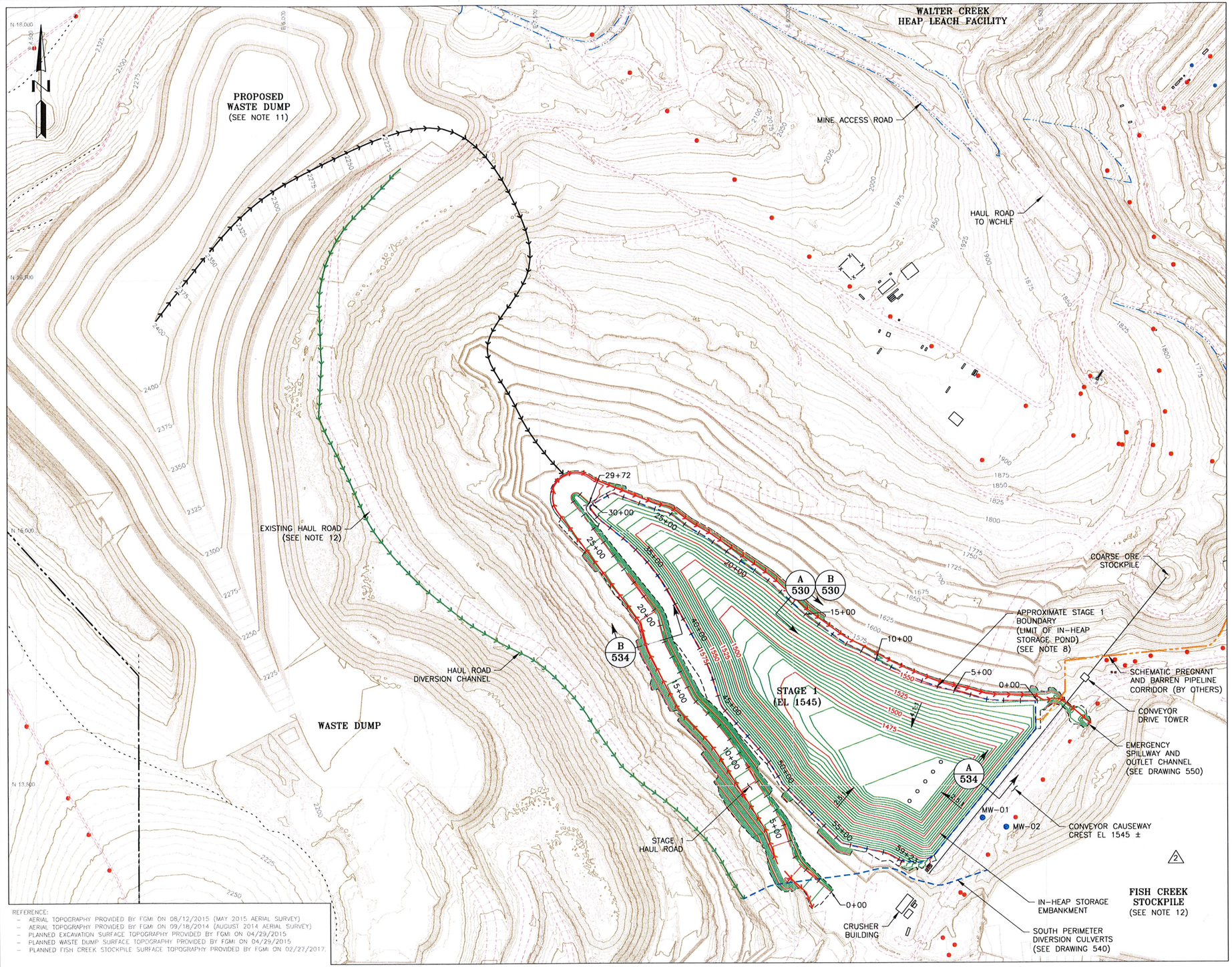
DRAWING PATH: C:\1\01\00089.99\CAD\CAD Dept\Drawings\DV101-00089-99_310-320.dwg
PRINTED BY: JASON REIVA PRINT TIME: 3/23/2017 1:42 PM




## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- PROPOSED INTER-STAGE DIVERSION
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING STRUCTURES
- VERTICAL SOLUTION EXTRACTION WELL (SEE DRAWING 512)
- MW-01 UNDERDRAIN MONITORING WELL (SEE DRAWING 200) 1

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, AND BOTTOM OF EROSION PROTECTION WITHIN ALL CHANNELS.
3. ALL UNSUITABLE MATERIALS ENCOUNTERED DURING CONSTRUCTION MUST BE REMOVED AND EITHER REPLACED WITH COMPACTED RANDOM FILL, SITE GRADING FILL, OR HAVE THE EXCAVATED SURFACE PREPARED AS THE ROUGH GRADE SURFACE.
4. SLOPES OF THE REGRADED PAD GROUND SURFACE CONTOURS SHALL NOT EXCEED 2.5H:1V.
5. ALL USABLE MATERIALS (PREPARED SUBGRADE MATERIAL, RIPRAP, ETC.) ENCOUNTERED BUT NOT USED DURING CONSTRUCTION SHALL BE REMOVED AND STOCKPILED FOR LATER USE AS REQUIRED BY FGMI.
6. THE EXISTING POWER LINE SHALL BE RELOCATED AS REQUIRED BY FGMI.
7. ALL EXISTING CULVERTS WITHIN THE LEACH PAD BASIN AREA SHALL BE REMOVED PRIOR TO THE START OF LEACH PAD REGRADING.
8. ABOVE THE STAGE 1 BOUNDARY, THE LEACH PAD WILL BE CONSTRUCTED WITH A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND SHALL BE CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM WITH LCRS.
9. SEE DRAWING 430 FOR THE STAGE 1 PERIMETER ROAD PROFILE AND HORIZONTAL CONTROL DATA.
10. SEE DRAWING 435 FOR THE STAGE 1 HAUL ROAD PROFILE AND HORIZONTAL CONTROL DATA.
11. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
12. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.

**PLAN**

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

## REVISION:

1 ADDED UNDERDRAIN MONITORING WELL LOCATIONS IN PLAN AND CALLOUT TO LEGEND.

2 REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

| REV | DATE | DESCRIPTION | APP'D / CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |

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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 1 GRADING PLAN

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 400 | 2 |

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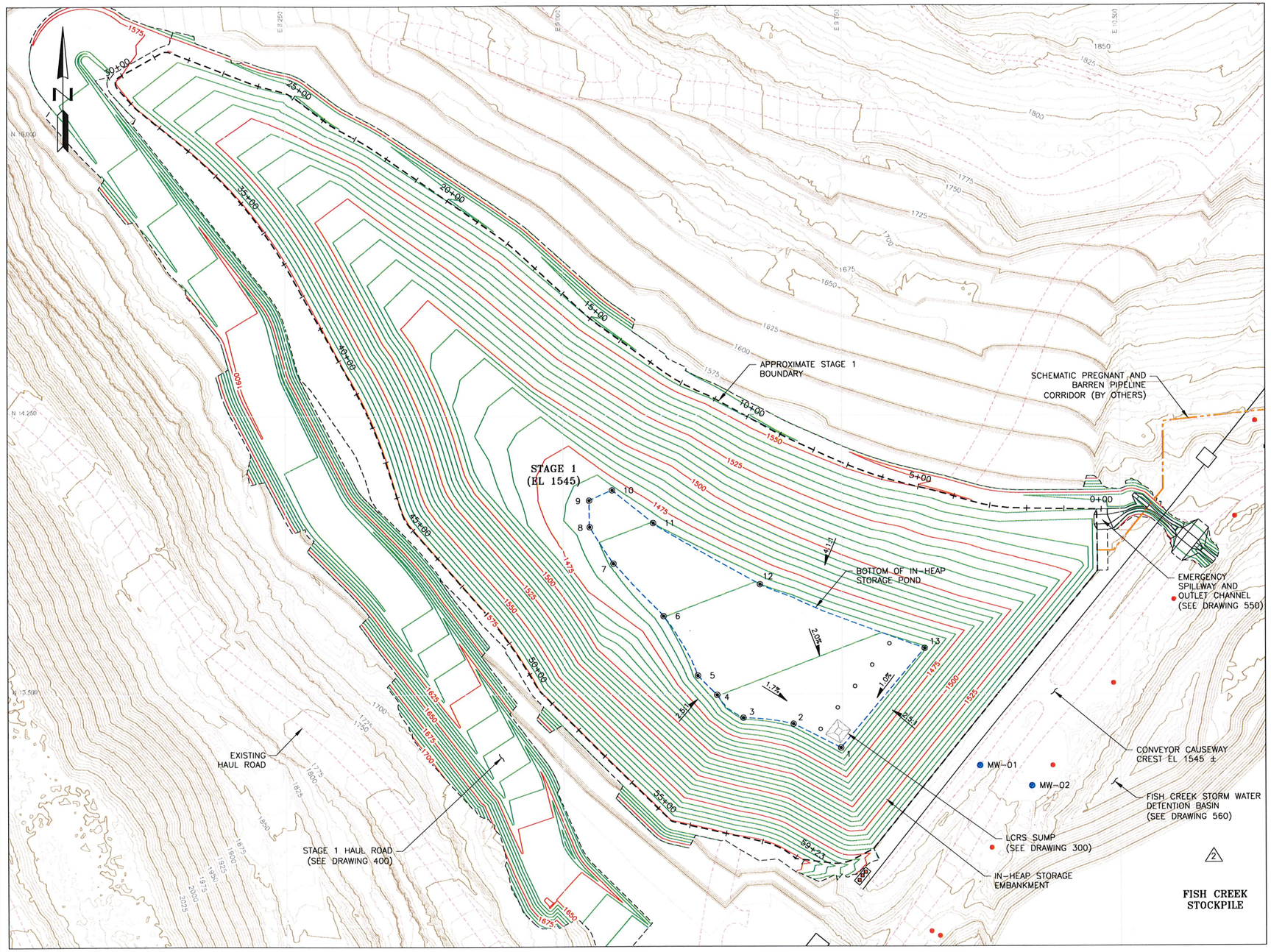


REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

## PLAN

150 0 150 300 FEET

1"=150' AT FULL SIZE (ANSI D)
1"=300' AT HALF SIZE (ANSI B)

## LEGEND:

- 2100 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 2100 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- UTILITY POLE
- EXISTING ROAD
- VERTICAL SOLUTION EXTRACTION WELL (SEE DRAWING 512)
- MW-01 UNDERDRAIN MONITORING WELL (SEE DRAWING 200) △1
- 1 SETTING OUT DATUM POINT (SEE NOTE 8)

### IN-HEAP STORAGE POND SETTING OUT DATA

| POINT No. | NORTHING (ft) | EASTING (ft) | ELEVATION (ft) |
|---|---|---|---|
| 1 | 13,357 | 9,746 | 1450.00 |
| 2 | 13,421 | 9,618 | 1452.13 |
| 3 | 13,437 | 9,483 | 1453.40 |
| 4 | 13,499 | 9,412 | 1455.06 |
| 5 | 13,551 | 9,361 | 1456.40 |
| 6 | 13,710 | 9,269 | 1460.05 |
| 7 | 13,852 | 9,135 | 1463.66 |
| 8 | 13,950 | 9,071 | 1465.95 |
| 9 | 14,022 | 9,070 | 1467.30 |
| 10 | 14,049 | 9,132 | 1467.37 |
| 11 | 13,961 | 9,242 | 1464.93 |
| 12 | 13,796 | 9,528 | 1459.80 |
| 13 | 13,624 | 9,974 | 1453.41 |

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, AND BOTTOM OF EROSION PROTECTION WITHIN ALL CHANNELS.
3. ALL UNSUITABLE MATERIALS ENCOUNTERED DURING CONSTRUCTION MUST BE REMOVED AND EITHER REPLACED WITH COMPACTED RANDOM FILL, SITE GRADING FILL, OR HAVE THE EXCAVATED SURFACE PREPARED AS THE ROUGH GRADE SURFACE.
4. SLOPES OF THE REGRADED PAD GROUND SURFACE CONTOURS SHALL NOT EXCEED 2.5H:1V.
5. ALL USABLE MATERIALS (PREPARED SUBGRADE MATERIAL, RIPRAP, ETC.) ENCOUNTERED BUT NOT USED DURING CONSTRUCTION SHALL BE REMOVED AND STOCKPILED FOR LATER USE AS REQUIRED BY FGMI.
6. SEE DRAWING 430 FOR THE STAGE 1 PERIMETER ROAD PROFILE AND HORIZONTAL CONTROL DATA.
7. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
8. THE SETTING OUT DATA REPRESENT THE BOTTOM OF THE IN-HEAP POND PRIMARY GEOMEMBRANE LINER (i.e. THE TOP OF THE UNDERLYING PREPARED SUBBASE) AT THE TOE OF THE IN-HEAP POND SIDE-SLOPE.
9. ALL SETTING OUT DATA SHALL BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINNER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

## REVISION:

△1 ADDED UNDERDRAIN MONITORING WELL LOCATIONS IN PLAN AND CALLOUT TO LEGEND.

△2 MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |


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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND GRADING PLAN

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 401 | 2 |

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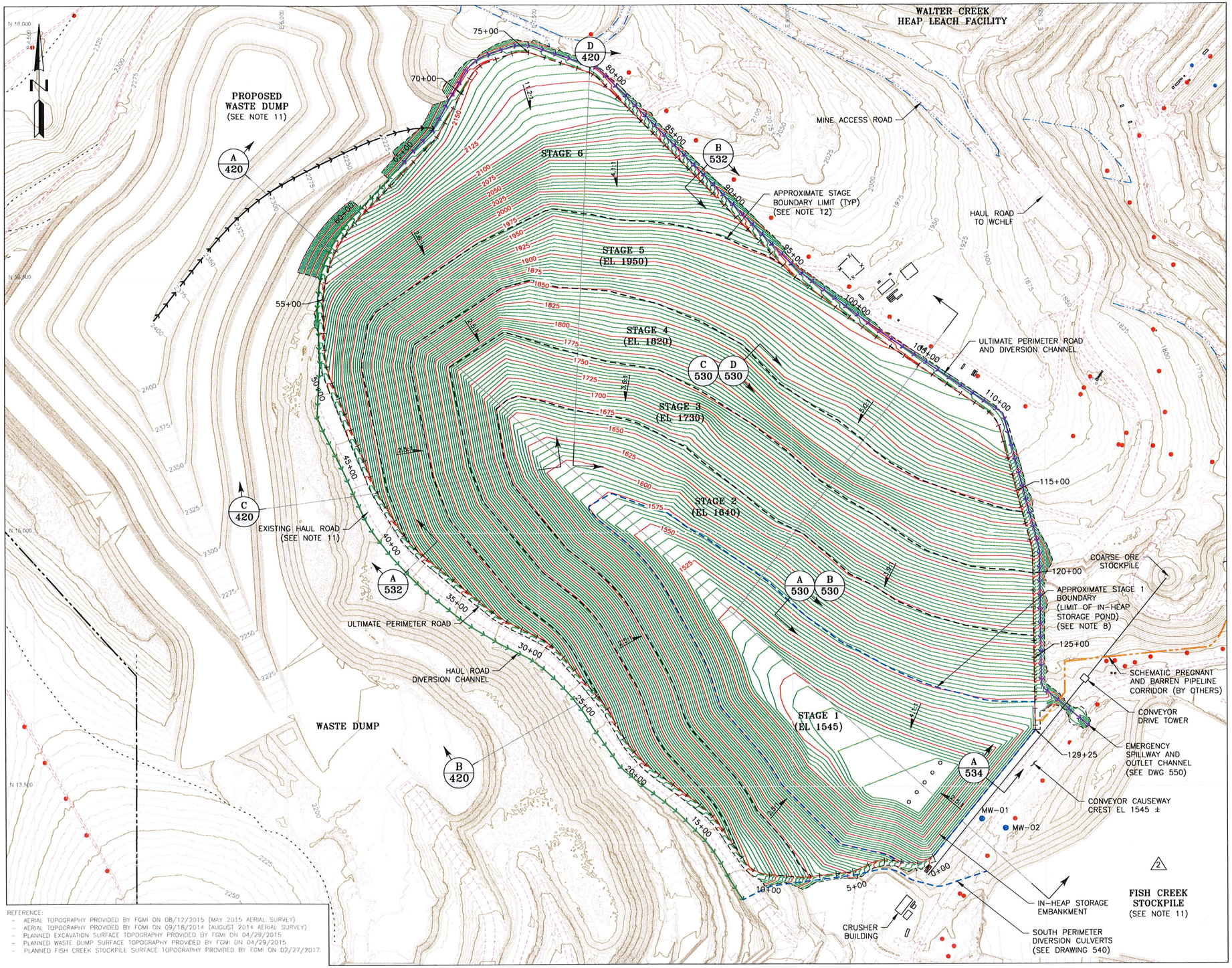


REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED NORTH PERIMETER DIVERSION AND OUTLET CHANNEL
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING STRUCTURES
- VERTICAL SOLUTION EXTRACTION WELL (SEE DRAWING 512)
- MW-01 UNDERDRAIN MONITORING WELL (SEE DRAWING 200) △1

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, AND BOTTOM OF EROSION PROTECTION WITHIN ALL CHANNELS.
3. ALL UNSUITABLE MATERIALS ENCOUNTERED DURING CONSTRUCTION MUST BE REMOVED AND EITHER REPLACED WITH COMPACTED RANDOM FILL, SITE GRADING FILL, OR HAVE THE EXCAVATED SURFACE PREPARED AS THE ROUGH GRADE SURFACE.
4. SLOPES OF THE REGRADED PAD GROUND SURFACE CONTOURS SHALL NOT EXCEED 2.5H:1V.
5. ALL USABLE MATERIALS (PREPARED SUBGRADE MATERIAL, RIPRAP, ETC.) ENCOUNTERED BUT NOT USED DURING CONSTRUCTION SHALL BE REMOVED AND STOCKPILED FOR LATER USE AS REQUIRED BY FGMI.
6. THE EXISTING POWER LINE SHALL BE RELOCATED AS REQUIRED BY FGMI.
7. ALL EXISTING CULVERTS WITHIN THE LEACH PAD BASIN AREA SHALL BE REMOVED PRIOR TO THE START OF LEACH PAD REGRADING.
8. ABOVE THE STAGE 1 BOUNDARY, THE LEACH PAD WILL BE CONSTRUCTED WITH A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND SHALL BE CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM WITH LCRS.
9. SEE DRAWINGS 480 AND 485 FOR THE ULTIMATE PERIMETER ROAD PROFILE AND HORIZONTAL CONTROL DATA, RESPECTIVELY.
10. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
11. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
12. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

## REVISION:

△1 ADDED UNDERDRAIN MONITORING WELL LOCATIONS IN PLAN AND CALLOUT TO LEGEND.

△2 REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

| REV | DATE | DESCRIPTION | APP'D CADD |
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| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
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ACTIVITY CODE N/A XREF N/A

CLIENT
FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT
FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE
STAGE GRADING PLAN THRU STAGE 6

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 410 | 2 |

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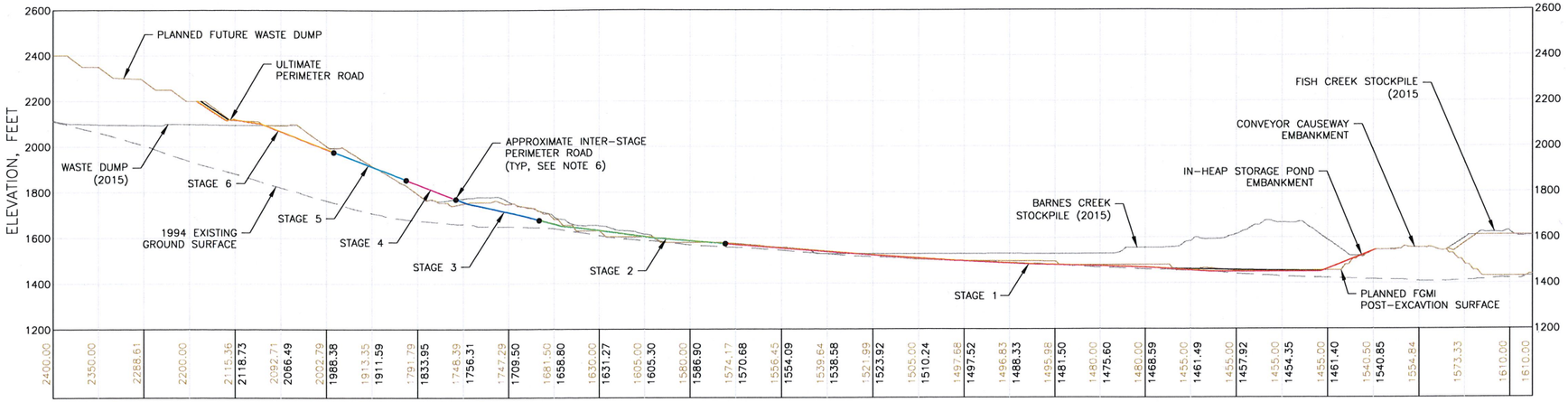
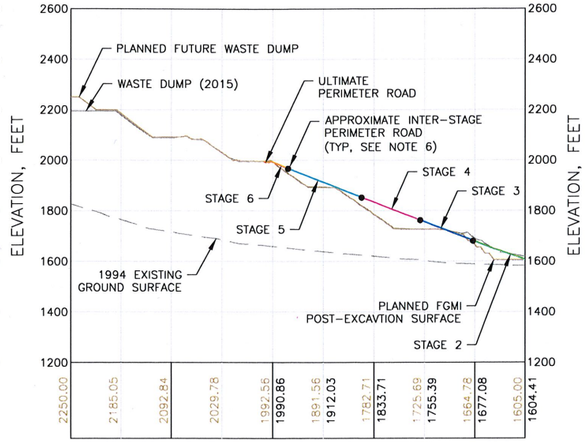
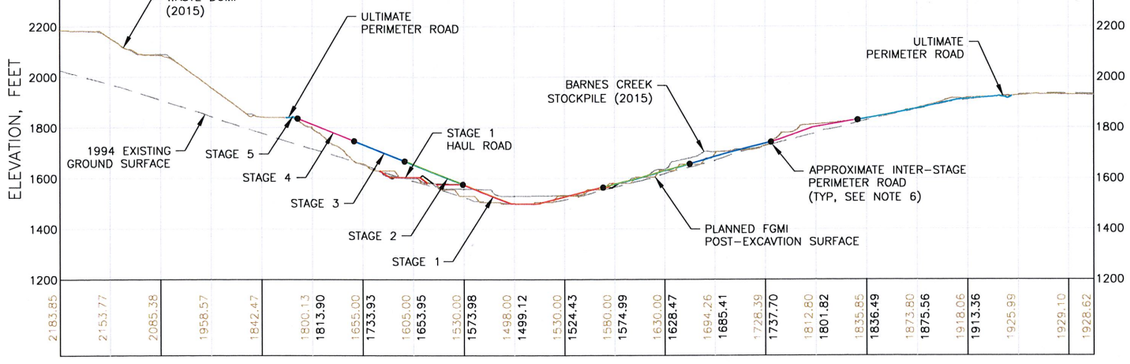


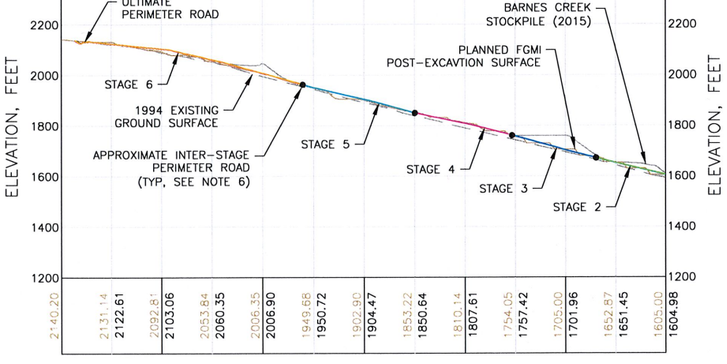

A 410 BARNES CREEK HEAP LEACH FACILITY LONGITUDINAL SECTION
FROM TOE OF IN-HEAP STORAGE POND THROUGH FACILITY




C 410 BARNES CREEK HEAP LEACH FACILITY
NORTHEASTERN BASIN SIDE-SLOPE




B 410 BARNES CREEK HEAP LEACH FACILITY LATERAL SECTION
THROUGH FULL FACILITY




D 410 BARNES CREEK HEAP LEACH FACILITY
NORTHWESTERN BASIN SIDE-SLOPE

**LEGEND:**

- 1994 EXISTING GROUND SURFACE
- 2015 EXISTING GROUND SURFACE
- PLANNED FGMI POST-EXCAVATION SURFACE
- STAGE 1 REGRADE SURFACE
- STAGE 2 REGRADE SURFACE
- STAGE 3 REGRADE SURFACE
- STAGE 4 REGRADE SURFACE
- STAGE 5 REGRADE SURFACE
- STAGE 6 REGRADE SURFACE
- 1455.00 — PLANNED FGMI POST-EXCAVATION GROUND SURFACE ELEVATION, FEET
- 1450.00 — REGRADE GROUND SURFACE ELEVATION, FEET

**NOTES:**

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL PERMAFROST ENCOUNTERED IN THE BARNES CREEK VALLEY MUST BE REMOVED AND REPLACED WITH SUITABLE GRADING FILL DURING SITE GRADING, AS DIRECTED BY FGMI OR THE ENGINEER.
3. ALL UNSUITABLE MATERIALS ENCOUNTERED DURING CONSTRUCTION MUST BE REMOVED AND EITHER REPLACED WITH COMPACTED RANDOM FILL, SITE GRADING FILL, OR HAVE THE EXCAVATED SURFACE PREPARED AS THE ROUGH GRADE SURFACE, AS DIRECTED BY FGMI OR THE ENGINEER.
4. SLOPES OF THE REGRADED PAD GROUND SURFACE CONTOURS SHALL NOT EXCEED 2.5H:1V.
5. ALL USABLE MATERIALS (PREPARED SUBGRADE MATERIAL, RIPRAP, ETC.) ENCOUNTERED BUT NOT USED DURING CONSTRUCTION SHALL BE REMOVED AND STOCKPILED FOR LATER USE AS REQUIRED BY FGMI. FGMI MUST BE NOTIFIED FOR SURVEY AND APPROVAL PRIOR TO REMOVAL AND STOCKPILING OF MATERIALS.
6. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17 /17

300 0 300 600 FEET
1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)
(TYPICAL SCALE FOR ALL SECTIONS)

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: TYPICAL GRADING SECTIONS

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 420 | 0 |

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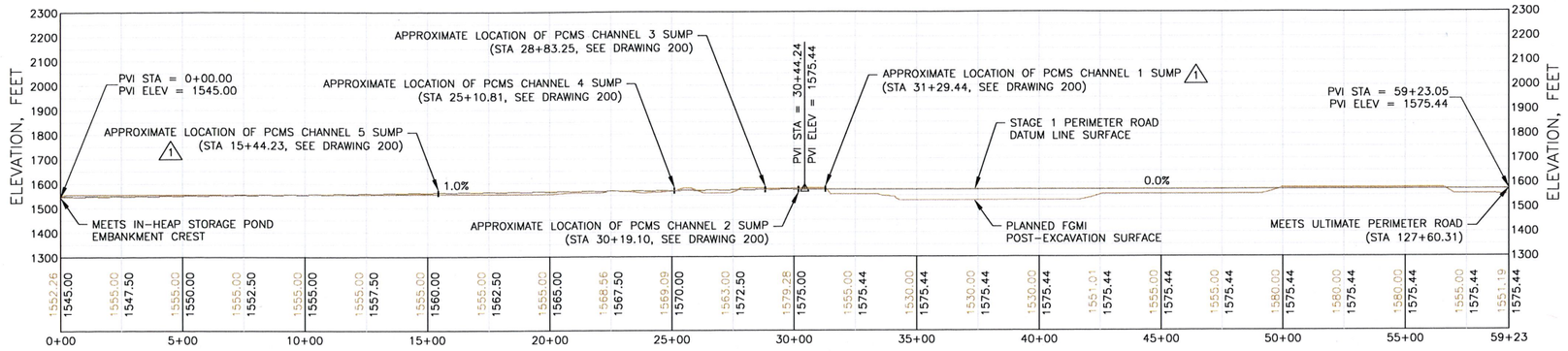


## STAGE 1 PERIMETER ROAD PROFILE

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

**LEGEND:**

| | |
|---|---|
| 2119.57 | PLANNED FGMI POST-EXCAVATION GROUND SURFACE ELEVATION, FEET |
| 2117.76 | STAGE 1 PERIMETER ROAD DATUM LINE ELEVATION, FEET |
| PC | POINT OF CURVATURE |
| PI | POINT OF INTERSECTION |
| RP | RADIAL POINT (CENTER OF CURVATURE) |
| PT | POINT OF TANGENCY |
| PVI | POINT OF VERTICAL INTERSECTION |

**NOTES:**

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL SETTING OUT DATA IS TO BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
3. SURVEY CONTROL POINTS SHALL BE PROVIDED BY FGMI.
4. THE STAGE 1 PERIMETER ROAD ALONG THE SOUTH SIDE OF THE VALLEY, FROM STATION 30+44.24 TO THE ALIGNMENT END, SHALL BE WITH A FLAT LONGITUDINAL GRADE TO MITIGATE STORMWATER RUNOFF IN THE AREA OF THE CONVEYOR TRANSFORMERS. THE PERIMETER ROAD IN THIS AREA SHALL BE GRADED WITH A MINIMUM 2 PERCENT CROSSFALL, TOWARD THE IN-HEAP POND, TO PROMOTE DRAINAGE.
5. THE PCMS CHANNELS SHALL TERMINATE AT SUMPS TO BE LOCATED BENEATH THE STAGE 1 PERIMETER ROAD. ANY FLOWS WITHIN THE PCMS CHANNELS WILL BE TRAINED INTO THE PCMS HEADER PIPES AND CONVEYED TO THE DISCHARGE POINTS WITHIN THE NORTHEAST CORNER OF THE IN-HEAP POND.

**REVISION:**

1 ADDED TWO PCMS SUMPS AND RENUMBERED OTHERS

### STAGE 1 PERIMETER ROAD HORIZONTAL CONTROL DATA

| DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|
| | 0+00.0 | 13,995.34 | 10,449.68 | | | |
| | 2+20.3 | 13,998.85 | 10,229.38 | | | |
| PC | 2+20.3 | 13,998.85 | 10,229.38 | | | |
| PI | 2+88.7 | 13,999.94 | 10,160.99 | | | |
| RP | | 14,498.79 | 10,237.35 | 500.0 | 15°35' | 11°29' |
| PT | 3+56.3 | 14,019.36 | 10,095.41 | | | |
| | 3+56.3 | 14,019.36 | 10,095.41 | | | |
| | 4+28.1 | 14,039.75 | 10,026.52 | | | |
| PC | 4+28.1 | 14,039.75 | 10,026.52 | | | |
| PI | 4+40.7 | 14,043.32 | 10,014.49 | | | |
| RP | | 13,560.33 | 9,884.57 | 500.0 | 2°52' | 11°29' |
| PT | 4+53.2 | 14,046.27 | 10,002.30 | | | |
| | 4+53.2 | 14,046.27 | 10,002.30 | | | |
| | 5+28.2 | 14,063.92 | 9,929.45 | | | |
| PC | 5+28.2 | 14,063.92 | 9,929.45 | | | |
| PI | 6+43.4 | 14,091.04 | 9,817.49 | | | |
| RP | | 15,230.18 | 10,211.99 | 1200.0 | 10°58' | 4°47' |
| PT | 7+57.9 | 14,138.97 | 9,712.74 | | | |
| | 7+57.9 | 14,138.97 | 9,712.74 | | | |
| | 8+85.7 | 14,192.14 | 9,596.53 | | | |
| PC | 8+85.7 | 14,192.14 | 9,596.53 | | | |
| PI | 9+04.6 | 14,200.03 | 9,579.27 | | | |
| RP | | 14,646.81 | 9,804.55 | 500.0 | 4°21' | 11°29' |
| PT | 9+23.6 | 14,209.22 | 9,562.66 | | | |
| | 9+23.6 | 14,209.22 | 9,562.66 | | | |
| | 11+05.4 | 14,297.16 | 9,403.56 | | | |
| PC | 11+05.4 | 14,297.16 | 9,403.56 | | | |
| PI | 11+14.8 | 14,301.70 | 9,395.34 | | | |
| RP | | 14,122.13 | 9,306.80 | 200.0 | 5°22' | 28°57' |
| PT | 11+24.1 | 14,305.45 | 9,386.74 | | | |
| | 11+24.1 | 14,305.45 | 9,386.74 | | | |
| | 11+51.0 | 14,316.21 | 9,362.09 | | | |
| PC | 11+51.0 | 14,316.21 | 9,362.09 | | | |
| PI | 12+04.9 | 14,337.71 | 9,312.76 | | | |
| RP | | 14,774.53 | 9,561.94 | 500.0 | 12°17' | 11°29' |
| PT | 12+58.3 | 14,369.23 | 9,269.14 | | | |
| | 12+58.3 | 14,369.23 | 9,269.14 | | | |
| | 14+70.0 | 14,493.23 | 9,097.49 | | | |
| PC | 14+70.0 | 14,493.23 | 9,097.49 | | | |
| PI | 14+99.6 | 14,510.55 | 9,073.51 | | | |
| RP | | 14,898.53 | 9,390.29 | 500.0 | 6°46' | 11°29' |
| PT | 15+29.1 | 14,530.58 | 9,051.75 | | | |
| | 15+29.1 | 14,530.58 | 9,051.75 | | | |
| | 17+81.4 | 14,701.44 | 8,866.04 | | | |
| PC | 17+81.4 | 14,701.44 | 8,866.04 | | | |
| PI | 17+93.7 | 14,709.71 | 8,857.05 | | | |
| RP | | 14,554.26 | 8,730.62 | 200.0 | 6°59' | 28°57' |
| PT | 18+05.8 | 14,716.83 | 8,847.12 | | | |

### STAGE 1 PERIMETER ROAD HORIZONTAL CONTROL DATA

| DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|
| | 18+05.8 | 14,716.83 | 8,847.12 | | | |
| | 18+87.5 | 14,764.40 | 8,780.75 | | | |
| PC | 18+87.5 | 14,764.40 | 8,780.75 | | | |
| PI | 18+89.1 | 14,765.33 | 8,779.44 | | | |
| RP | | 14,601.83 | 8,664.24 | 200.0 | 0°55' | 28°57' |
| PT | 18+90.7 | 14,766.25 | 8,778.11 | | | |
| | 18+90.7 | 14,766.25 | 8,778.11 | | | |
| | 22+33.5 | 14,961.41 | 8,496.32 | | | |
| PC | 22+33.5 | 14,961.41 | 8,496.32 | | | |
| PI | 22+60.6 | 14,976.82 | 8,474.08 | | | |
| RP | | 14,550.37 | 8,211.64 | 500.0 | 6°12' | 11°29' |
| PT | 22+87.6 | 14,989.73 | 8,450.30 | | | |
| | 22+87.6 | 14,989.73 | 8,450.30 | | | |
| | 23+52.2 | 15,020.60 | 8,393.47 | | | |
| PC | 23+52.2 | 15,020.60 | 8,393.47 | | | |
| PI | 23+76.5 | 15,032.19 | 8,372.14 | | | |
| RP | | 15,284.22 | 8,536.66 | 300.0 | 9°15' | 19°11' |
| PT | 24+00.7 | 15,047.05 | 8,352.95 | | | |
| | 24+00.7 | 15,047.05 | 8,352.95 | | | |
| | 24+94.8 | 15,104.71 | 8,278.51 | | | |
| PC | 24+94.8 | 15,104.71 | 8,278.51 | | | |
| PI | 25+16.8 | 15,118.19 | 8,261.11 | | | |
| RP | | 15,025.65 | 8,217.28 | 100.0 | 24°50' | 60°00' |
| PT | 25+38.2 | 15,123.12 | 8,239.65 | | | |
| | 25+38.2 | 15,123.12 | 8,239.65 | | | |
| | 25+64.0 | 15,128.89 | 8,214.47 | | | |
| PC | 25+64.0 | 15,128.89 | 8,214.47 | | | |
| PI | 26+00.5 | 15,137.06 | 8,178.90 | | | |
| RP | | 15,616.22 | 8,326.33 | 500.0 | 8°21' | 11°29' |
| PT | 26+36.9 | 15,150.30 | 8,144.89 | | | |
| | 26+36.9 | 15,150.30 | 8,144.89 | | | |
| | 28+53.4 | 15,228.86 | 7,943.16 | | | |
| PC | 28+53.4 | 15,228.86 | 7,943.16 | | | |
| PI | 28+67.5 | 15,234.01 | 7,929.95 | | | |
| RP | | 15,200.91 | 7,932.28 | 30.0 | 50°36' | 0°00' |
| PT | 28+79.9 | 15,227.07 | 7,917.58 | | | |
| | 28+79.9 | 15,227.07 | 7,917.58 | | | |
| | 30+08.6 | 15,164.03 | 7,805.36 | | | |
| PC | 30+08.6 | 15,164.03 | 7,805.36 | | | |
| PI | 30+33.4 | 15,151.84 | 7,783.67 | | | |
| RP | | 15,146.59 | 7,815.16 | 20.0 | 102°25' | 0°00' |
| PT | 30+44.3 | 15,133.28 | 7,800.23 | | | |
| | 30+44.3 | 15,133.28 | 7,800.23 | | | |
| | 31+01.1 | 15,090.88 | 7,838.06 | | | |
| PC | 31+01.1 | 15,090.88 | 7,838.06 | | | |
| PI | 31+17.6 | 15,078.59 | 7,849.03 | | | |
| RP | | 15,224.04 | 7,987.29 | 200.0 | 9°25' | 28°57' |
| PT | 31+34.0 | 15,068.27 | 7,861.86 | | | |

### STAGE 1 PERIMETER ROAD HORIZONTAL CONTROL DATA

| DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|
| | 31+34.0 | 15,068.27 | 7,861.86 | | | |
| | 32+71.3 | 14,982.14 | 7,968.82 | | | |
| PC | 32+71.3 | 14,982.14 | 7,968.82 | | | |
| PI | 35+06.2 | 14,834.84 | 8,151.76 | | | |
| RP | | 14,047.47 | 7,216.23 | 1200.0 | 22°09' | 4°47' |
| PT | 37+35.2 | 14,629.43 | 8,265.67 | | | |
| | 37+35.2 | 14,629.43 | 8,265.67 | | | |
| | 40+86.5 | 14,322.23 | 8,436.02 | | | |
| PC | 40+86.5 | 14,322.23 | 8,436.02 | | | |
| PI | 41+96.0 | 14,226.49 | 8,489.11 | | | |
| RP | | 13,788.77 | 7,474.03 | 1100.0 | 11°22' | 5°13' |
| PT | 43+04.7 | 14,122.16 | 8,522.29 | | | |
| | 43+04.7 | 14,122.16 | 8,522.29 | | | |
| | 44+45.1 | 13,988.37 | 8,564.84 | | | |
| PC | 44+45.1 | 13,988.37 | 8,564.84 | | | |
| PI | 44+66.1 | 13,968.37 | 8,571.20 | | | |
| RP | | 14,018.67 | 8,660.14 | 100.0 | 23°42' | 60°00' |
| PT | 44+86.5 | 13,952.62 | 8,585.06 | | | |
| | 44+86.5 | 13,952.62 | 8,585.06 | | | |
| | 47+88.5 | 13,725.87 | 8,784.57 | | | |
| PC | 47+88.5 | 13,725.87 | 8,784.57 | | | |
| PI | 48+89.8 | 13,649.81 | 8,851.50 | | | |
| RP | | 12,999.25 | 7,958.72 | 1100.0 | 10°32' | 5°13' |
| PT | 49+90.6 | 13,562.79 | 8,903.40 | | | |
| | 49+90.6 | 13,562.79 | 8,903.40 | | | |
| | 50+45.7 | 13,515.42 | 8,931.66 | | | |
| PC | 50+45.7 | 13,515.42 | 8,931.66 | | | |
| PI | 50+49.4 | 13,512.27 | 8,933.54 | | | |
| RP | | 13,530.79 | 8,957.42 | 30.0 | 13°57' | 0°00' |
| PT | 50+53.0 | 13,509.67 | 8,936.12 | | | |
| | 50+53.0 | 13,509.67 | 8,936.12 | | | |
| | 52+85.3 | 13,344.73 | 9,099.69 | | | |
| | 52+85.3 | 13,344.73 | 9,099.69 | | | |
| | 55+43.6 | 13,163.89 | 9,284.14 | | | |
| PC | 55+43.6 | 13,163.89 | 9,284.14 | | | |
| PI | 55+49.3 | 13,159.90 | 9,288.21 | | | |
| RP | | 13,178.17 | 9,298.15 | 20.0 | 31°49' | 0°00' |
| PT | 55+54.7 | 13,158.66 | 9,293.77 | | | |
| | 55+54.7 | 13,158.66 | 9,293.77 | | | |
| | 57+87.5 | 13,107.78 | 9,520.90 | | | |
| PC | 57+87.5 | 13,107.78 | 9,520.90 | | | |
| PI | 58+50.4 | 13,094.03 | 9,582.32 | | | |
| RP | | 12,815.04 | 9,455.33 | 300.0 | 23°42' | 19°11' |
| PT | 59+11.6 | 13,056.74 | 9,633.04 | | | |
| | 59+11.6 | 13,056.74 | 9,633.04 | | | |
| | 59+23.1 | 13,049.94 | 9,642.28 | | | |

STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 1 PERIMETER ROAD PROFILE AND HORIZONTAL CONTROL DATA

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 430 | 1 |

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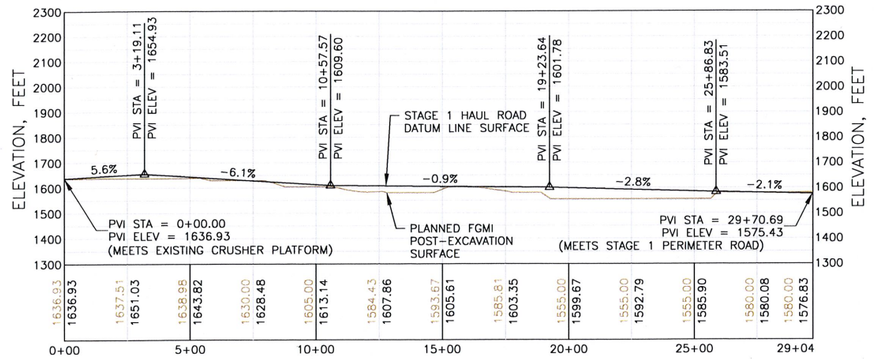



## STAGE 1 HAUL ROAD PROFILE

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

### STAGE 1 HAUL ROAD
### HORIZONTAL CONTROL DATA

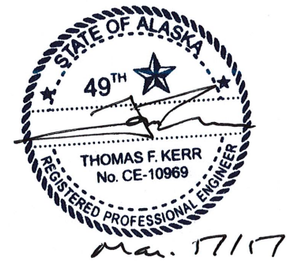
| DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|
| | 0+00.0 | 12,872.56 | 9,222.30 | | | |
| | 2+25.2 | 13,019.89 | 9,051.95 | | | |
| PC | 2+25.2 | 13,019.89 | 9,051.95 | | | |
| PI | 2+80.3 | 13,055.92 | 9,010.29 | | | |
| RP | | 13,171.16 | 9,182.78 | 200.0 | 30°48' | 28°57' |
| PT | 3+32.7 | 13,108.21 | 8,992.95 | | | |
| | 3+32.7 | 13,108.21 | 8,992.95 | | | |
| | 4+88.3 | 13,255.91 | 8,943.97 | | | |
| PC | 4+88.3 | 13,255.91 | 8,943.97 | | | |
| PI | 5+13.3 | 13,279.61 | 8,936.11 | | | |
| RP | | 13,192.96 | 8,754.14 | 200.0 | 14°14' | 28°57' |
| PT | 5+38.0 | 13,300.65 | 8,922.67 | | | |
| | 5+38.0 | 13,300.65 | 8,922.67 | | | |
| | 10+46.3 | 13,728.99 | 8,648.95 | | | |
| PC | 10+46.3 | 13,728.99 | 8,648.95 | | | |
| PI | 11+48.9 | 13,815.43 | 8,593.72 | | | |
| RP | | 13,190.52 | 7,806.31 | 1000.0 | 11°43' | 5°44' |
| PT | 12+50.8 | 13,888.86 | 8,522.08 | | | |
| | 12+50.8 | 13,888.86 | 8,522.08 | | | |
| | 13+91.0 | 13,989.21 | 8,424.16 | | | |
| PC | 13+91.0 | 13,989.21 | 8,424.16 | | | |
| PI | 14+21.7 | 14,011.16 | 8,402.75 | | | |
| RP | | 14,128.88 | 8,567.32 | 200.0 | 17°26' | 28°57' |
| PT | 14+51.9 | 14,038.51 | 8,388.90 | | | |
| | 14+51.9 | 14,038.51 | 8,388.90 | | | |
| | 17+73.7 | 14,325.67 | 8,243.45 | | | |
| PC | 17+73.7 | 14,325.67 | 8,243.45 | | | |
| PI | 17+94.8 | 14,344.43 | 8,233.95 | | | |
| RP | | 14,416.04 | 8,421.87 | 200.0 | 12°00' | 28°57' |
| PT | 18+15.7 | 14,364.76 | 8,228.55 | | | |
| | 18+15.7 | 14,364.76 | 8,228.55 | | | |
| | 19+31.0 | 14,476.26 | 8,198.97 | | | |
| PC | 19+31.0 | 14,476.26 | 8,198.97 | | | |
| PI | 19+69.1 | 14,513.10 | 8,189.20 | | | |
| RP | | 14,424.98 | 8,005.66 | 200.0 | 21°35' | 28°57' |
| PT | 20+06.3 | 14,543.76 | 8,166.57 | | | |

## LEGEND:

| | |
|---|---|
| 2119.57 | PLANNED FGMI POST-EXCAVATION GROUND SURFACE ELEVATION, FEET |
| 2117.76 | STAGE 1 HAUL ROAD DATUM LINE ELEVATION, FEET |
| PC | POINT OF CURVATURE |
| PI | POINT OF INTERSECTION |
| RP | RADIAL POINT (CENTER OF CURVATURE) |
| PT | POINT OF TANGENCY |
| PVI | POINT OF VERTICAL INTERSECTION |

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL SETTING OUT DATA IS TO BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
3. SURVEY CONTROL POINTS SHALL BE PROVIDED BY FGMI.
4. THE INITIAL PORTION OF THE STAGE 1 HAUL ROAD IS INTENDED TO SLOPE UPWARD PRIOR TO DROPPING INTO THE BASIN AND TYING INTO THE STAGE 1 PERIMETER. THIS INITIAL PORTION IS ENVISIONED TO FORM THE STAGE 2 PAD PERIMETER IN THIS AREA.




Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
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DISCLAIMER
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ACTIVITY CODE: N/A
XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 1 HAUL ROAD
PROFILE AND HORIZONTAL CONTROL DATA

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 435 | 0 |


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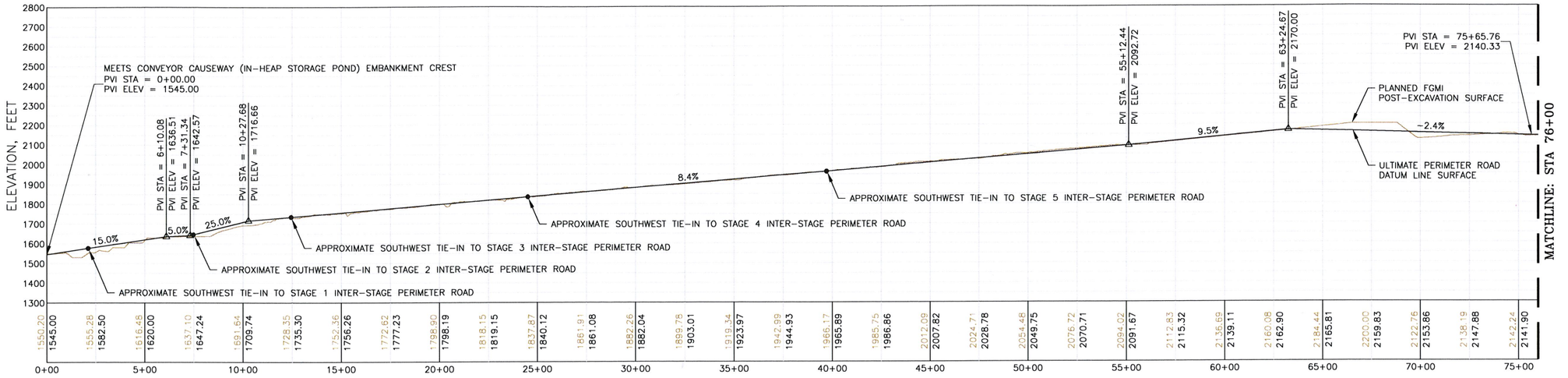


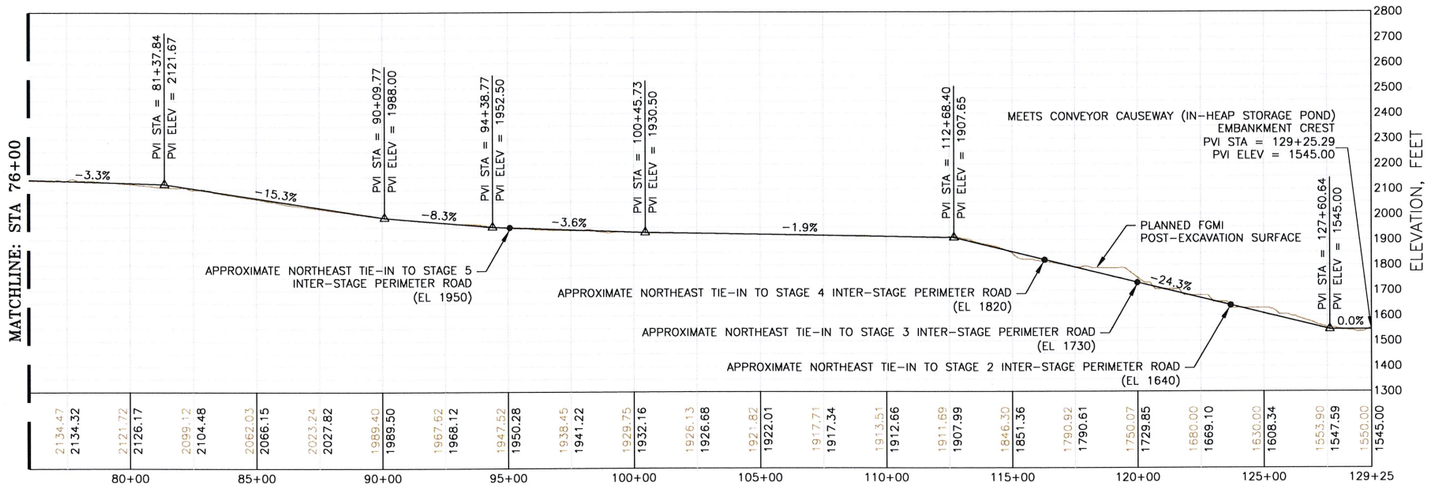


## ULTIMATE PERIMETER ROAD PROFILE

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

**LEGEND:**

| | |
|---|---|
| 2119.57 | PLANNED FGMI POST-EXCAVATION GROUND SURFACE ELEVATION, FEET |
| 2117.76 | ULTIMATE PERIMETER ROAD DATUM LINE ELEVATION, FEET |
| PVI | POINT OF VERTICAL INTERSECTION |

**NOTES:**

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL SETTING OUT DATA IS TO BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
3. SURVEY CONTROL POINTS SHALL BE PROVIDED BY FGMI.
4. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | CADD JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |


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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: ULTIMATE PERIMETER ROAD PROFILE

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 480 | 0 |


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## ULTIMATE PERIMETER ROAD HORIZONTAL CONTROL DATA

| DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|
| | 0+00.0 | 13,086.07 | 9,833.69 | | | |
| | 0+60.5 | 13,057.95 | 9,780.12 | | | |
| PC | 0+60.5 | 13,057.95 | 9,780.12 | | | |
| PI | 0+91.6 | 13,043.50 | 9,752.58 | | | |
| RP | | 13,146.50 | 9,733.64 | 100.0 | 34°33' | 60°00' |
| PT | 1+20.8 | 13,047.21 | 9,721.70 | | | |
| | 1+20.8 | 13,047.21 | 9,721.70 | | | |
| | 2+67.3 | 13,064.71 | 9,576.30 | | | |
| PC | 2+67.3 | 13,064.71 | 9,576.30 | | | |
| PI | 3+24.3 | 13,071.52 | 9,519.66 | | | |
| RP | | 12,866.14 | 9,552.41 | 200.0 | 31°51' | 28°57' |
| PT | 3+78.4 | 13,047.43 | 9,467.94 | | | |
| | 3+78.4 | 13,047.43 | 9,467.94 | | | |
| | 4+62.5 | 13,011.92 | 9,391.73 | | | |
| PC | 4+62.5 | 13,011.92 | 9,391.73 | | | |
| PI | 4+94.8 | 12,998.26 | 9,362.42 | | | |
| RP | | 13,193.21 | 9,307.27 | 200.0 | 18°22' | 28°57' |
| PT | 5+26.6 | 12,994.54 | 9,330.30 | | | |
| | 5+26.6 | 12,994.54 | 9,330.30 | | | |
| | 7+60.8 | 12,967.56 | 9,097.61 | | | |
| PC | 7+60.8 | 12,967.56 | 9,097.61 | | | |
| PI | 7+89.2 | 12,964.29 | 9,069.41 | | | |
| RP | | 13,265.56 | 9,063.06 | 300.0 | 10°49' | 19°11' |
| PT | 8+17.5 | 12,966.37 | 9,041.09 | | | |
| | 8+17.5 | 12,966.37 | 9,041.09 | | | |
| | 10+03.1 | 12,979.96 | 8,855.98 | | | |
| PC | 10+03.1 | 12,979.96 | 8,855.98 | | | |
| PI | 11+00.0 | 12,987.07 | 8,759.27 | | | |
| RP | | 13,179.43 | 8,870.62 | 200.0 | 51°44' | 28°57' |
| PT | 11+83.7 | 13,067.39 | 8,704.95 | | | |
| | 11+83.7 | 13,067.39 | 8,704.95 | | | |
| | 13+90.0 | 13,238.32 | 8,589.36 | | | |
| PC | 13+90.0 | 13,238.32 | 8,589.36 | | | |
| PI | 15+10.5 | 13,338.15 | 8,521.86 | | | |
| RP | | 12,846.21 | 8,009.49 | 700.0 | 19°32' | 8°12' |
| PT | 16+28.7 | 13,409.65 | 8,424.86 | | | |
| | 16+28.7 | 13,409.65 | 8,424.86 | | | |
| | 19+33.8 | 13,590.71 | 8,179.26 | | | |
| PC | 19+33.8 | 13,590.71 | 8,179.26 | | | |
| PI | 20+86.4 | 13,681.26 | 8,056.42 | | | |
| RP | | 14,154.15 | 8,594.62 | 700.0 | 24°36' | 8°12' |
| PT | 22+34.3 | 13,814.72 | 7,982.43 | | | |
| | 22+34.3 | 13,814.72 | 7,982.43 | | | |
| | 23+56.2 | 13,921.36 | 7,923.30 | | | |
| PC | 23+56.2 | 13,921.36 | 7,923.30 | | | |
| PI | 24+06.3 | 13,965.13 | 7,899.03 | | | |
| RP | | 13,678.91 | 7,486.01 | 500.0 | 11°26' | 11°29' |
| PT | 24+56.0 | 14,003.22 | 7,866.57 | | | |

## ULTIMATE PERIMETER ROAD HORIZONTAL CONTROL DATA

| DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|
| | 24+56.0 | 14,003.22 | 7,866.57 | | | |
| | 27+96.6 | 14,262.42 | 7,645.67 | | | |
| PC | 27+96.6 | 14,262.42 | 7,645.67 | | | |
| PI | 28+96.3 | 14,338.38 | 7,580.95 | | | |
| RP | | 13,938.11 | 7,265.12 | 500.0 | 22°34' | 11°29' |
| PT | 29+93.6 | 14,383.66 | 7,492.03 | | | |
| | 29+93.6 | 14,383.66 | 7,492.03 | | | |
| | 31+95.9 | 14,475.48 | 7,311.74 | | | |
| PC | 31+95.9 | 14,475.48 | 7,311.74 | | | |
| PI | 32+49.8 | 14,499.94 | 7,263.70 | | | |
| RP | | 14,921.03 | 7,538.65 | 500.0 | 12°18' | 11°29' |
| PT | 33+03.3 | 14,534.08 | 7,221.99 | | | |
| | 33+03.3 | 14,534.08 | 7,221.99 | | | |
| | 38+58.9 | 14,886.00 | 6,791.96 | | | |
| PC | 38+58.9 | 14,886.00 | 6,791.96 | | | |
| PI | 39+42.4 | 14,938.83 | 6,727.40 | | | |
| RP | | 15,272.95 | 7,108.62 | 500.0 | 18°57' | 11°29' |
| PT | 40+24.3 | 15,009.76 | 6,683.49 | | | |
| | 40+24.3 | 15,009.76 | 6,683.49 | | | |
| | 42+59.4 | 15,209.68 | 6,559.73 | | | |
| PC | 42+59.4 | 15,209.68 | 6,559.73 | | | |
| PI | 43+00.8 | 15,244.93 | 6,537.91 | | | |
| RP | | 15,472.87 | 6,984.86 | 500.0 | 9°29' | 11°29' |
| PT | 43+42.1 | 15,283.28 | 6,522.19 | | | |
| | 43+42.1 | 15,283.28 | 6,522.19 | | | |
| | 50+16.3 | 15,907.12 | 6,266.56 | | | |
| PC | 50+16.3 | 15,907.12 | 6,266.56 | | | |
| PI | 51+06.1 | 15,990.20 | 6,232.52 | | | |
| RP | | 16,096.71 | 6,729.22 | 500.0 | 20°22' | 11°29' |
| PT | 51+94.0 | 16,079.93 | 6,229.50 | | | |
| | 51+94.0 | 16,079.93 | 6,229.50 | | | |
| | 54+38.9 | 16,324.77 | 6,221.29 | | | |
| PC | 54+38.9 | 16,324.77 | 6,221.29 | | | |
| PI | 57+46.7 | 16,632.41 | 6,210.96 | | | |
| RP | | 16,349.09 | 6,945.88 | 725.0 | 46°01' | 7°55' |
| PT | 60+21.1 | 16,853.50 | 6,425.12 | | | |
| | 60+21.1 | 16,853.50 | 6,425.12 | | | |
| | 66+80.7 | 17,327.26 | 6,884.00 | | | |
| PC | 66+80.7 | 17,327.26 | 6,884.00 | | | |
| PI | 67+06.9 | 17,346.11 | 6,902.26 | | | |
| RP | | 17,466.40 | 6,740.34 | 200.0 | 14°57' | 28°57' |
| PT | 67+32.9 | 17,369.04 | 6,915.04 | | | |
| | 67+32.9 | 17,369.04 | 6,915.04 | | | |
| | 71+09.0 | 17,697.61 | 7,098.15 | | | |
| PC | 71+09.0 | 17,697.61 | 7,098.15 | | | |
| PI | 71+85.1 | 17,764.11 | 7,135.21 | | | |
| RP | | 17,600.25 | 7,272.85 | 200.0 | 41°41' | 28°57' |
| PT | 72+54.5 | 17,789.14 | 7,207.12 | | | |

## ULTIMATE PERIMETER ROAD HORIZONTAL CONTROL DATA

| DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|
| | 72+54.5 | 17,789.14 | 7,207.12 | | | |
| | 73+94.9 | 17,835.28 | 7,339.70 | | | |
| PC | 73+94.9 | 17,835.28 | 7,339.70 | | | |
| PI | 74+62.9 | 17,857.64 | 7,403.95 | | | |
| RP | | 17,646.39 | 7,405.44 | 200.0 | 37°34' | 28°57' |
| PT | 75+26.0 | 17,836.19 | 7,468.51 | | | |
| | 75+26.0 | 17,836.19 | 7,468.51 | | | |
| | 78+57.8 | 17,731.56 | 7,783.36 | | | |
| PC | 78+57.8 | 17,731.56 | 7,783.36 | | | |
| PI | 79+05.8 | 17,716.42 | 7,828.90 | | | |
| RP | | 17,541.76 | 7,720.29 | 200.0 | 26°59' | 28°57' |
| PT | 79+52.0 | 17,682.27 | 7,862.62 | | | |
| | 79+52.0 | 17,682.27 | 7,862.62 | | | |
| | 95+69.2 | 16,531.41 | 8,998.71 | | | |
| PC | 95+69.2 | 16,531.41 | 8,998.71 | | | |
| PI | 95+84.7 | 16,520.37 | 9,009.61 | | | |
| RP | | 16,671.91 | 9,141.04 | 200.0 | 8°52' | 28°57' |
| PT | 96+00.1 | 16,511.14 | 9,022.07 | | | |
| | 96+00.1 | 16,511.14 | 9,022.07 | | | |
| | 101+84.0 | 16,163.82 | 9,491.43 | | | |
| PC | 101+84.0 | 16,163.82 | 9,491.43 | | | |
| PI | 101+95.9 | 16,156.77 | 9,500.96 | | | |
| RP | | 16,565.74 | 9,788.85 | 500.0 | 2°43' | 11°29' |
| PT | 102+07.7 | 16,150.18 | 9,510.81 | | | |
| | 102+07.7 | 16,150.18 | 9,510.81 | | | |
| | 109+81.9 | 15,719.65 | 10,154.27 | | | |
| PC | 109+81.9 | 15,719.65 | 10,154.27 | | | |
| PI | 110+57.2 | 15,677.78 | 10,216.85 | | | |
| RP | | 15,553.43 | 10,043.05 | 200.0 | 41°16' | 28°57' |
| PT | 111+26.0 | 15,605.04 | 10,236.28 | | | |
| | 111+26.0 | 15,605.04 | 10,236.28 | | | |
| | 118+40.1 | 14,915.07 | 10,420.57 | | | |
| PC | 118+40.1 | 14,915.07 | 10,420.57 | | | |
| PI | 118+65.2 | 14,890.88 | 10,427.03 | | | |
| RP | | 14,863.46 | 10,227.34 | 200.0 | 14°16' | 28°57' |
| PT | 118+89.9 | 14,865.84 | 10,427.33 | | | |
| | 118+89.9 | 14,865.84 | 10,427.33 | | | |
| | 129+25.3 | 13,830.56 | 10,439.64 | | | |

**LEGEND:**

| | |
|---|---|
| PC | POINT OF CURVATURE |
| PI | POINT OF INTERSECTION |
| RP | RADIAL POINT (CENTER OF CURVATURE) |
| PT | POINT OF TANGENCY |

**NOTES:**

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL SETTING OUT DATA IS TO BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINEER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.
3. SURVEY CONTROL POINTS SHALL BE PROVIDED BY FGMI.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D / CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: ULTIMATE PERIMETER ROAD HORIZONTAL CONTROL DATA

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 485 | 0 |


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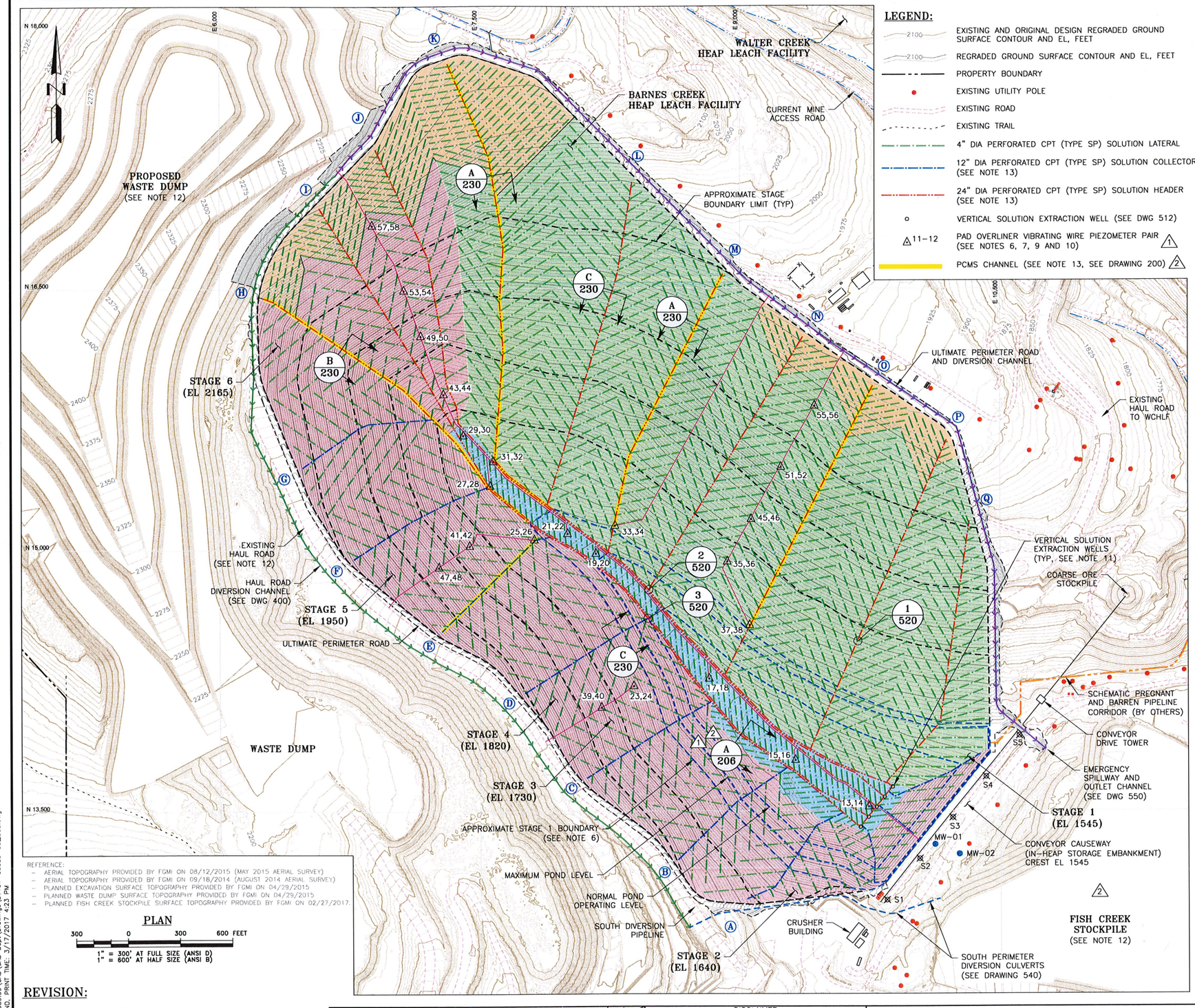


## LEGEND:

- EXISTING AND ORIGINAL DESIGN REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- EXISTING UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- 4" DIA PERFORATED CPT (TYPE SP) SOLUTION LATERAL
- 12" DIA PERFORATED CPT (TYPE SP) SOLUTION COLLECTOR (SEE NOTE 13)
- 24" DIA PERFORATED CPT (TYPE SP) SOLUTION HEADER (SEE NOTE 13)
- VERTICAL SOLUTION EXTRACTION WELL (SEE DWG 512)
- 11-12 PAD OVERLINER VIBRATING WIRE PIEZOMETER PAIR (SEE NOTES 6, 7, 9 AND 10) /1
- PCMS CHANNEL (SEE NOTE 13, SEE DRAWING 200) /2
- MW-01 UNDERDRAIN MONITORING WELL (SEE DRAWING 200) /1
- PIEZOMETER CABLE CONDUIT WITH 12" DIA PERFORATED CPT (TYPE SP) BACKUP PIPE (SEE NOTE 9)
- S1 SETTLEMENT MONUMENT
- A SOLUTION COLLECTOR/HEADER DESIGNATION
- <10% SLOPE ALONG THE VALLEY BOTTOM A/520 B/520
- 5H:1V TO 10H:1V PAD SLOPES ABOVE IN-HEAP POND C/520 D/520
- 4H:1V TO 4.5H:1V PAD SLOPES C/520 D/520
- 2.5H:1V PAD SLOPES C/520 D/520

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, AND BOTTOM OF EROSION PROTECTION WITHIN ALL CHANNELS.
3. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED FABRICATED COUPLERS, TEES OR WYES. ALL CPT CONNECTIONS WHICH CHANGE DIAMETER, SLOPE OR ALIGNMENT MUST BE WIRE-TIED AROUND BOTH ENDS OF THE CONNECTIONS.
4. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
5. THE BARNES CREEK HEAP LEACH FACILITY CONTAINS FOUR RELATIVELY DISTINCT PAD SLOPES AS INDICATED BY THE SHADED AREAS. REFER TO TABLE 1 ON DWG 520 FOR THE SOLUTION LATERAL PIPE SPACING WITHIN EACH AREA.
6. ABOVE THE STAGE 1 BOUNDARY THE LEACH PAD CONTAINS A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND IS CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM WITH LCRS. REFER TO THE SECTIONS AND DETAILS FOR THE LEACH PAD LINER SYSTEMS.
7. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
8. THE INTER-STAGE BOUNDARIES ARE APPROXIMATE AND SHALL BE FINALIZED PRIOR TO CONSTRUCTION OF EACH STAGE.
9. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH THE SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN-HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT THREE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY, BUT ARE NOT SHOWN FOR CLARITY. A 12" DIAMETER PERFORATED CPT PIPE SHALL BE INSTALLED ADJACENT TO EACH PIEZOMETER CABLE CONDUIT TO ALLOW MANUAL PIEZOMETER READINGS, IF NEEDED, SHOULD PROBLEMS BE EXPERIENCED WITH THE INSTALLED PIEZOMETERS.
10. REFER TO DRAWING 720 FOR INSTRUMENTATION SECTIONS AND DETAILS.
11. REFER TO DRAWINGS 510 AND 512 FOR THE VERTICAL SOLUTION COLLECTION WELLS SECTIONS AND DETAILS. REFER TO DRAWING 514 FOR THE TEMPORARY INCLINED SOLUTION EXTRACTION WELLS PLAN AND SECTIONS (NOT SHOWN FOR CLARITY).
12. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
13. THE SOLUTION COLLECTORS (12" DIAMETER PERFORATED CPT) AND HEADERS (24" DIAMETER PERFORATED CPT) SHALL GENERALLY BE INSTALLED /2 WITHIN 20' OF THE LOCATIONS SHOWN IN PLAN. SOLUTION COLLECTORS AND HEADERS OVERLYING PCMS CHANNELS SHALL FOLLOW THE PCMS CHANNEL HORIZONTAL CONTROL DATA ON DRAWINGS 204 AND 205.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH
THOMAS F. KERR No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Ma. 17/17

## REVISION:

/1 ADDED A/220 CALLOUT. MODIFIED LEGEND DESCRIPTION. ADDED UNDERDRAIN MONITORING/PUMPING WELL LOCATIONS IN PLAN AND CALLOUT TO LEGEND.

/2 REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY. ADDED PCMS CHANNEL LOCATIONS. MODIFIED NOTE 13 AND PIEZOMETERS. MODIFIED SOLUTION COLLECTION PIPEWORK AROUND PCMS CHANNEL 4. MOVED PREVIOUS SECTION A/220 TO DRAWING 206.

| REV | DATE | DESCRIPTION | APP'D / CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SOLUTION COLLECTION PIPEWORK AND INSTRUMENTATION PLAN

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 500 | 2 |

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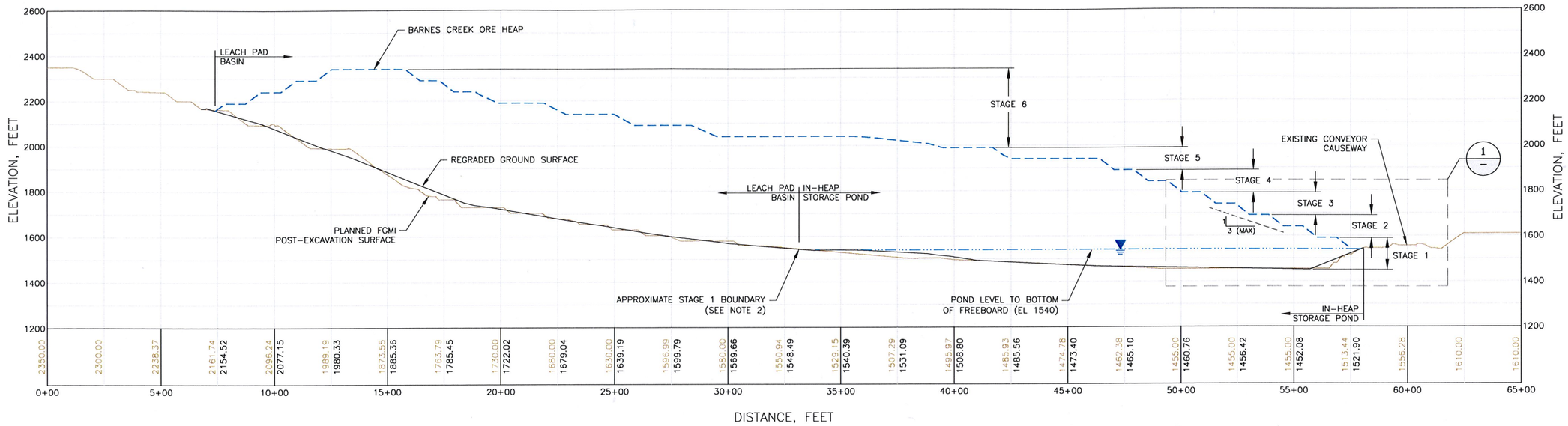
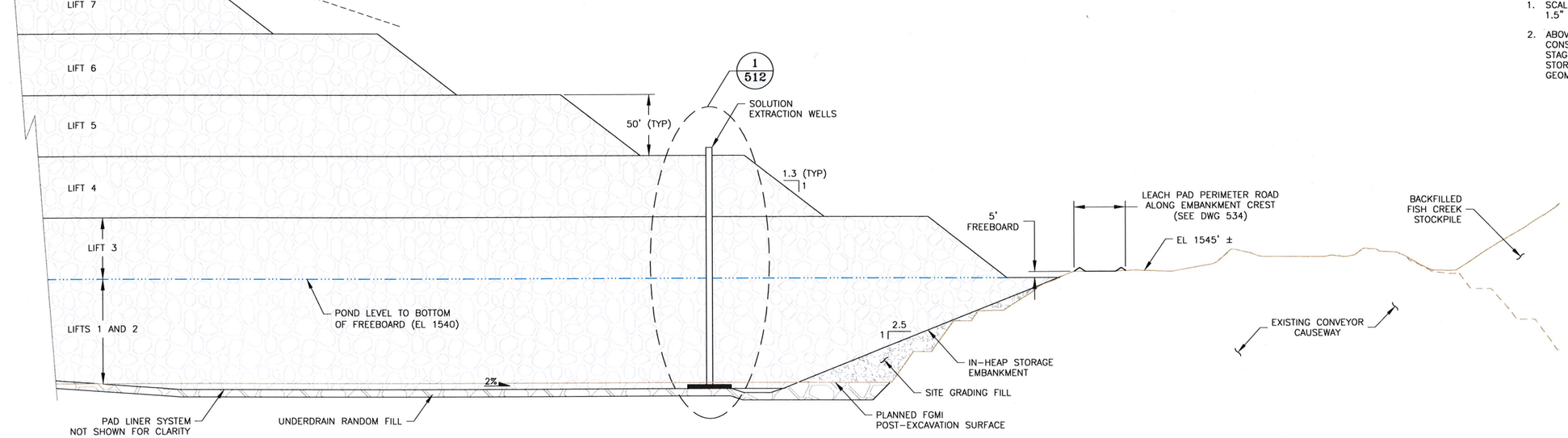



A / 610 TYPICAL BARNES CREEK HEAP LEACH FACILITY

250 0 250 500 FEET
1" = 250' AT FULL SIZE (ANSI D)
1" = 500' AT HALF SIZE (ANSI B)




1 / – TYPICAL IN-HEAP STORAGE POND AND EMBANKMENT

50 0 50 100 FEET
1" = 50' AT FULL SIZE (ANSI D)
1" = 100' AT HALF SIZE (ANSI B)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI–D) AND 1.5" ON A HALF SIZE PLOT (ANSI–B).
2. ABOVE THE STAGE 1 BOUNDARY, THE LEACH PAD WILL BE CONSTRUCTED WITH A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN–HEAP STORAGE POND AND SHALL BE CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.

STATE OF ALASKA 49TH
THOMAS F. KERR No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: VERTICAL SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 1 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 510 | 0 |

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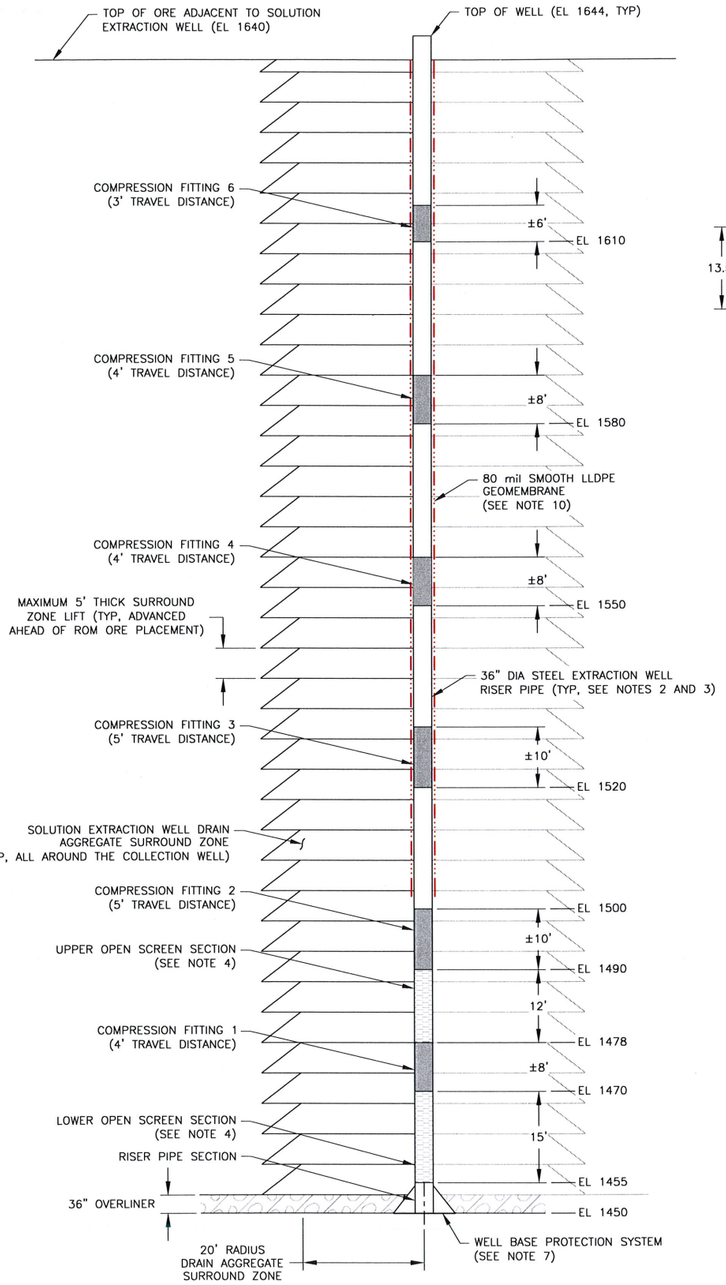
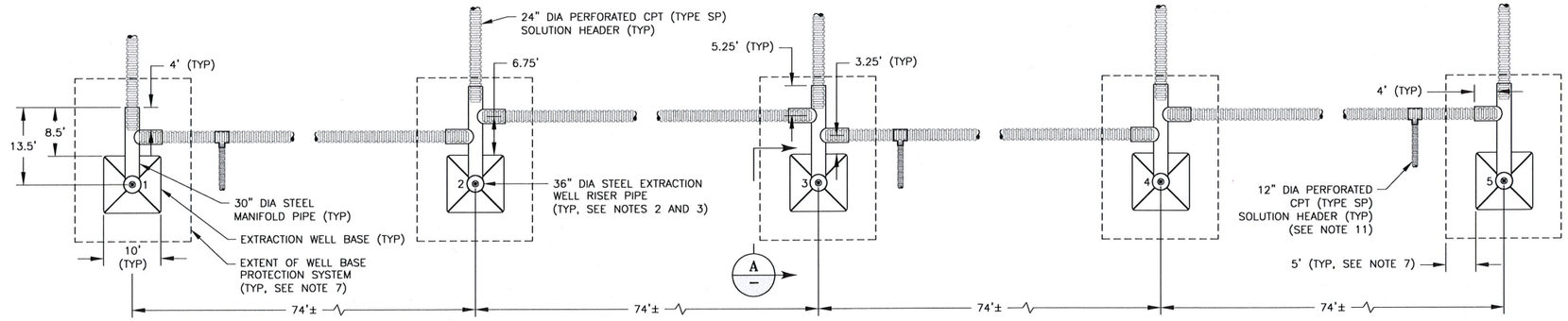



A/– 1/510 TYPICAL VERTICAL SOLUTION EXTRACTION WELL AND DRAIN AGGREGATE SURROUND ZONE
(SEE NOTE 5)



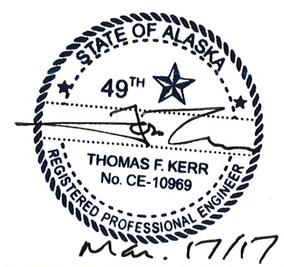

2/– VERTICAL SOLUTION EXTRACTION WELLS AND CONNECTIONS TO SOLUTION HEADERS AND COLLECTORS
(SEE DRAWING 514)

TABLE 1
VERTICAL SOLUTION EXTRACTION WELLS SETTING OUT DATA

| ID | NORTHING | EASTING | ELEVATION |
|---|---|---|---|
| 1 | 13,406.88 | 9,691.00 | 1451.34 |
| 2 | 13,464.25 | 9,737.70 | 1452.08 |
| 3 | 13,522.62 | 9,784.39 | 1452.83 |
| 4 | 13,578.98 | 9,831.09 | 1453.57 |
| 5 | 13,636.35 | 9,877.79 | 1454.31 |

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI–D) AND 1.5" ON A HALF SIZE PLOT (ANSI–B).
2. ALL STEEL IN THE SOLUTION EXTRACTION WELL ASSEMBLIES SHALL BE ASTM 139 HIGH STRENGTH LOW ALLOY (A606 TYPE 4) STEEL.
3. BLANK PIPE USED IN THE WELL ASSEMBLIES SHALL HAVE A WALL THICKNESS OF 0.4375–INCHES. PERFORATED PIPE SHALL HAVE A WALL THICKNESS OF 0.5–INCHES.
4. CIRCUMFERENTIAL SCREEN SLOTS SHALL BE MILL–CUT BY THE PIPE MANUFACTURER. SEE DRAWING 514 FOR THE SCREEN SLOT ARRANGEMENT.
5. COMPRESSION FITTINGS PROVIDED BY ROSCOE MOSS (OR APPROVED EQUIVALENT) SHALL BE 0.5" THICK STEEL PLATE, SINGLE–STYLE FITTINGS. THESE SHALL BE IN FULL EXTENSION BEFORE AND DURING PLACEMENT OF THE SOLUTION EXTRACTION WELL DRAIN AGGREGATE SURROUND ZONE.
6. SOLUTION EXTRACTION WELL RISER PIPE SECTIONS SHALL BE JOINED BY PENETRATION WELDS OR USING WELDED JOINT SLEEVE SEGMENTS, PLANS FOR JOINING THE WELL PIPE SEGMENTS SHALL BE SUBMITTED FOR REVIEW TO THE ENGINEER PRIOR TO INSTALLATION.
7. THE WELL BASE PROTECTION SYSTEM SHALL CONSIST OF 4 LAYERS OF 80 mil DOUBLE–SIDE TEXTURED LLDPE GEOMEMBRANE AND 1 LAYER OF CONVEYOR BELT (OR APPROVED EQUIVALENT). THE PROTECTION SYSTEM SHALL BE PLACED BENEATH THE WELL ASSEMBLY BASE PLATES AND ASSOCIATED STEEL PIPES AND SHALL EXTEND A MINIMUM OF 5' BEYOND THESE COMPONENTS IN ALL DIRECTIONS.
8. THE WELL SCREEN, CASING, COMPRESSION FITTINGS AND OTHER CONNECTIONS SHALL BE INSTALLED SUCH THAT THE SOLUTION EXTRACTION WELLS MAINTAIN THEIR VERTICAL ALIGNMENTS.
9. AS EACH SOLUTION EXTRACTION WELL IS CONSTRUCTED, THE DRAIN AGGREGATE SURROUND ZONE SHALL BE PLACED SUCH THAT THE DRAIN AGGREGATE IS AT AN EQUAL ELEVATION ENTIRELY AROUND THE WELL AT ALL TIMES. THE CONTRACTOR'S PLACEMENT METHODS SHALL NOT CAUSE ANY SEGMENT OF THE WELL COMPONENTS TO SHIFT OR CAUSE THE WELL SCREEN, CASING, COMPRESSION FITTINGS OR OTHER JOINTS TO DEFLECT FROM THE VERTICAL POSITION.
10. A LAYER OF 80 mil SMOOTH LLDPE GEOMEMBRANE SHALL BE WRAPPED AROUND THE CIRCUMFERENCE OF EACH SOLID WELL CASING BEFORE INSTALLATION OF THE DRAINAGE AGGREGATE SURROUND ZONE TO REDUCE FRICTION AGAINST THE RISER PIPE. THE GEOMEMBRANE WRAP SHALL NOT EXTEND OVER THE SCREEN SECTION NOR OVER COMPRESSION FITTINGS 1 AND 2.
11. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED FABRICATED COUPLERS, TEES OR WYES. ALL CPT CONNECTIONS WHICH CHANGE DIAMETER, SLOPE OR ALIGNMENT MUST BE WIRE–TIED AROUND BOTH ENDS OF THE CONNECTIONS.
12. THE SETTING OUT DATA REPRESENT THE BOTTOM OF THE IN–HEAP POND PRIMARY GEOMEMBRANE LINER (i.e. THE TOP OF THE UNDERLYING PREPARED SUBBASE) AT THE CENTERS OF THE VERTICAL SOLUTION EXTRACTION WELLS.
13. ALL SETTING OUT DATA SHALL BE APPROVED BY THE ENGINEER AND/OR FGMI IN THE FIELD PRIOR TO CONSTRUCTION. IF THE EXISTING TOPOGRAPHY DIFFERS FROM THAT SHOWN ON THE DRAWINGS, THE ENGINNER SHALL ADJUST THE DESIGN FOR UNFORESEEN CONDITIONS.




12.5 0 12.5 25 FEET
1" = 12.5' AT FULL SIZE (ANSI D)
1" = 25' AT HALF SIZE (ANSI B)
(TYPICAL SCALE FOR ALL SECTIONS)

| REV | DATE | DESCRIPTION | APP'D CADD |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: VERTICAL SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 2 OF 3

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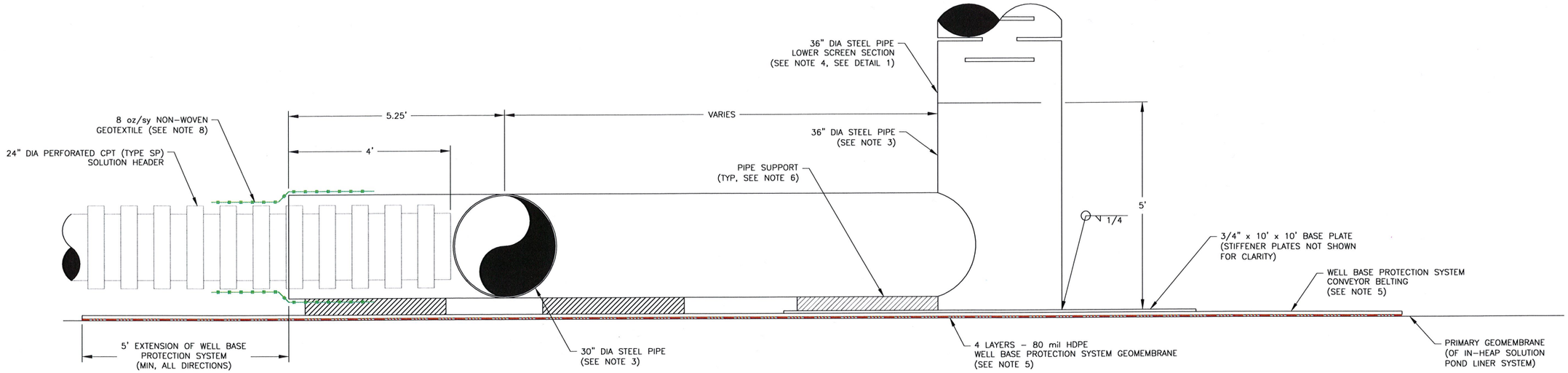
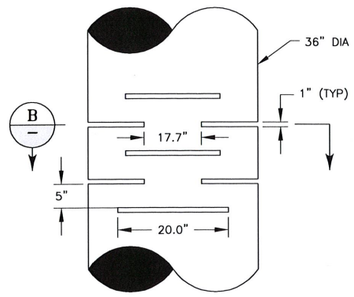



A — VERTICAL SOLUTION EXTRACTION WELL BASE ASSEMBLY AND PROTECTION SYSTEM

NTS

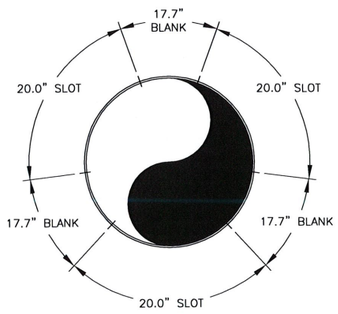
## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. ALL STEEL IN THE SOLUTION EXTRACTION WELL ASSEMBLIES SHALL BE ASTM 139 HIGH STRENGTH LOW ALLOY (A606 TYPE 4) STEEL.
3. BLANK PIPE USED IN THE WELL ASSEMBLIES SHALL HAVE A WALL THICKNESS OF 0.4375-INCHES. PERFORATED PIPE SHALL HAVE A WALL THICKNESS OF 0.5-INCHES.
4. CIRCUMFERENTIAL SCREEN SLOTS SHALL BE MILL-CUT BY THE PIPE MANUFACTURER. SEE DRAWING 514 FOR THE SCREEN SLOT ARRANGEMENT.
5. THE WELL BASE PROTECTION SYSTEM SHALL CONSIST OF 4 LAYERS OF 80 MIL DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE AND 1 LAYER OF CONVEYOR BELT (OR APPROVED EQUIVALENT). THE PROTECTION SYSTEM SHALL BE PLACED BENEATH THE WELL ASSEMBLY BASE PLATES AND ASSOCIATED STEEL PIPES AND SHALL EXTEND A MINIMUM OF 5' BEYOND THESE COMPONENTS IN ALL DIRECTIONS.
6. PIPE SUPPORTS (WOOD TIMBERS OR APPROVED OTHER) SHALL BE INSTALLED UNDER THE STEEL PIPES CONNECTING TO THE VERTICAL SOLUTION COLLECTION WELLS TO SUPPORT THE PIPES DURING INSTALLATION OF OVERLINER DRAIN AGGREGATE AND TO PROTECT THE PIPE AND WELL BASE ASSEMBLY WELDS.
7. OVERLINER DRAIN AGGREGATE SHALL BE WORKED BETWEEN THE PIPE SUPPORTS UNDER THE WELL PIPE ASSEMBLIES.
8. TWO LAYERS OF GEOTEXTILE SHALL BE WRAPPED AND SECURED OVER THE JUNCTIONS OF PERFORATED CPT INTO THE WELL BASE PIPE ASSEMBLIES.



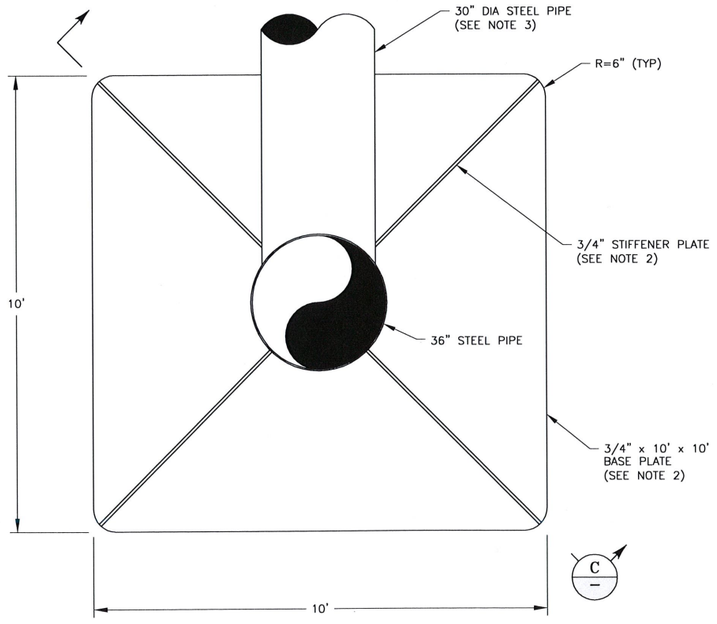

1 — WELL SCREEN SLOT ARRANGEMENT

NTS



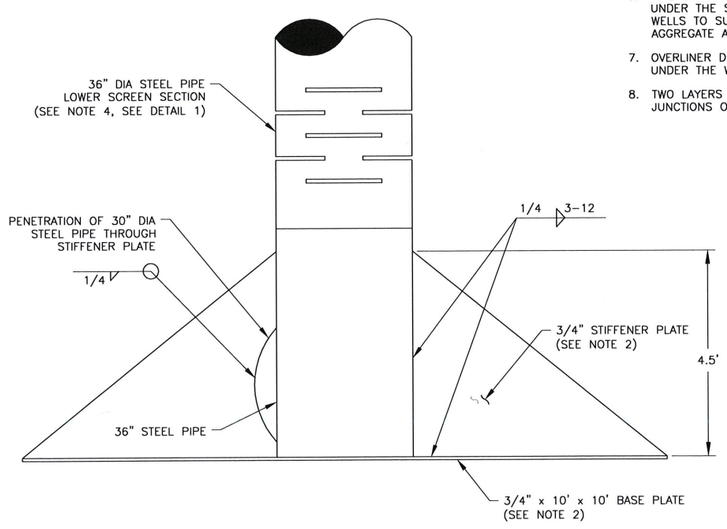

B — PLAN OF CIRCUMFERENTIAL SCREEN SLOT ARRANGEMENT

NTS




2 — VERTICAL SOLUTION EXTRACTION WELL BASE

NTS




C — VERTICAL SOLUTION EXTRACTION WELL BASE

NTS

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

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ACTIVITY CODE: N/A | XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: VERTICAL SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 3 OF 3

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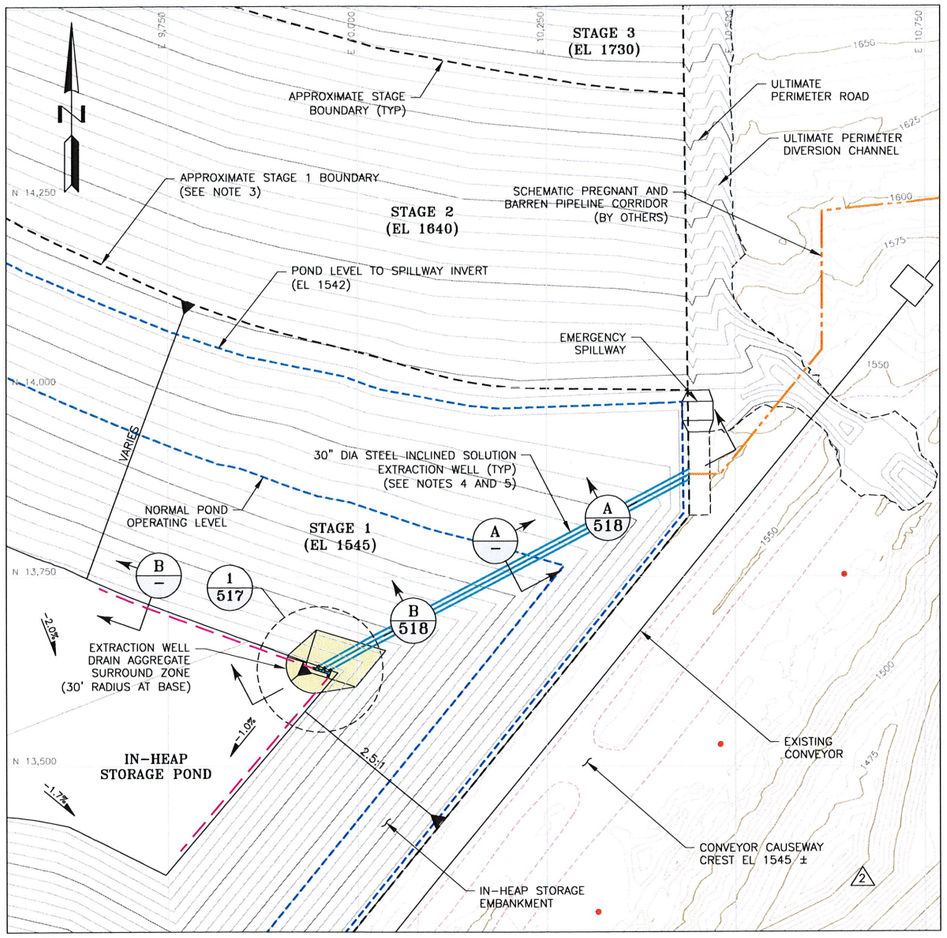



REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

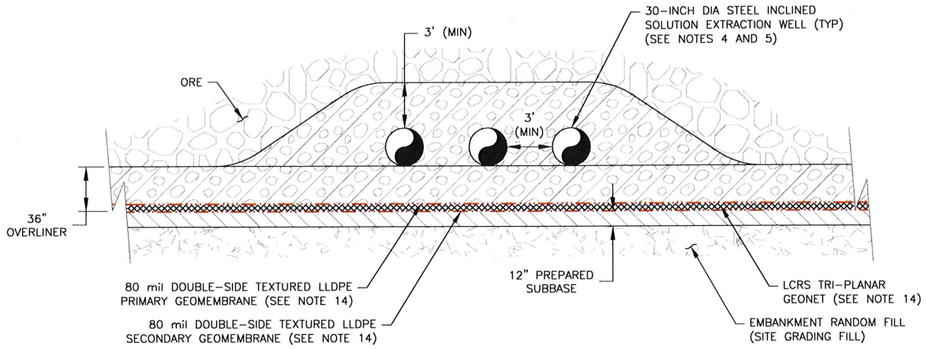
## PLAN

100 0 100 200 FEET

1" = 100' AT FULL SIZE (ANSI D)
1" = 200' AT HALF SIZE (ANSI B)

## LEGEND:

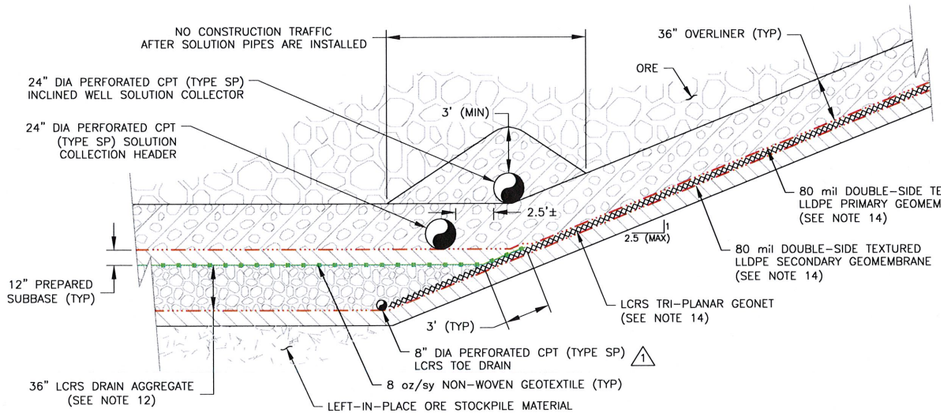
- 1700 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 1700 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- UTILITY POLE
- EXISTING ROAD
- 36" DIA STEEL INCLINED SOLUTION EXTRACTION WELL
- 24" DIA PERFORATED CPT (TYPE SP) INCLINED WELL SOLUTION COLLECTOR




A/– A/517 A/518 TYPICAL INCLINED SOLUTION EXTRACTION WELLS ALONG UPSTREAM NORTHEASTERN ABUTMENT OF IN-HEAP STORAGE EMBANKMENT

5 0 5 10 FEET

1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)




B/– B/517 TYPICAL INCLINED WELL SOLUTION COLLECTOR ALONG TOE OF IN-HEAP STORAGE POND

5 0 5 10 FEET

1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B SHEET).
2. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
3. ABOVE THE STAGE 1 BOUNDARY, THE LEACH PAD WILL BE CONSTRUCTED WITH A SINGLE GEOMEMBRANE LINER. BELOW THE STAGE 1 BOUNDARY, THE LEACH PAD EXISTS AS THE IN-HEAP STORAGE POND AND SHALL BE CONSTRUCTED WITH A DOUBLE GEOMEMBRANE LINER SYSTEM AND LCRS.
4. 30-INCH BLANK STEEL PIPES USED IN THE WELL ASSEMBLIES SHALL BE 30-INCH O.D. PIPE FABRICATED WITH ASTM 139 HIGH STRENGTH LOW ALLOY STEEL (A606 TYPE 4). BLANK PIPE SHALL HAVE A WALL THICKNESS OF 0.562-INCHES. PIPE SHALL BE FABRICATED BY ROSCOE MOSS OR APPROVED EQUIVALENT.
5. PERFORATED PIPES USED IN THE WELL ASSEMBLIES SHALL BE 29-INCH O.D. "SUPERFLO" SHUTTER SCREEN FABRICATED USING ASTM 139 HIGH STRENGTH LOW ALLOY STELL (A606 TYPE 4). SHUTTER SCREENS SHALL BE FABRICATED BY ROSCOE MOSS OR APPROVED EQUIVALENT. PIPE WALL THICKNESS SHALL BE 0.4375-INCHES.
6. SOLUTION EXTRACTION WELL PIPE SECTIONS SHALL BE JOINED BY PENETRATION WELDS OR USING WELDED JOINT SLEEVE SEGMENTS, PLANS FOR JOINING THE WELL PIPE SEGMENTS SHALL BE SUBMITTED FOR REVIEW TO THE ENGINEER PRIOR TO INSTALLATION.
7. TWO LAYERS OF GEOTEXTILE SHALL BE WRAPPED AND SECURED AROUND THE JUNCTIONS OF PERFORATED CPT INTO THE WELL MANIFOLD ASSEMBLY.
8. THE PERFORATED INCLINED WELL SOLUTION COLLECTORS SHALL BE EXTENDED APPROXIMATELY 300' ALONG THE TOES OF THE IN-HEAP POND SLOPES, FROM THE NORTHEAST CORNER OF THE POND, ON THE SURFACE OF THE OVERLINER LAYER.
9. ALL CPT PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL SOLUTION COLLECTION PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
10. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
11. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER THE 24" DIA CPT SOLUTION COLLECTORS.
12. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.
13. A STOPPING RING SHALL BE WELDED INSIDE THE BOTTOM OF EACH INCLINED EXTRACTION WELL TO PREVENT PUMPS FROM GETTING LODGED IN THE PIPE BENDS.
14. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT. (2)

## REVISION:

(1) REMOVED CROSS-REFERENCE FOR SECTION B. ADDED LCRS TOE DRAIN

(2) ADDED NOTE 14 SPECIFYING GEOMEMBRANE AND GEONET. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| B | 12/09/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: INCLINED SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 1 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 516 | 2 |

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

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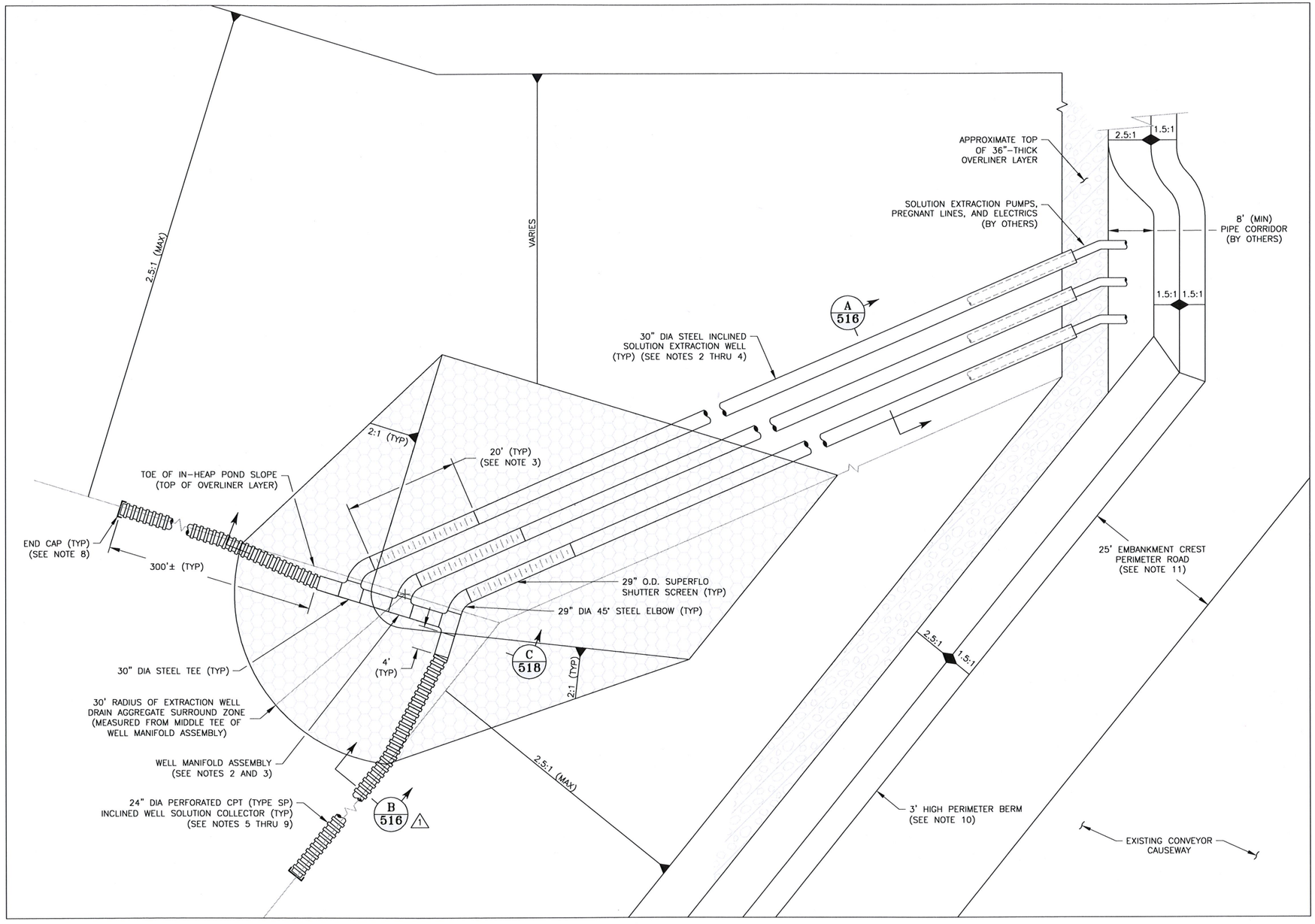


1/516 SCHEMATIC INCLINED SOLUTION EXTRACTION WELLS WITH MANIFOLD ASSEMBLY AND PERORATED CPT SOLUTION COLLECTORS

10 0 10 20 FEET

1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)

## REVISION:

1 REVISED CALLOUT DIRECTION FOR B/516.

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B SHEET).
2. 30-INCH BLANK STEEL PIPES USED IN THE WELL ASSEMBLIES SHALL BE 30-INCH O.D. PIPE FABRICATED WITH ASTM 139 HIGH STRENGTH LOW ALLOY STEEL (A606 TYPE 4). BLANK PIPE SHALL HAVE A WALL THICKNESS OF 0.562-INCHES. PIPE SHALL BE FABRICATED BY ROSCOE MOSS OR APPROVED EQUIVALENT.
3. PERFORATED PIPES USED IN THE WELL ASSEMBLIES SHALL BE 29-INCH O.D. "SUPERFLO" SHUTTER SCREEN FABRICATED USING ASTM 139 HIGH STRENGTH LOW ALLOY STELL (A606 TYPE 4). SHUTTER SCREENS SHALL BE FABRICATED BY ROSCOE MOSS OR APPROVED EQUIVALENT. PIPE WALL THICKNESS SHALL BE 0.4375-INCHES.
4. SOLUTION EXTRACTION WELL PIPE SECTIONS SHALL BE JOINED BY PENETRATION WELDS OR USING WELDED JOINT SLEEVE SEGMENTS, PLANS FOR JOINING THE WELL PIPE SEGMENTS SHALL BE SUBMITTED FOR REVIEW TO THE ENGINEER PRIOR TO INSTALLATION.
5. TWO LAYERS OF GEOTEXTILE SHALL BE WRAPPED AND SECURED OVER THE JUNCTIONS OF PERFORATED CPT INTO THE WELL MANIFOLD ASSEMBLY.
6. THE PERFORATED INCLINED WELL SOLUTION COLLECTORS SHALL BE EXTENDED APPROXIMATELY 300' ALONG THE TOES OF THE IN-HEAP POND SLOPES, FROM THE NORTHEAST CORNER OF THE POND, ON THE SURFACE OF THE OVERLINER LAYER.
7. ALL CPT PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL SOLUTION COLLECTION PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
8. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
9. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER THE 24" DIA SOLUTION COLLECTORS.
10. THE PAD PERIMETER BERM SHALL BE CONSTRUCTED WITH A 12" THICK LAYER OF COMPACTED PREPARED SUBBASE MATERIAL ALONG THE SLOPING FACE OF THE BERM INTERIOR TO THE LEACH PAD. THE UNDERLYING PORTION OF THE PAD PERIMETER BERM SHALL BE CONSTRUCTED OF COMPACTED SITE GRADING FILL UNLESS OTHERWISE AUTHORIZED BY FGMI.
11. THE EMBANKMENT CREST PERIMETER ROAD SHALL BE GRADED AWAY FROM THE LEACH PAD PERIMETER BERM BY SHAPING THE WEARING COURSE SURFACE. A MINIMUM WEARING COURSE THICKNESS OF 12" SHALL BE MAINTAINED.
12. A STOPPING RING SHALL BE WELDED INSIDE THE BOTTOM OF EACH INCLINED EXTRACTION WELL TO PREVENT PUMPS FROM GETTING LODGED IN THE PIPE BENDS.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D / CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| B | 12/09/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |


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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: INCLINED SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 2 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 517 | 1 |

LAST SAVED BY: CBEANLAND
DRAWING PATH: G:\101\00089.99\CAD\CAD Dept\Drawings\DV101-00089-99_516-518.dwg
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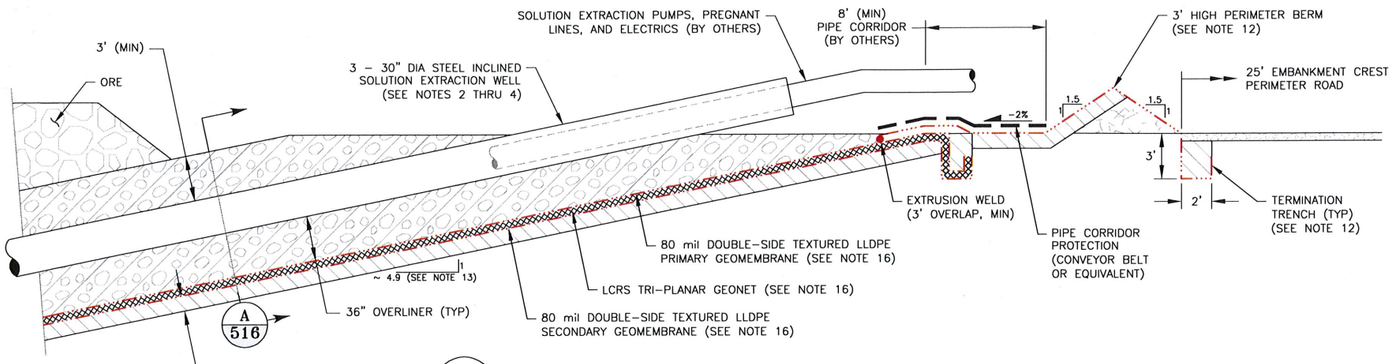
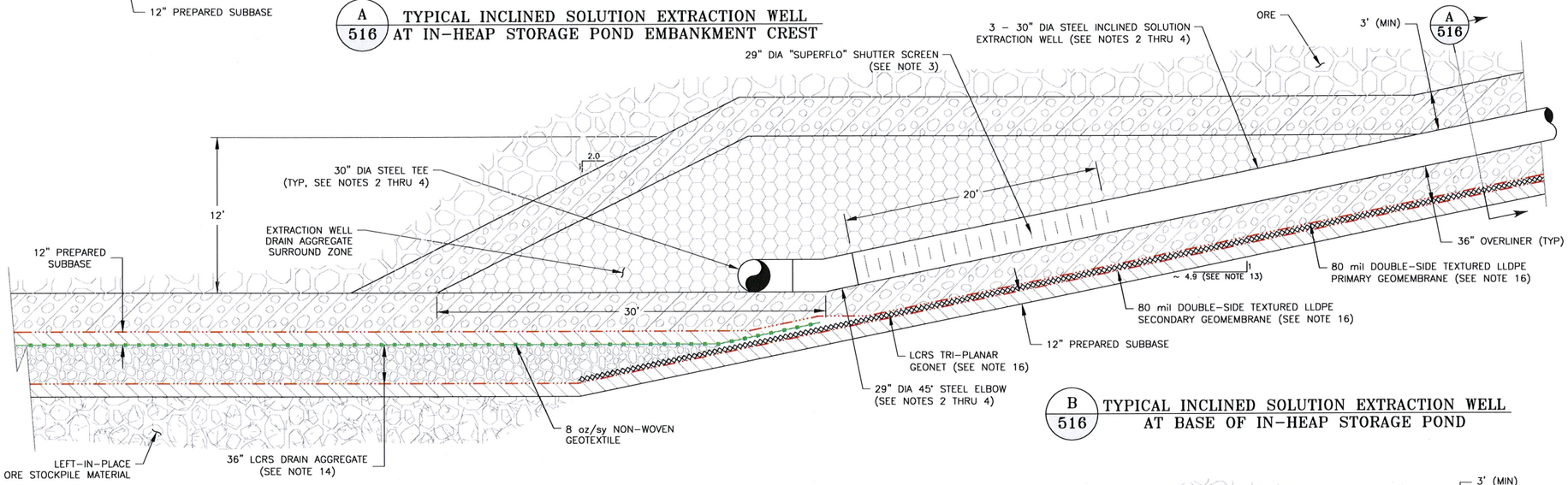
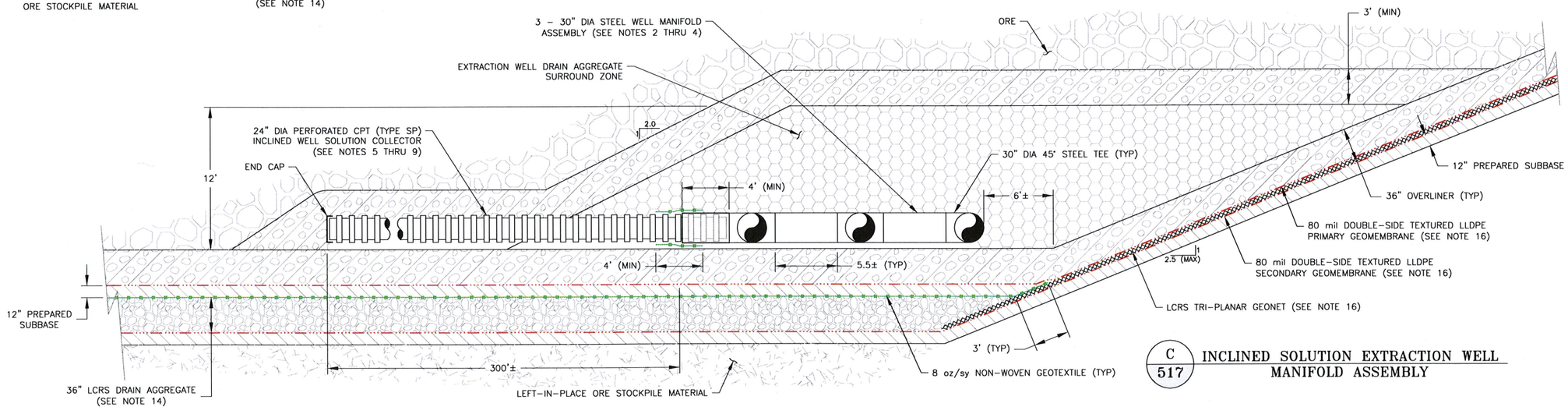



A 516 TYPICAL INCLINED SOLUTION EXTRACTION WELL AT IN-HEAP STORAGE POND EMBANKMENT CREST




B 516 TYPICAL INCLINED SOLUTION EXTRACTION WELL AT BASE OF IN-HEAP STORAGE POND




C 517 INCLINED SOLUTION EXTRACTION WELL MANIFOLD ASSEMBLY

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B SHEET).
2. 30-INCH BLANK STEEL PIPES USED IN THE WELL ASSEMBLIES SHALL BE 30-INCH O.D. PIPE FABRICATED WITH ASTM 139 HIGH STRENGTH LOW ALLOY STEEL (A606 TYPE 4). BLANK PIPE SHALL HAVE A WALL THICKNESS OF 0.562-INCHES. PIPE SHALL BE FABRICATED BY ROSCOE MOSS OR APPROVED EQUIVALENT.
3. PERFORATED PIPES USED IN THE WELL ASSEMBLIES SHALL BE 29-INCH O.D. "SUPERFLO" SHUTTER SCREEN FABRICATED USING ASTM 139 HIGH STRENGTH LOW ALLOY STELL (A606 TYPE 4). SHUTTER SCREENS SHALL BE FABRICATED BY ROSCOE MOSS OR APPROVED EQUIVALENT. PIPE WALL THICKNESS SHALL BE 0.4375-INCHES.
4. SOLUTION EXTRACTION WELL PIPE SECTIONS SHALL BE JOINED BY PENETRATION WELDS OR USING WELDED JOINT SLEEVE SEGMENTS, PLANS FOR JOINING THE WELL PIPE SEGMENTS SHALL BE SUBMITTED FOR REVIEW TO THE ENGINEER PRIOR TO INSTALLATION.
5. TWO LAYERS OF GEOTEXTILE SHALL BE WRAPPED AND SECURED OVER THE JUNCTIONS OF PERFORATED CPT INTO THE WELL MANIFOLD ASSEMBLY.
6. THE PERFORATED INCLINED WELL SOLUTION COLLECTORS SHALL BE EXTENDED APPROXIMATELY 300' ALONG THE TOES OF THE IN-HEAP POND SLOPES, FROM THE NORTHEAST CORNER OF THE POND, ON THE SURFACE OF THE OVERLINER LAYER.
7. ALL CPT PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED COUPLERS. ALL CPT PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL SOLUTION COLLECTION PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
8. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
9. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER THE 24" DIA SOLUTION COLLECTORS.
10. THE PAD PERIMETER BERM SHALL BE CONSTRUCTED WITH A 12" THICK LAYER OF COMPACTED PREPARED SUBBASE MATERIAL ALONG THE SLOPING FACE OF THE BERM INTERIOR TO THE LEACH PAD. THE UNDERLYING PORTION OF THE PAD PERIMETER BERM SHALL BE CONSTRUCTED OF COMPACTED SITE GRADING FILL UNLESS OTHERWISE AUTHORIZED BY FGMI.
11. THE EMBANKMENT CREST PERIMETER ROAD SHALL BE GRADED AWAY FROM THE LEACH PAD PERIMETER BERM BY SHAPING THE WEARING COURSE SURFACE. A MINIMUM WEARING COURSE THICKNESS OF 12" SHALL BE MAINTAINED.
12. THE STAGE 1 TERMINATION TRENCH SHALL BE BACKFILLED WITH COMPACTED PREPARED SUBBASE TO MITIGATE EXTERNAL WATERS FROM ENTERING THE LCRS.
13. THE SLOPE PORTRAYED IN SECTIONS A AND B IS THE APPROXIMATE SLOPE ALONG THE INCLINED WELL ALIGNMENT WHICH CUTS ACROSS THE NORTHEASTERN SLOPE OF THE IN-HEAP STORAGE POND.
14. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL, IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.
15. A STOPPING RING SHALL BE WELDED INSIDE THE BOTTOM OF EACH INCLINED EXTRACTION WELL TO PREVENT PUMPS FROM GETTING LODGED IN THE PIPE BENDS.
16. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.

## REVISION:

1 ADDED NOTE 16 SPECIFYING GEOMEMBRANE AND GEONET.

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| B | 12/09/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |

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ACTIVITY CODE N/A
XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: INCLINED SOLUTION EXTRACTION WELLS SECTIONS AND DETAILS, SHEET 3 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 518 | 1 |

STATE OF ALASKA 49TH
THOMAS F. KERR No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
ma. 17/17

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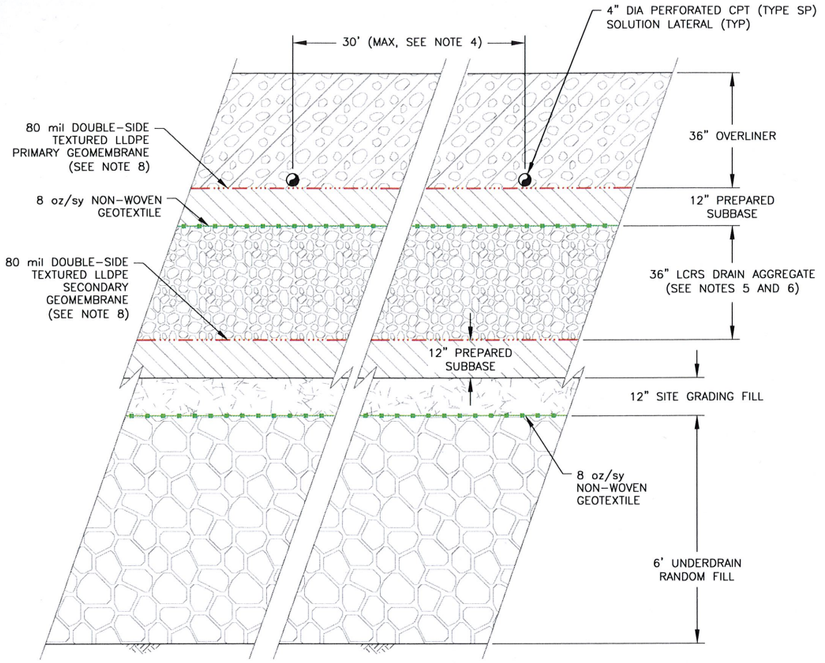
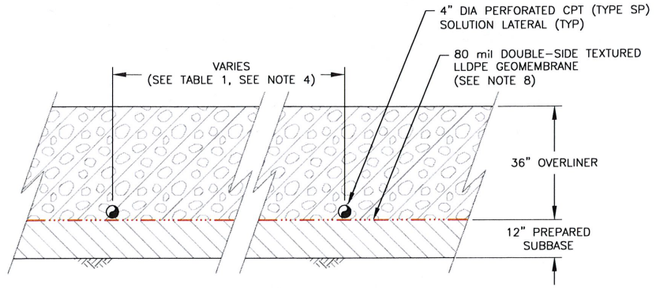
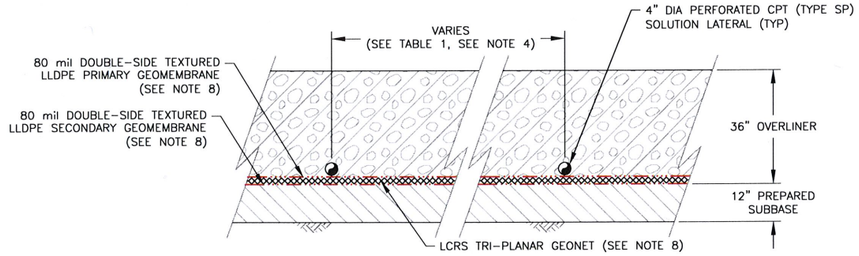



A / 500 TYPICAL IN-HEAP STORAGE POND LINER SYSTEM ON SLOPES LESS THAN 10% (SHOWN ABOVE UNDERDRAIN RANDOM FILL)



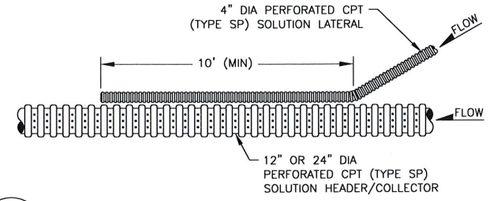

C / 500 C / 530 C / 532 TYPICAL PAD LINER SYSTEM ABOVE IN-HEAP STORAGE POND




D / 500 D / 530 D / 534 TYPICAL IN-HEAP STORAGE POND LINER SYSTEM ON SLOPES OF 10% OR GREATER

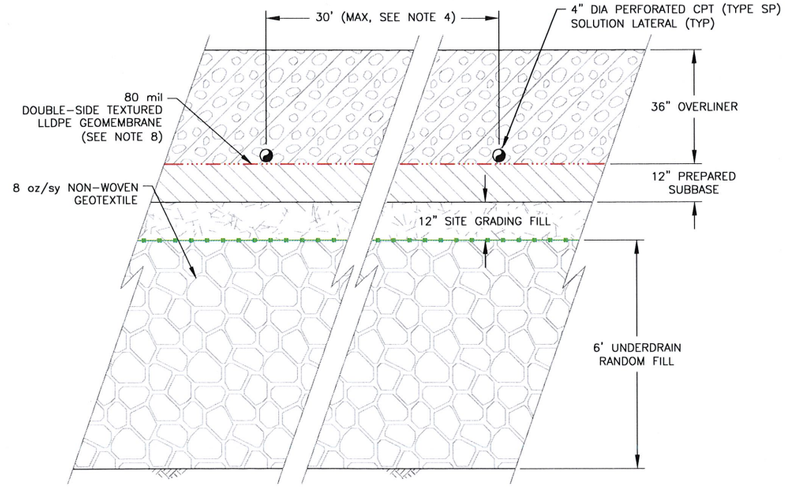
| TABLE 1 | | | |
|---|---|---|---|
| 4" DIA PERFORATED CPT SOLUTION LATERAL SPACING | | | |
| PAD SLOPES | | LOCATION | SPACING (ft) |
| < 10% | | BASE OF IN-HEAP POND | 30 |
| 5H:1V to 10H:1V | | UPPER NORTHWEST, NORTHEAST VALLEY | 35 |
| 4H:1V – 4.5H:1V | | NORTHEAST VALLEY | 55 |
| 2.5H:1V | | SOUTHWEST VALLEY | 80 |

* SEE NOTE 4

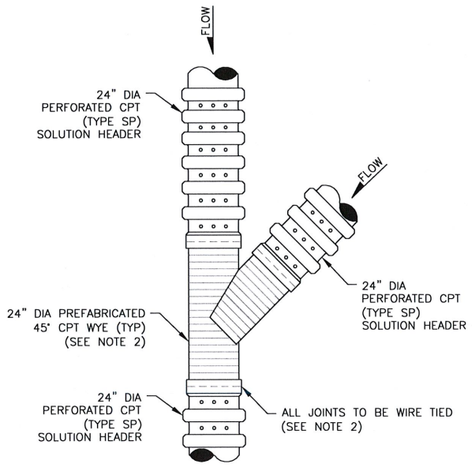



1 / 500 TYPICAL 4" DIA SOLUTION LATERAL TIE-IN TO 12" OR 24" DIA CPT SOLUTION PIPE

NTS

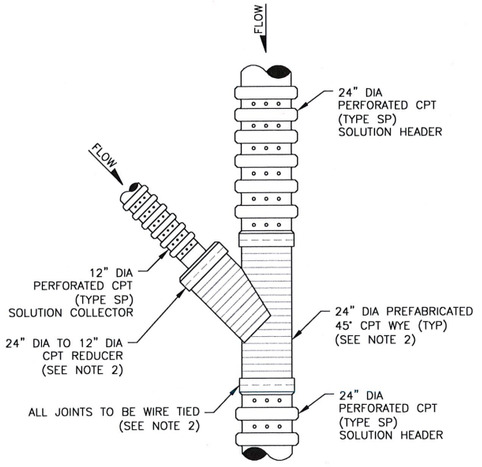



B / 500 TYPICAL PAD LINER SYSTEM OVER UNDERDRAIN RANDOM FILL ABOVE STAGE 1




2 / 500 TYPICAL DOUBLE TO SINGLE SOLUTION HEADER TRANSITION

NTS




3 / 500 TYPICAL SOLUTION COLLECTOR TO HEADER TRANSITION

NTS

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B SHEET).
2. ALL PIPE CONNECTIONS SHALL BE FORMED USING MANUFACTURER SUPPLIED TEES, WYES AND COUPLERS. ALL PIPE CONNECTIONS SHALL BE SECURED TO THE PIPE USING WIRE TIES AROUND BOTH ENDS OF THE CONNECTIONS. ALL SOLUTION COLLECTION PIPEWORK CONNECTED WITH SPLIT COUPLERS SHALL HAVE THE COUPLER SEAM ROTATED TO THE SIDE OF THE PIPE.
3. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
4. THE BARNES CREEK HEAP LEACH FACILITY CONTAINS FOUR RELATIVELY DISTINCT PAD SLOPES AS INDICATED BY THE SHADED AREAS IN PLAN ON DWG 500. EACH SLOPE HAS A CORRESPONDING SOLUTION LATERAL SPACING AS SPECIFIED IN TABLE 1.
5. WITHIN THE AREA OF THE LEACH PAD COVERED WITH AN LCRS, WHERE THE PAD SLOPES ARE LESS THAN 10%, A 36"-THICK LAYER OF LCRS DRAIN AGGREGATE, OR ALTERNATE CONFIGURATION IF PREFERRED BY FGMI (SEE NOTE 11), WILL BE INCLUDED. A SERIES OF LCRS COLLECTOR PIPES WILL BE ENCAPSULATED WITHIN THE DRAIN AGGREGATE LAYER AND WILL REPORT TO THE LCRS SUMP. SEE DRAWING 300.
6. THE 36"-THICK LAYER OF LCRS DRAIN AGGREGATE MAY BE REPLACED WITH OVERLINER MATERIAL, IF PREFERRED BY FGMI, PROVIDED THE TRI-PLANAR LCRS GEONET (IN-HEAP POND SIDE-SLOPES) IS EXTENDED BENEATH THE FLOOR OF THE IN-HEAP POND BETWEEN THE LCRS OVERLINER MATERIAL AND THE 8 oz/sy NON-WOVEN GEOTEXTILE.
7. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER 12" DIA OR GREATER CPT SOLUTION PIPES. CPT SOLUTION PIPES OF LESS THAN 12" DIA SHALL BE ENCAPSULATED WITHIN THE 36"-THICK LAYER OF OVERLINER. SEE DRAWING 235 FOR TYPICAL SECTIONS ASSOCIATED WITH THE LARGER DIAMETER SOLUTION COLLECTION HEADERS.
8. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.

## REVISION:

1 ADDED NOTE 8 SPECIFYING GEOMEMBRANE AND GEONET.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

2 0 2 4 FEET
1" = 2' AT FULL SIZE (ANSI D)
1" = 4' AT HALF SIZE (ANSI B)
(TYPICAL SCALE FOR ALL SECTIONS)

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
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ACTIVITY CODE N/A. XREF N/A.

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SOLUTION PIPEWORK AND LINER SYSTEM SECTIONS AND DETAILS

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 520 | 1 |

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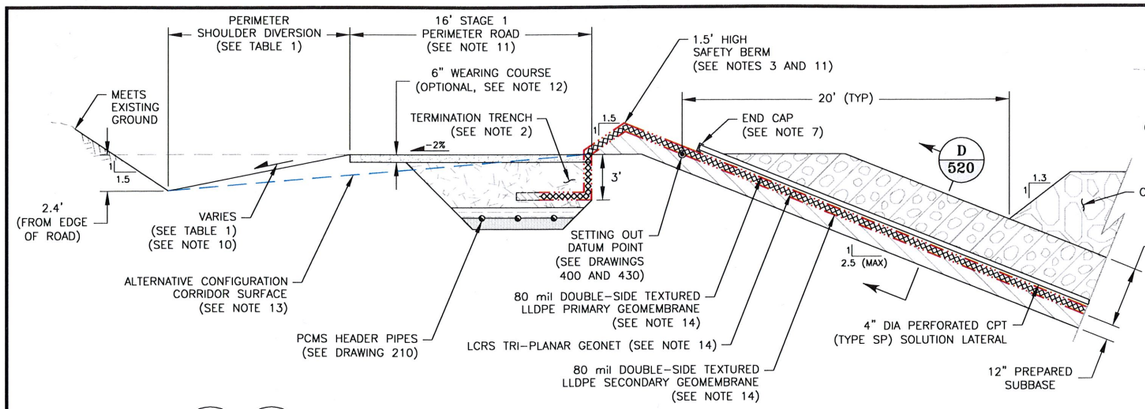
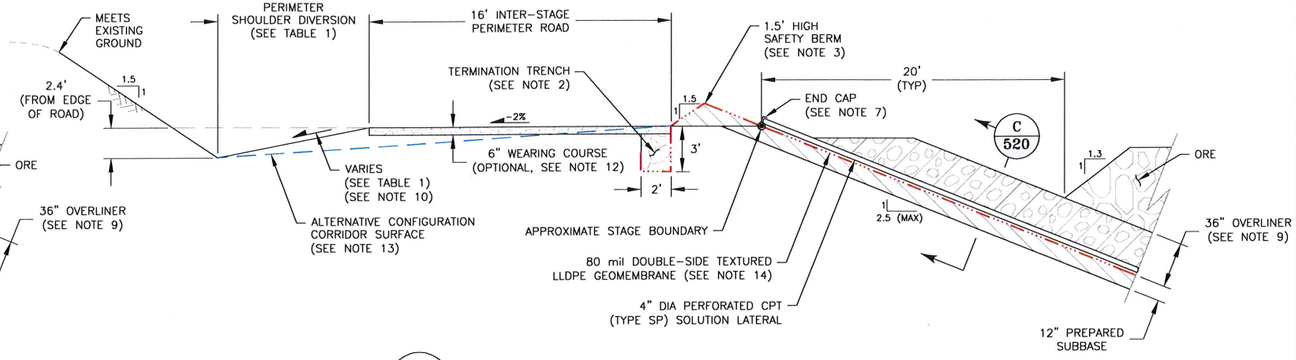
PRINTED BY: CHAD BEANLAND, PRINT TIME: 3/17/2017 4:23 PM




A/400 A/410 TYPICAL STAGE 1 NORTH PERIMETER ROAD

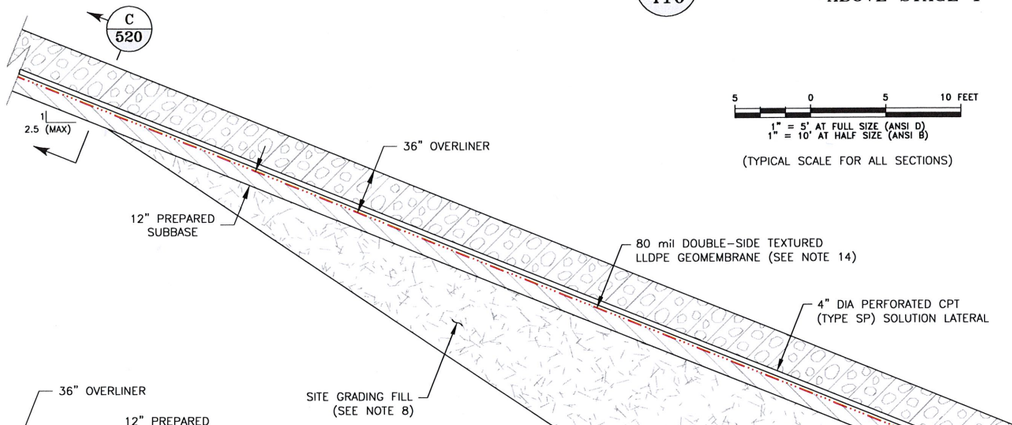
(SEE NOTE 11)




C/410 TYPICAL INTER-STAGE PERIMETER ROAD ABOVE STAGE 1

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. TERMINATION TRENCH BACKFILL SHALL TYPICALLY BE CONSTRUCTED OF COMPACTED SITE GRADING FILL. THE STAGE 1 GEOSYNTHETICS TERMINATION TRENCH ALONG THE NORTH PERIMETER ROAD SHALL TYPICALLY BE INCORPORATED AND ANCHORED INTO THE BACKFILL OF THE PCMS HEADER PIPES. A MINIMUM 6"-THICK LAYER OF COMPACTED PREPARED SUBBASE SHALL BE CONSTRUCTED OVER THE GEOSYNTHETICS TERMINATION TO MITIGATE EXTERNAL WATERS FROM ENTERING THE LCRS.
3. THE INTER-STAGE PERIMETER ROAD SAFETY BERM SHALL BE CONSTRUCTED OF PREPARED SUBBASE.
4. THE INTER-STAGE PERIMETER ROAD SHALL TYPICALLY BE GRADED AWAY FROM THE LEACH PAD AND SAFETY BERM BY SHAPING THE WEARING COURSE SURFACE. A MINIMUM WEARING COURSE THICKNESS OF 6" SHALL BE MAINTAINED. SEE NOTE 11.
5. PERIODIC MAINTENANCE OF THE INTER-STAGE PERIMETER ROAD SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
6. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
7. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
8. TO EXTEND THE VALLEY FILL LEACH PAD LINING SYSTEM, SITE GRADING FILL SHALL BE PLACED AND COMPACTED TO FORM THE LEACH PAD SURFACE. THE REQUIRED GRADING FILL WILL TYPICALLY BE GENERATED FROM CUT ASSOCIATED WITH THE SUBSEQUENT STAGE PERIMETER ROAD AND/OR BASIN GRADING.
9. THE EXTENT OF THE LEACH PAD OVERLINER LAYER, BEYOND THE FOOTPRINT OF THE OPERATIONAL PERIOD HEAP CONFIGURATION, IS INTENDED TO PROTECT THE LEACH PAD GEOMEMBRANE LINER AND ACCOMMODATE THE REGRADED HEAP AT CLOSURE (BASED ON ASSUMED HEAP CLOSURE REGRADING TO A CONSISTENT 3H:1V OVERALL SLOPE, BY OTHERS).
10. SHOULDER WIDTHS ARE INCLUDED ALONG MOST INTER-STAGE PERIMETER ROADS TO FACILITATE CONSTRUCTION AND ACCOMMODATE LARGER PEAK FLOWS ALONG THE ROAD CORRIDOR. THE INTER-STAGE ROADS WILL TYPICALLY BE GRADED DOWN FROM THE HIGH POINTS NEAR THE UPPER END OF EACH STAGE PERIMETER AS SHOWN ON DRAWING 140. ALONG INTER-STAGE ROADS FLOWING TO THE NORTH, THE ROAD SHOULDERS WILL TYPICALLY BE SLOPED AT 20 PERCENT AWAY FROM THE ROAD, WHILE ALONG THE INTER-STAGE ROADS FLOWING TO THE SOUTH THE SHOULDERS WILL TYPICALLY BE SHAPED AS AN EXTENSION OF THE ROAD 2-PERCENT CROSS-FALL AWAY FROM THE PAD PERIMETER. SHOULDER WIDTHS WILL VARY AS SHOWN IN TABLE 1. SEE NOTES 11 AND 13.
11. THE STAGE 1 INTER-STAGE PERIMETER ROAD, ALONG THE SOUTHWEST SIDE OF THE IN-HEAP POND, SHALL BE CONSTRUCTED WITHOUT A SAFETY BERM AND THE ROAD SHALL BE GRADED WITH CROSS-FALL TOWARD THE IN-HEAP POND.
12. WEARING COURSE IS INTENDED TO IMPROVE THE TRAFFICABILITY OF THE INTER-STAGE ROAD SURFACE; HOWEVER, PLACEMENT OF WEARING COURSE ALONG THE INTER-STAGE PERIMETER ROADS IS OPTIONAL AND MAY BE REMOVED IF PREFERRED BY FGMI. WEARING COURSE MATERIAL SHALL NOT BE PLACED ALONG THE INTER-STAGE PERIMETER ROADS WITHOUT CONFIRMATION FROM FGMI.
13. THE ALTERNATE CONFIGURATION OF THE INTER-STAGE PERIMETER CORRIDOR MAY BE ADOPTED IF PREFFERED BY FGMI. THE CONSTANT CROSS-FALL OF THE ALTERNATE CONFIGURATION SURFACE IS INTENDED TO FACILITATE MAINTENANCE OF THE PERIMETER CORRIDORS, BUT WILL AFFECT TRAFFICABILITY ALONG THE CORRIDORS. THE ALTERNATE CONFIGURATIONS FOR THE STAGES 1 THRU 3 NORTH PERIMETER CORRIDORS WILL RESULT IN CROSS-FALLS OF APPROXIMATELY 9% TO 10%. ALONG THE STAGE 1 NORTH PERIMETER CORRIDOR, THE ALTERNATE CONFIGURATION SHALL NOT BE INCORPORATED WITHIN 10' OF THE PCMS SUMP LOCATIONS.
14. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT.




5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

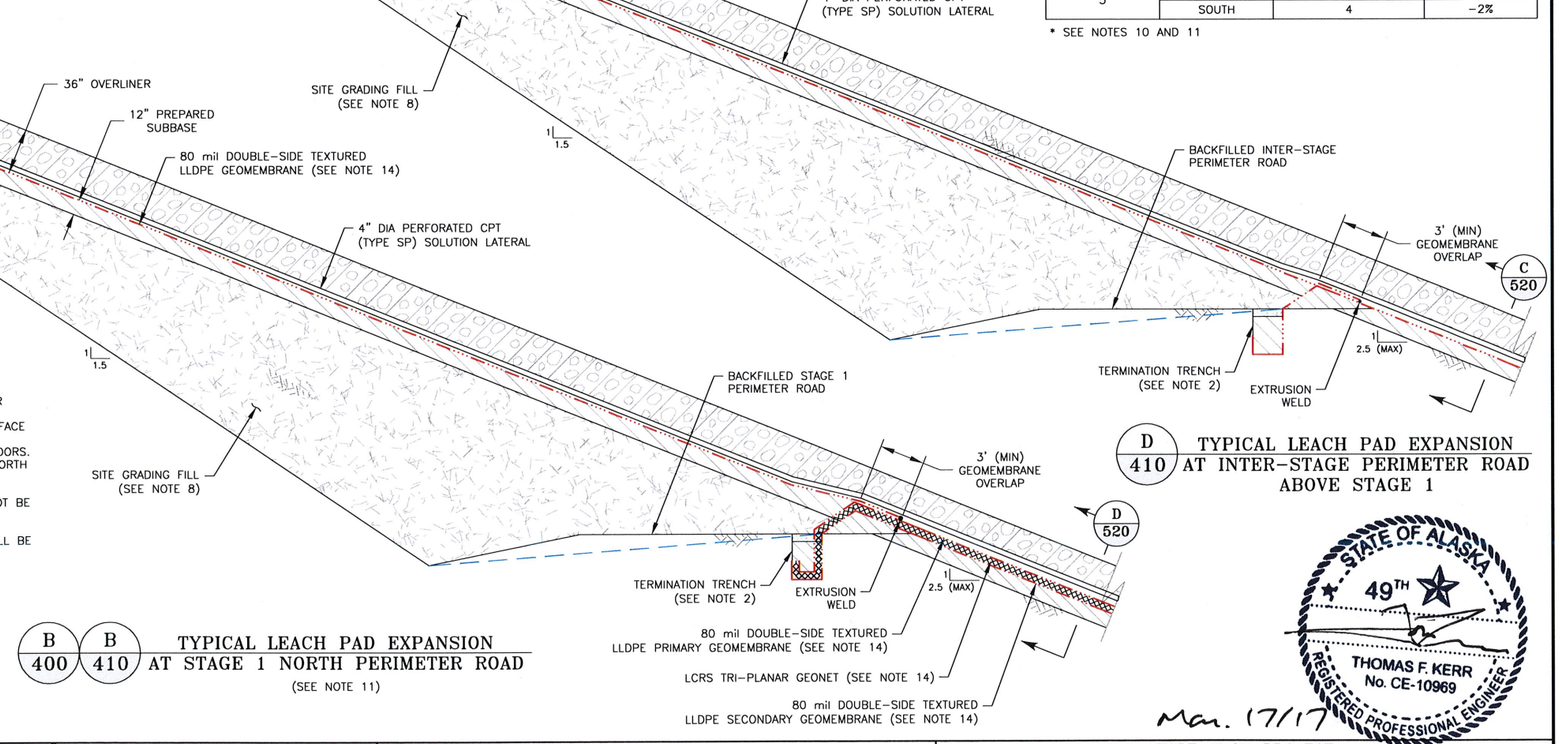
(TYPICAL SCALE FOR ALL SECTIONS)

**TABLE 1**

**INTER-STAGE PERIMETER SHOULDER DIVERSION DESIGN DATA**

| STAGE | SIDE | SHOULDER WIDTH * (ft) | SHOULDER SLOPE * |
|---|---|---|---|
| 1 | NORTH | 12 | -20% |
| | SOUTH | NA | NA |
| 2 | NORTH | 11 | -20% |
| | SOUTH | 4 | -2% |
| 3 | NORTH | 9 | -20% |
| | SOUTH | 4 | -2% |
| 4 | NORTH | 4 | -2% |
| | SOUTH | 4 | -2% |
| 5 | NORTH | 4 | -2% |
| | SOUTH | 4 | -2% |

* SEE NOTES 10 AND 11




D/410 TYPICAL LEACH PAD EXPANSION AT INTER-STAGE PERIMETER ROAD ABOVE STAGE 1

B/400 B/410 TYPICAL LEACH PAD EXPANSION AT STAGE 1 NORTH PERIMETER ROAD

(SEE NOTE 11)

## REVISION:

1. ADDED NOTE 14 SPECIFYING GEOMEMBRANE AND GEONET.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR CB |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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: PAD PERIMETER
SECTIONS AND DETAILS, SHEET 1 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 530 | 1 |

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17


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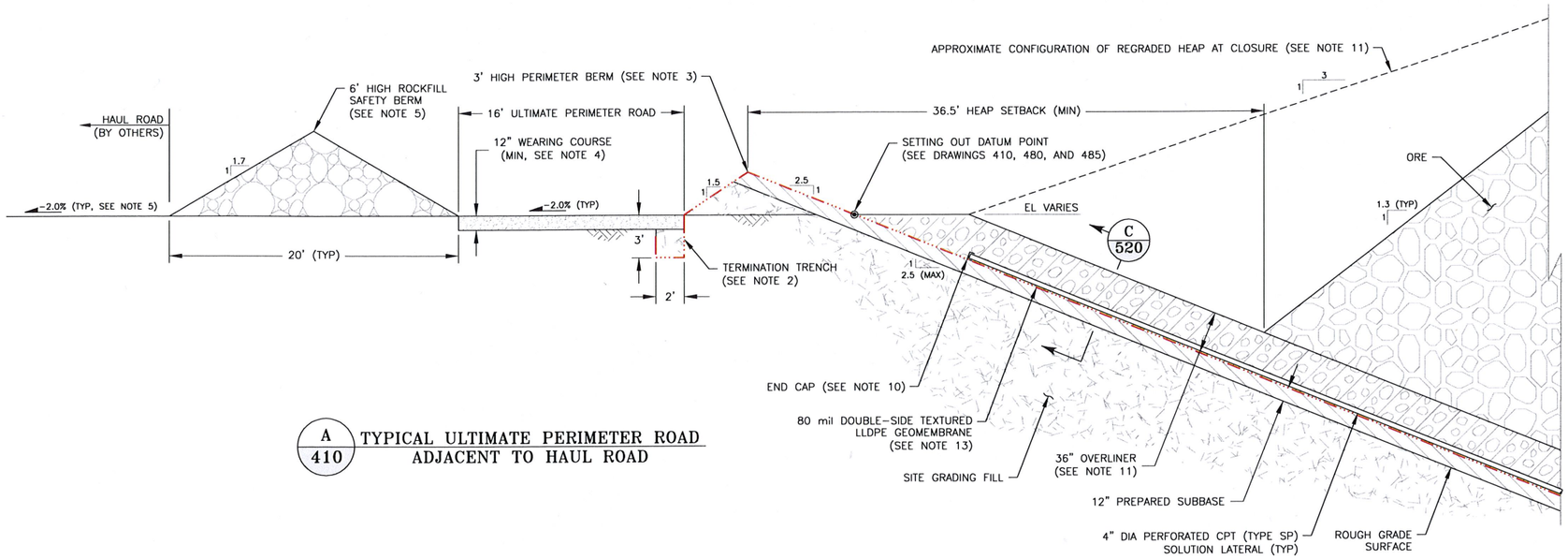
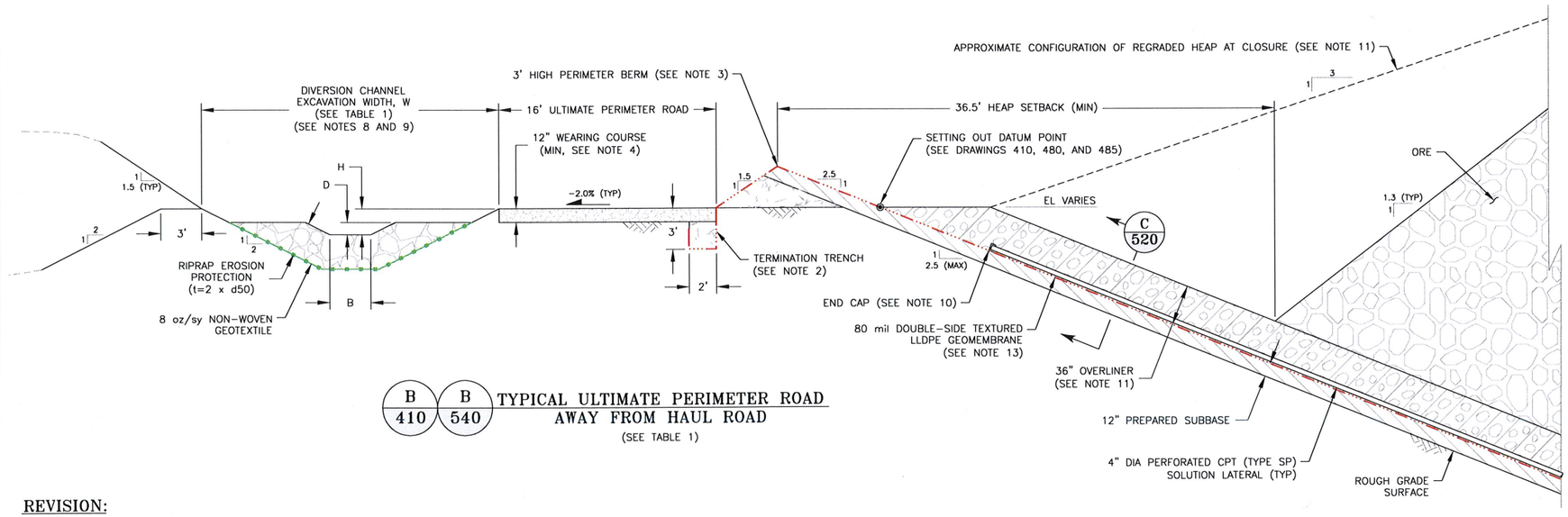



A 410 TYPICAL ULTIMATE PERIMETER ROAD ADJACENT TO HAUL ROAD




B 410 B 540 TYPICAL ULTIMATE PERIMETER ROAD AWAY FROM HAUL ROAD (SEE TABLE 1)

**TABLE 1** ⚠1

**ULTIMATE PERIMETER DIVERSION CHANNEL DESIGN DATA**

| STATION | EXCAVATION TOP WIDTH W (ft) | BOTTOM WIDTH B (ft) | CHANNEL DEPTH H (ft) | RIPRAP DEPTH D (ft) | RIPRAP THICKNESS t (ft) | RIPRAP d50 (in) |
|---|---|---|---|---|---|---|
| SOUTH PERIMETER DIVERSION CHANNEL (SEE NOTE 12) | | | | | | |
| 10+81* TO 24+52 | 29.4 | 4.0 | 3.0 | 2.0 | 3.0 | 18.0 |
| 24+57 TO 55+12 | 28.4 | 3.0 | 3.0 | 2.0 | 3.0 | 18.0 |
| 55+12 TO 63+25 | 17.7 | 3.0 | 2.0 | 1.0 | 1.5 | 9.0 |
| NORTH PERIMETER DIVERSION CHANNEL ** | | | | | | |
| 63+25 TO 69+76 | 20.7 | 2.0 | 3.0 | 2.0 | 1.5 | 9.0 |
| 69+81 TO 75+66 | 22.1 | 3.0 | 3.1 | 2.1 | 1.5 | 9.0 |
| 75+66 TO 81+33 | 21.7 | 3.0 | 3.0 | 2.0 | 1.5 | 9.0 |
| 81+38 TO 94+39 | 30.4 | 5.0 | 3.0 | 2.0 | 3.0 | 18.0 |
| 94+39 TO 112+65 | 23.7 | 5.0 | 3.0 | 2.0 | 1.5 | 9.0 |
| 112+68 TO 119+96 | 29.4 | 6.0 | 2.5 | 1.5 | 3.0 | 18.0 |
| 119+99 TO 123+66 | 30.4 | 7.0 | 2.5 | 1.5 | 3.0 | 18.0 |
| 123+69 TO 127+61 | 31.4 | 8.0 | 2.5 | 1.5 | 3.0 | 18.0 |

* STATION 10+81 REPRESENTS THE SOUTH PERIMETER DIVERSION CULVERT INLET.

** THE REQUIRED DIMENSIONS AND EROSION PROTECTION FOR THE END STATIONING OF THE NORTH PERIMETER DIVERSION CHANNEL SHALL BE MAINTAINED TO ITS CONFLUENCE WITH THE EMERGENCY SPILLWAY OUTLET CHANNEL.

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. TERMINATION TRENCH BACKFILL SHALL TYPICALLY BE CONSTRUCTED OF COMPACTED SITE GRADING FILL.
3. THE PAD PERIMETER BERM SHALL BE CONSTRUCTED WITH A 12" THICK LAYER OF COMPACTED PREPARED SUBBASE MATERIAL ALONG THE SLOPING FACE OF THE BERM INTERIOR TO THE LEACH PAD. THE UNDERLYING PORTION OF THE PAD PERIMETER BERM SHALL BE CONSTRUCTED OF COMPACTED SITE GRADING FILL UNLESS OTHERWISE AUTHORIZED BY FGMI.
4. THE ULTIMATE PERIMETER ROAD SHALL BE GRADED AWAY FROM THE LEACH PAD PERIMETER BERM BY SHAPING THE WEARING COURSE SURFACE. A MINIMUM WEARING COURSE THICKNESS OF 12" SHALL BE MAINTAINED.
5. THE SAFETY BERM SEPARATING THE PERIMETER ROAD AND HAUL ROAD SHALL BE CONSTRUCTED OF PERVIOUS ROCKFILL TO PROMOTE SURFACE RUNOFF THROUGH THE BERM AND AWAY FROM THE LEACH PAD. THE HAUL ROAD SHALL BE GRADED AWAY FROM THE ROCKFILL SAFETY BERM.
6. PERIODIC MAINTENANCE OF THE PAD PERIMETER ROAD AND DIVERSION CHANNEL SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
7. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
8. WHERE THE ULTIMATE PERIMETER DIVERSION CHANNEL EXCAVATION ENCOUNTERS COMPETENT ROCK, AS DETERMINED BY THE ENGINEER, RIPRAP EROSION PROTECTION MAY NOT BE REQUIRED. NO ADDITIONAL CHANNEL EXCAVATION, BEYOND THE REQUIRED FLOW CROSS-SECTION (I.E. CHANNEL BOTTOM WIDTH, B, AND CHANNEL DEPTH, H), WILL BE CONDUCTED WITHOUT PRIOR AUTHORIZATION FROM THE ENGINEER AND/OR FGMI. REFER TO TABLE 1 FOR THE CHANNEL DIMENSION REQUIREMENTS.
9. ANY PORTIONS OF THE CHANNEL CROSS-SECTION CONSTRUCTED IN FILL SHALL INCLUDE A MINIMUM CREST WIDTH OF 3'.
10. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
11. THE EXTENT OF THE LEACH PAD OVERLINER LAYER, BEYOND THE FOOTPRINT OF THE OPERATIONAL PERIOD HEAP CONFIGURATION, IS INTENDED TO PROTECT THE LEACH PAD GEOMEMBRANE LINER AND ACCOMMODATE THE REGRADED HEAP AT CLOSURE (BASED ON ASSUMED HEAP CLOSURE REGRADING TO A CONSISTENT 3H:1V OVERALL SLOPE, BY OTHERS).
12. BETWEEN THE SOUTH PERIMETER DIVERSION CULVERT AND STA 63+25, THE SOUTH CHANNEL IS TO BE INCORPORATED INTO THE UPSTREAM SIDE OF THE EXISTING HAUL ROAD.
13. ⚠1 ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT.

## REVISION:

⚠1 REMOVED SOUTH PERIMETER OUTLET CHANNEL. ADDED NOTE 13 SPECIFYING GEOMEMBRANE.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

(TYPICAL SCALE FOR ALL SECTIONS)

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: PAD PERIMETER
SECTIONS AND DETAILS, SHEET 2 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 532 | 1 |

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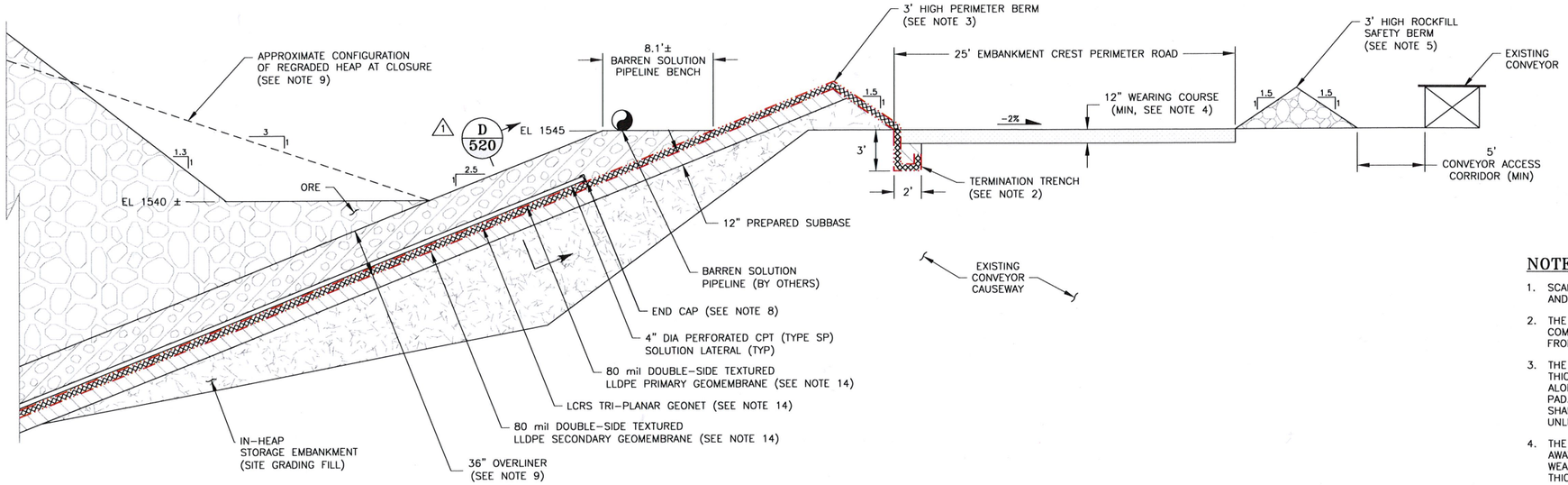
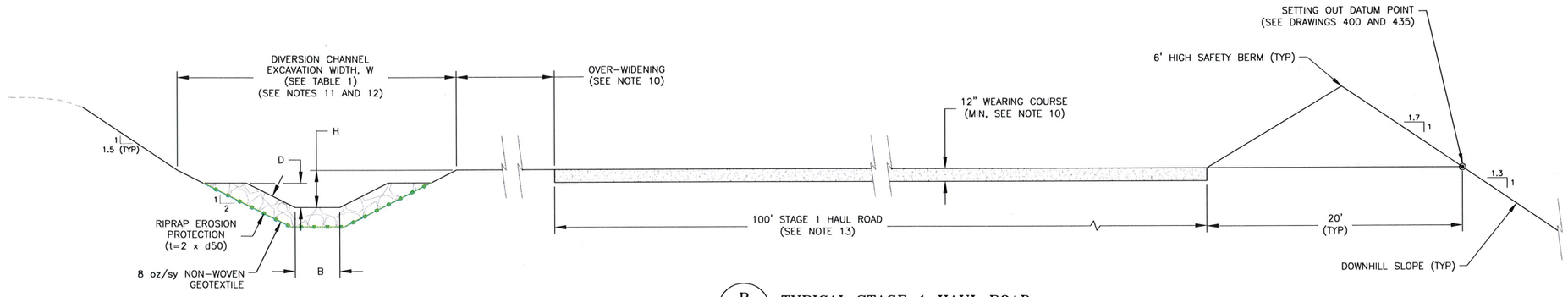



A/400 A/410 TYPICAL LEACH PAD PERIMETER ROAD ALONG IN-HEAP STORAGE POND EMBANKMENT CREST

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)




B/400 TYPICAL STAGE 1 HAUL ROAD

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

| TABLE 1 | | | | | | |
|---|---|---|---|---|---|---|
| STAGE 1 HAUL ROAD DIVERSION CHANNEL DESIGN DATA | | | | | | |
| STATION | EXCAVATION TOP WIDTH W (ft) | BOTTOM WIDTH B (ft) | CHANNEL DEPTH H (ft) | RIPRAP DEPTH D (ft) | RIPRAP THICKNESS t (ft) | RIPRAP d50 (in) |
| 3+00 TO 24+73 | 19.7 | 3.0 | 2.5 | 1.5 | 1.5 | 9.0 |
| 24+73 TO 29+05 | 20.7 | 4.0 | 2.5 | 1.5 | 1.5 | 9.0 |

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE STAGE 1 TERMINATION TRENCH SHALL BE BACKFILLED WITH COMPACTED PREPARED SUBBASE TO MITIGATE EXTERNAL WATERS FROM ENTERING THE LCRS.
3. THE PAD PERIMETER BERM SHALL BE CONSTRUCTED WITH A 12" THICK LAYER OF COMPACTED PREPARED SUBBASE MATERIAL ALONG THE SLOPING FACE OF THE BERM INTERIOR TO THE LEACH PAD. THE UNDERLYING PORTION OF THE PAD PERIMETER BERM SHALL BE CONSTRUCTED OF COMPACTED SITE GRADING FILL UNLESS OTHERWISE AUTHORIZED BY FGMI.
4. THE EMBANKMENT CREST PERIMETER ROAD SHALL BE GRADED AWAY FROM THE LEACH PAD PERIMETER BERM BY SHAPING THE WEARING COURSE SURFACE. A MINIMUM WEARING COURSE THICKNESS OF 12" SHALL BE MAINTAINED.
5. THE SAFETY BERM SEPARATING THE PERIMETER ROAD AND CONVEYOR ACCESS CORRIDOR SHALL BE CONSTRUCTED OF PERVIOUS ROCKFILL TO PROMOTE SURFACE RUNOFF THROUGH THE BERM AND AWAY FROM THE LEACH PAD.
6. PERIODIC MAINTENANCE OF THE PERIMETER ROAD SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
7. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
8. WHERE CPT SOLUTION PIPES TERMINATE, THEY SHALL BE FITTED WITH MANUFACTURER SUPPLIED END CAPS. ALL PIPEWORK, INCLUDING TERMINATION POINTS, SHALL BE SURVEYED FOR FUTURE IDENTIFICATION.
9. THE EXTENT OF THE LEACH PAD OVERLINER LAYER, BEYOND THE FOOTPRINT OF THE OPERATIONAL PERIOD HEAP CONFIGURATION, IS INTENDED TO PROTECT THE LEACH PAD GEOMEMBRANE LINER AND ACCOMMODATE THE REGRADED HEAP AT CLOSURE (BASED ON ASSUMED HEAP CLOSURE REGRADING TO A CONSISTENT 3H:1V OVERALL SLOPE, BY OTHERS).
10. THE STAGE 1 HAUL ROAD WEARING COARSE DESIGN IS BY OTHERS.
11. THE STAGE 1 HAUL ROAD DIVERSION CHANNEL WILL BE CONSTRUCTED ADJACENT TO THE UPSTREAM SLOPE. WHERE THE ROAD IS ENTIRELY IN FILL, THE ROADWAY PLATFORM WILL BE OVER-WIDENED SUCH THAT RUNOFF FROM THE HAUL ROAD FLOWS INTO THE CHANNEL.
12. BEGINNING AT STATION 25+87, THE STAGE 1 HAUL ROAD DIVERSION CHANNEL BOTTOM SLOPE IS 0.8%, DIMINISHING THE CHANNEL DEPTH TO ZERO BY STATION 28+37. DOWNSTREAM OF THIS POINT, THE FLOWS WILL BE CONVEYED IN A STEEPENED SHOULDER ALONG THE HAUL ROAD AND DISCHARGE INTO THE STAGE 1 PERIMETER ROAD FLOWING NORTH.
13. BEYOND STATION 3+20, THE STAGE 1 HAUL ROAD WILL BE DECOMMISSIONED TO CONSTRUCT THE STAGE 2 PAD AREA.
14. ALL 80-mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE SHALL BE GSE OR SOLMAX STANDARD PRODUCT, OR ENGINEER APPROVED EQUIVALENT. ALL TRI-PLANAR LCRS GEONET SHALL BE GSE TENDRAIN 300, OR ENGINEER APPROVED EQUIVALENT. 2

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

## REVISION:

1 ADDED CALLOUT FOR D/520.

2 ADDED NOTE 14 SPECIFYING GEOMEMBRANE AND GEONET.

| REV | DATE | DESCRIPTION | APP'D / CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
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CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: PAD PERIMETER
SECTIONS AND DETAILS, SHEET 3 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
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| DRAWN BY | CB | DV101 | 00089.99 | 534 | 2 |


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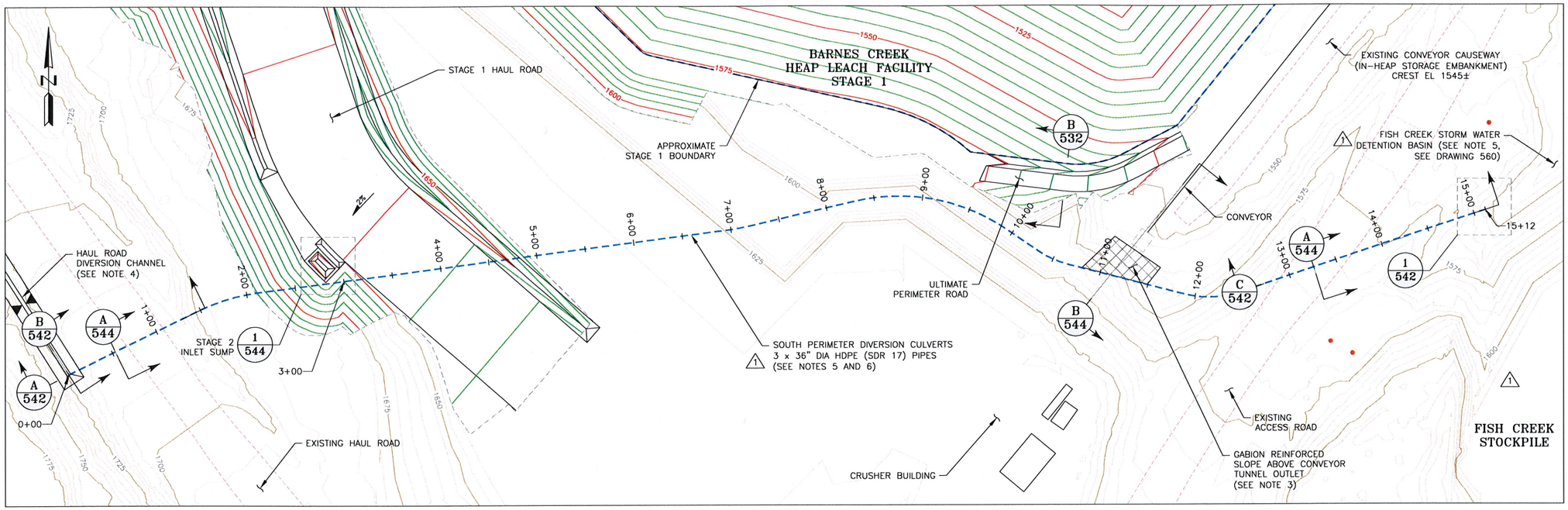



REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

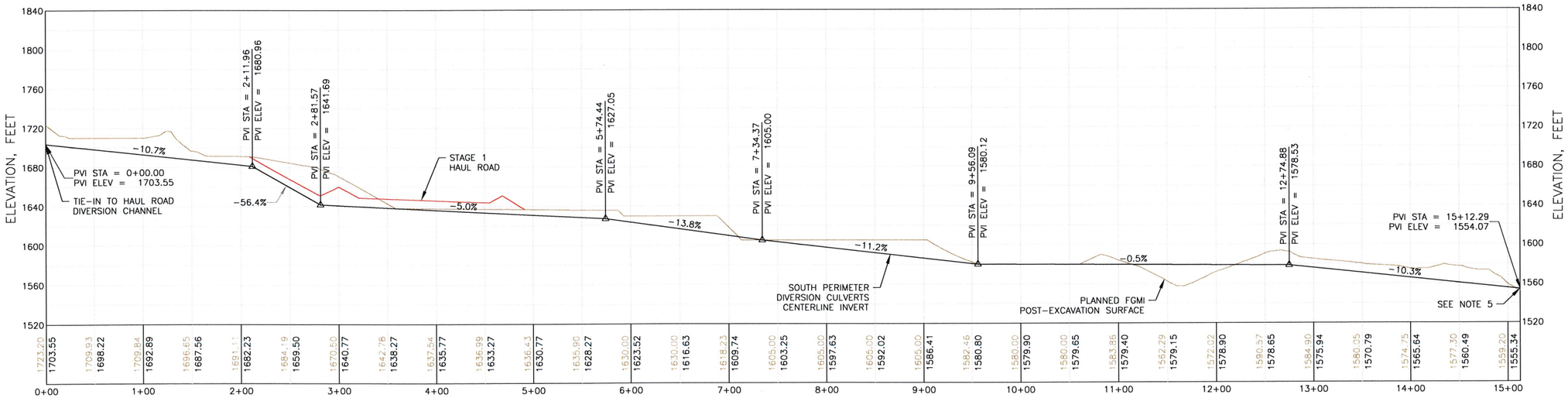
PLAN

60 0 60 120 FEET

1" = 60' AT FULL SIZE (ANSI D)
1" = 120' AT HALF SIZE (ANSI B)

## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET (SEE NOTE 2)
- EXISTING DRAINAGE/DIVERSION
- EXISTING ROAD
- UTILITY POLE (SEE NOTE 7)
- 2119.57 — PLANNED FGMI POST-EXCAVATION GROUND SURFACE ELEVATION, FEET
- 2117.76 — SOUTH PERIMETER DIVERSION CULVERTS DATUM LINE ELEVATION, FEET
- PVI — POINT OF VERTICAL INTERSECTION




PROFILE

60 0 60 120 FEET

1" = 60' AT FULL SIZE (ANSI D)
1" = 120' AT HALF SIZE (ANSI B)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF ROAD SURFACES AND BOTTOM OF EROSION PROTECTION FOR ALL DIVERSION CHANNELS.
3. GABION REINFORCEMENT WILL BE UTILIZED TO SUPPORT THE CULVERT CORRIDOR REQUIRED ABOVE THE CONVEYOR TUNNEL OUTLET FROM BENEATH THE CRUSHER BUILDING. REFER TO DRAWING 544 FOR MORE INFORMATION.
4. REFER TO TABLE 1 ON DRAWING 532 FOR THE HAUL ROAD DIVERSION DESIGN DATA.
5. (1) THE SOUTH PERIMETER DIVERSION CULVERTS SHALL DISCHARGE INTO THE SOUTHWESTERN END OF THE FISH CREEK DETENTION BASIN. THE CULVERT OUTLET TRANSITION TO THE FISH CREEK BASIN SHALL BE FIELD FIT.
6. REFER TO DRAWING 544 FOR THE SOUTH PERIMETER DIVERSION CULVERTS HORIZONTAL CONTROL DATA.
7. PERIODIC MAINTENANCE OF THE CULVERT INLETS AND OUTLETS SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
8. EXISTING POWER LINES SHALL BE RELOCATED AS REQUIRED BY FGMI.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

## REVISION:

(1) REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY. REPLACED NOTE 5.

| REV | DATE | DESCRIPTION | APP'D / CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
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XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SOUTH PERIMETER DIVERSION CULVERTS PLAN AND PROFILE

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| DRAWN BY | CB | DV101 | 00089.99 | 540 | 1 |

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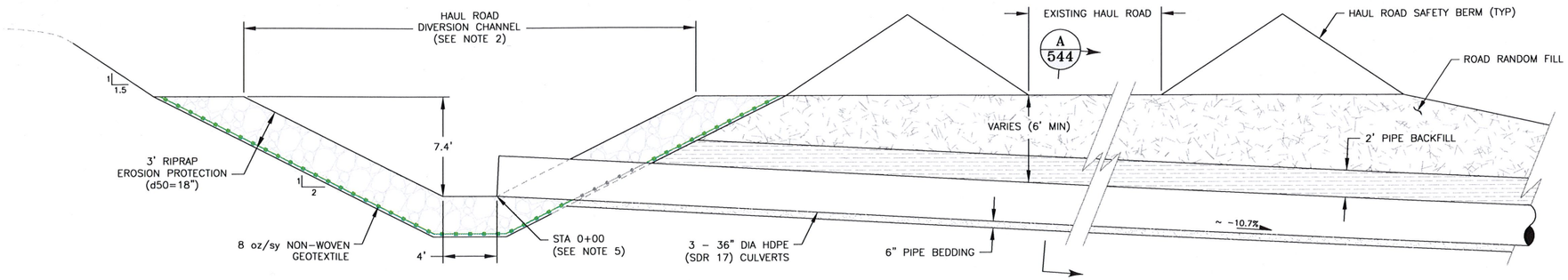
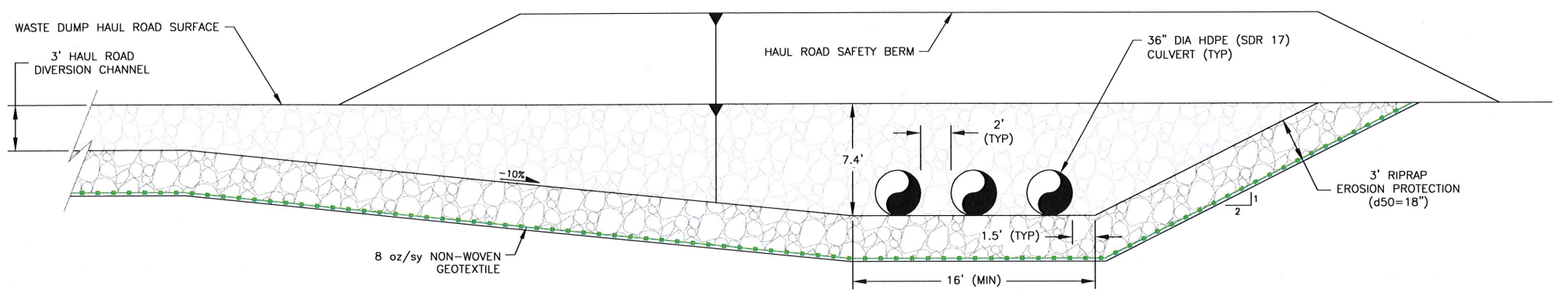


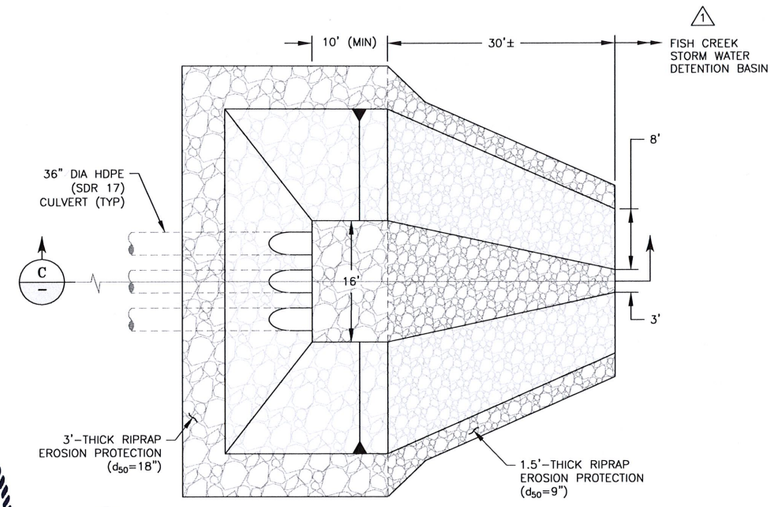

A/540 UPSTREAM INLET TO SOUTH PERIMETER DIVERSION CULVERTS
LONGITUDINAL SECTION
(SEE NOTE 4)

**NOTES:**

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. REFER TO TABLE 1 ON DRAWING 532 FOR THE CHANNEL SIZING DATA.
3. THE SOUTH PERIMETER DIVERSION CULVERTS SHALL DISCHARGE INTO THE SOUTHWESTERN END OF THE FISH CREEK DETENTION BASIN. THE CULVERT OUTLET TRANSITION TO THE FISH CREEK BASIN SHALL BE FIELD FIT. (1)
4. PERIODIC MAINTENANCE OF THE CULVERT INLETS AND OUTLETS SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
5. THE SETTING OUT DATA REPRESENTS THE INVERT OF THE CENTER PIPE. REFER TO TABLE 1 ON DRAWING 544 FOR THE HORIZONTAL CONTROL DATA AND THE PROFILE ON DRAWING 540 FOR THE VERTICAL CONTROL DATA.

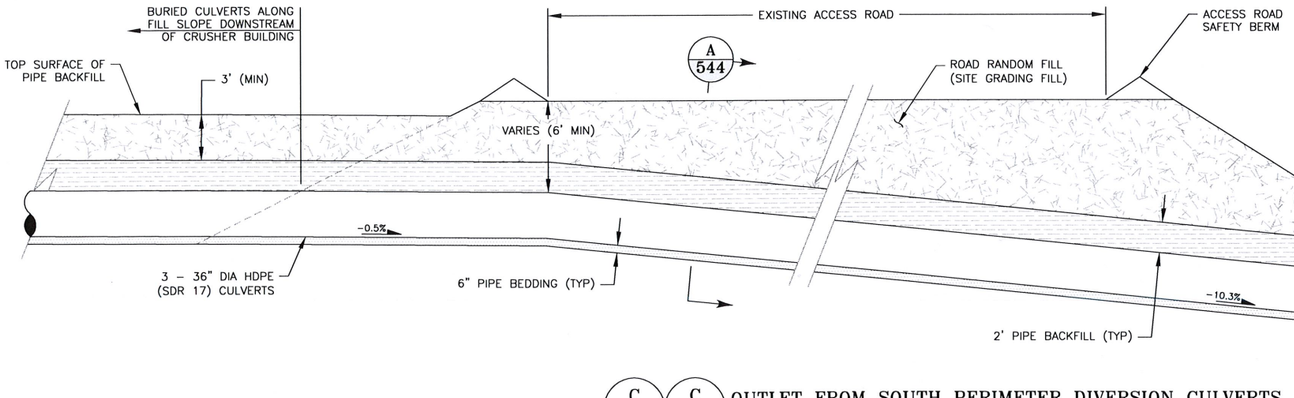



B/540 UPSTREAM INLET TO SOUTH PERIMETER DIVERSION CULVERTS
TRANSVERSE SECTION
(SEE NOTE 4)

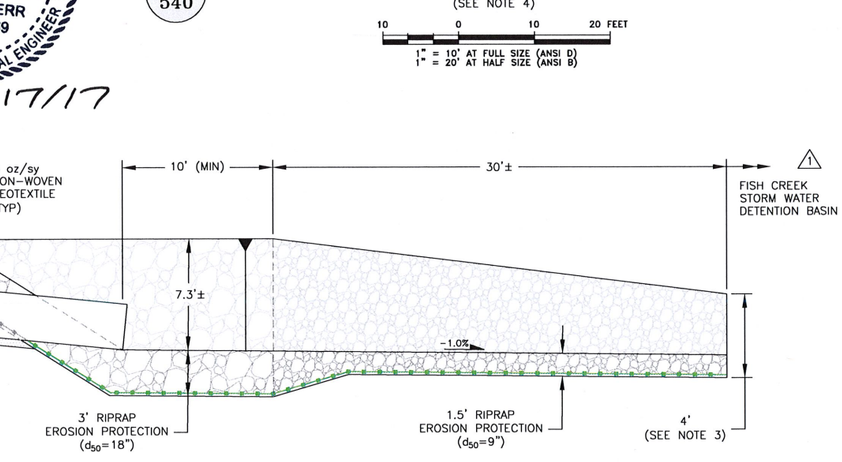



1/540 OUTLET FROM SOUTH PERIMETER DIVERSION CULVERTS
(SEE NOTE 4)

10 0 10 20 FEET
1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)




C/- C/540 OUTLET FROM SOUTH PERIMETER DIVERSION CULVERTS
LONGITUDINAL SECTION
(SEE NOTE 4)




STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

**REVISION:**

(1) REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED OUTLET FROM SOUTH PERIMETER DIVERSION CULVERTS. REPLACED NOTE 3.

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

| REV | DATE | DESCRIPTION | APP'D / CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SOUTH PERIMETER DIVERSION CULVERTS
SECTIONS AND DETAILS, SHEET 1 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 542 | 1 |

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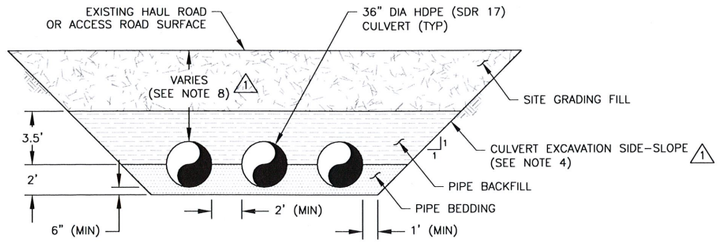
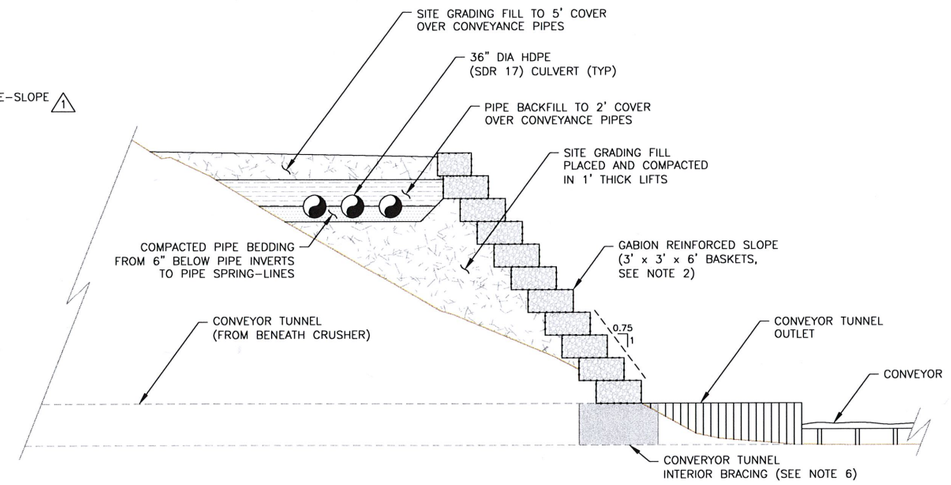


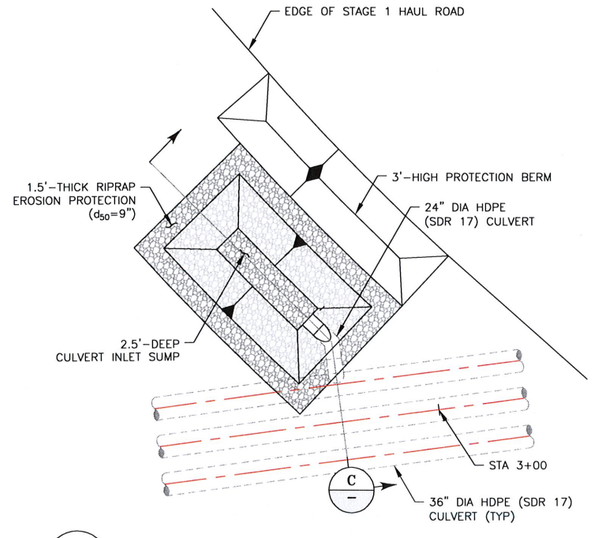

A/540 A/542 TYPICAL EXCAVATION AND BACKFILL FOR SOUTH PERIMETER DIVERSION CULVERTS
(SEE NOTES 5 AND 7)

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)



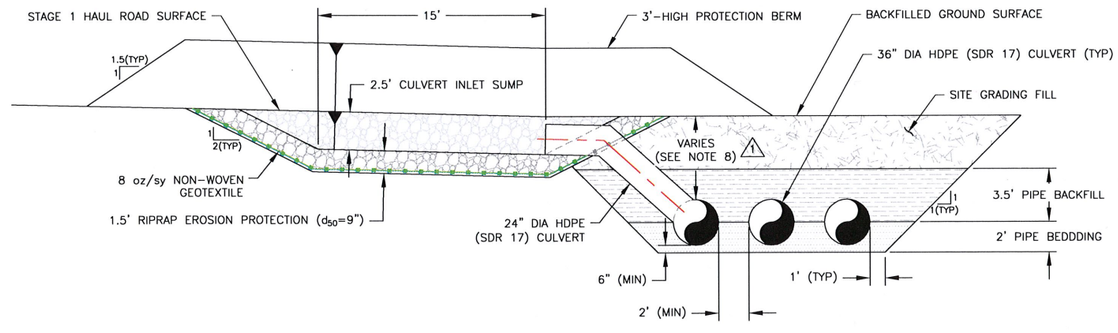

B/540 CONVEYANCE PIPES ON REINFORCED STEEPENED SLOPE ADJACENT TO CRUSHER BUILDING
(SEE NOTES 5 AND 7)

10 0 10 20 FEET
1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)




1/540 STAGE 2 CULVERT INLET SUMP TO SOUTH PERIMETER DIVERSION CULVERTS

10 0 10 20 FEET
1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)




C/- STAGE 2 INLET TO SOUTH PERIMETER DIVERSION CULVERTS
(SEE NOTES 5 AND 7)

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

TABLE 1

SOUTH PERIMETER DIVERSION CULVERTS
HORIZONTAL CONTROL DATA

| ID | DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|---|
| L1 | | 0+00.0 | 12,833.67 | 8,698.81 | | | |
| | | 1+48.5 | 12,898.83 | 8,832.29 | | | |
| C1 | PC | 1+48.5 | 12,898.83 | 8,832.29 | 200.0 | 18°34' | 28°57' |
| | PI | 1+81.2 | 12,913.17 | 8,861.67 | | | |
| | RP | | 12,719.10 | 8,920.03 | | | |
| | PT | 2+13.4 | 12,917.41 | 8,894.09 | | | |
| L2 | | 2+13.4 | 12,917.41 | 8,894.09 | | | |
| | | 6+98.0 | 12,980.27 | 9,374.60 | | | |
| C2 | PC | 6+98.0 | 12,980.27 | 9,374.60 | 200.0 | 5°37' | 28°57' |
| | PI | 7+07.8 | 12,981.55 | 9,384.32 | | | |
| | RP | | 13,178.58 | 9,348.66 | | | |
| | PT | 7+17.6 | 12,983.76 | 9,393.88 | | | |
| L3 | | 7+17.6 | 12,983.76 | 9,393.88 | | | |
| | | 8+29.6 | 13,009.09 | 9,503.00 | | | |
| C3 | PC | 8+29.6 | 13,009.09 | 9,503.00 | 200.0 | 47°18' | 28°57' |
| | PI | 9+17.2 | 13,028.89 | 9,588.31 | | | |
| | RP | | 12,814.27 | 9,548.22 | | | |
| | PT | 9+94.7 | 12,979.63 | 9,660.71 | | | |
| L4 | | 9+94.7 | 12,979.63 | 9,660.71 | | | |
| | | 10+32.0 | 12,958.61 | 9,691.62 | | | |
| C4 | PC | 10+32.0 | 12,958.61 | 9,691.62 | 100.0 | 19°44' | 60°00' |
| | PI | 10+49.4 | 12,948.83 | 9,706.00 | | | |
| | RP | | 13,041.29 | 9,747.87 | | | |
| | PT | 10+66.5 | 12,944.47 | 9,722.85 | | | |
| L5 | | 10+66.5 | 12,944.47 | 9,722.85 | | | |
| | | 11+88.8 | 12,913.87 | 9,841.27 | | | |
| C5 | PC | 11+88.8 | 12,913.87 | 9,841.27 | 100.0 | 32°45' | 60°00' |
| | PI | 12+18.2 | 12,906.52 | 9,869.73 | | | |
| | RP | | 13,010.69 | 9,866.29 | | | |
| | PT | 12+46.0 | 12,915.73 | 9,897.63 | | | |
| L6 | | 12+46.0 | 12,915.73 | 9,897.63 | | | |
| | | 15+12.3 | 12,999.18 | 10,150.53 | | | |

NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. GABION REINFORCEMENT WILL BE UTILIZED TO SUPPORT THE FILL SLOPE OF THE CULVERT CROSSING REQUIRED ABOVE THE CONVEYOR TUNNEL OUTLET FROM BENEATH THE CRUSHER BUILDING. THREE-FOOT (3') BY 3' BY 6' GABION CAGES BACKFILLED WITH DRAIN ROCK ARE ENVISIONED.
3. REFER TO TABLE 1 ON DRAWING 532 FOR THE CHANNEL SIZING DATA.
4. THE FINAL CULVERT EXCAVATION SIDE SLOPES SHALL BE BASED ON FGMI SAFETY REQUIREMENTS OR BE DETERMINED IN THE FIELD BY THE ON-SITE ENGINEER BASED ON THE EXISTING SOIL CONDITIONS ENCOUNTERED. 1
5. PERIODIC MAINTENANCE OF THE CULVERT CROSSINGS SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
6. THE CONVEYOR TUNNEL WILL NOT EXPERIENCE GREATER LOADS THAN THOSE BENEATH THE FULL HEIGHT OF THE CRUSHER PLATFORM; HOWEVER, IT IS RECOMMENDED THAT INTERIOR BRACING OF THE TUNNEL BE INCORPORATED PRIOR TO CONSTRUCTION OF THE GABION-REINFORCED SLOPE.
7. THE SETTING OUT DATA REPRESENTS THE INVERT OF THE CENTER PIPE. REFER TO PROFILE ON DRAWING 540 FOR VERTICAL CONTROL DATA.
8. THE DEPTH OF SITE GRADING FILL USED TO BACKFILL THE CULVERT EXCAVATION WILL VARY. A MINIMUM OF 6' TOTAL COVER TO THE CROWN OF THE PIPES SHALL BE INCORPORATED WHEREVER THE CULVERTS PASS BENEATH THE EXISTING HAUL ROAD OR ACCESS ROAD, A MINIMUM OF 5' TOTAL COVER BENEATH THE PAD PERIMETER ROAD AND OTHER AREAS WHERE ROAD TRAFFIC MAY BE EXPERIENCED ABOVE THE FILL, AND A MINIMUM OF 4' TOTAL COVER IN ALL OTHER LOCATIONS EXCEPT THE INLET AND OUTLET POINTS. 1

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
mar.17/17

REVISION:

1 MODIFIED NOTE 4 AND ADDED NOTE 8.
MODIFIED NOTE CALLOUTS.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SOUTH PERIMETER DIVERSION CULVERTS SECTIONS AND DETAILS, SHEET 2 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 544 | 1 |

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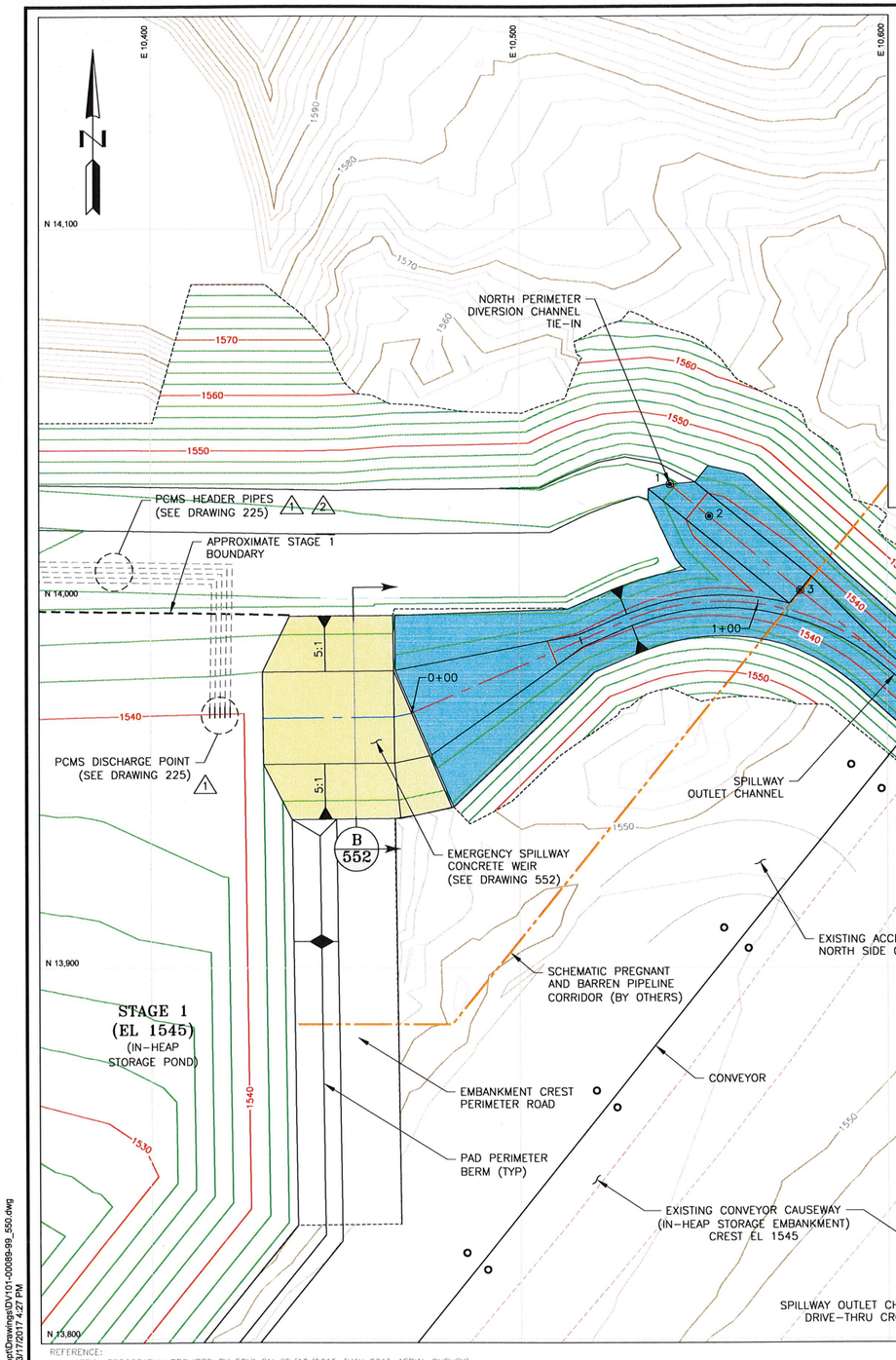


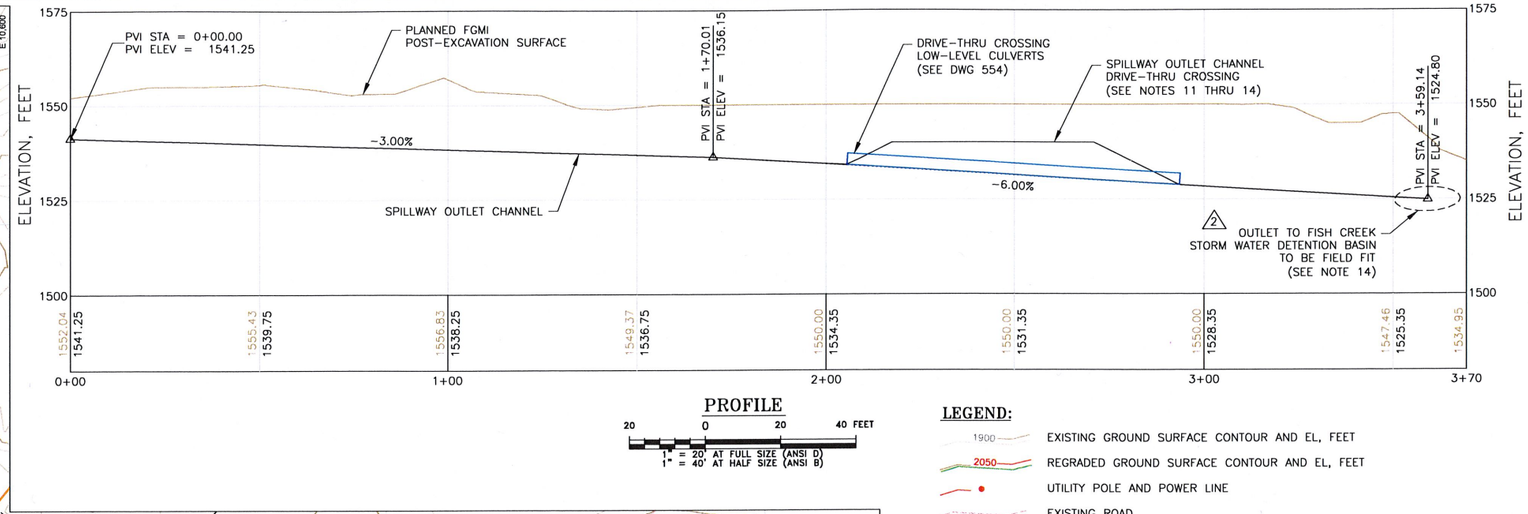


PROFILE

20 0 20 40 FEET

1" = 20' AT FULL SIZE (ANSI D)
1" = 40' AT HALF SIZE (ANSI B)

PLAN

20 0 20 40 FEET

1" = 20' AT FULL SIZE (ANSI D)
1" = 40' AT HALF SIZE (ANSI B)

## LEGEND:

- 1900 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 2050 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- UTILITY POLE AND POWER LINE
- EXISTING ROAD
- △ POINT OF VERTICAL INTERSECTION (PVI, PROFILE ONLY)
- ◉1 SETTING OUT DATA POINT AND DESIGNATION (SEE DRAWING 551)
- EMERGENCY SPILLWAY REINFORCED CONCRETE (SEE NOTES 13 AND 15) △1
- SPILLWAY OUTLET CHANNEL GROUTED RIPRAP (SEE NOTE 13)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, THE TOP OF THE EMERGENCY SPILLWAY CONCRETE EROSION PROTECTION, AND THE BOTTOM OF EROSION PROTECTION WITHIN ALL OTHER CHANNELS.
3. EXISTING POWER LINES SHALL BE RELOCATED AS REQUIRED BY FGMI.
4. THE ROUGH GRADE SURFACE SHALL BE SHAPED AND COMPACTED TO PROVIDE A SUITABLE SURFACE TO RECEIVE RIPRAP.
5. THE PORTIONS OF THE CHANNEL CROSS-SECTION CONSTRUCTED IN FILL SHALL INCLUDE A MINIMUM CREST WIDTH OF 3'.
6. PERIODIC MAINTENANCE OF THE SPILLWAY AND OUTLET CHANNEL SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
7. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
8. REFER TO DRAWING 551 FOR ADDITIONAL SPILLWAY OUTLET CHANNEL AND NORTH PERIMETER DIVERSION CHANNEL TIE-IN SETTING OUT DATA.
9. THE SETTING OUT DATA POINTS REPRESENT THE CENTERLINE, BOTTOM OF EROSION PROTECTION, FOR THE NORTH PERIMETER DIVERSION CHANNEL.
10. REFER TO TABLE 1 ON DRAWING 552 FOR THE IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL DESIGN DATA.
11. THE SPILLWAY OUTLET CHANNEL DRIVE-THRU CROSSING HAS BEEN SIZED TO ACCOMMODATE LIGHT VEHICLE TRAFFIC ONLY.
12. FROM STATION 1+10 TO STATION 2+17.48, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE EXTENDED TO A MINIMUM ELEVATION OF 1543.
13. BETWEEN THE REINFORCED CONCRETE WEIR SECTION AND THE DRIVE-THRU CROSSING, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE UNDERLAIN BY A LAYER OF 80 mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE. THE CHANNEL GEOMEMBRANE SHALL BE CONNECTED TO THE DOWNSTREAM EDGE OF THE REINFORCED CONCRETE WEIR AS SHOWN ON DETAIL 2 ON DRAWING 552.
14. AS REQUESTED BY FGMI, EROSION PROTECTION HAS BEEN REMOVED FROM THE SPILLWAY OUTLET CHANNEL DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET TO THE FISH CREEK STOCKPILE AREA. THIS IS NOT EXPECTED TO AFFECT THE INTEGRITY OF THE IN-HEAP STORAGE POND EMBANKMENT; HOWEVER, MAINTENANCE OF THE DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET WILL BE REQUIRED THROUGH THE LIFE OF THE FACILITY.
15. THE IN-HEAP STORAGE POND PRIMARY AND SECONDARY GEOMEMBRANE SHALL BE CONNECTED TO THE UPSTREAM EDGE OF THE △1 EMERGENCY SPILLWAY REINFORCED CONCRETE WEIR USING THE BATTEN BAR CONNECTION SHOWN ON DETAIL 3 ON DRAWING 552. THE TOP EDGES OF THE ATTACHED PRIMARY AND SECONDARY GEOMEMBRANE SHALL BE EXTRUSION WELDED TOGETHER FOR THE ENTIRE WIDTH OF THE SPILLWAY.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

## REVISION:

△1 ADDED PCMS HEADER PIPES AND DISCHARGE POINT. ADDED NOTE 15.

△2 REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY. ADDED TWO PCMS HEADER PIPES.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |


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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL PLAN AND PROFILE, STAGE 1

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 550 | 2 |

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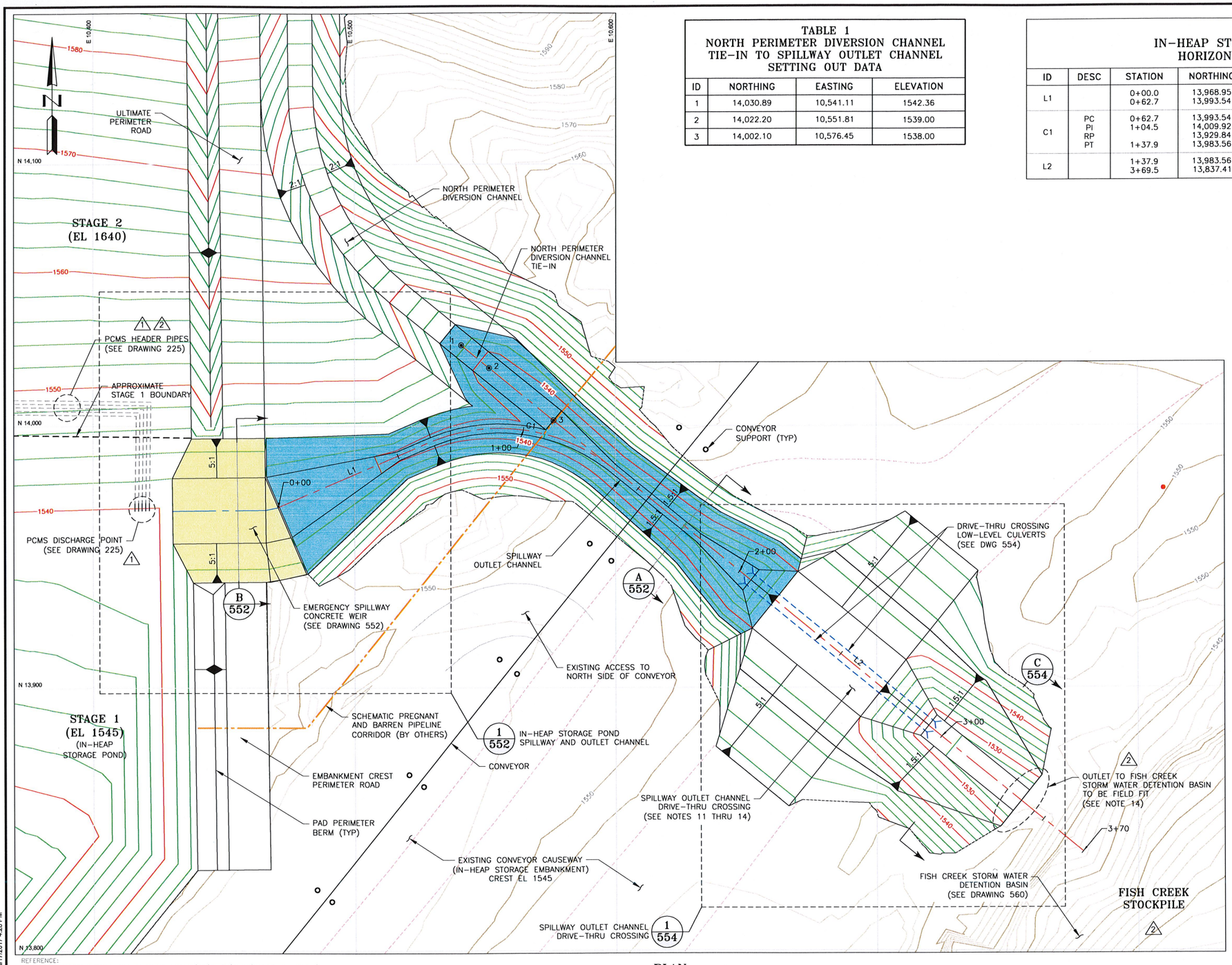


REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

PLAN

20 0 20 40 FEET

1" = 20' AT FULL SIZE (ANSI D)
1" = 40' AT HALF SIZE (ANSI B)

**TABLE 1**
**NORTH PERIMETER DIVERSION CHANNEL TIE-IN TO SPILLWAY OUTLET CHANNEL SETTING OUT DATA**

| ID | NORTHING | EASTING | ELEVATION |
|---|---|---|---|
| 1 | 14,030.89 | 10,541.11 | 1542.36 |
| 2 | 14,022.20 | 10,551.81 | 1539.00 |
| 3 | 14,002.10 | 10,576.45 | 1538.00 |

**TABLE 2**
**IN-HEAP STORAGE POND SPILLWAY HORIZONTAL CONTROL DATA**

| ID | DESC | STATION | NORTHING | EASTING | RADIUS | DELTA | D.O.C. |
|---|---|---|---|---|---|---|---|
| L1 | | 0+00.0<br>0+62.7 | 13,968.95<br>13,993.54 | 10,470.76<br>10,528.46 | | | |
| C1 | PC<br>PI<br>RP<br>PT | 0+62.7<br>1+04.5<br><br>1+37.9 | 13,993.54<br>14,009.92<br>13,929.84<br>13,983.56 | 10,528.46<br>10,566.90<br>10,555.61<br>10,599.31 | 69.2 | 62°13' | 92°27' |
| L2 | | 1+37.9<br>3+69.5 | 13,983.56<br>13,837.41 | 10,599.31<br>10,778.98 | | | |

**LEGEND:**

- 1900 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 2050 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- UTILITY POLE AND POWER LINE
- EXISTING ROAD
- 1 SETTING OUT DATA POINT AND DESIGNATION
- EMERGENCY SPILLWAY REINFORCED CONCRETE (SEE NOTES 13 AND 15) △1
- SPILLWAY OUTLET CHANNEL GROUTED RIPRAP (SEE NOTE 13)

**NOTES:** △1

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, THE TOP OF THE EMERGENCY SPILLWAY CONCRETE EROSION PROTECTION, AND THE BOTTOM OF EROSION PROTECTION WITHIN ALL OTHER CHANNELS.
3. EXISTING POWER LINES SHALL BE RELOCATED AS REQUIRED BY FGMI.
4. THE ROUGH GRADE SURFACE SHALL BE SHAPED AND COMPACTED TO PROVIDE A SUITABLE SURFACE TO RECEIVE RIPRAP.
5. THE PORTIONS OF THE CHANNEL CROSS-SECTION CONSTRUCTED IN FILL SHALL INCLUDE A MINIMUM CREST WIDTH OF 3'.
6. PERIODIC MAINTENANCE OF THE SPILLWAY AND OUTLET CHANNEL SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
7. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
8. REFER TO DRAWING 550 FOR THE SPILLWAY OUTLET CHANNEL PROFILE.
9. THE SETTING OUT DATA POINTS REPRESENT THE CENTERLINE, BOTTOM OF EROSION PROTECTION, FOR THE NORTH PERIMETER DIVERSION CHANNEL.
10. REFER TO TABLE 1 ON DRAWING 552 FOR THE IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL DESIGN DATA.
11. THE SPILLWAY OUTLET CHANNEL DRIVE-THRU CROSSING HAS BEEN SIZED TO ACCOMMODATE LIGHT VEHICLE TRAFFIC ONLY.
12. FROM STATION 1+10 TO STATION 2+17.48, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE EXTENDED TO A MINIMUM ELEVATION OF 1543.
13. BETWEEN THE REINFORCED CONCRETE WEIR SECTION AND THE DRIVE-THRU CROSSING, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE UNDERLAIN BY A LAYER OF 80 mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE. THE CHANNEL GEOMEMBRANE SHALL BE CONNECTED TO THE DOWNSTREAM EDGE OF THE REINFORCED CONCRETE WEIR AS SHOWN ON DETAIL 2 ON DRAWING 552.
14. AS REQUESTED BY FGMI, EROSION PROTECTION HAS BEEN REMOVED FROM THE SPILLWAY OUTLET CHANNEL DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET TO THE FISH CREEK STOCKPILE AREA. THIS IS NOT EXPECTED TO AFFECT THE INTEGRITY OF THE IN-HEAP STORAGE POND EMBANKMENT; HOWEVER, MAINTENANCE OF THE DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET WILL BE REQUIRED THROUGH THE LIFE OF THE FACILITY.
15. THE IN-HEAP STORAGE POND PRIMARY AND SECONDARY GEOMEMBRANE SHALL BE CONNECTED TO THE UPSTREAM EDGE OF THE EMERGENCY SPILLWAY REINFORCED CONCRETE WEIR USING THE BATTEN BAR CONNECTION SHOWN ON DETAIL 3 ON DRAWING 552. THE TOP EDGES OF THE ATTACHED PRIMARY AND SECONDARY GEOMEMBRANE SHALL BE EXTRUSION WELDED TOGETHER FOR THE ENTIRE WIDTH OF THE SPILLWAY.

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

**REVISION:**

△1 ADDED PCMS HEADER PIPES AND DISCHARGE POINT.
ADDED NOTE 15.

△2 REMOVED SOUTH PERIMETER OUTLET CHANNEL.
MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.
ADDED TWO PCMS HEADER PIPES.

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | CADD / JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |


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ACTIVITY CODE N/A
XREF N/A

CLIENT
FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT
FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE
IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL PLAN, STAGE 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 551 | 2 |

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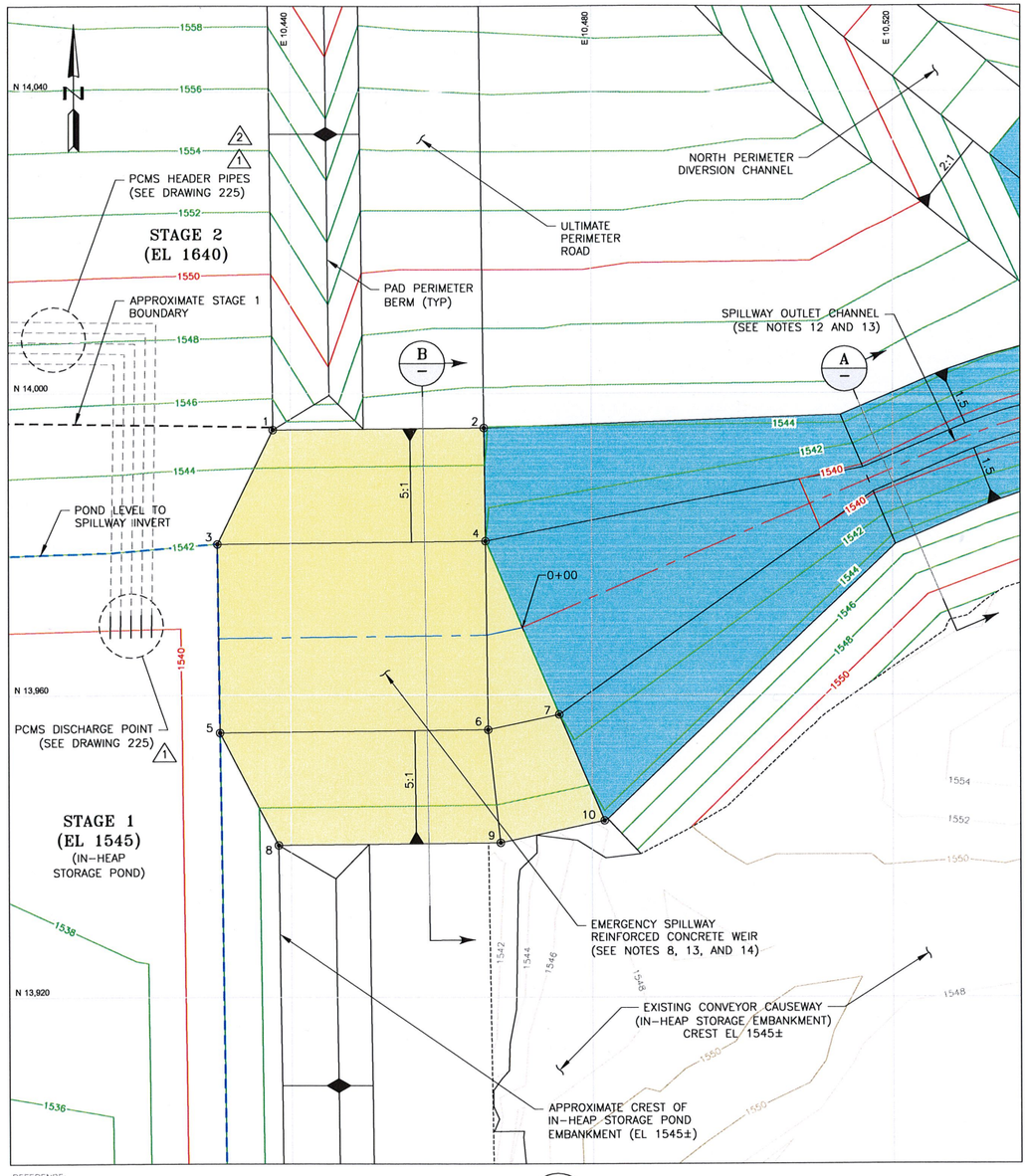



REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

1 / 551 — IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL

10 0 10 20 FEET
1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)

**LEGEND:**

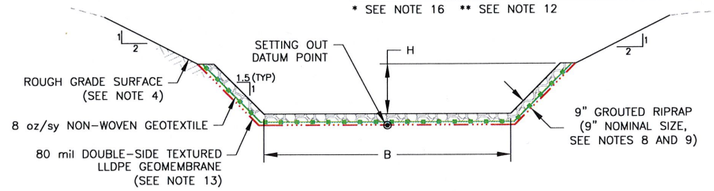
- 1900 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 2050 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- EMERGENCY SPILLWAY REINFORCED CONCRETE (SEE NOTES 13 AND 14) △1
- SPILLWAY OUTLET CHANNEL GROUTED RIPRAP (SEE NOTE 13)
- ◉1 SETTING OUT DATA POINT AND DESIGNATION (SEE NOTE 15, SEE TABLE 2)

**TABLE 1**

**EMERGENCY SPILLWAY OUTLET CHANNEL DESIGN DATA**

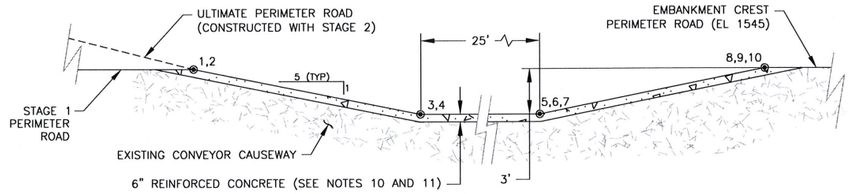
| STATION | BOTTOM WIDTH, B (ft) | CHANNEL DEPTH H (ft) | SECTION |
|---|---|---|---|
| 0+00 | 25* | 3 | A, DWG 552 |
| 0+00 to 0+50 | TRANSITION | TRANSITION | A, DWG 552 |
| 0+50 to 1+10 | 3 | 4.3 | A, DWG 552 |
| 1+10 TO 1+38 | VARIES* | VARIES** | A, DWG 552 |
| 1+38 TO 1+80 | 3 | VARIES** | A, DWG 552 |
| 1+80 TO 2+05 | TRANSITION | VARIES** | A, DWG 552 |
| 2+05 | 10 | 8.1 | A, DWG 552 |
| 2+05 TO 2+93 | CROSSING | CROSSING | A & B, DWG 556 |
| 2+93 TO 3+00 | 10 | VARIES** | C, DWG 554 |
| 3+00 TO 3+39 | 10 | TRANSITION | C, DWG 554 |
| 3+39 | 10 | 4.3 | C, DWG 554 |

* SEE NOTE 16 ** SEE NOTE 12




A / — | A / 550 | A / 551 — TYPICAL GROUTED RIPRAP-LINED SPILLWAY OUTLET CHANNEL UPSTREAM OF DRIVE-THRU CROSSING
(SEE TABLE 1)

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)



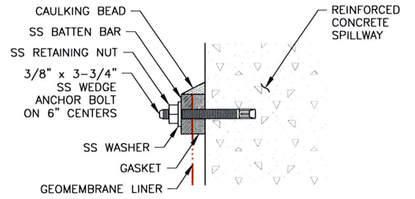

B / — | B / 550 | B / 551 — EMERGENCY SPILLWAY REINFORCED CONCRETE WEIR
(SEE NOTES 13 AND 14)

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

**TABLE 2**

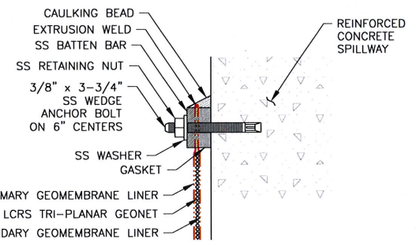
**EMERGENCY SPILLWAY CONCRETE WEIR SETTING OUT DATA**

| ID | NORTHING | EASTING | ELEVATION |
|---|---|---|---|
| 1 | 13,995.2 | 10,437.7 | 1545.0 |
| 2 | 13,995.4 | 10,465.7 | 1545.0 |
| 3 | 13,980.0 | 10,430.3 | 1542.0 |
| 4 | 13,980.4 | 10,465.9 | 1542.0 |
| 5 | 13,955.0 | 10,430.6 | 1542.0 |
| 6 | 13,955.4 | 10,466.2 | 1542.0 |
| 7 | 13,957.4 | 10,475.7 | 1542.0 |
| 8 | 13,940.1 | 10,438.3 | 1545.0 |
| 9 | 13,940.5 | 10,467.8 | 1545.0 |
| 10 | 13,943.4 | 10,481.7 | 1545.0 |




2 / — SPILLWAY DOWNSTREAM GEOMEMBRANE BATTEN BAR CONNECTION
(SEE NOTE 13)

0.25 0 0.25 0.5 FEET
1" = 0.25' AT FULL SIZE (ANSI D)
1" = 0.5' AT HALF SIZE (ANSI B)




3 / — SPILLWAY UPSTREAM GEOMEMBRANE BATTEN BAR CONNECTION
(SEE NOTE 13)

0.25 0 0.25 0.5 FEET
1" = 0.25' AT FULL SIZE (ANSI D)
1" = 0.5' AT HALF SIZE (ANSI B)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, THE TOP OF THE EMERGENCY SPILLWAY CONCRETE EROSION PROTECTION, AND THE BOTTOM OF EROSION PROTECTION WITHIN ALL OTHER CHANNELS.
3. EXISTING POWER LINES SHALL BE RELOCATED AS REQUIRED BY FGMI.
4. THE ROUGH GRADE SURFACE SHALL BE SHAPED AND COMPACTED TO PROVIDE A SUITABLE SURFACE TO RECEIVE RIPRAP.
5. THE PORTIONS OF THE CHANNEL CROSS SECTION CONSTRUCTED IN FILL SHALL HAVE A MINIMUM 3-FT-WIDE CREST.
6. PERIODIC MAINTENANCE OF THE SPILLWAY AND OUTLET CHANNEL SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
7. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
8. IMMEDIATELY DOWNSTREAM OF THE REINFORCED CONCRETE SPILLWAY, THE OUTLET CHANNEL EROSION PROTECTION SHALL BE A 9"-THICK LAYER OF GROUTED RIPRAP. THE TRANSITION FROM CONCRETE TO RIPRAP SHOULD BE SMOOTH TO PREVENT FLOW FROM BACKING UP WITHIN THE SPILLWAY.
9. THE GROUT THICKNESS IS TO BE MEASURED FROM THE BOTTOM OF THE RIPRAP LAYER. THE FINISHED SURFACE IS INTENDED TO BE ROUGH WITH EXPOSED RIPRAP PROTRUDING APPROXIMATELY ONE-THIRD OF THE LAYER THICKNESS.
10. COMPRESSIVE STRENGTH OF CONCRETE SHALL BE f'c 3,000 PSI.
11. CONCRETE REINFORCEMENT SHALL BE PLACED ON A 12" GRID AND HAVE A MINIMUM 3" CONCRETE COVER. ASTM A615, GRADE 40 BARS SHALL BE USED.
12. FROM STATION 1+10 TO STATION 2+17.48, THE SPILLWAY OUTLET CHANNEL RIPRAP GROUTED EROSION PROTECTION SHALL BE EXTENDED TO A MINIMUM ELEVATION OF 1543.
13. BETWEEN THE REINFORCED CONCRETE WEIR SECTION AND THE DRIVE-THRU CROSSING, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE UNDERLAIN BY A LAYER OF 80 mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE. THE CHANNEL GEOMEMBRANE SHALL BE CONNECTED TO THE DOWNSTREAM EDGE OF THE REINFORCED CONCRETE WEIR AS SHOWN ON DETAIL 2.
14. △1 THE IN-HEAP STORAGE POND PRIMARY AND SECONDARY GEOMEMBRANE SHALL BE CONNECTED TO THE UPSTREAM EDGE OF THE EMERGENCY SPILLWAY REINFORCED CONCRETE WEIR USING THE BATTEN BAR CONNECTION SHOWN IN DETAIL 3. THE TOP EDGES OF THE ATTACHED PRIMARY AND SECONDARY GEOMEMBRANE SHALL BE EXTRUSION WELDED TOGETHER FOR THE ENTIRE WIDTH OF THE SPILLWAY.
15. THE SETTING OUT DATA POINTS REPRESENT THE TOP OF THE EMERGENCY SPILLWAY EROSION PROTECTION.
16. FROM APPROXIMATELY STATION 1+10 TO STATION 1+38, THE SPILLWAY OUTLET CHANNEL BOTTOM WIDTH SHALL BE WIDENED TO ACCOMMODATE THE CONFLUENCE WITH THE NORTH PERIMETER DIVERSION CHANNEL.

## REVISION:

△1 ADDED PCMS HEADER PIPES AND DISCHARGE POINT. MODIFIED NOTE 14. ADDED NOTE CALLOUT IN LEGEND. ADDED DETAIL 3.

△2 ADDED TWO PCMS HEADER PIPES.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D / CADD |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND SPILLWAY SECTIONS AND DETAILS, SHEET 1 OF 3

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 552 | 2 |

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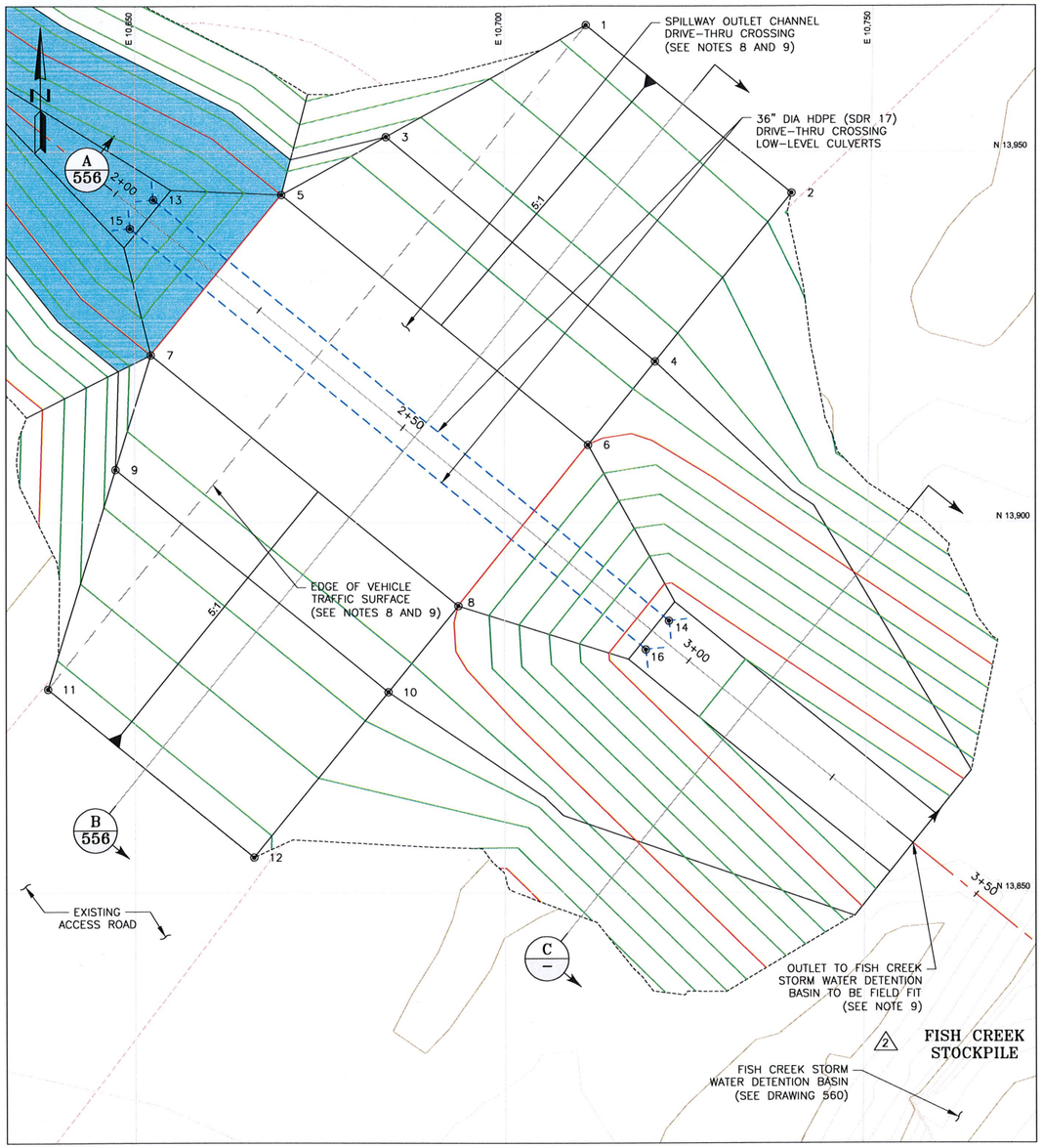



REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017

1/550 1/551 SPILLWAY OUTLET CHANNEL DRIVE–THRU CROSSING

10 0 10 20 FEET
1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)

## LEGEND:

- 1900 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 2050 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- EXISTING ROAD
- SPILLWAY OUTLET CHANNEL GROUTED RIPRAP (SEE NOTE 12)
- 1 — SETTING OUT DATA POINT AND DESIGNATION (SEE NOTE 4, SEE TABLE 1)

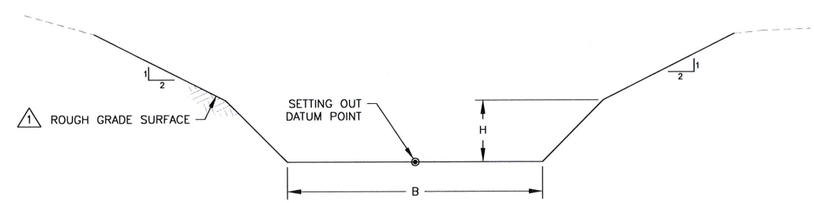
## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI–D) AND 1.5" ON A HALF SIZE PLOT (ANSI–B).
2. THE REGRADED CONTOURS REPRESENT THE TOP OF THE LEACH PAD LINER WITHIN THE PAD, TOP OF ROAD SURFACES, THE TOP OF THE EMERGENCY SPILLWAY CONCRETE EROSION PROTECTION, AND THE BOTTOM OF EROSION PROTECTION WITHIN ALL OTHER CHANNELS.
3. EXISTING POWER LINES SHALL BE RELOCATED AS REQUIRED BY FGMI.
4. SETTING OUT DATA POINTS 1 THRU 12 REPRESENT THE TOP OF THE DRIVE–THRU CROSSING ROAD SURFACE, WHILE SETTING OUT DATA POINTS 13 THRU 16 REPRESENT THE INLET AND OUTLET INVERTS OF THE DRIVE–THRU CROSSING LOW–LEVEL CULVERTS.
5. THE PORTIONS OF THE CHANNEL CROSS SECTION CONSTRUCTED IN FILL SHALL HAVE A MINIMUM 3–FT–WIDE CREST.
6. PERIODIC MAINTENANCE OF THE SPILLWAY AND OUTLET CHANNEL SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
7. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
8. THE SPILLWAY OUTLET CHANNEL DRIVE–THRU CROSSING HAS BEEN SIZED TO ACCOMMODATE LIGHT VEHICLE TRAFFIC ONLY.
9. AS REQUESTED BY FGMI, EROSION PROTECTION HAS BEEN REMOVED FROM THE SPILLWAY OUTLET CHANNEL DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET TO THE FISH CREEK STOCKPILE AREA. THIS IS NOT EXPECTED TO AFFECT THE INTEGRITY OF THE IN–HEAP STORAGE POND EMBANKMENT; HOWEVER, MAINTENANCE OF THE DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET WILL BE REQUIRED THROUGH THE LIFE OF THE FACILITY.
10. BETWEEN THE REINFORCED CONCRETE WEIR SECTION AND THE DRIVE–THRU CROSSING, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE UNDERLAIN BY A LAYER OF 80 mil DOUBLE–SIDE TEXTURED LLDPE GEOMEMBRANE. THE CHANNEL GEOMEMBRANE SHALL BE CONNECTED TO THE DOWNSTREAM EDGE OF THE REINFORCED CONCRETE WEIR AS SHOWN ON DETAIL 2 ON DRAWING 552.
11. REFER TO TABLE 1 ON DRAWING 552 FOR THE IN–HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL DESIGN DATA.
12. FROM STATION 1+10 TO STATION 2+17.48, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE EXTENDED TO A MINIMUM ELEVATION OF 1543.

### TABLE 1

SPILLWAY OUTLET CHANNEL DRIVE–THRU CROSSING SETTING OUT DATA

| ID | NORTHING | EASTING | ELEVATION |
|---|---|---|---|
| 1 | 13,967.1 | 10,711.1 | 1548.8 |
| 2 | 13,944.4 | 10,739.0 | 1548.8 |
| 3 | 13,952.0 | 10,684.0 | 1543.0 |
| 4 | 13,921.7 | 10,720.5 | 1543.0 |
| 5 | 13,944.2 | 10,669.9 | 1540.0 |
| 6 | 13,910.4 | 10,711.4 | 1540.0 |
| 7 | 13,922.5 | 10,652.2 | 1540.0 |
| 8 | 13,888.7 | 10,693.7 | 1540.0 |
| 9 | 13,907.1 | 10,647.4 | 1543.0 |
| 10 | 13,877.1 | 10,684.3 | 1543.0 |
| 11 | 13,877.5 | 10,638.2 | 1548.8 |
| 12 | 13,854.8 | 10,666.1 | 1548.8 |
| 13 | 13,943.5 | 10,652.6 | 1534.1 |
| 14 | 13,886.7 | 10,722.4 | 1528.7 |
| 15 | 13,939.6 | 10,649.4 | 1534.2 |
| 16 | 13,882.8 | 10,719.2 | 1528.7 |




C/– C/550 C/551 TYPICAL SPILLWAY OUTLET CHANNEL DOWNSTREAM OF DRIVE–THRU CROSSING

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

## REVISION:

1. REMOVED NOTES 4 AND 10 AND RENUMBERED NOTES 13 AND 14.
2. REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

| REV | DATE | DESCRIPTION | APP'D CADD |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN–HEAP STORAGE POND SPILLWAY SECTIONS AND DETAILS, SHEET 2 OF 3

| DESIGNED BY | DRAWN BY | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| JTR | CB | DV101 | 00089.99 | 554 | 2 |

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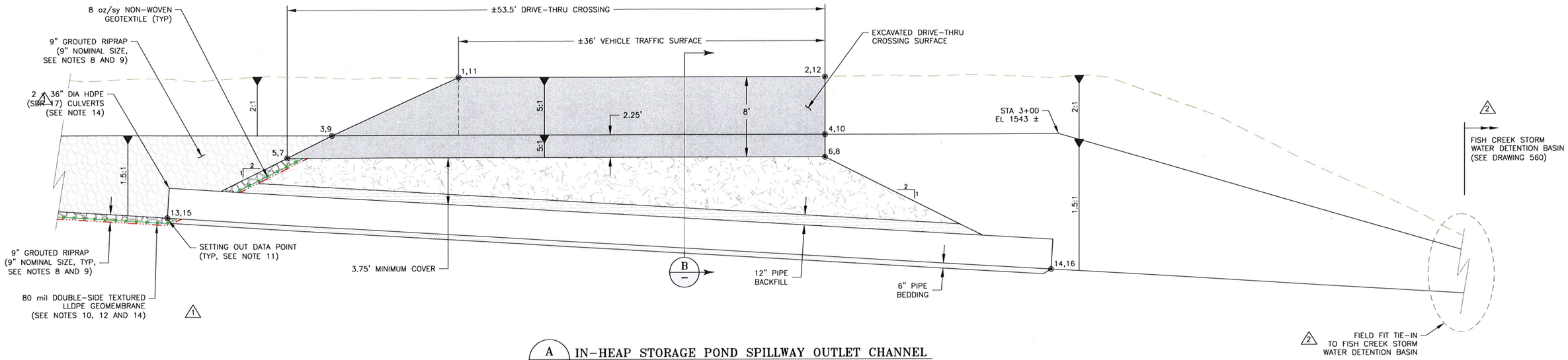
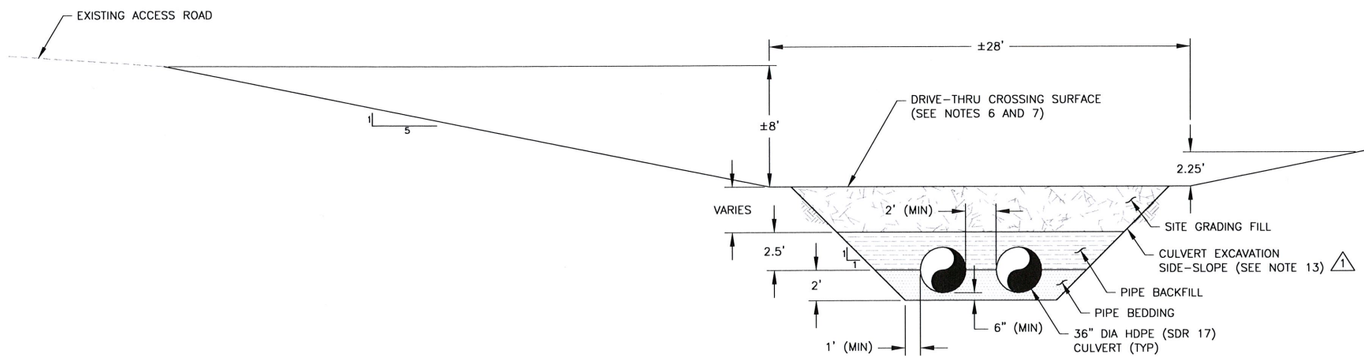



A / 554 IN-HEAP STORAGE POND SPILLWAY OUTLET CHANNEL
DRIVE-THRU CROSSING AND LOW-LEVEL CULVERTS

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)




B / — B / 554 SPILLWAY OUTLET CHANNEL
DRIVE-THRU CROSSING

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)

## NOTES: △1

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE ROUGH GRADE SURFACE SHALL BE SHAPED AND COMPACTED TO PROVIDE A SUITABLE SURFACE TO RECEIVE RIPRAP.
3. PERIODIC MAINTENANCE OF THE SPILLWAY AND OUTLET CHANNEL SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
4. MAINTENANCE OF ALL FILL SLOPES WILL BE REQUIRED FOR THE LIFE OF THE FACILITY. ANY REVEGETATION OR SEDIMENT CONTROL FOR THESE SLOPES SHALL BE PROVIDED BY FGMI.
5. REFER TO TABLE 1 ON DRAWING 552 FOR THE IN-HEAP STORAGE POND SPILLWAY AND OUTLET CHANNEL DESIGN DATA.
6. THE SPILLWAY OUTLET CHANNEL DRIVE-THRU CROSSING HAS BEEN SIZED TO ACCOMMODATE LIGHT VEHICLE TRAFFIC ONLY.
7. AS REQUESTED BY FGMI, EROSION PROTECTION HAS BEEN REMOVED FROM THE SPILLWAY OUTLET CHANNEL DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET TO THE FISH CREEK STOCKPILE AREA. THIS IS NOT EXPECTED TO AFFECT THE INTEGRITY OF THE IN-HEAP STORAGE POND EMBANKMENT; HOWEVER, MAINTENANCE OF THE DRIVE THROUGH CROSSING AND DOWNSTREAM OUTLET WILL BE REQUIRED THROUGH THE LIFE OF THE FACILITY.
8. THE GROUT THICKNESS IS TO BE MEASURED FROM THE BOTTOM OF THE RIPRAP LAYER. THE FINISHED SURFACE IS INTENDED TO BE ROUGH WITH EXPOSED RIPRAP PROTRUDING APPROXIMATELY ONE-THIRD OF THE LAYER THICKNESS.
9. UPSTREAM OF THE SPILLWAY OUTLET CHANNEL DRIVE-THRU CROSSING, THE GROUTED RIPRAP EROSION PROTECTION SHALL BE EXTENDED TO A MINIMUM ELEVATION OF 1543.
10. BETWEEN THE REINFORCED CONCRETE WEIR SECTION AND THE DRIVE-THRU CROSSING, THE SPILLWAY OUTLET CHANNEL GROUTED RIPRAP EROSION PROTECTION SHALL BE UNDERLAIN BY A LAYER OF 80 mil DOUBLE-SIDE TEXTURED LLDPE GEOMEMBRANE. THE CHANNEL GEOMEMBRANE SHALL BE CONNECTED TO THE DOWNSTREAM EDGE OF THE REINFORCED CONCRETE WEIR AS SHOWN ON DRAWING 552.
11. SETTING OUT DATA POINTS 1 THRU 12 REPRESENT THE TOP OF THE DRIVE-THRU CROSSING ROAD SURFACE, WHILE SETTING OUT DATA POINTS 13 THRU 16 REPRESENT THE INLET AND OUTLET INVERTS OF THE DRIVE-THRU CROSSING LOW-LEVEL CULVERTS. REFER TO DETAIL 1 AND TABLE 1 ON DRAWING 554 FOR MORE INFORMATION.
12. THE 36" DIAMETER HDPE CULVERTS SHALL BE BOOTED TO THE 80 mil LLDPE GEOMEMBRANE PRIOR TO PLACEMENT OF THE GROUTED RIPRAP EROSION PROTECTION.
13. THE FINAL CULVERT EXCAVATION SIDE SLOPES SHALL BE BASED ON FGMI SAFETY REQUIREMENTS OR BE DETERMINED IN THE FIELD BY THE ON-SITE ENGINEER BASED ON THE EXISTING SOIL CONDITIONS ENCOUNTERED.
14. THE LOW-LEVEL CULVERT PENETRATION THROUGH THE CHANNEL GEOMEMBRANE LINER, AND THE ASSOCIATED PIPE BOOT, SHALL BE PREFABRICATED PRIOR TO THE ACTUAL FIELD INSTALLATION SUCH THAT, WHEN INSTALLED, THE PIPE PENETRATION (AND SECTION OF SLOTTED HDPE PIPE) WILL BE LOCATED APPROXIMATELY FLUSH WITH THE LOW-POINT OF THE BOTTOM OF THE CHANNEL.

STATE OF ALASKA 49TH
THOMAS F. KERR No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17 17

## REVISION:

△1 ADDED NOTES 13 AND 14 AND ADDED CALLOUTS.

△2 ADDED CALLOUTS FOR FISH CREEK STORM WATER DETENTION BASIN.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: IN-HEAP STORAGE POND SPILLWAY SECTIONS AND DETAILS, SHEET 3 OF 3

| DESIGNED BY | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|
| JTR | DV101 | 00089.99 | 556 | 2 |
| DRAWN BY: CB | | | | |

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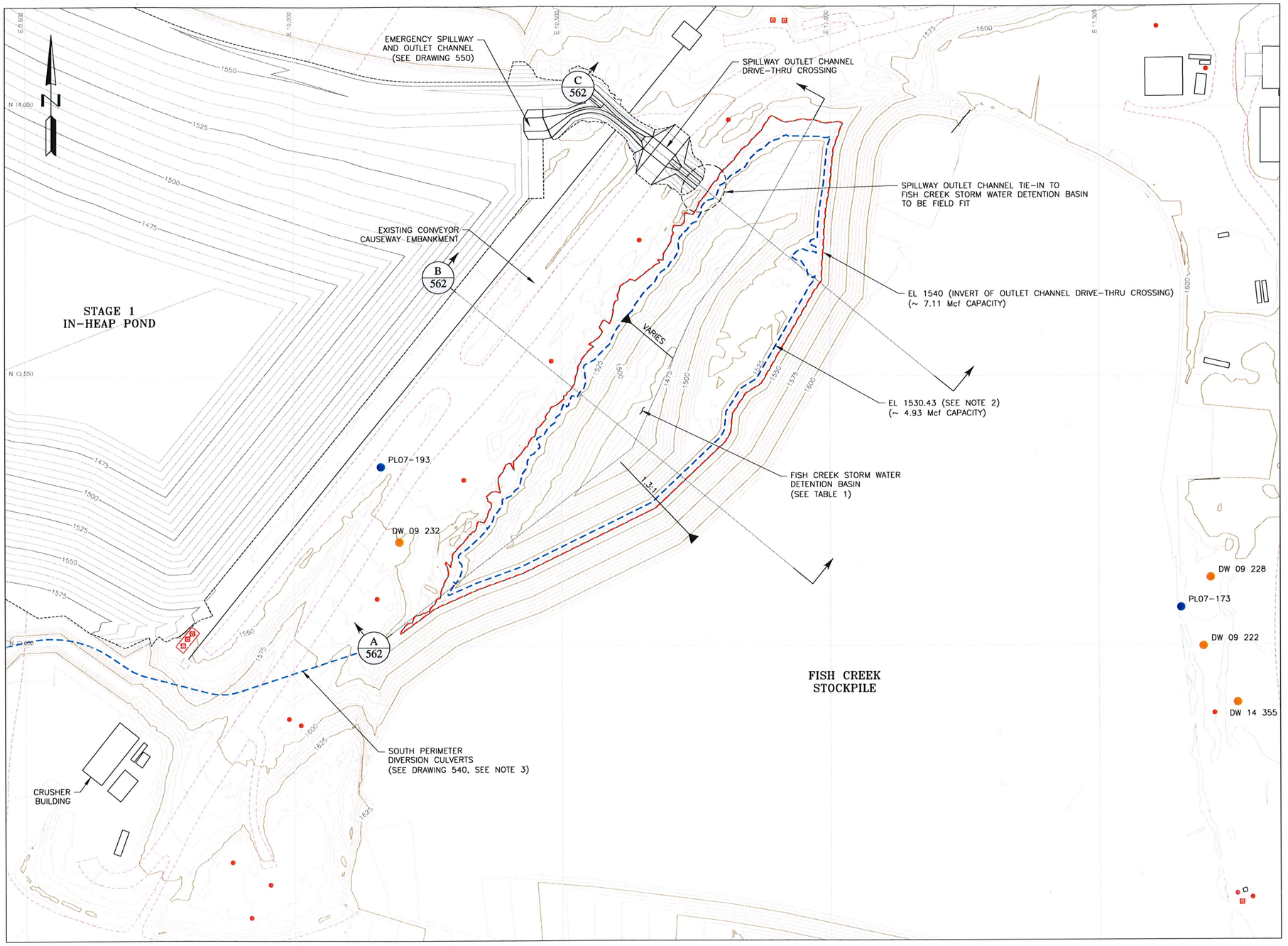


## LEGEND:

- 1500 — EXISTING GROUND SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- 1500 — GRADING SURFACE CONTOUR AND ELEVATION, FEET (MSL)
- EXISTING ROAD
- EXISTING STRUCTURE
- EXISTING FENCE
- EXISTING UTILITY POLE
- TAILING (CONFIGURATION AS OF AUGUST 2015)
- PL07-173 EXISTING MONITORING WELL (FGMI)
- DW 09 228 EXISTING DEWATERING WELL (FGMI)

**TABLE 1**

**STAGE STORAGE**
**FISH CREEK STORM WATER DETENTION BASIN**

| ELEV (ft) | AREA (sq. ft.) | DEPTH (ft) | CONIC INC. VOL. (cu. ft.) | CONIC TOTAL VOL. (cu. ft.) |
|---|---|---|---|---|
| 1455 | 76.16 | N/A | | 0.00 |
| 1460 | 1,455.24 | 5 | 2,429.29 | 2,429.29 |
| 1465 | 3,949.51 | 5 | 12,780.16 | 15,209.45 |
| 1470 | 7,394.52 | 5 | 28,021.85 | 43,231.30 |
| 1475 | 13,407.80 | 5 | 49,592.44 | 92,823.74 |
| 1480 | 21,446.71 | 5 | 86,300.03 | 179,123.78 |
| 1485 | 31,151.78 | 5 | 129,759.96 | 308,883.74 |
| 1490 | 42,941.16 | 5 | 184,856.14 | 493,739.88 |
| 1495 | 54,946.28 | 5 | 243,520.85 | 737,260.73 |
| 1500 | 69,576.38 | 5 | 308,743.26 | 1,046,003.99 |
| 1505 | 86,551.81 | 5 | 389,483.95 | 1,435,487.94 |
| 1510 | 104,354.77 | 5 | 476,879.28 | 1,912,367.22 |
| 1515 | 121,306.06 | 5 | 563,921.53 | 2,476,288.75 |
| 1520 | 139,607.60 | 5 | 650,777.22 | 3,127,065.97 |
| 1525 | 169,702.64 | 5 | 763,071.09 | 3,890,137.06 |
| 1530 | 204,531.02 | 5 | 952,319.66 | 4,842,456.72 |
| **1530.43** | – | – | – | **4,929,784.25** |
| 1535 | 226,389.65 | 5 | 1,082,310.59 | 5,924,767.30 |
| 1540 | 252,207.41 | 5 | 1,190,173.39 | 7,114,940.70 |

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE CONTROLLING DESIGN STORM EVENT FOR RUNOFF REPORTING TO THE FISH CREEK STORM WATER DETENTION BASIN IS THE 100-yr/24-hr JANUARY RAIN-ON-SNOW EVENT WHICH RESULTS IN A REPORTING VOLUME OF APPROXIMATELY 4.93 Mcf.
3. THE SOUTH PERIMETER DIVERSION CULVERTS SHALL DISCHARGE INTO THE SOUTHWESTERN END OF THE FISH CREEK STORM WATER DETENTION BASIN. THE CULVERT OUTLET TRANSITION TO THE FISH CREEK BASIN SHALL BE FIELD FIT.
4. MAINTENANCE OF THE FISH CREEK STORM WATER DETENTION BASIN SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

PLAN

100 0 100 200 FEET

1" = 100' AT FULL SIZE (ANSI D)
1" = 200' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

| REV | DATE | DESCRIPTION | APP'D | CADD |
|---|---|---|---|---|
| 0 | 03/17/17 | ISSUED FOR CONSTRUCTION | JTR | CB |

DISCLAIMER

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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: FISH CREEK STORM WATER DETENTION BASIN PLAN

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 560 | 0 |

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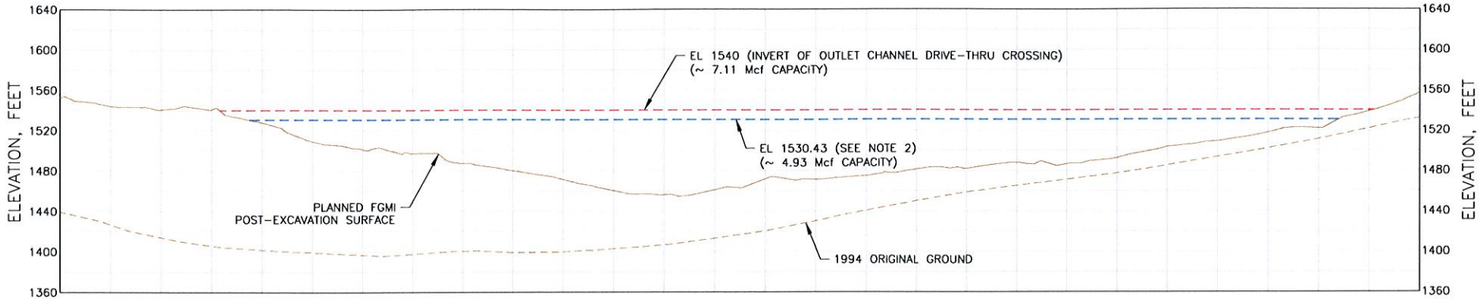
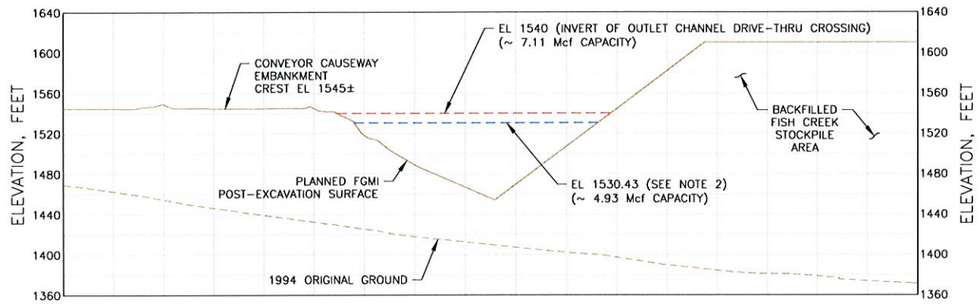


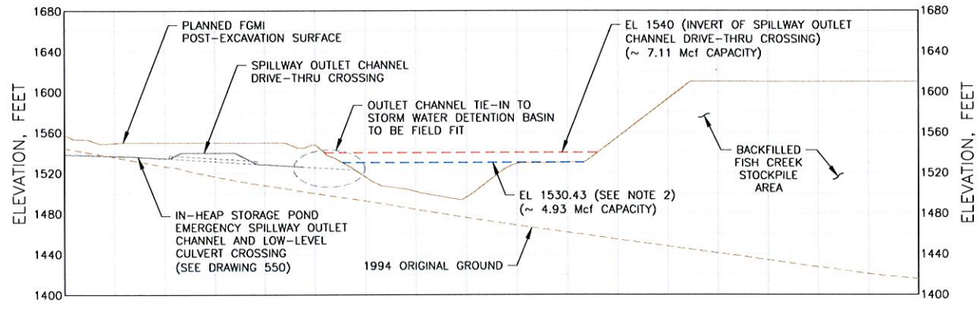

A / 560 FISH CREEK STORM WATER DETENTION BASIN
LONGITUDINAL SECTION

75 0 75 150 FEET
1" = 75' AT FULL SIZE (ANSI D)
1" = 150' AT HALF SIZE (ANSI B)




B / 560 FISH CREEK STORM WATER DETENTION BASIN
SOUTHWEST LATERAL SECTION

75 0 75 150 FEET
1" = 75' AT FULL SIZE (ANSI D)
1" = 150' AT HALF SIZE (ANSI B)




C / 560 FISH CREEK STORM WATER DETENTION BASIN
NORTHEAST LATERAL SECTION

75 0 75 150 FEET
1" = 75' AT FULL SIZE (ANSI D)
1" = 150' AT HALF SIZE (ANSI B)

## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI-D) AND 1.5" ON A HALF SIZE PLOT (ANSI-B).
2. THE CONTROLLING DESIGN STORM EVENT FOR RUNOFF REPORTING TO THE FISH CREEK STORM WATER DETENTION BASIN IS THE 100-yr/24-hr JANUARY RAIN-ON-SNOW EVENT WHICH RESULTS IN A REPORTING VOLUME OF APPROXIMATELY 4.93 Mcf.
3. THE SOUTH PERIMETER DIVERSION CULVERTS SHALL DISCHARGE INTO THE SOUTHWESTERN END OF THE FISH CREEK STORM WATER DETENTION BASIN. THE CULVERT OUTLET TRANSITION TO THE FISH CREEK BASIN SHALL BE FIELD FIT.
4. MAINTENANCE OF THE FISH CREEK STORM WATER DETENTION BASIN SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.

Mar. 24/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| 0 | 03/24/17 | ISSUED FOR CONSTRUCTION | CADD JTR CB |

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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: FISH CREEK STORM WATER DETENTION BASIN TYPICAL SECTIONS

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
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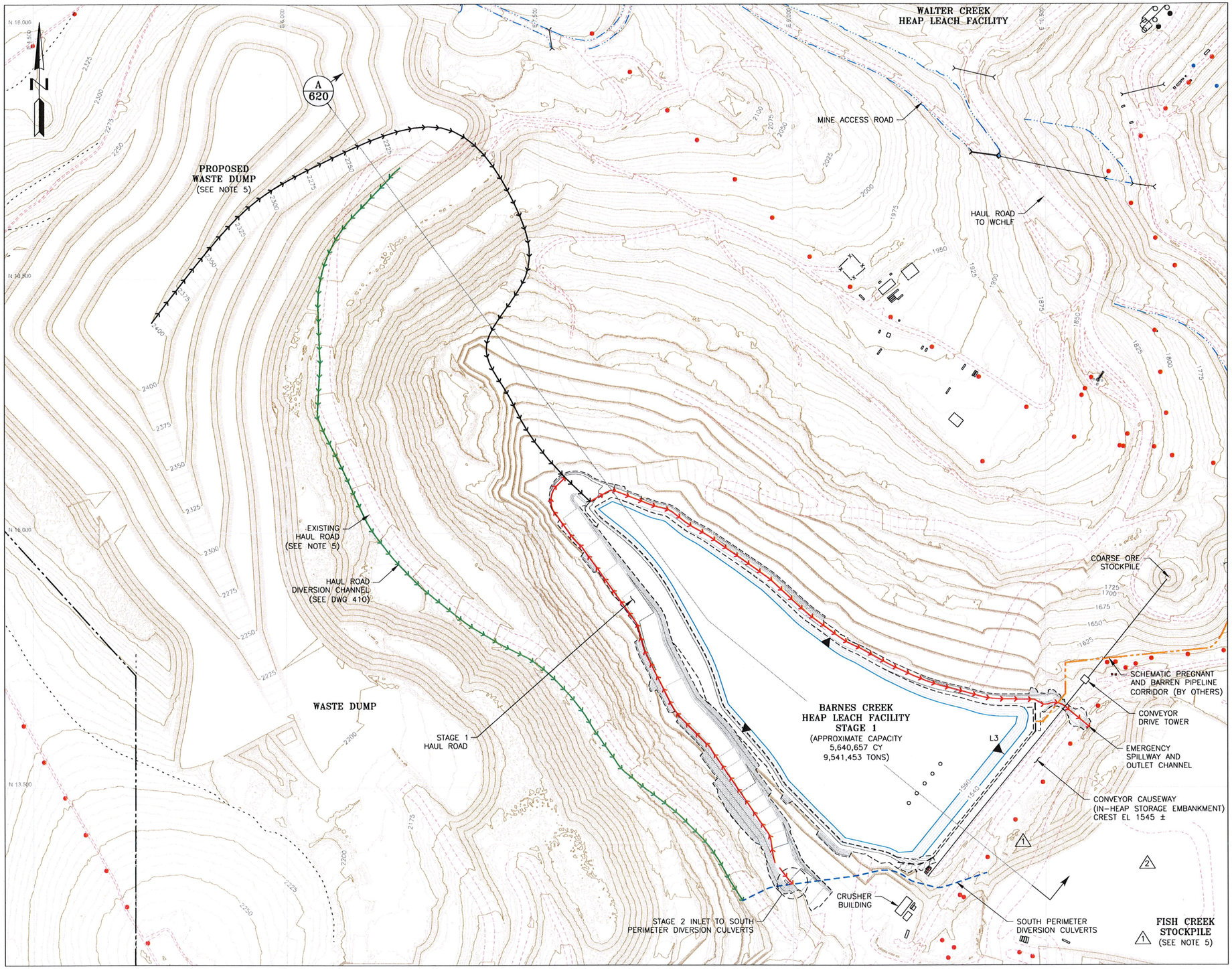


## LEGEND:

- 2100 — EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- 1550 — HEAP SURFACE CONTOUR AND EL, FEET
- 2100 — REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- PROPOSED INTER-STAGE DIVERSION
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- SOLUTION EXTRACTION WELL
- L3 — HEAP LIFT DESIGNATION (SEE NOTE 6) △1
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE APPROXIMATE HEAP CAPACITY IS BASED ON AN ORE DENSITY OF 125.3 lb/ft$^3$.
3. REFER TO SECTION A ON DRAWING 620 FOR THE RELATIONSHIPS OF HEAP ELEVATION, HEAP LIFT, AND STAGE BOUNDARY.
4. THE APPROXIMATE HEAP CAPACITY CONSIDERS LOADING ROUGHLY UP TO ONE LIFT HIGHER THAN THE HIGH-POINT ELEVATION OF EACH RESPECTIVE INTER-STAGE PERIMETER ROAD NEAR THE UPPER END OF THE VALLEY.
5. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
6. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.

## REVISION:

△1 CORRECTED NOTE CALLOUTS. REMOVED UNDERDRAIN MONITORING WELL AND CALLOUT.

△2 REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

**PLAN**

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17, 17

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |

DISCLAIMER
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 1
SCHEMATIC HEAP CONFIGURATION

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 600 | 2 |

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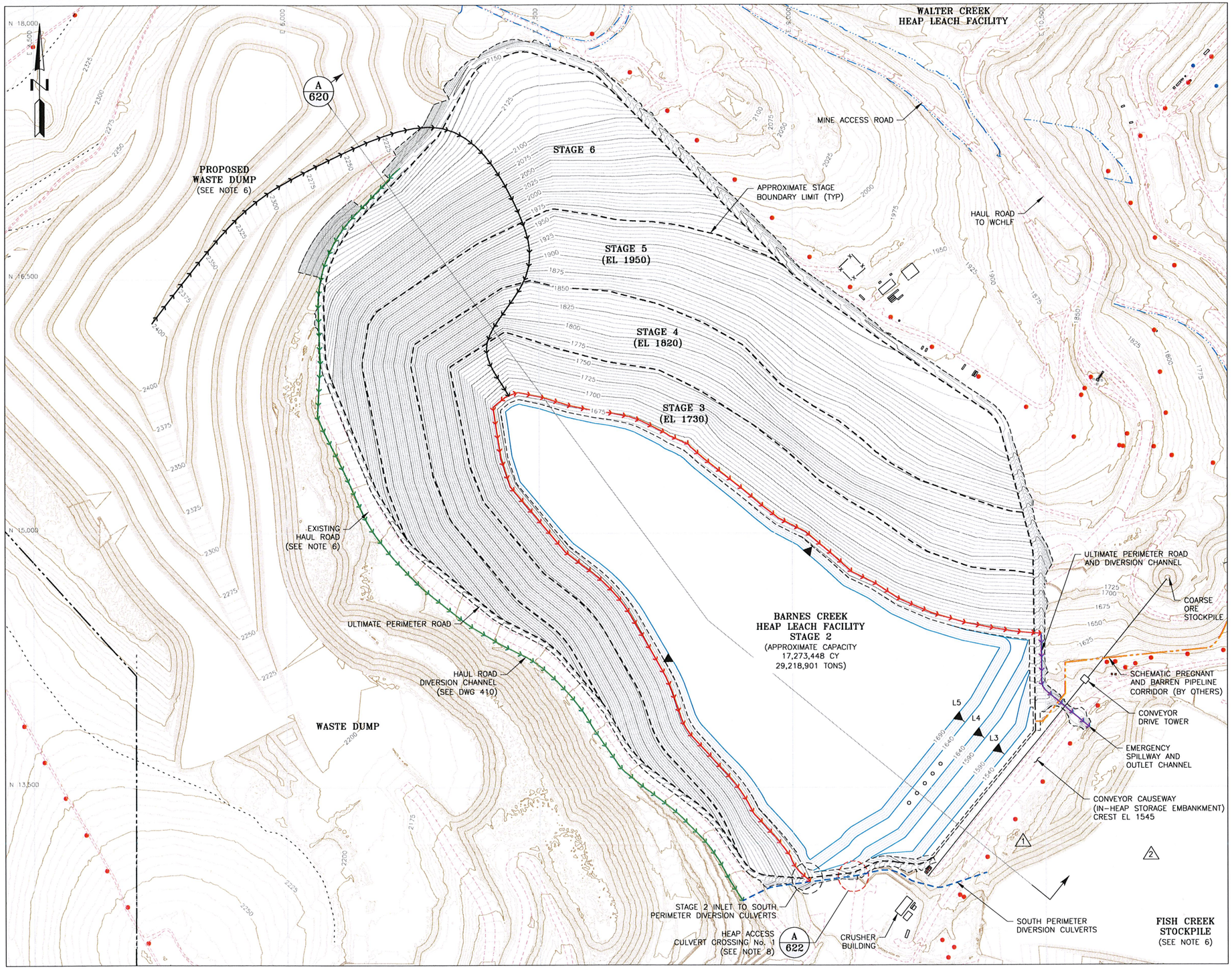


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED NORTH PERIMETER DIVERSION AND OUTLET CHANNEL
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- PROPOSED INTER-STAGE DIVERSION
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- SOLUTION EXTRACTION WELL
- L3 HEAP LIFT DESIGNATION (SEE NOTE 7)
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE APPROXIMATE HEAP CAPACITY IS BASED ON AN ORE DENSITY OF 125.3 lb/ft$^3$.
3. REFER TO SECTION A ON DRAWING 620 FOR THE RELATIONSHIPS OF HEAP ELEVATION, HEAP LIFT, AND STAGE BOUNDARY.
4. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
5. THE APPROXIMATE HEAP CAPACITY CONSIDERS LOADING ROUGHLY UP TO ONE LIFT HIGHER THAN THE HIGH-POINT ELEVATION OF EACH RESPECTIVE INTER-STAGE PERIMETER ROAD NEAR THE UPPER END OF THE VALLEY.
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
8. HEAP ACCESS CULVERT CROSSINGS NOS. 1 THRU 4 WILL BE REQUIRED NEAR THE SOUTHWEST CORNERS OF THE STAGES 2, 3, 4, AND 5 LEACH PADS, RESPECTIVELY, TO ALLOW HAUL ACCESS FOR LOADING THE ORE HEAP. THESE CULVERT CROSSINGS WILL BE REQUIRED WITH EACH RESPECTIVE AREA OF PAD DEVELOPMENT, BUT MAY BE REMOVED ONCE THE SUBSEQUENT OR FINAL HEAP ACCESS POINT IS DEVELOPED FOR LOADING.

## REVISION:

1. REMOVED UNDERDRAIN MONITORING WELL AND CALLOUT.
2. REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY).
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY).
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015.
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015.
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | CADD JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR CB |

DISCLAIMER
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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 2 (EL 1640)
SCHEMATIC HEAP CONFIGURATION

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 602 | 2 |

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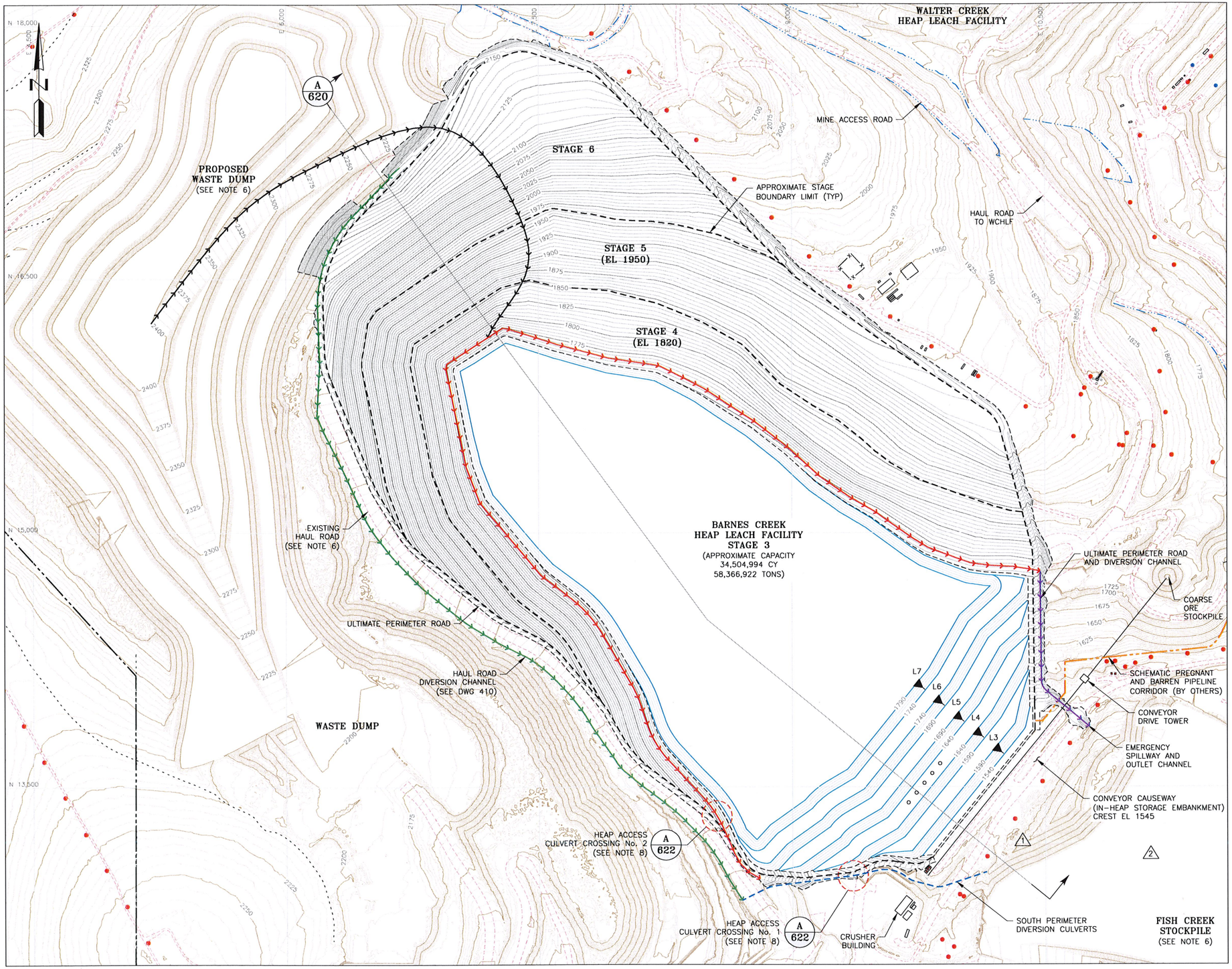


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED NORTH PERIMETER DIVERSION AND OUTLET CHANNEL
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- PROPOSED INTER-STAGE DIVERSION
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- SOLUTION EXTRACTION WELL
- L3 — HEAP LIFT DESIGNATION (SEE NOTE 7)
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE APPROXIMATE HEAP CAPACITY IS BASED ON AN ORE DENSITY OF 125.3 lb/ft$^3$.
3. REFER TO SECTION A ON DRAWING 620 FOR THE RELATIONSHIPS OF HEAP ELEVATION, HEAP LIFT, AND STAGE BOUNDARY.
4. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
5. THE APPROXIMATE HEAP CAPACITY CONSIDERS LOADING ROUGHLY UP TO ONE LIFT HIGHER THAN THE HIGH-POINT ELEVATION OF EACH RESPECTIVE INTER-STAGE PERIMETER ROAD NEAR THE UPPER END OF THE VALLEY.
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
8. HEAP ACCESS CULVERT CROSSINGS NOS. 1 THRU 4 WILL BE REQUIRED NEAR THE SOUTHWEST CORNERS OF THE STAGES 2, 3, 4, AND 5 LEACH PADS, RESPECTIVELY, TO ALLOW HAUL ACCESS FOR LOADING THE ORE HEAP. THESE CULVERT CROSSINGS WILL BE REQUIRED WITH EACH RESPECTIVE AREA OF PAD DEVELOPMENT, BUT MAY BE REMOVED ONCE THE SUBSEQUENT OR FINAL HEAP ACCESS POINT IS DEVELOPED FOR LOADING.

## REVISION:

1. REMOVED UNDERDRAIN MONITORING WELL AND CALLOUT.
2. REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

**PLAN**

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR CB |


DISCLAIMER
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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 3 (EL 1730) SCHEMATIC HEAP CONFIGURATION

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 604 | 2 |

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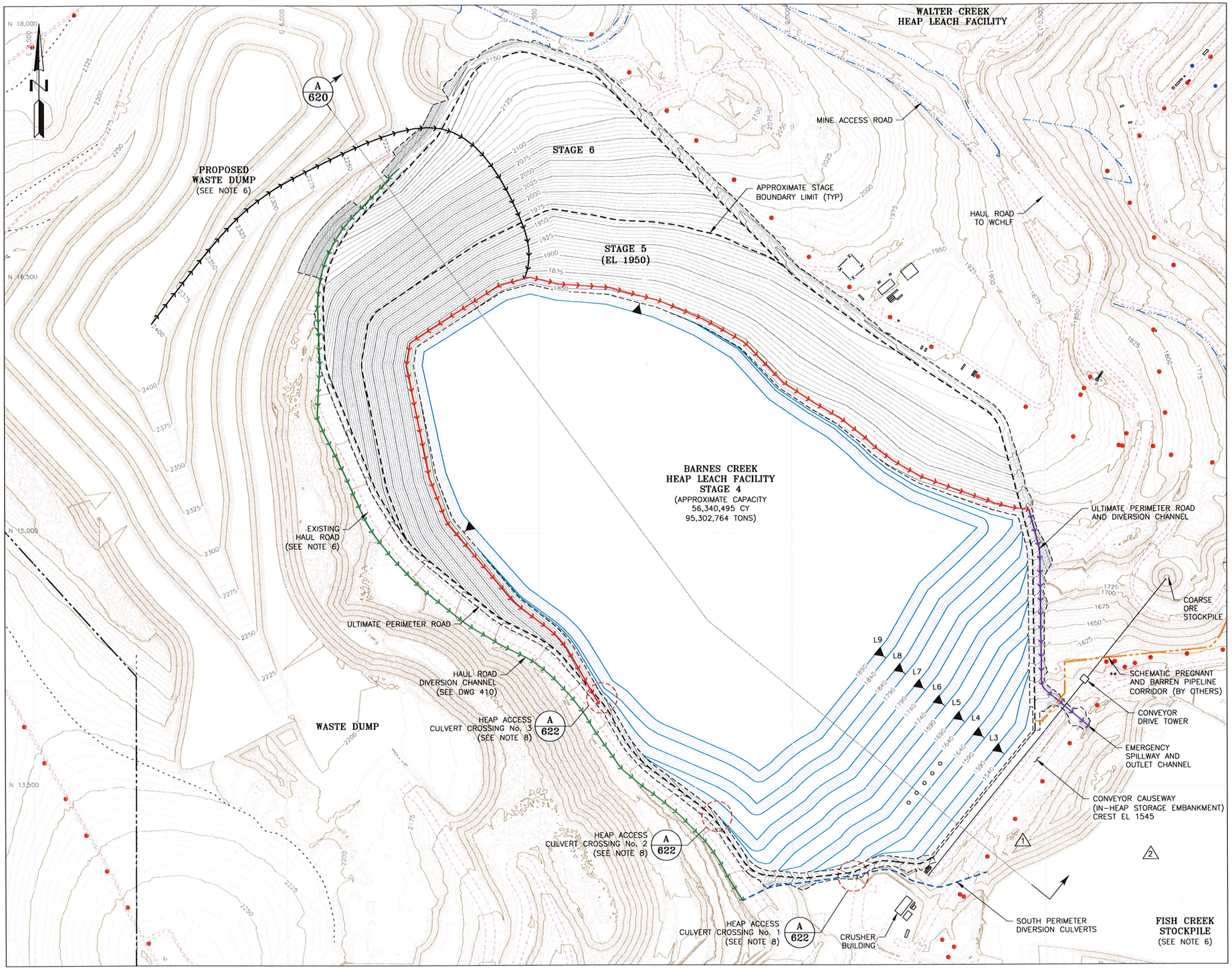


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED NORTH PERIMETER DIVERSION AND OUTLET CHANNEL
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- PROPOSED INTER-STAGE DIVERSION
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- SOLUTION EXTRACTION WELL
- L3 HEAP LIFT DESIGNATION (SEE NOTE 7)
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE APPROXIMATE HEAP CAPACITY IS BASED ON AN ORE DENSITY OF 125.3 lb/ft$^3$.
3. REFER TO SECTION A ON DRAWING 620 FOR THE RELATIONSHIPS OF HEAP ELEVATION, HEAP LIFT, AND STAGE BOUNDARY.
4. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
5. THE APPROXIMATE HEAP CAPACITY CONSIDERS LOADING ROUGHLY UP TO ONE LIFT HIGHER THAN THE HIGH-POINT ELEVATION OF EACH RESPECTIVE INTER-STAGE PERIMETER ROAD NEAR THE UPPER END OF THE VALLEY.
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
8. HEAP ACCESS CULVERT CROSSINGS NOS. 1 THRU 4 WILL BE REQUIRED NEAR THE SOUTHWEST CORNERS OF THE STAGES 2, 3, 4, AND 5 LEACH PADS, RESPECTIVELY, TO ALLOW HAUL ACCESS FOR LOADING THE ORE HEAP. THESE CULVERT CROSSINGS WILL BE REQUIRED WITH EACH RESPECTIVE AREA OF PAD DEVELOPMENT, BUT MAY BE REMOVED ONCE THE SUBSEQUENT OR FINAL HEAP ACCESS POINT IS DEVELOPED FOR LOADING.

## REVISION:

1. REMOVED UNDERDRAIN MONITORING WELL AND CALLOUT.
2. REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

**PLAN**

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER
Mar. 17/17

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY).
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY).
- PLANNED EXCAVATION SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |


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ACTIVITY CODE: N/A
XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 4 (EL 1820)
SCHEMATIC HEAP CONFIGURATION

| DESIGNED BY | DRAWN BY | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| JTR | CB | DV101 | 00089.99 | 606 | 2 |

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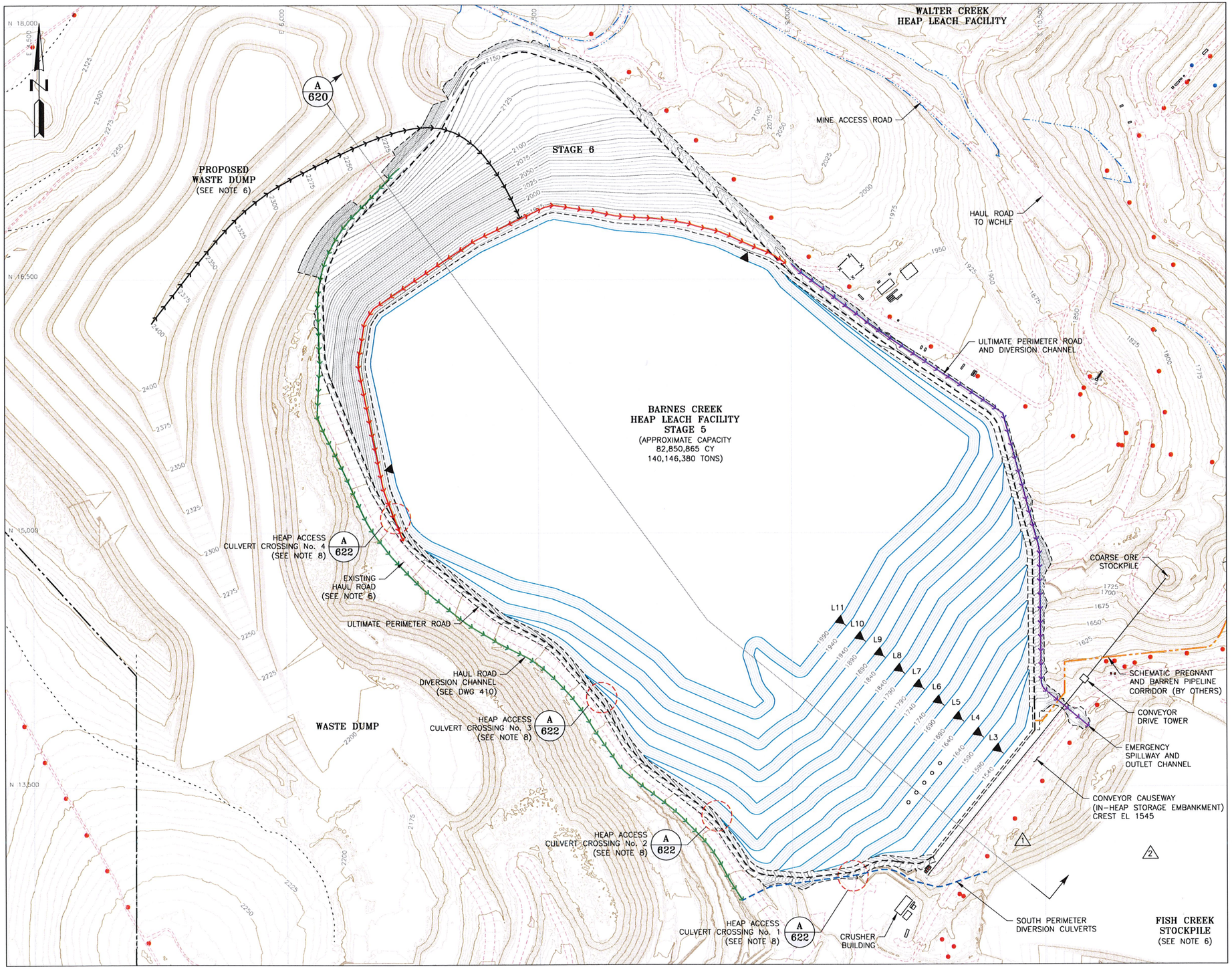


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED NORTH PERIMETER DIVERSION AND OUTLET CHANNEL
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- PROPOSED INTER-STAGE DIVERSION
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- SOLUTION EXTRACTION WELL
- L3 — HEAP LIFT DESIGNATION (SEE NOTE 7)
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE APPROXIMATE HEAP CAPACITY IS BASED ON AN ORE DENSITY OF 125.3 $lb/ft^3$.
3. REFER TO SECTION A ON DRAWING 620 FOR THE RELATIONSHIPS OF HEAP ELEVATION, HEAP LIFT, AND STAGE BOUNDARY.
4. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
5. THE APPROXIMATE HEAP CAPACITY CONSIDERS LOADING ROUGHLY UP TO ONE LIFT HIGHER THAN THE HIGH-POINT ELEVATION OF EACH RESPECTIVE INTER-STAGE PERIMETER ROAD NEAR THE UPPER END OF THE VALLEY.
6. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.
7. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
8. HEAP ACCESS CULVERT CROSSINGS NOS. 1 THRU 4 WILL BE REQUIRED NEAR THE SOUTHWEST CORNERS OF THE STAGES 2, 3, 4, AND 5 LEACH PADS, RESPECTIVELY, TO ALLOW HAUL ACCESS FOR LOADING THE ORE HEAP. THESE CULVERT CROSSINGS WILL BE REQUIRED WITH EACH RESPECTIVE AREA OF PAD DEVELOPMENT, BUT MAY BE REMOVED ONCE THE SUBSEQUENT OR FINAL HEAP ACCESS POINT IS DEVELOPED FOR LOADING.

## REVISION:

1. REMOVED UNDERDRAIN MONITORING WELL AND CALLOUT.
2. REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
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| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR / CB |


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ACTIVITY CODE: N/A
XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 5 (EL 1950)
SCHEMATIC HEAP CONFIGURATION

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 608 | 2 |

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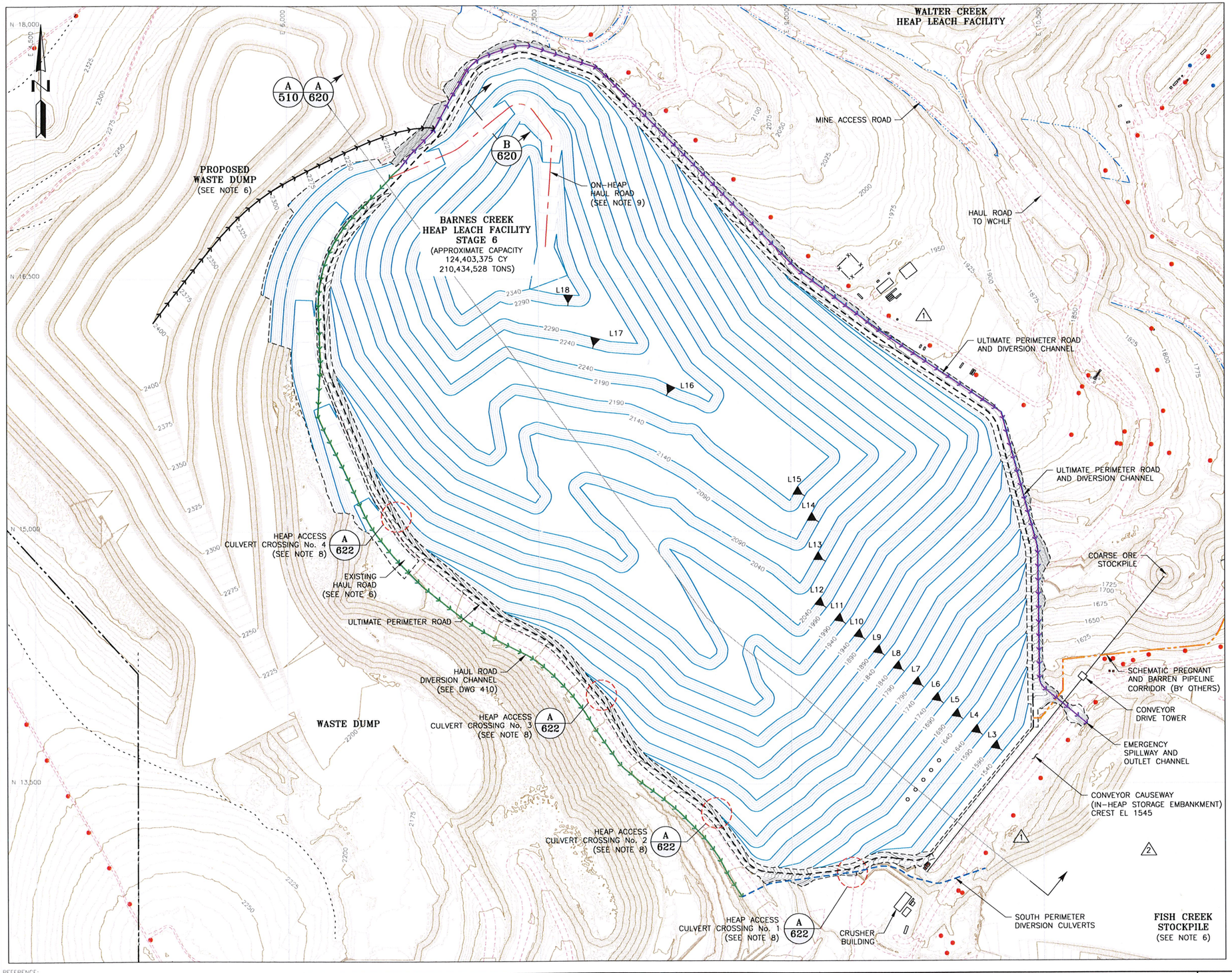


## LEGEND:

- EXISTING GROUND SURFACE CONTOUR AND EL, FEET
- HEAP SURFACE CONTOUR AND EL, FEET
- REGRADED GROUND SURFACE CONTOUR AND EL, FEET
- PROPERTY BOUNDARY
- PROPOSED NORTH PERIMETER DIVERSION AND OUTLET CHANNEL
- PROPOSED HAUL ROAD DIVERSION CHANNEL
- PROPOSED INTER-STAGE DIVERSION
- NATURAL DRAINAGE (OR DIVERSION BY OTHERS)
- EXISTING DRAINAGE/DIVERSION
- UTILITY POLE
- EXISTING ROAD
- EXISTING TRAIL
- EXISTING CULVERT
- SOLUTION EXTRACTION WELL
- L3 — HEAP LIFT DESIGNATION (SEE NOTE 7)
- EXISTING STRUCTURES

## NOTES:

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE APPROXIMATE HEAP CAPACITY IS BASED ON AN ORE DENSITY OF 125.3 lb/ft$^3$.
3. REFER TO SECTION A ON DRAWING 620 FOR THE RELATIONSHIPS OF HEAP ELEVATION, HEAP LIFT, AND STAGE BOUNDARY.
4. THE STAGE ELEVATIONS IN BRACKETS REPRESENT THE APPROXIMATE LOW POINTS AT THE NORTHEASTERN END OF THE RESPECTIVE INTER-STAGE PERIMETER ROADS.
5. THE APPROXIMATE HEAP CAPACITY CONSIDERS LOADING ROUGHLY UP TO ONE LIFT HIGHER THAN THE HIGH-POINT ELEVATION OF EACH RESPECTIVE INTER-STAGE PERIMETER ROAD NEAR THE UPPER END OF THE VALLEY.
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7. LIFTS 1 AND 2 ARE LOCATED WITHIN THE IN-HEAP STORAGE POND.
8. HEAP ACCESS CULVERT CROSSINGS NOS. 1 THRU 4 WILL BE REQUIRED NEAR THE SOUTHWEST CORNERS OF THE STAGES 2, 3, 4, AND 5 LEACH PADS, RESPECTIVELY, TO ALLOW HAUL ACCESS FOR LOADING THE ORE HEAP. THESE CULVERT CROSSINGS WILL BE REQUIRED WITH EACH RESPECTIVE AREA OF PAD DEVELOPMENT, BUT MAY BE REMOVED ONCE THE SUBSEQUENT OR FINAL HEAP ACCESS POINT IS DEVELOPED FOR LOADING.
9. THE ON-HEAP HAUL ROAD HAS BEEN CONFIGURED WITH A 10-PERCENT CONSTANT GRADE.

## REVISION:

1. REMOVED UNDERDRAIN MONITORING WELL AND CALLOUT.
2. REMOVED SOUTH PERIMETER OUTLET CHANNEL. MODIFIED FISH CREEK STOCKPILE TOPOGRAPHY.

PLAN

300 0 300 600 FEET

1" = 300' AT FULL SIZE (ANSI D)
1" = 600' AT HALF SIZE (ANSI B)

STATE OF ALASKA
49TH
THOMAS F. KERR
No. CE-10969
REGISTERED PROFESSIONAL ENGINEER

Mar. 17/17

REFERENCE:
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 08/12/2015 (MAY 2015 AERIAL SURVEY)
- AERIAL TOPOGRAPHY PROVIDED BY FGMI ON 09/18/2014 (AUGUST 2014 AERIAL SURVEY)
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- PLANNED WASTE DUMP SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 04/29/2015
- PLANNED FISH CREEK STOCKPILE SURFACE TOPOGRAPHY PROVIDED BY FGMI ON 02/27/2017.

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| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CADD |
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ACTIVITY CODE: N/A
XREF: N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: STAGE 6 (FINAL)
SCHEMATIC HEAP CONFIGURATION

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 610 | 2 |

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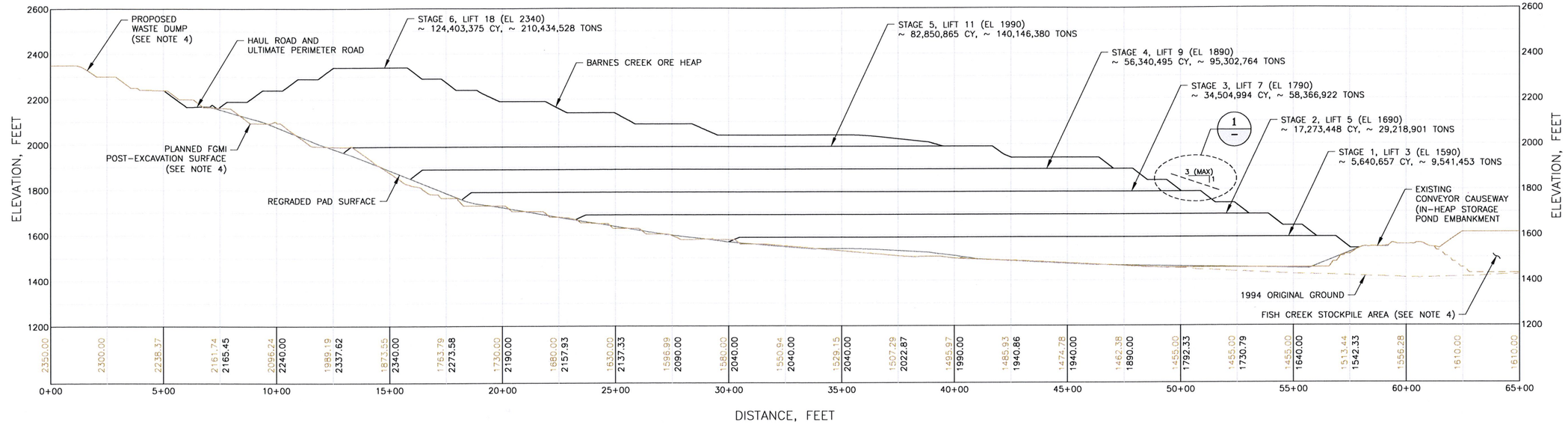
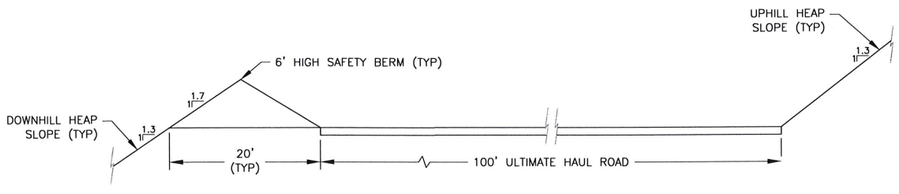



A 600 A 602 A 604 A 606 A 608 A 610 SCHEMATIC STAGED HEAP CONFIGURATION

250 0 250 500 FEET
1" = 250' AT FULL SIZE (ANSI D)
1" = 500' AT HALF SIZE (ANSI B)

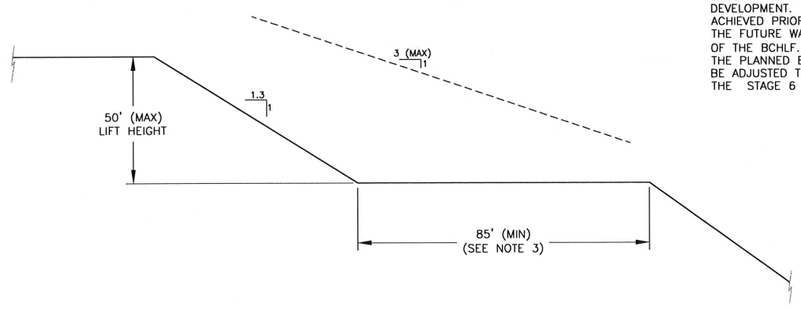
**NOTES:**

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. THE APPROXIMATE HEAP CAPACITY IS CUMULATIVE AND BASED ON AN ORE DENSITY OF 125.3 lb/ft$^3$.
3. THE HEAP BENCH WIDTH IS DEPENDENT ON THE LIFT THICKNESS SUCH THAT THE MAXIMUM 3H:1V OVERALL HEAP SLOPE IS NOT EXCEEDED.
4. FGMI'S PLANNED EXCAVATION SURFACE TO REMOVE ORE FROM THE BARNES CREEK STOCKPILE, ITS PLANNED FUTURE WASTE DUMP CONFIGURATION, AND THE CURRENT BACKFILLED CONFIGURATION OF THE FISH CREEK STOCKPILE AREA HAVE BEEN INCORPORATED INTO THE BASE TOPOGRAPHY FOR THE BARNES CREEK HEAP LEACH FACILITY DEVELOPMENT. THE EXCAVATION SURFACE IS ENVISIONED TO BE ACHIEVED PRIOR TO THE START OF THE BCHLF CONSTRUCTION, WHILE THE FUTURE WASTE DUMP IS EXPECTED TO BE LOADED OVER THE LIFE OF THE BCHLF. FGMI'S EXISTING HAUL ROAD ON THE SOUTH SIDE OF THE PLANNED BCHLF, AND THE PLANNED WASTE DUMP, WILL NEED TO BE ADJUSTED TO ALLOW HAUL ACCESS AROUND THE UPPER END OF THE STAGE 6 LEACH PAD.




B 610 TYPICAL ON-HEAP HAUL ROAD

10 0 10 20 FEET
1" = 10' AT FULL SIZE (ANSI D)
1" = 20' AT HALF SIZE (ANSI B)




1 – TYPICAL HEAP LIFT AND BENCH GEOMETRY

30 0 30 60 FEET
1" = 30' AT FULL SIZE (ANSI D)
1" = 60' AT HALF SIZE (ANSI B)

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
mar. 17/17

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SCHEMATIC HEAP
SECTIONS AND DETAILS, SHEET 1 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 620 | 0 |

LAST SAVED BY: DWMEETINGS
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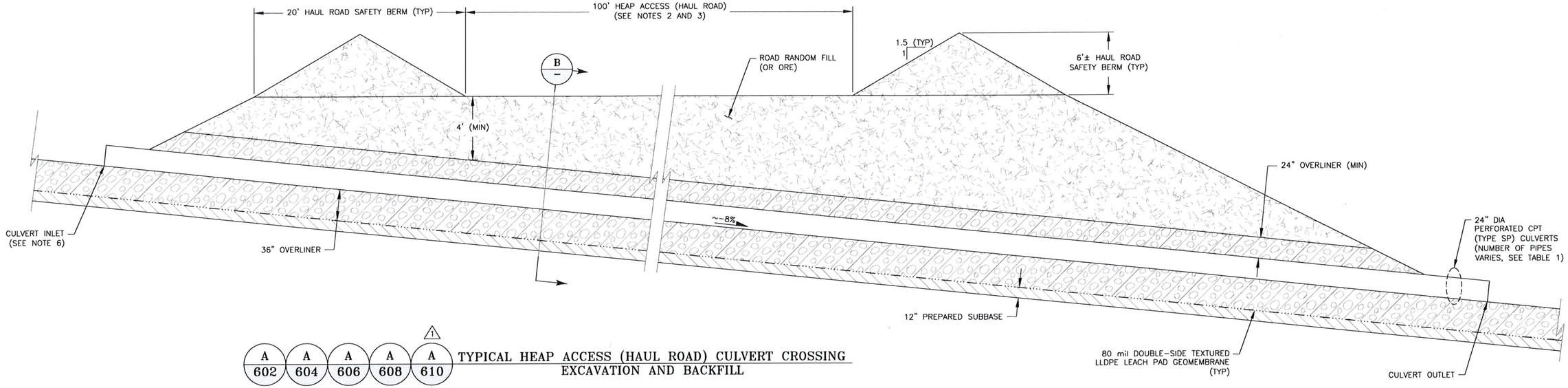
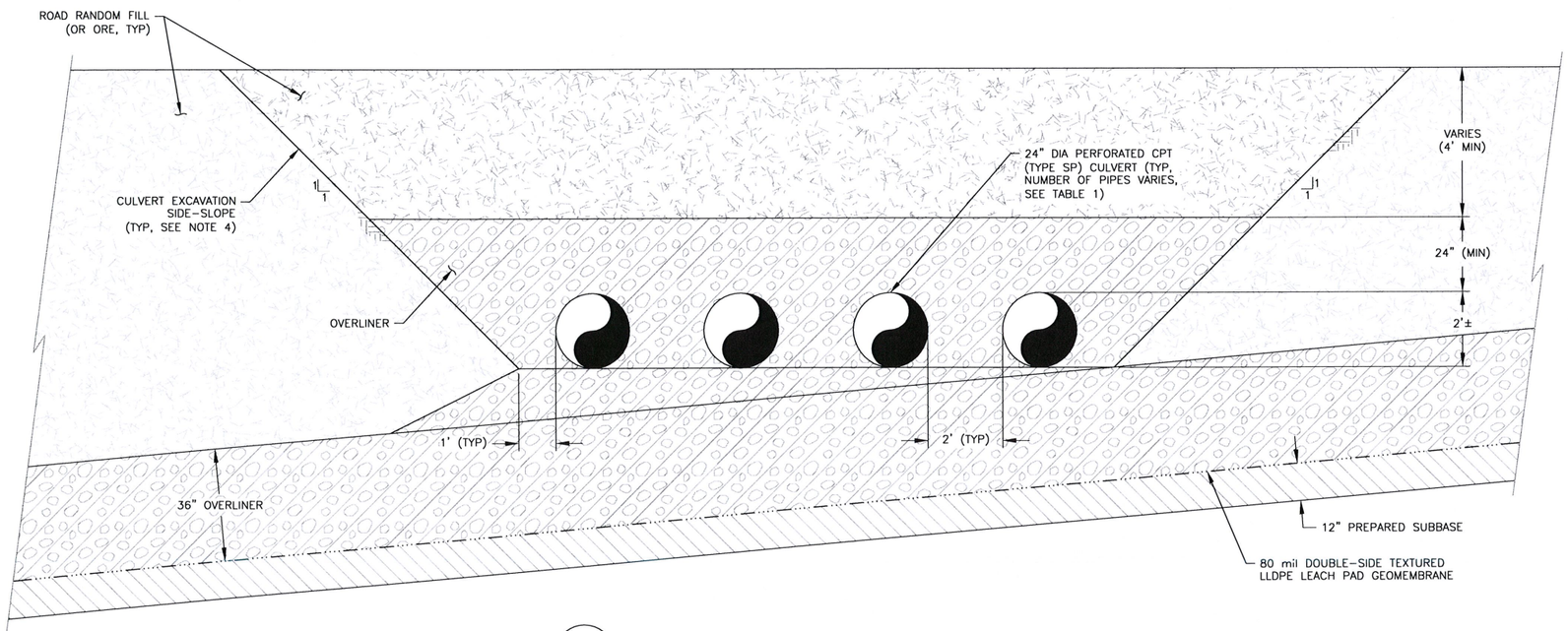



A 602 | A 604 | A 606 | A 608 | A 610 — TYPICAL HEAP ACCESS (HAUL ROAD) CULVERT CROSSING EXCAVATION AND BACKFILL

1

5 0 5 10 FEET
1" = 5' AT FULL SIZE (ANSI D)
1" = 10' AT HALF SIZE (ANSI B)




B – TYPICAL HEAP ACCESS (HAUL ROAD) CULVERT CROSSING EXCAVATION AND BACKFILL

2 0 2 4 FEET
1" = 2' AT FULL SIZE (ANSI D)
1" = 4' AT HALF SIZE (ANSI B)

**TABLE 1**
**IN-HEAP CULVERT CROSSING DESIGN DATA**

| IN-HEAP CULVERT CROSSING | STAGE | NUMBER OF PIPES | INLET NORTHING | INLET EASTING | OUTLET NORTHING | OUTLET EASTING |
|---|---|---|---|---|---|---|
| 1 | 2 | 4 | 13,018 | 9,282 | 13,090 | 9,505 |
| 2 | 3 | 4 | 13,033 | 8,786 | 12,999 | 8,992 |
| 3 | 4 | 3 | 13,916 | 7,961 | 13,730 | 8,077 |
| 4 | 5 | 3 | 14,904 | 6,819 | 14,762 | 6,991 |

* SEE NOTES 2 AND 3

**NOTES:**

1. SCALE BAR MEASURES 3" WHEN PLOTTED AT FULL SIZE (ANSI-D) AND 1.5" WHEN PLOTTED AT HALF SIZE (ANSI-B).
2. HEAP ACCESS CULVERT CROSSINGS WILL BE REQUIRED NEAR THE SOUTHWEST CORNERS OF THE STAGES 2, 3, 4, AND 5 LEACH PADS, RESPECTIVELY, TO ALLOW HAUL ACCESS FOR LOADING THE ORE HEAP. THESE CULVERT CROSSINGS WILL BE REQUIRED WITH EACH RESPECTIVE AREA OF PAD DEVELOPMENT. CULVERT NO. 1 WILL ALSO ALLOW VEHICLE ACCESS TO THE VERTICAL SOLUTION EXTRACTION WELLS THAT WILL DAYLIGHT THROUGH THE LIFT 4 BENCH ON THE FRONT FACE OF THE ORE HEAP AND WILL BE MAINTAINED FOR THE LIFE OF THE FACILITY. THE REMAINING CULVERTS BUT MAY BE REMOVED ONCE THE SUBSEQUENT OR FINAL HEAP ACCESS POINT IS DEVELOPED FOR LOADING.
3. NORTHING/EASTING DATA FOR IN-HEAP CULVERT CROSSINGS IS APPROXIMATE AND WILL BE ADJUSTED AS REQUIRED TO SUIT THE HEAP HAUL ROAD DESIGN (BY OTHERS).
4. THE FINAL CULVERT EXCAVATION SIDE-SLOPES SHALL BE DETERMINED IN THE FIELD BY FGMI BASED ON THE EXISTING SOIL CONDITIONS ENCOUNTERED.
5. PERIODIC MAINTENANCE OF THE CULVERT CROSSING INLETS AND OUTLETS SHALL BE REQUIRED FOR THE LIFE OF THE FACILITY.
6. THE IN-HEAP CULVERT CROSSINGS SHALL BE CONSTRUCTED ENTIRELY WITHIN THE GEOMEMBRANE LINED AREA OF THE HEAP LEACH PAD TO PROVIDE CONTAINMENT OF SOLUTIONS.

STATE OF ALASKA 49TH THOMAS F. KERR No. CE-10969 REGISTERED PROFESSIONAL ENGINEER
Mar. 17 17

**REVISION:**

1 ADDED CALLOUT FOR SECTION A TO DRAWING 610.

| REV | DATE | DESCRIPTION | APP'D |
|---|---|---|---|
| | | | CADD |
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR / CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR / CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR / CB |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ACTIVITY CODE N/A | XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

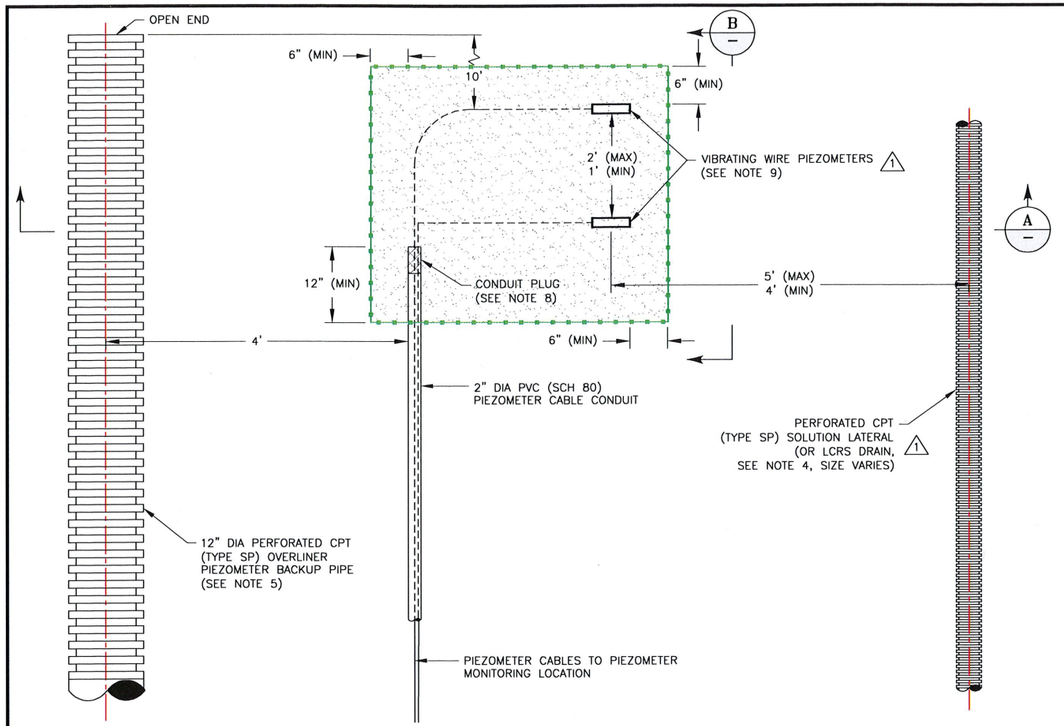
Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: SCHEMATIC HEAP
SECTIONS AND DETAILS, SHEET 2 OF 2

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 622 | 1 |


LAST SAVED BY: DVMEETINGS
DRAWING PATH: C:\101\00089.99\CAD\CAD Dept\Drawings\DV101-00089-99_622.dwg
PRINTED BY: CHAD BEANLAND, PRINT TIME: 3/17/2017 4:35 PM




1 — TYPICAL PIEZOMETER PLACEMENT

NTS

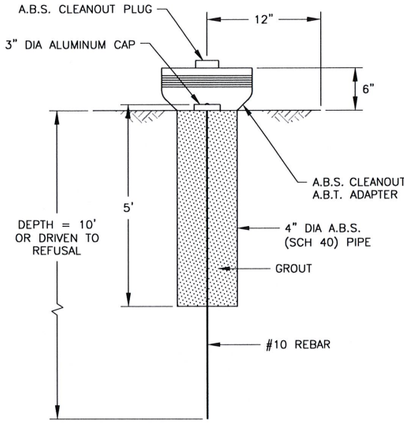
## TABLE 1 (2)

### VIBRATING WIRE PIEZOMETER SETTING OUT DATA

| ID | DESCRIPTION | PIEZOMETER MODEL | NORTHING | EASTING | DWG/FIG REFERENCE | COMMENT |
|---|---|---|---|---|---|---|
| 1,2 | LCRS | 4500AL–170kPa | 13,396.06 | 9,733.13 | 300 | STAGE 1 |
| 3,4 | LCRS | 4500AL–170kPa | 13,401.45 | 9,745.06 | 300 | STAGE 1 |
| 5,6 | LCRS | 4500AL–170kPa | 13,626.57 | 9,818.47 | 300 | STAGE 1 |
| 7,8 | LCRS | 4500AL–170kPa | 13,641.92 | 9,477.10 | 300 | STAGE 1 |
| 9,10 | LCRS | 4500AL–170kPa | 14,059.82 | 9,031.11 | 300 | STAGE 1 |
| 11,12 | LCRS | 4500AL–170kPa | 14,528.46 | 8,596.85 | 300 | STAGE 1 |
| 13,14 | OVERLINER | 4500S–350kPa | 13,523.62 | 9,739.83 | 500, 6.02 | STAGE 1 |
| 15,16 | OVERLINER | 4500S–350kPa | 13,790.49 | 9,315.51 | 500, 6.02 | STAGE 1 |
| 17,18 | OVERLINER | 4500S–350kPa | 14,257.75 | 8,819.22 | 500, 6.02 | STAGE 1 |
| 19,20 | OVERLINER | 4500AL–70kPa | 14,796.77 | 8,169.48 | 500, 6.02 | STAGE 1 |
| 21,22 | OVERLINER | 4500AL–70kPa | 15,091.73 | 8,006.78 | 500, 6.02 | STAGE 1 |
| 23,24 | OVERLINER | 4500AL–70kPa | 14,215.66 | 8,386.32 | 500, 6.02 | STAGE 2 |
| 25,26 | OVERLINER | 4500AL–70kPa | 15,055.90 | 7,815.88 | 500, 6.02 | STAGE 2 |
| 27,28 | OVERLINER | 4500AL–70kPa | 15,365.12 | 7,561.50 | 500, 6.02 | STAGE 2 |
| 29,30 | OVERLINER | 4500AL–70kPa | 15,655.97 | 7,406.46 | 500, 6.02 | STAGE 2 |
| 31,32 | OVERLINER | 4500AL–70kPa | 15,506.92 | 7,579.78 | 500, 6.02 | STAGE 2 |
| 33,34 | OVERLINER | 4500AL–70kPa | 15,137.62 | 8,279.49 | 500, 6.02 | STAGE 2 |
| 35,36 | OVERLINER | 4500AL–70kPa | 14,930.28 | 8,924.58 | 500, 6.02 | STAGE 2 |
| 37,38 | OVERLINER | 4500AL–70kPa | 14,564.58 | 9,052.33 | 500, 6.02 | STAGE 2 |
| 39,40 | OVERLINER | 4500AL–70kPa | 14,092.62 | 8,199.80 | 500, 6.03 | STAGE 3 |
| 41,42 | OVERLINER | 4500AL–70kPa | 15,019.93 | 7,442.41 | 500, 6.03 | STAGE 3 |
| 43,44 | OVERLINER | 4500AL–70kPa | 15,892.37 | 7,292.07 | 500, 6.03 | STAGE 3 |
| 45,46 | OVERLINER | 4500AL–70kPa | 15,174.11 | 9,062.46 | 500, 6.03 | STAGE 3 |
| 47,48 | OVERLINER | 4500AL–70kPa | 14,897.00 | 7,263.53 | 500, 6.04 | STAGE 4 |
| 49,50 | OVERLINER | 4500AL–70kPa | 16,227.59 | 7,160.61 | 500, 6.04 | STAGE 4 |
| 51,52 | OVERLINER | 4500AL–70kPa | 15,473.31 | 9,235.20 | 500, 6.04 | STAGE 4 |
| 53,54 | OVERLINER | 4500AL–70kPa | 16,489.86 | 7,063.33 | 500, 6.05 | STAGE 5 |
| 55,56 | OVERLINER | 4500AL–70kPa | 15,824.42 | 9,429.54 | 500, 6.05 | STAGE 5 |
| 57,58 | OVERLINER | 4500AL–70kPa | 16,863.89 | 6,880.05 | 500, 6.06 | STAGE 6 |

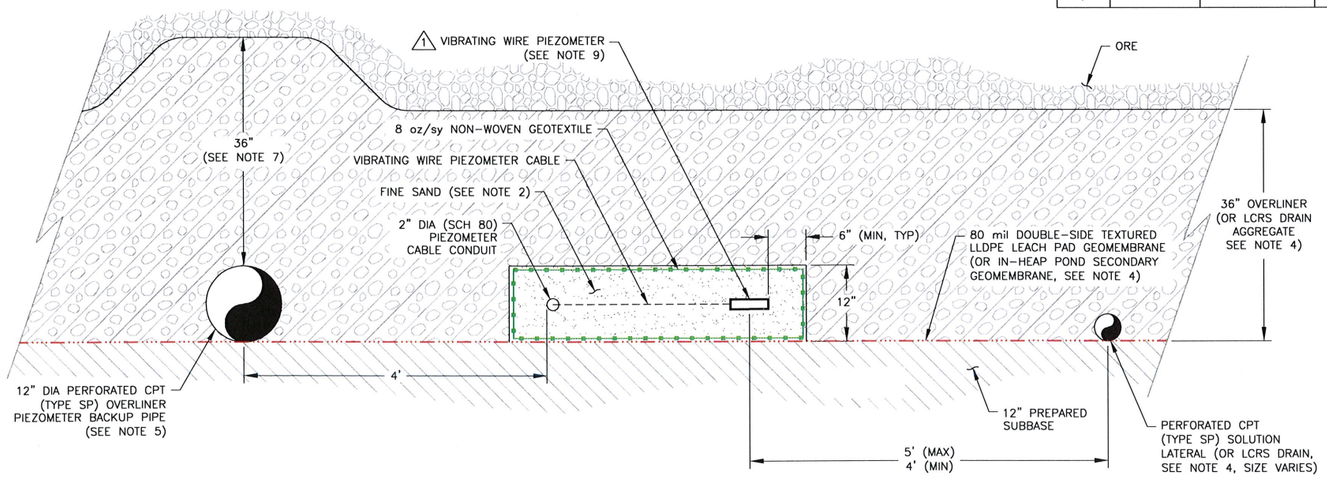
## NOTES:

1. SCALE BAR MEASURES 3" ON A FULL SIZE PLOT (ANSI–D) AND 1.5" ON A HALF SIZE PLOT (ANSI–B).
2. SAND SHALL BE CLEAN OTTAWA SILICA SAND, OR EQUIVALENT, WITH MAXIMUM PARTICLE SIZE NO GREATER THAN 1/16" AND A MAXIMUM OF 3 PERCENT PASSING THE NO. 200 SIEVE.
3. SEE DRAWING 300 FOR THE LCRS VIBRATING WIRE PIEZOMETER PLAN AND DRAWING 500 FOR THE OVERLINER VIBRATING WIRE PIEZOMETER PLAN AND SETTLEMENT MONUMENT PLAN.
4. SECTIONS A AND B SPECIFICALLY PRESENT INSTALLATIONS WITHIN THE OVERLINER LAYER; HOWEVER PIEZOMETER INSTALLATIONS WITHIN THE LCRS DRAIN AGGREGATE WILL BE SIMILAR. FOR PIEZOMETER INSTALLATIONS WITHIN THE LCRS, THE 36" OVERLINER MAY BE REPLACED WITH 36" LCRS DRAIN AGGREGATE (SEE DRAWING 300) AND THE 4" DIA SOLUTION LATERAL IS REPLACED WITH AN 8" DIA LCRS TOE DRAIN OR 4" DIA LCRS LATERAL. THE 12" DIA PERFORATED BACKUP PIPE IS ONLY REQUIRED ADJACENT TO THE PIEZOMETER CABLE CONDUIT IN THE OVERLINER LAYER. (1)
5. EACH PIEZOMETER PAIR SHALL INCLUDE SEPARATE CABLES AND ONE CABLE CONDUIT. THE PIEZOMETER CABLE CONDUITS SHALL GENERALLY RUN ALONG THE TOE OF THE NORTHEAST VALLEY SLOPE OR BETWEEN LATERAL PIPES TO AVOID INTERFERENCE WITH THE SOLUTION COLLECTION PIPES TO THE EXTENT POSSIBLE. THE PIEZOMETER CONDUITS AND CABLES SHALL REPORT TO CENTRAL READOUT BOXES TO BE LOCATED ON THE IN–HEAP STORAGE POND EMBANKMENT CREST FOR STAGE 1 PIEZOMETERS, AND AT THREE DISTINCT LOCATIONS ON THE UPSTREAM SIDE OF THE PAD FOR STAGES 2 THROUGH 6. THE STAGE 2 THROUGH 6 READOUT BOXES WILL BE RELOCATED AS THE PAD IS BUILT INCREMENTALLY UP THE VALLEY, BUT ARE NOT SHOWN FOR CLARITY. A 12" DIAMETER PERFORATED CPT PIPE SHALL BE INSTALLED ADJACENT TO EACH PIEZOMETER CABLE CONDUIT TO ALLOW MANUAL PIEZOMETER READINGS, IF NEEDED, SHOULD PROBLEMS BE EXPERIENCED WITH THE INSTALLED PIEZOMETERS.
6. PIEZOMETER CABLE LEADS SHALL BE INSTALLED WITH SLACK TO ACCOUNT FOR POTENTIAL MOVEMENT OF THE CABLE CONDUITS DURING HEAP LOADING. PIEZOMETER CABLE LEAD LENGTH QUANTITIES FOR INSTALLATION HAVE BEEN INCREASED BY 10 PERCENT TO ACCOUNT FOR THIS.
7. A MINIMUM OF 36" OVERLINER COVER SHALL BE MAINTAINED OVER 12" DIA OR GREATER CPT PIPES. NO CONSTRUCTION TRAFFIC SHALL BE ALLOWED TO PASS OVER THESE AREAS AFTER THE HEADER PIPES ARE INSTALLED. CPT SOLUTION PIPES OF LESS THAN 12" DIA SHALL BE ENCAPSULATED WITHIN THE 36"–THICK LAYER OF OVERLINER. (1)
8. THE ENDS OF THE CABLE CONDUITS SHALL BE PLUGGED WITH NON–WOVEN GEOTEXTILE WRAPPED AROUND THE PIEZOMETER CABLES TO MITIGATE LCRS SAND ENTERING THE CONDUITS.
9. VIBRATING WIRE PIEZOMETERS 1 THRU 12 SHALL BE GEOKON MODEL 4500AL–170kPa (OR EQUIVALENT), PIEZOMETERS 13 THROUGH 18 SHALL BE GEOKON MODEL 4500S–350kPa (OR EQUIVALENT), AND PIEZOMETERS 19 THRU 58 SHALL BE GEOKON MODEL 4500AL–70kPa (OR EQUIVALENT). (1) (2)




2 — SETTLEMENT MONUMENT

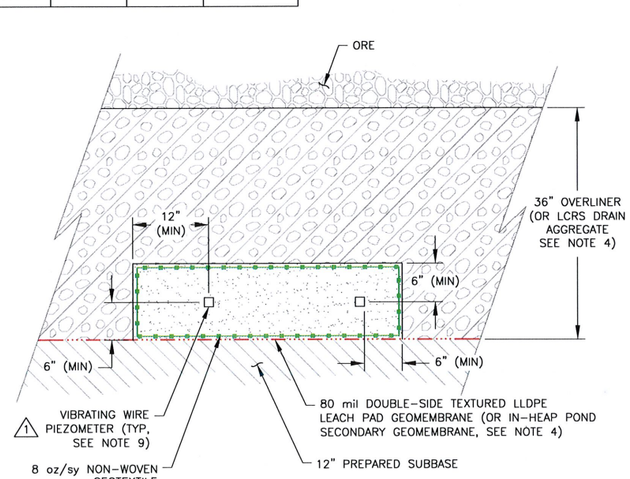
NTS




A — PIEZOMETER INSTALLATION IN OVERLINER

(SEE NOTE 4)

1" = 1' AT FULL SIZE (ANSI D)
1" = 2' AT HALF SIZE (ANSI B)




B — PIEZOMETER INSTALLATION IN OVERLINER

(SEE NOTE 4)

1" = 1' AT FULL SIZE (ANSI D)
1" = 2' AT HALF SIZE (ANSI B)

Mar. 17/17

## REVISION:

(1) MODIFIED CPT DRAIN CALLOUTS AS SIZE VARIES. MODIFIED NOTES 4 AND 7. ADDED NOTE 9.

(2) MODIFIED PIEZOMETER LOCATIONS FOR 23 THROUGH 48. ADDED 49 THROUGH 58. ADDED PIEZOMETER MODELS TO TABLE AND FIGURE REFERENCES.

| REV | DATE | DESCRIPTION | APP'D CADD |
|---|---|---|---|
| A | 10/10/16 | ISSUED FOR CLIENT REVIEW | JTR CB |
| 0 | 01/27/17 | ISSUED FOR CONSTRUCTION | JTR CB |
| 1 | 02/07/17 | REISSUED FOR CONSTRUCTION | JTR CB |
| 2 | 03/17/17 | REISSUED FOR CONSTRUCTION | JTR CB |

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ACTIVITY CODE N/A XREF N/A

CLIENT: FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

PROJECT: FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE: INSTRUMENTATION SECTIONS AND DETAILS

| DESIGNED BY | JTR | LOCATION | PROJECT NUMBER | DRAWING NUMBER | REVISION |
|---|---|---|---|---|---|
| DRAWN BY | CB | DV101 | 00089.99 | 720 | 2 |

LAST SAVED BY: JREIVA
DRAWING PATH: G:\101\00089.99\CAD\CAD Dept\Drawings\DV101-00089-99_720.dwg
PRINTED BY: CHAD BEANLAND, PRINT TIME: 3/17/2017 4:45 PM

# Appendices

A *Alaska Dam Safety Program, Hazard Potential Classification and Jurisdictional Review for Fort Knox Barnes Creek Heap Leach Facility Dam* (Approved by ADNR)

B Select Information from Knight Piésold Empirical Testing of 600-mm-dia ADS N-12 CPT

C Stability Analyses Results and Calculations

D Settlement Analysis Results and Supporting Information

E Peggs, et al., 2005, Assessment of Maximum Allowable Strains in Polyethylene and Polypropylene Geomembranes

F Operational Period Process Water Balance Figures

Knight Piésold
CONSULTING

# Appendix A

# Alaska Dam Safety Program, Hazard Potential Classification and Jurisdictional Review for Fort Knox Barnes Creek Heap Leach Facility Dam (Approved by ADNR)

Print Form

# Alaska Dam Safety Program

## HAZARD POTENTIAL CLASSIFICATION AND JURISDICTIONAL REVIEW

This form is used to review and indicate the hazard potential classification of an artificial barrier in accordance with 11 AAC 93.157 and to determine if the barrier is a dam under the jurisdiction of the Alaska dam safety regulations, based on the definition articulated under Alaska Statute 46.17.900 (3), and summarized as follows:

"Dam" includes an artificial barrier, and its appurtenant works, which may impound or divert water and which...

- has or will have an impounding capacity at maximum water storage elevation of 50 acre-feet and is at least 10 feet in height measured from the lowest point at either the upstream or downstream toe of the dam to the crest of the dam; or
- is at least 20 feet in height measured from the lowest point at either the upstream or downstream toe of the dam to the crest of the dam; or
- poses a threat to lives and property as determined by the department after an inspection.

In accordance with 11 AAC 93.151, an artificial barrier with a Class I or Class II designation is determined to meet the third definition of a dam, regardless of its geometry.

***Please complete items 1 through 21. Attach additional information as necessary. This form must be certified and stamped on page 3 by an Alaska-registered professional engineer, qualified in accordance with 11 AAC 93.193.***

1. Name of barrier: Fort Knox Barnes Creek Heap Leach Facility Dam

   National Inventory of Dams (NID) number: _____ (Assigned by Department)

   Name of stream: Barnes Creek

   General location and region: Fort Knox Mine, Fairbanks, AK

   Legal location: Township 2N Range 2E Section S8, 9, 16, 17 Meridian Fairbanks

   Purpose and type of barrier: Solution storage / membrane-faced compacted rockfill

   This barrier is: ☐ Existing ☒ Proposed ☐ Under construction

   Current hazard potential classification: ☐ I ☐ II ☐ III ☒ Not assigned

2. Owner: Fairbanks Gold Mining Inc. (FGMI)

   Address: #1 Fort Knox Road

   PO Box 73726

   Fairbanks, AK 99707-3726

   Contact name: Ms. Bartly Kleven

   Phone: (907) 490-2207

3. Is barrier federally owned, or regulated by the Federal Energy Regulatory Commission?

   ☐ Yes (stop here) ☒ No (complete form)

4. Maximum crest height of barrier: 95 * feet
   Measured from: ☒ Upstream toe ☐ Downstream toe ☐ Offstream toe
   Basis of height: ☒ Conceptual design drawing ☐ Detailed design drawing
   ☐ As-built drawing ☐ Field measurement ☐ NID data

5. Maximum impoundment volume: 383 (in interstitial pore spaces) acre-feet
   Surface area of reservoir at maximum storage: 37 (in ore heap at el. 1542 ft) acres
   Average depth of reservoir above bottom of barrier: 45 ** feet (live storage)
   Basis of volume estimate: ☐ Surface area multiplied by average depth
   ☐ Bathymetry
   ☐ NID data
   ☒ Other: ACAD calculation from drawings and average ore porosity

6. Downstream development: ☒ Yes ☐ No ☐ Unknown
   Type of development (check all that apply):
   - ☐ Homes
   - ☐ School
   - ☐ Community halls, churches, etc.
   - ☐ Industrial or commercial property
   - ☐ Major highway
   - ☐ Primary roads
   - ☐ Secondary or rural roads
   - ☐ Railroads
   - ☐ Power or communication utilities
   - ☐ Water or wastewater treatment facilities or lines
   - ☐ Overnight campgrounds
   - ☐ Public parks or trails
   - ☐ Fish hatchery or processor
   - ☒ Barrier owner's property or facilities
   - ☐ Other utilities: _____
   - ☒ Other development: Tailing Storage Facility

   Basis of observations: ☒ Ground reconnaissance ☐ Aerial reconnaissance
   ☒ Aerial photo ☒ Other: FGMI information and general site knowledge
   Date of observations: June 13, 2016

7. Proximity of development to downstream channel (add maps or other information as necessary):
   Distance downstream from barrier: ~28.5 m to Cheena Hot Springs Cabin
   Distance from stream bed: ~ 2 m to Fish Creek Channel below TSF
   Relative elevation above streambed: ~ 395 ft from dam crest to Fish Creek below TSF

8. Is development in the inundation zone of a flood from an uncontrolled release of water from the barrier?
   ☐ Yes ☒ No ☐ Unknown

9. Was a dam break analysis conducted? ☐ Yes ☒ No
   Basis of determining inundation zone:
   - ☐ Simplified DAMBRK model
   - ☐ DAMBRK model
   - ☐ NWS FLDWAV model
   - ☐ HEC-1 model
   - ☐ Other: _____

   (Please attach calculations)
   Maximum depth and velocity of flow through development: _____

10. Is development at risk from improper operation or a "sunny day" failure?
    ☐ Yes ☒ No ☐ Unknown

11. Is development at risk from an incremental increase in the flood if the barrier fails under flood conditions?
    ☐ Yes ☒ No ☐ Unknown
    Flood condition evaluated: ☒ 100 year ☐ ½ PMF ☐ PMF ☐ Other: _____

* Maximum crest height of barrier taken from upstream toe because the downstream side of the barrier is fully buttressed.
** The average depth of the reservoir above the bottom of barrier is based on the full impoundment volume to the invert of the emergency spillway (solution plus ore) divided by the surface area at that elevation (el. 1542 ft).

12. Could an uncontrolled release cause other significant property damage or loss? ☐ Yes ☒ No ☐ Unknown

Description: _____

13. Could an uncontrolled release effect public health? ☐ Yes ☒ No ☐ Unknown

Description: _____

14. Is the reservoir created by the barrier the primary water supply for a community of more than 500 residents? ☐ Yes ☒ No ☐ Unknown

Is a backup water supply available? ☐ Yes ☐ No ☐ Unknown ☒ N/A

15. Is barrier located on waters important to anadromous fish? ☐ Yes ☒ No ☐ Unknown

Are anadromous fish waters at risk of damage or loss if an uncontrolled release occurs? ☐ Yes ☐ No ☐ Unknown ☒ N/A

16. Proposed hazard potential classification: ☐ Class I (High) ☐ Class II (Significant) ☒ Class III (Low)

17. Basis of classification:
- ☐ Quantitative - Numerical dam break analysis conducted
- ☒ Qualitative - Limited engineering calculations
- ☐ Preliminary - No engineering calculations

18. Comments: This facility is located near the headwaters of the Barnes Creek valley, just upstream of the Fort Knox Tailing Storage Facility (TSF). It is located in the catchment area reporting to the TSF and the open pit, therefore the amount of runoff intercepted by the facilities will be unchanged. The impoundment of this facility will be small relative to that of the TSF.

19. Certified by: Thomas F. Kerr (Print name)

Date: June 14, 2016

Company: Knight Piésold and Co.

Phone: (303) 629-8788

Engineer's Seal and Signature

June 14/16.

Notes:
1. *This form must be certified and stamped by an Alaska-registered professional engineer qualified in accordance with 11 AAC 93.193.*
2. *The information presented in this form may be overruled based on current data that reveals a higher level of confidence in the quality of information necessary to make the appropriate determinations.*
3. *Anadromous fish waters are determined in accordance with 11 AAC 195.010 (a).*
4. *Alaska dam safety regulations are articulated under 11 AAC 93.151 through 11 AC 93.291 (Article 3).*

**FOR DEPARTMENT USE ONLY**

Jurisdictional Status of Barrier:

☒ Dam under state jurisdiction

Reasons:
- ☒ Height
- ☒ Height and storage volume
- ☐ Hazard potential classification
- ☐ Anadromous fish stream
- ☐ Other: _____

☐ Barrier is not a dam under state jurisdiction

Reasons:
- ☐ Height
- ☐ Height and storage volume
- ☐ Hazard potential classification
- ☐ Federal ownership or regulation
- ☐ Other: _____

Concur with proposed hazard potential classification: ☒ Yes ☐ No

Hazard potential classification based on current information: ☒ Yes ☐ No

Official hazard potential classification:

☐ Class I (High) ☐ Class II (Significant) ☒ Class III (Low)

Comments: NID number will be assigned when COA to Construct a Dam is issued.

Reviewed by: Charles F. Cobb

Title: State Dam Safety Engineer

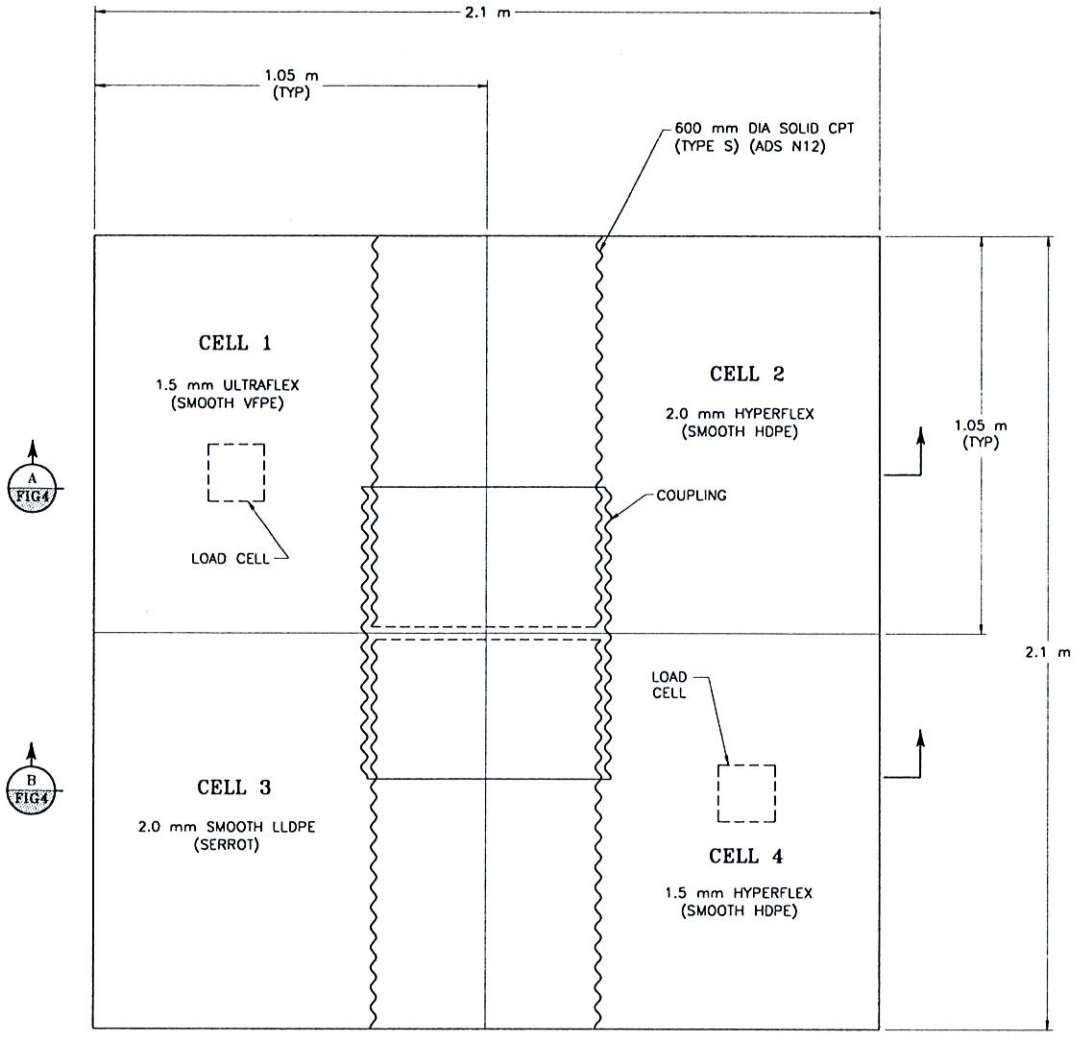
Signature: CFCobb

Date: Oct. 17, 2016

Knight Piésold
CONSULTING

# Appendix B

# Select Information from Knight Piésold Empirical Testing of 600-mm-dia ADS N-12 CPT



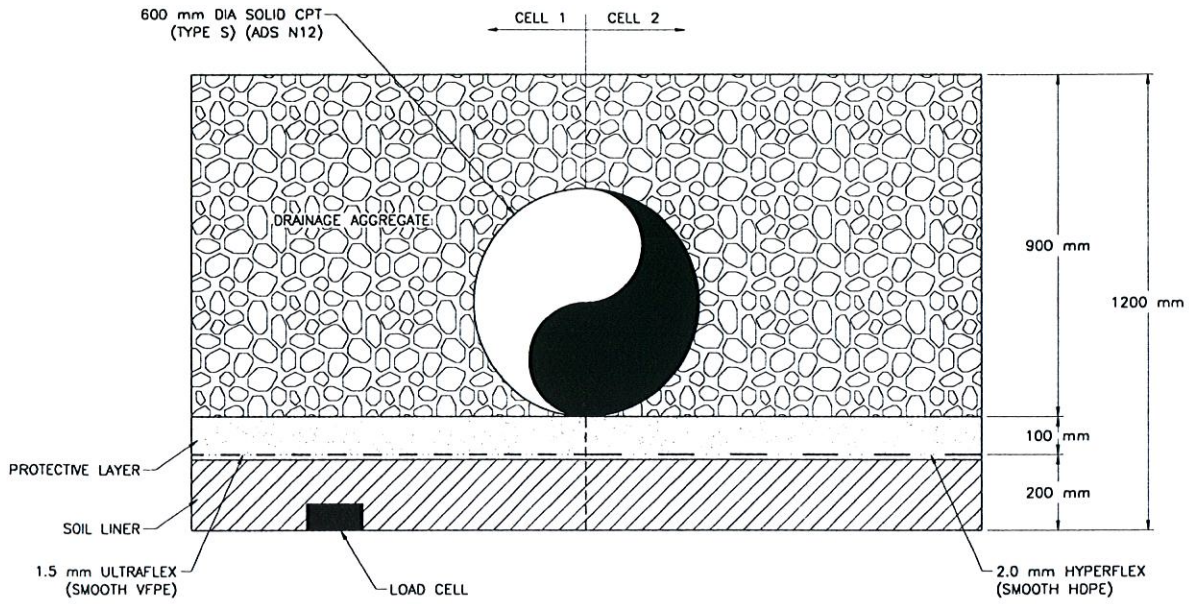

PLAN LAYOUT OF TEST SETUP

200 0 200 400 mm

1:20

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| CLIENT: MINERA YANACOCHA S.R.L. | | | | | | | |
| PROJECT: 150 METER HEAP HEIGHT EVALUATION REPORT | | | | | | | |
| TITLE: TEST CONFIGURATION NO. 2 SHEET 1 OF 2 | | | | | | | |
| Knight Piésold CONSULTING | | | | | | | |
| DESIGNED BY | RA | CHECKED BY | | DATE | FILENAME | FIGURE No. | REV. |
| DRAWN BY | WA | APPROVED BY | | 11/17/00 | 606F441B | 3 | B |

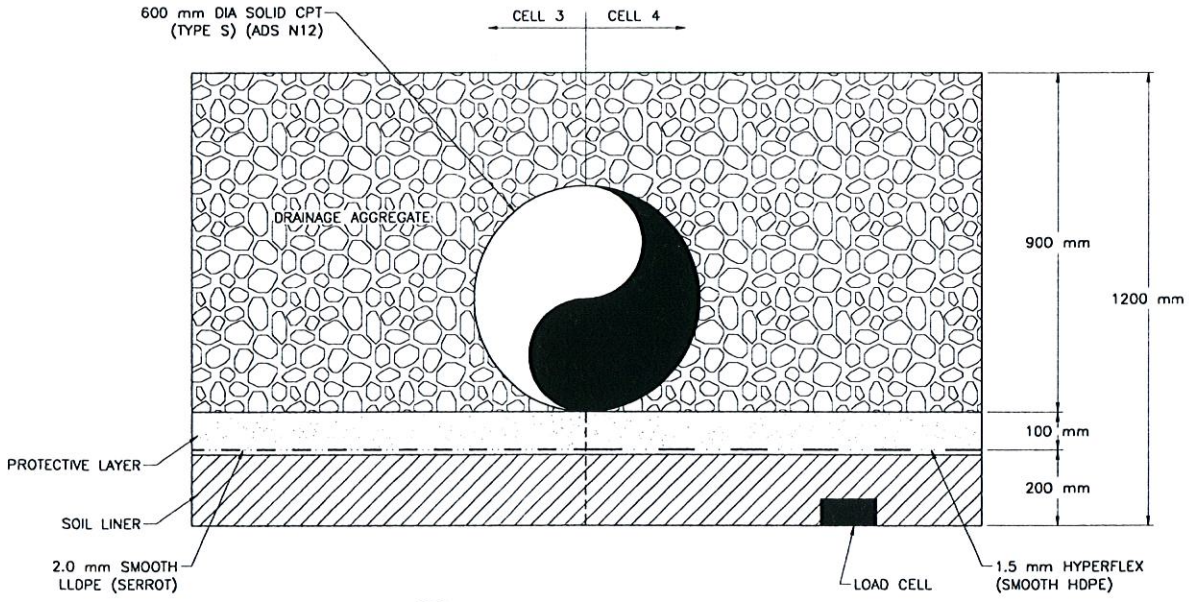
PLOT SCALE 1:2M | XREF. NO | NA




A FIG3 LAYOUT OF TEST SETUP

200 0 200 400 mm

1:20




B FIG3 LAYOUT OF TEST SETUP

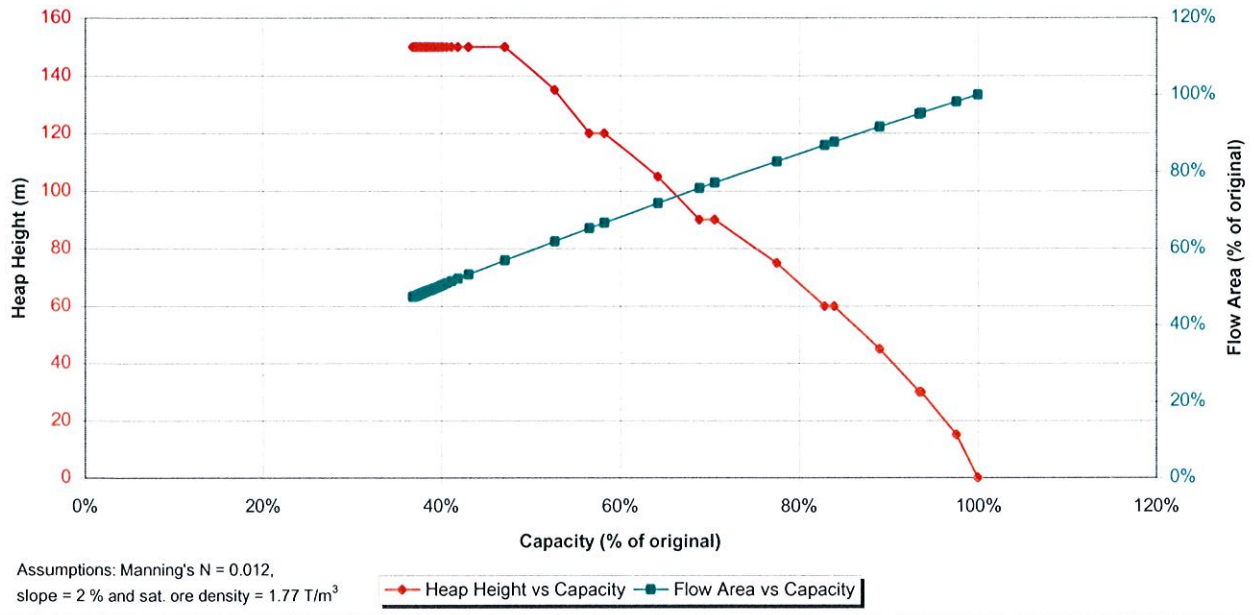
200 0 200 400 mm

1:20

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| CLIENT | MINERA YANACOCHA S.R.L. | | | | | | |
| PROJECT | 150 METER HEAP HEIGHT EVALUATION REPORT | | | | | | |
| TITLE | TEST CONFIGURATION NO. 2<br>SHEET 2 OF 2 | | | | | | |
| | Knight Piésold CONSULTING | | | | | | |
| DESIGNED BY | RA | CHECKED BY | | DATE | FILENAME | FIGURE No. | REV. |
| DRAWN BY | WA | APPROVED BY | | 11/17/00 | 606F453B | 4 | B |

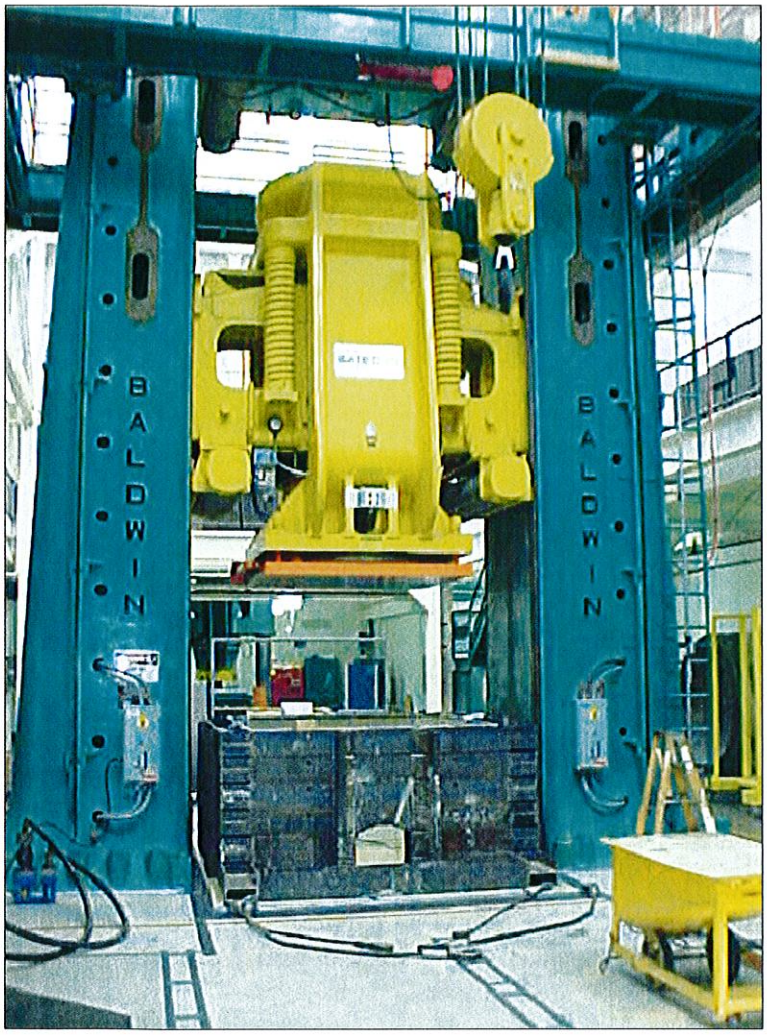
PLOT SCALE 1:2M | XREF. NO | NA

## FIGURE 16
### TEST CONFIGURATION 2 - ADS PIPE
### HEAP HEIGHT vs PIPE CAPACITY

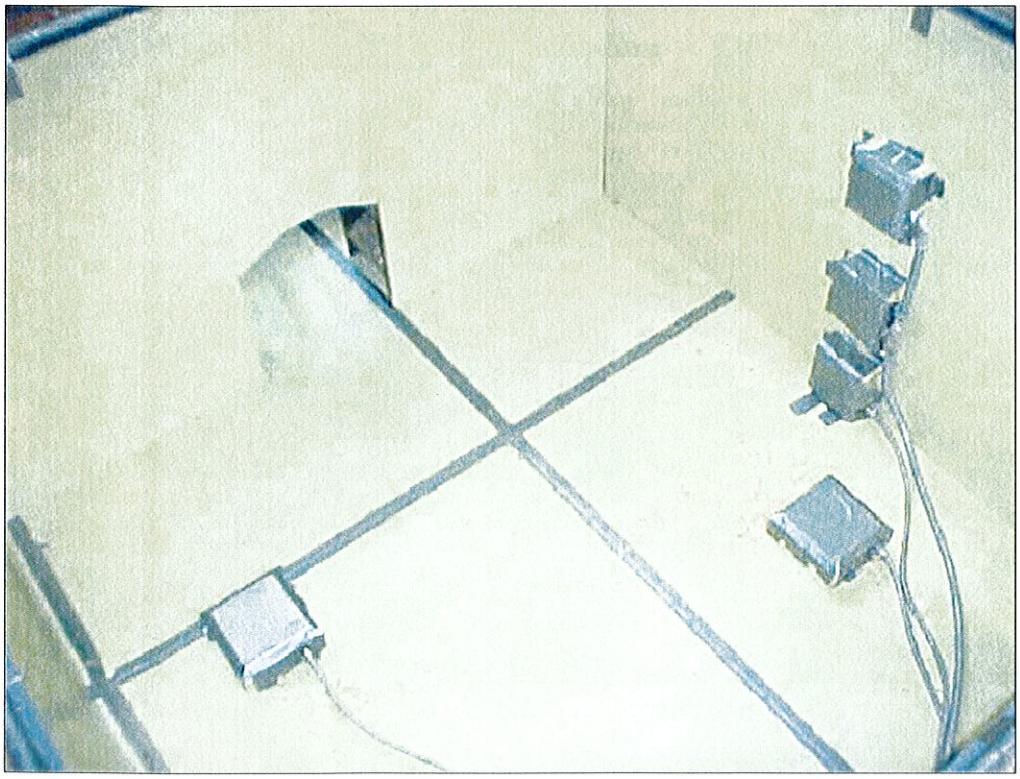



Test Date: 08/29/10 Start Time: 09:38:49

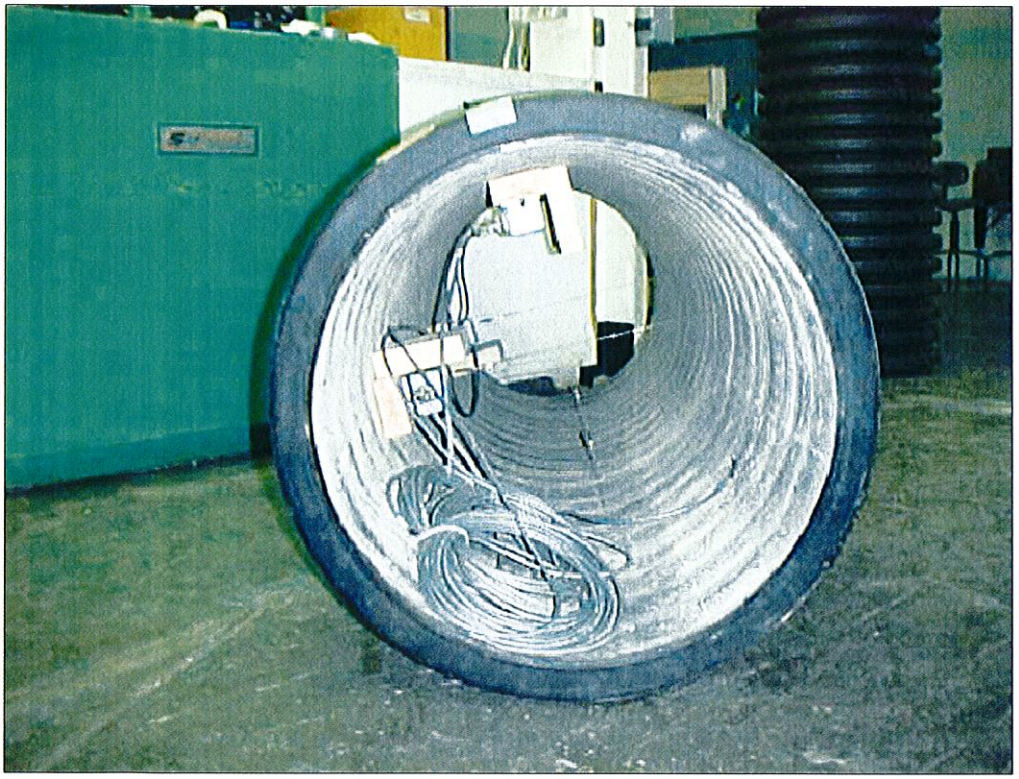
| Ellapsed Time (hr) | Applied Load (lb) | Applied Load (psi) | Approximate Heap Height (m) | Average Vertical Deflection (in) | Average Horizontal Deflection (in) | Flow Area (sf) | Flow Area (% of orig) | Equivalent Diameter (ft) | Equivalent Diameter (in) | Flow Carrying Capacity (cfs) | Flow Carrying Capacity (% of orig) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.000 | 938 | 0.2 | 0 | 0.000 | 0.000 | 3.14 | 100% | 2.00 | 24.00 | 34.66 | 100% |
| 0.283 | 217,656 | 42.0 | 15 | 0.916 | 0.492 | 3.08 | 98% | 1.98 | 23.78 | 33.82 | 98% |
| 0.567 | 431,719 | 83.3 | 30 | 1.806 | 0.722 | 2.99 | 95% | 1.95 | 23.42 | 32.47 | 94% |
| 0.650 | 431,719 | 83.3 | 30 | 1.865 | 0.732 | 2.99 | 95% | 1.95 | 23.40 | 32.39 | 93% |
| 0.967 | 650,000 | 125.4 | 45 | 2.804 | 0.923 | 2.88 | 92% | 1.92 | 22.98 | 30.87 | 89% |
| 1.217 | 856,875 | 165.3 | 60 | 3.913 | 1.150 | 2.76 | 88% | 1.87 | 22.48 | 29.11 | 84% |
| 1.350 | 859,375 | 165.8 | 60 | 4.120 | 1.174 | 2.73 | 87% | 1.86 | 22.37 | 28.73 | 83% |
| 1.600 | 1,069,531 | 206.3 | 75 | 5.121 | 1.225 | 2.60 | 83% | 1.82 | 21.82 | 26.88 | 78% |
| 1.900 | 1,285,469 | 248.0 | 90 | 6.324 | 1.125 | 2.42 | 77% | 1.76 | 21.07 | 24.49 | 71% |
| 2.067 | 1,287,813 | 248.4 | 90 | 6.595 | 1.061 | 2.38 | 76% | 1.74 | 20.88 | 23.91 | 69% |
| 2.317 | 1,494,063 | 288.2 | 105 | 7.397 | 0.916 | 2.26 | 72% | 1.69 | 20.34 | 22.29 | 64% |
| 2.600 | 1,716,250 | 331.1 | 120 | 8.464 | 0.723 | 2.09 | 67% | 1.63 | 19.60 | 20.20 | 58% |
| 2.750 | 1,718,125 | 331.4 | 120 | 8.756 | 0.653 | 2.05 | 65% | 1.62 | 19.39 | 19.62 | 57% |
| 3.000 | 1,921,250 | 370.6 | 135 | 9.443 | 0.481 | 1.94 | 62% | 1.57 | 18.88 | 18.28 | 53% |
| 3.267 | 2,142,031 | 413.2 | 150 | 10.470 | 0.239 | 1.79 | 57% | 1.51 | 18.11 | 16.36 | 47% |
| 4.267 | 2,138,438 | 412.5 | 150 | 11.270 | 0.078 | 1.67 | 53% | 1.46 | 17.51 | 14.95 | 43% |
| 5.267 | 2,140,000 | 412.8 | 150 | 11.509 | 0.032 | 1.64 | 52% | 1.44 | 17.33 | 14.54 | 42% |
| 6.267 | 2,138,594 | 412.5 | 150 | 11.656 | 0.005 | 1.62 | 51% | 1.43 | 17.21 | 14.28 | 41% |
| 7.267 | 2,141,406 | 413.1 | 150 | 11.763 | -0.016 | 1.60 | 51% | 1.43 | 17.13 | 14.10 | 41% |
| 8.267 | 2,139,063 | 412.6 | 150 | 11.852 | -0.031 | 1.59 | 51% | 1.42 | 17.06 | 13.95 | 40% |
| 9.267 | 2,142,656 | 413.3 | 150 | 11.927 | -0.046 | 1.58 | 50% | 1.42 | 17.01 | 13.84 | 40% |
| 10.267 | 2,140,000 | 412.8 | 150 | 11.992 | -0.058 | 1.57 | 50% | 1.41 | 16.96 | 13.73 | 40% |
| 11.267 | 2,145,625 | 413.9 | 150 | 12.050 | -0.069 | 1.56 | 50% | 1.41 | 16.91 | 13.62 | 39% |
| 12.267 | 2,144,531 | 413.7 | 150 | 12.101 | -0.078 | 1.55 | 49% | 1.41 | 16.87 | 13.54 | 39% |
| 13.267 | 2,141,250 | 413.0 | 150 | 12.147 | -0.085 | 1.55 | 49% | 1.40 | 16.84 | 13.47 | 39% |
| 14.267 | 2,144,531 | 413.7 | 150 | 12.190 | -0.094 | 1.54 | 49% | 1.40 | 16.80 | 13.39 | 39% |
| 15.267 | 2,138,438 | 412.5 | 150 | 12.226 | -0.099 | 1.53 | 49% | 1.40 | 16.77 | 13.32 | 38% |
| 16.267 | 2,140,938 | 413.0 | 150 | 12.264 | -0.105 | 1.53 | 49% | 1.40 | 16.75 | 13.28 | 38% |
| 17.267 | 2,141,719 | 413.1 | 150 | 12.298 | -0.112 | 1.52 | 49% | 1.39 | 16.72 | 13.22 | 38% |
| 18.267 | 2,142,656 | 413.3 | 150 | 12.329 | -0.118 | 1.52 | 48% | 1.39 | 16.69 | 13.16 | 38% |
| 19.267 | 2,139,844 | 412.8 | 150 | 12.360 | -0.125 | 1.52 | 48% | 1.39 | 16.67 | 13.11 | 38% |
| 20.267 | 2,143,125 | 413.4 | 150 | 12.388 | -0.131 | 1.51 | 48% | 1.39 | 16.65 | 13.07 | 38% |
| 21.267 | 2,144,844 | 413.7 | 150 | 12.416 | -0.136 | 1.51 | 48% | 1.39 | 16.63 | 13.03 | 38% |
| 22.267 | 2,139,375 | 412.7 | 150 | 12.442 | -0.140 | 1.50 | 48% | 1.38 | 16.61 | 12.99 | 37% |
| 23.267 | 2,142,500 | 413.3 | 150 | 12.469 | -0.146 | 1.50 | 48% | 1.38 | 16.59 | 12.95 | 37% |
| 24.267 | 2,140,000 | 412.8 | 150 | 12.492 | -0.150 | 1.50 | 48% | 1.38 | 16.57 | 12.90 | 37% |
| 25.267 | 2,141,250 | 413.0 | 150 | 12.514 | -0.155 | 1.49 | 48% | 1.38 | 16.55 | 12.86 | 37% |
| 26.267 | 2,140,938 | 413.0 | 150 | 12.537 | -0.161 | 1.49 | 47% | 1.38 | 16.53 | 12.82 | 37% |
| 27.267 | 2,140,938 | 413.0 | 150 | 12.559 | -0.164 | 1.49 | 47% | 1.38 | 16.51 | 12.78 | 37% |




Testing Apparatus



Typical Load Cell Configuration
Two 34,00–kg Load Cells
Attached To The Floor And
Three 9,00–kg Load Cells
Evenly Spaced Along The Wall
(Test Configuration #1)

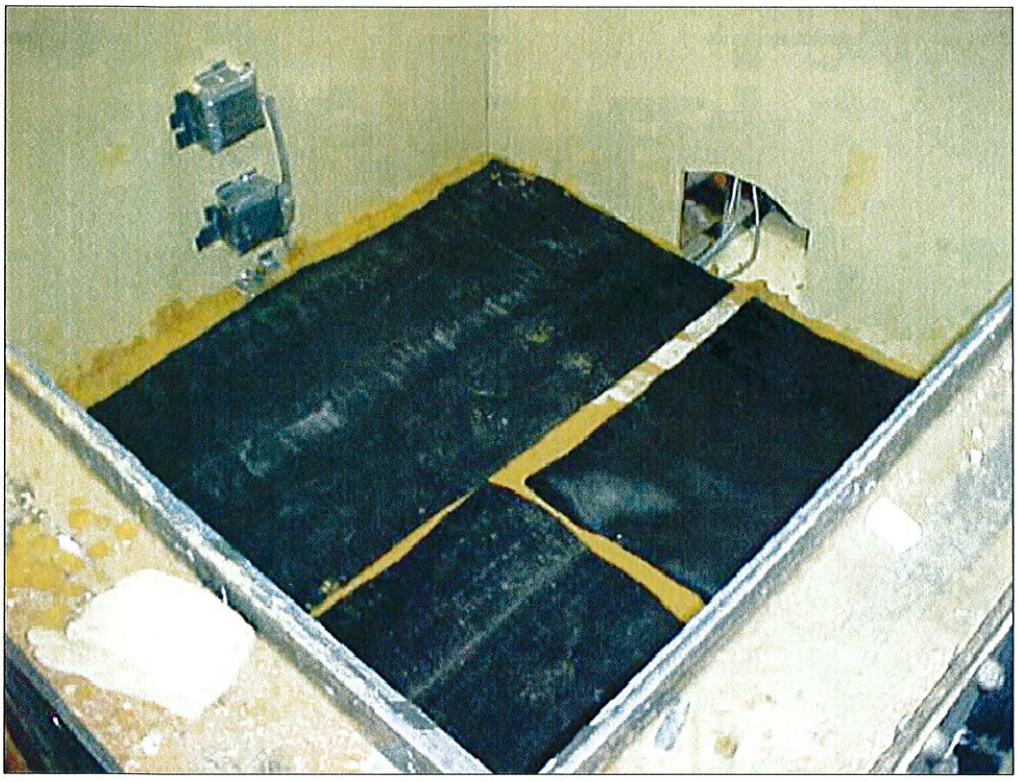


Typical Instrumented Pipe
(Test Configuration #1)

Spreading of Soil Liner
Material
(Test Configuration #1)

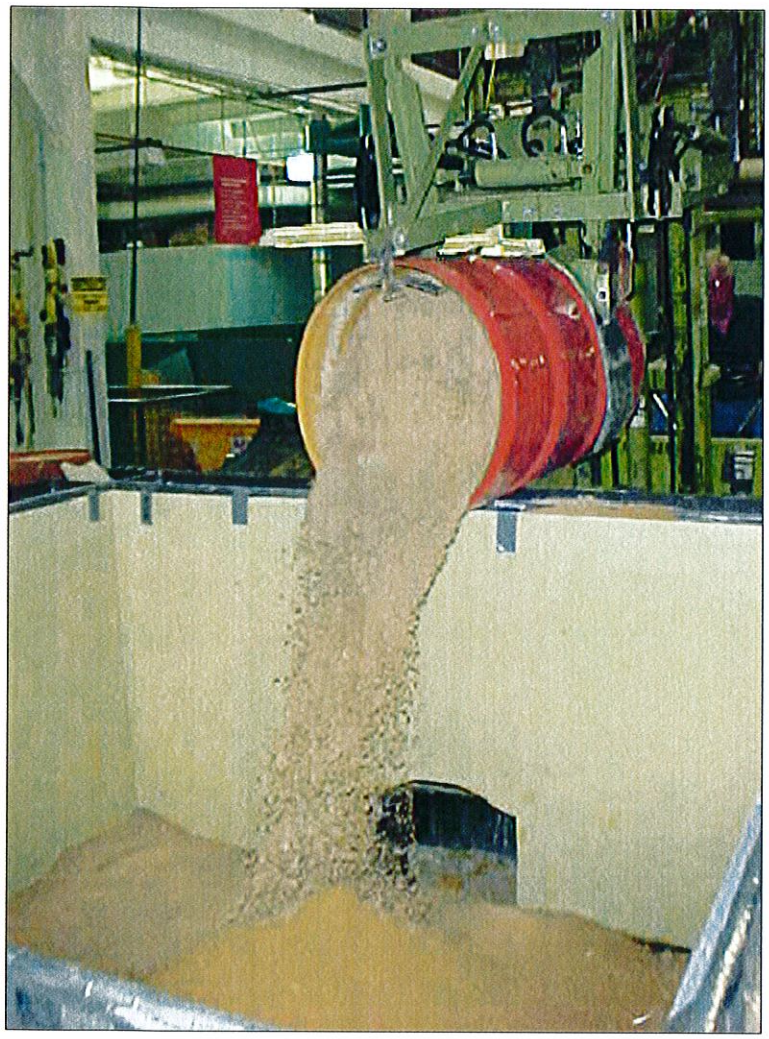
Compaction Of
Soil Liner Material Using
Air–Compressor Pogo–Tamper
(Test Configuration #1)

Soil Liner Material
Compacted Surface
(Test Configuration #1)

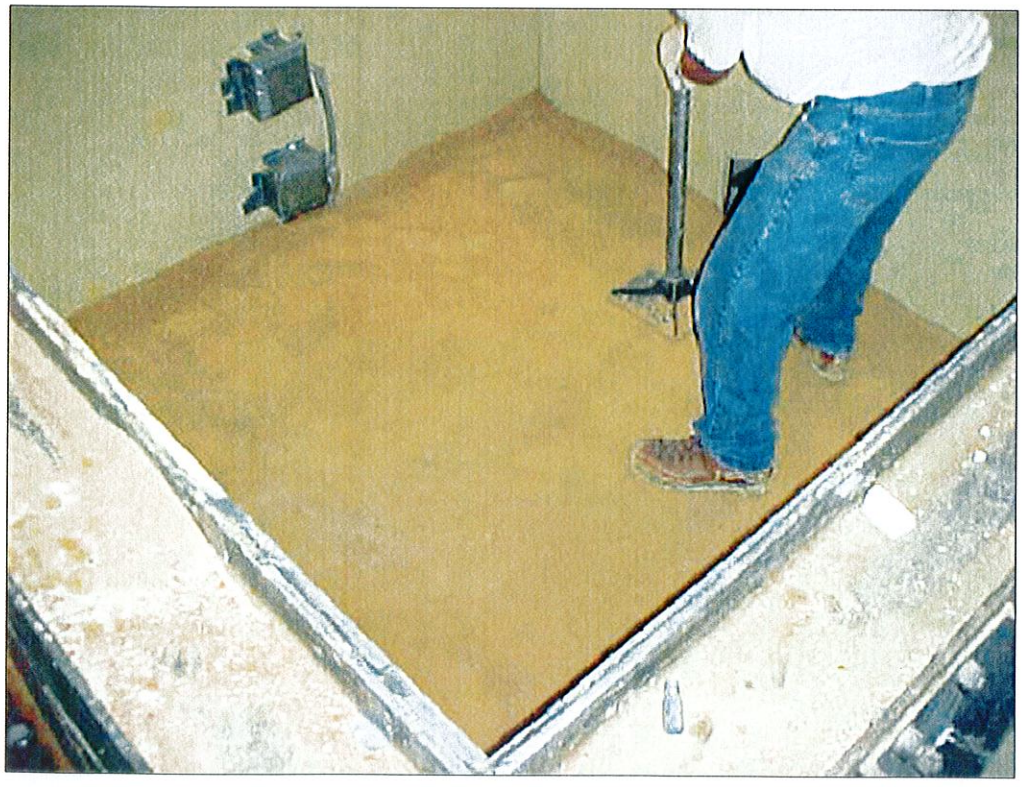


Deployed Geomembrane
(Test Configuration #1)

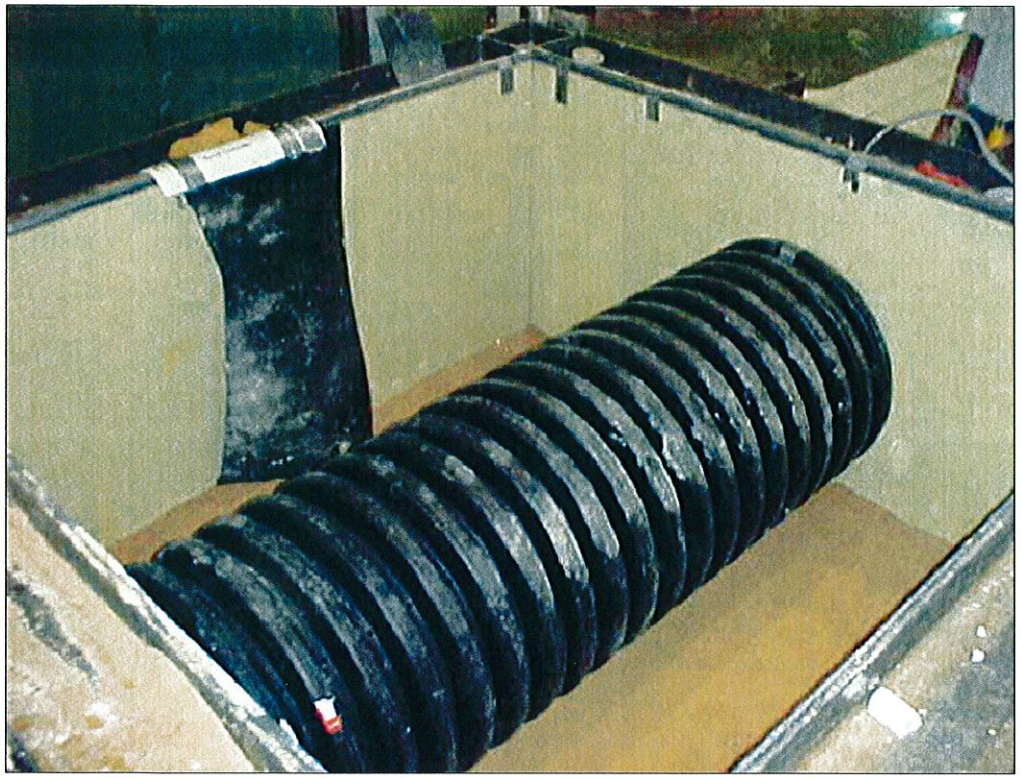
Deployed Geomembrane
And Soil Liner Material
Thickness
(Test Configuration #1)



Deployment Of Protective Layer Material (Test Configuration #1)

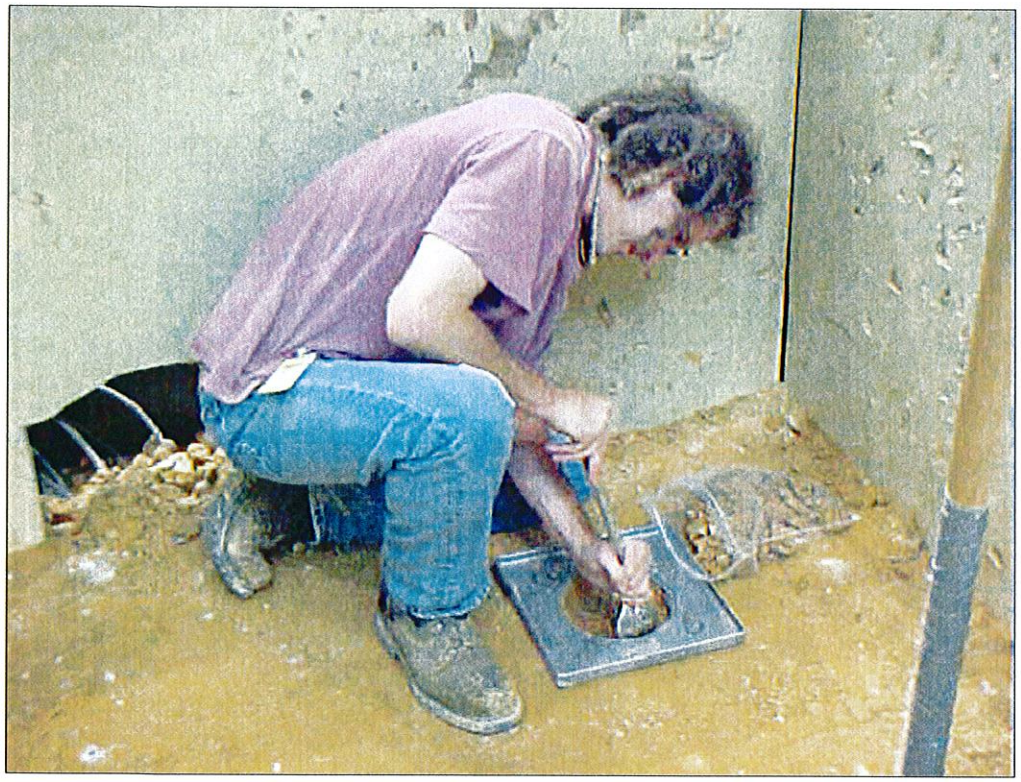


Protective Layer Material Compacted Surface (Test Configuration #1)

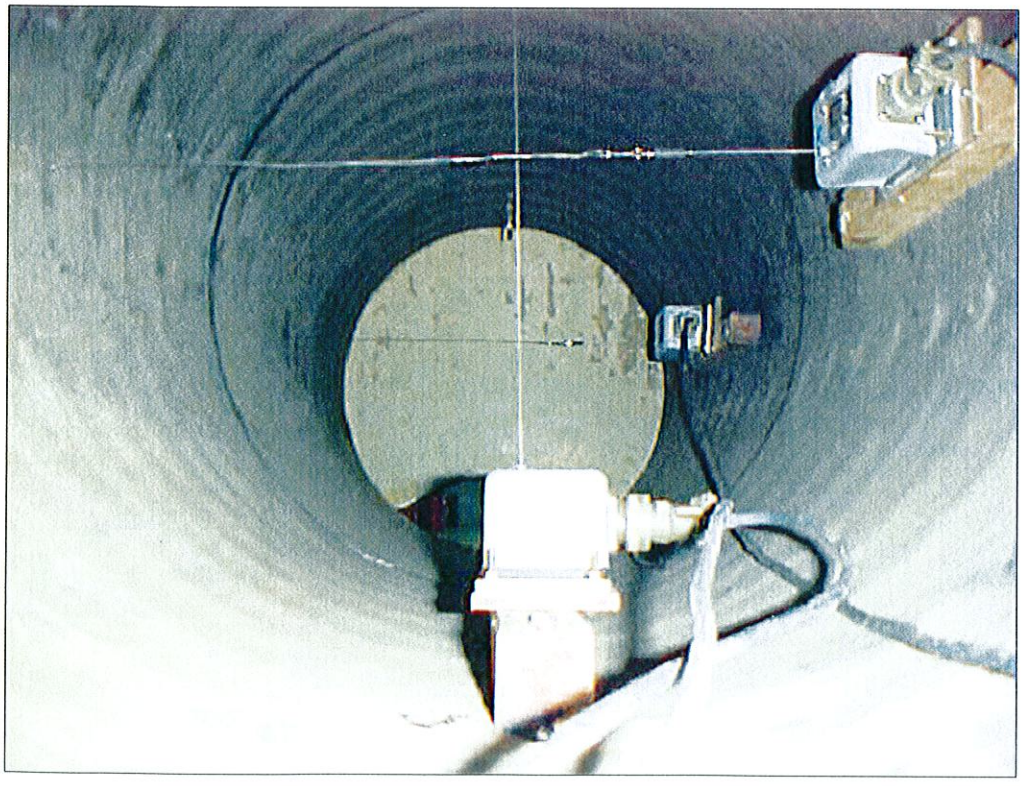


Instrumented Pipe
Placed In Soil Box
(Test Configuration #1)

Completed Soil Box
Configuration Ready For
Loading
(Test Configuration #1)



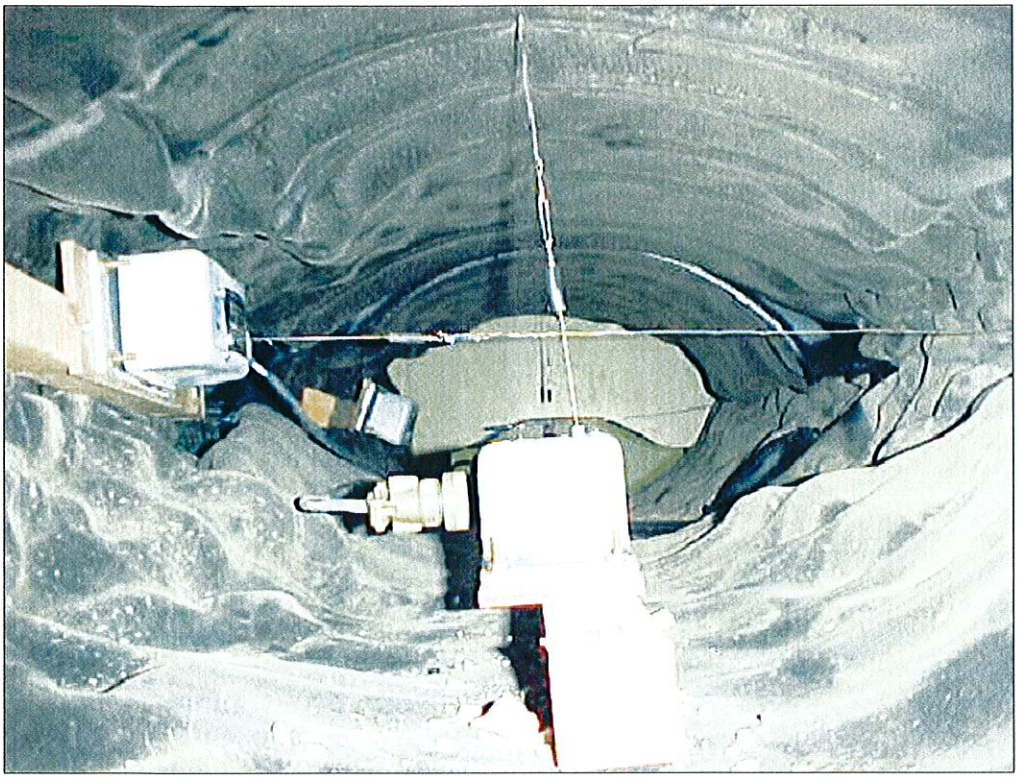
Sandcone Density Test
(Test Configuration #2)



Instrumented Pipe Interior
At Start Of Loading
(Test Configuration #2)

Instrumented Pipe Interior
As Dimpling Of Pipe Begins
(Test Configuration #2)

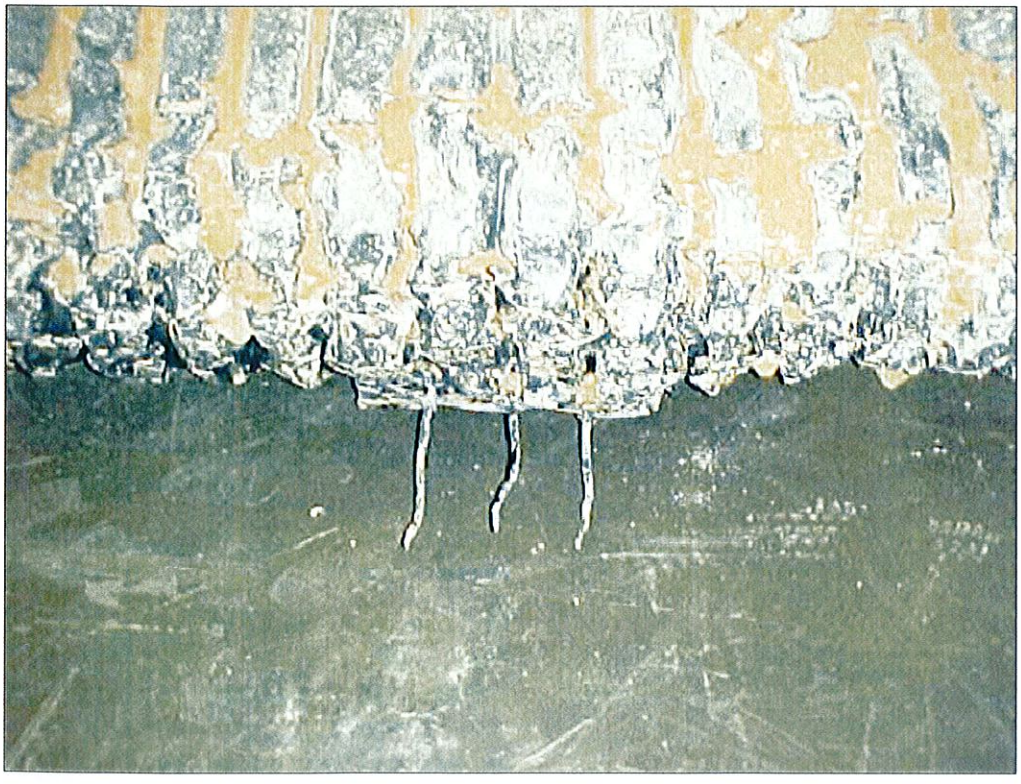
Instrumented Pipe Interior
As Pipe Is Deflected And
Dimpling Continues Showing
Signs Of Pipe Separating
From Coupler
(Test Configuration #2)



Instrumented Pipe Interior
At Final Stages Of Loading
(Test Configuration #2)

Instrumented Pipe Interior
At final Stages Of Loading
(Test Configuration #2)

Deformed Pipe After Being Separated From Soil Liner Material
(Test Configuration #2)



Deformed Pipe After Being Separated From Soil Liner Material Shows Coupler To Be Still Intact
(Test Configuration #2)

# Appendix C

## Stability Analyses Results and Calculations

C-1 Interface Shear Strength Envelopes
C-2 Static Slope Stability Figures
C-3 Estimation of Critical Yield Acceleration (Ky) Figures
C-4 Pseudo-Static Slope Stability Figures
C-5 Estimation of Representative Peak Horizontal Ground Accelerations (PHGA) for Potential Sliding Mass

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# Appendix C-1

# Stability Analyses Results and Calculations

# Interface Shear Strength Envelopes

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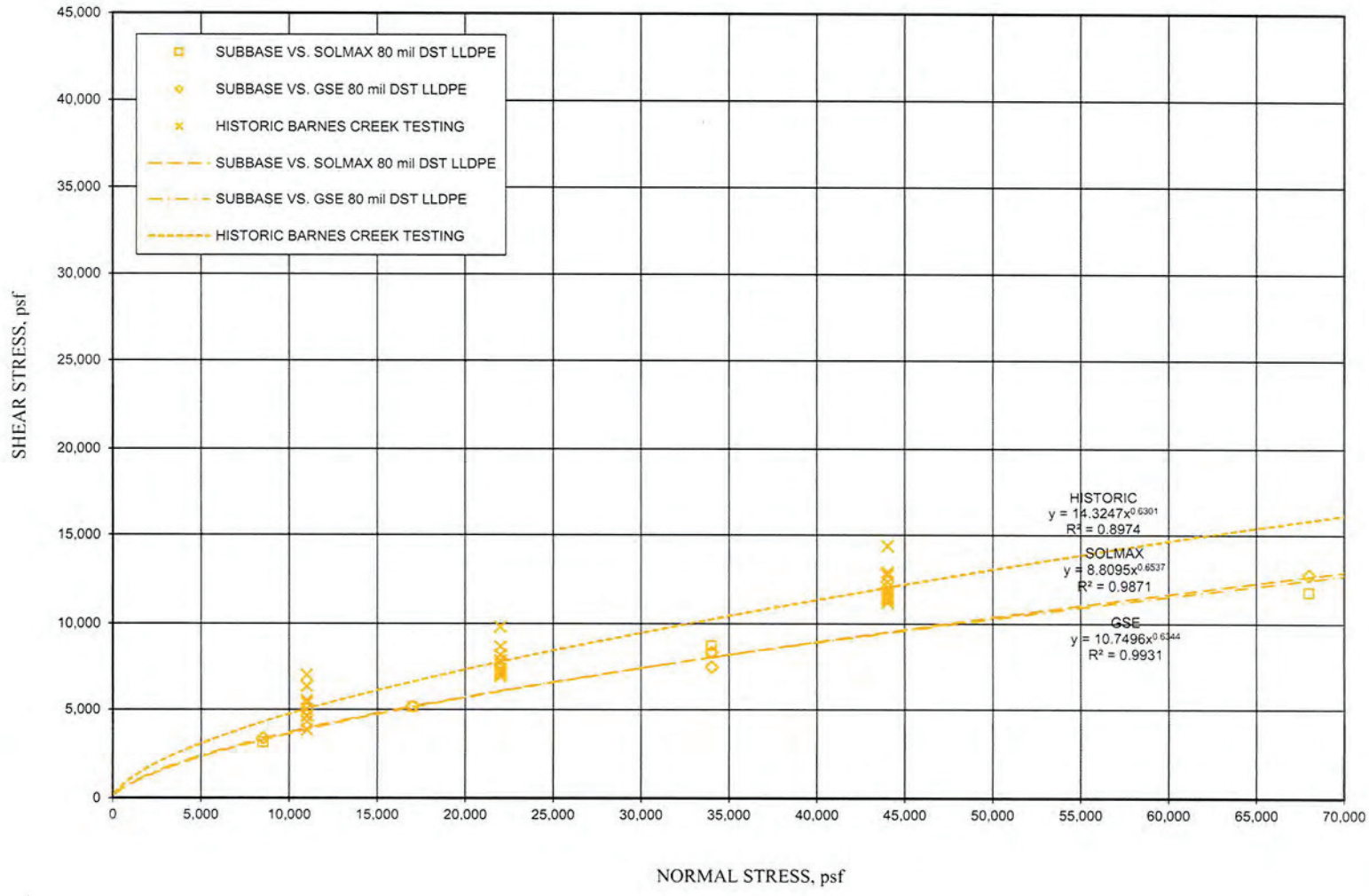
# Figure 1




PROJECT
FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE
SUBBASE INTERFACE
DIRECT SHEAR FAILURE ENVELOPES

CLIENT
FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | PD |
|---|---|
| DRAWN BY | CB |

| LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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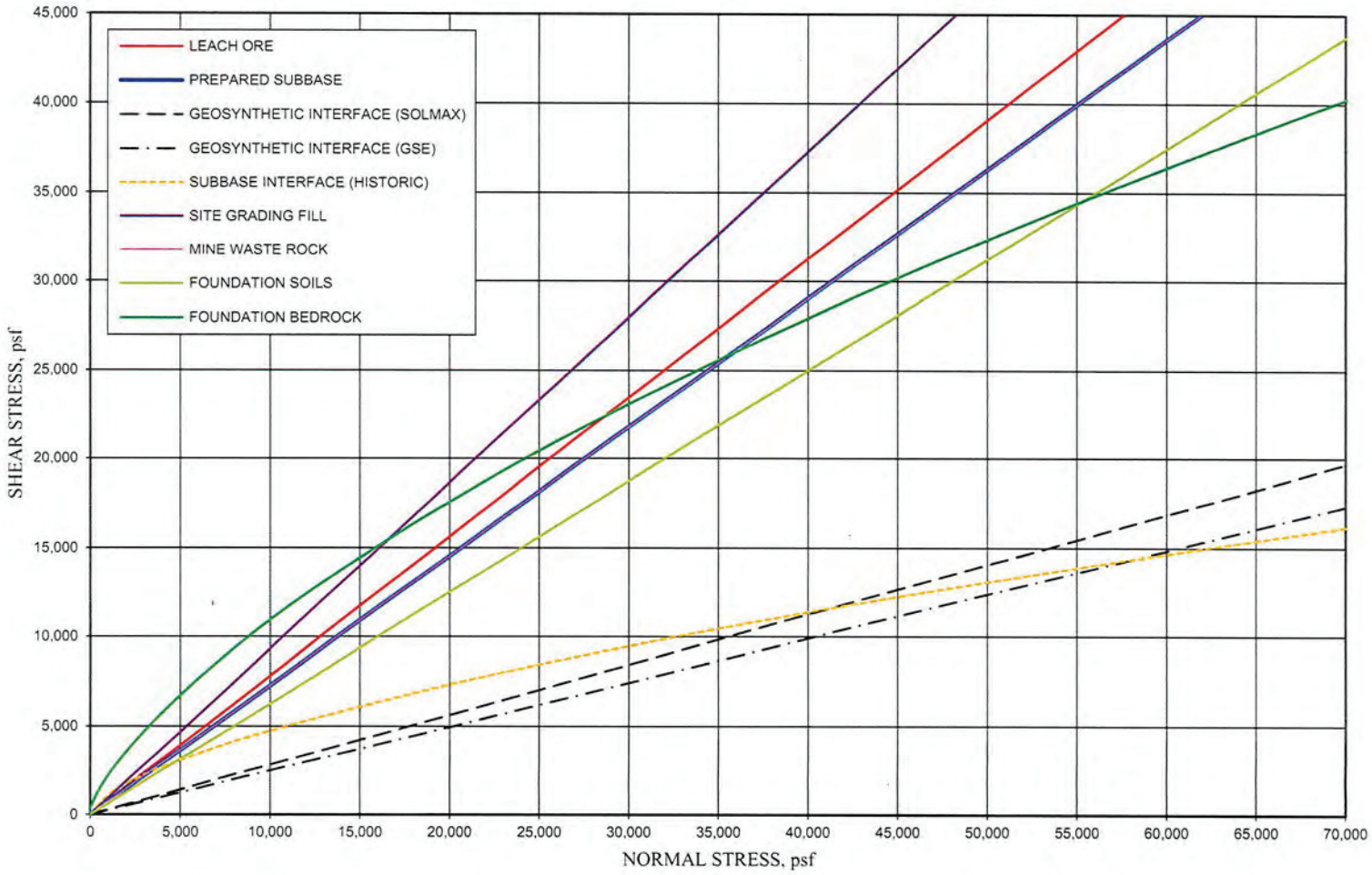
# Figure 2




PROJECT
FORT KNOX PROJECT
BARNES CREEK HEAP LEACH FACILITY FINAL DESIGN

TITLE
SHEAR STRENGTH FAILURE ENVELOPES

CLIENT
FAIRBANKS GOLD MINING, INC.

Knight Piésold CONSULTING

| DESIGNED BY | LOCATION | PROJECT NUMBER | FIGURE NUMBER | REVISION |
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| DRAWN BY | | | | |
| CB | | | | |
| ACTIVITY CODE | FILE NUMBER | | | |
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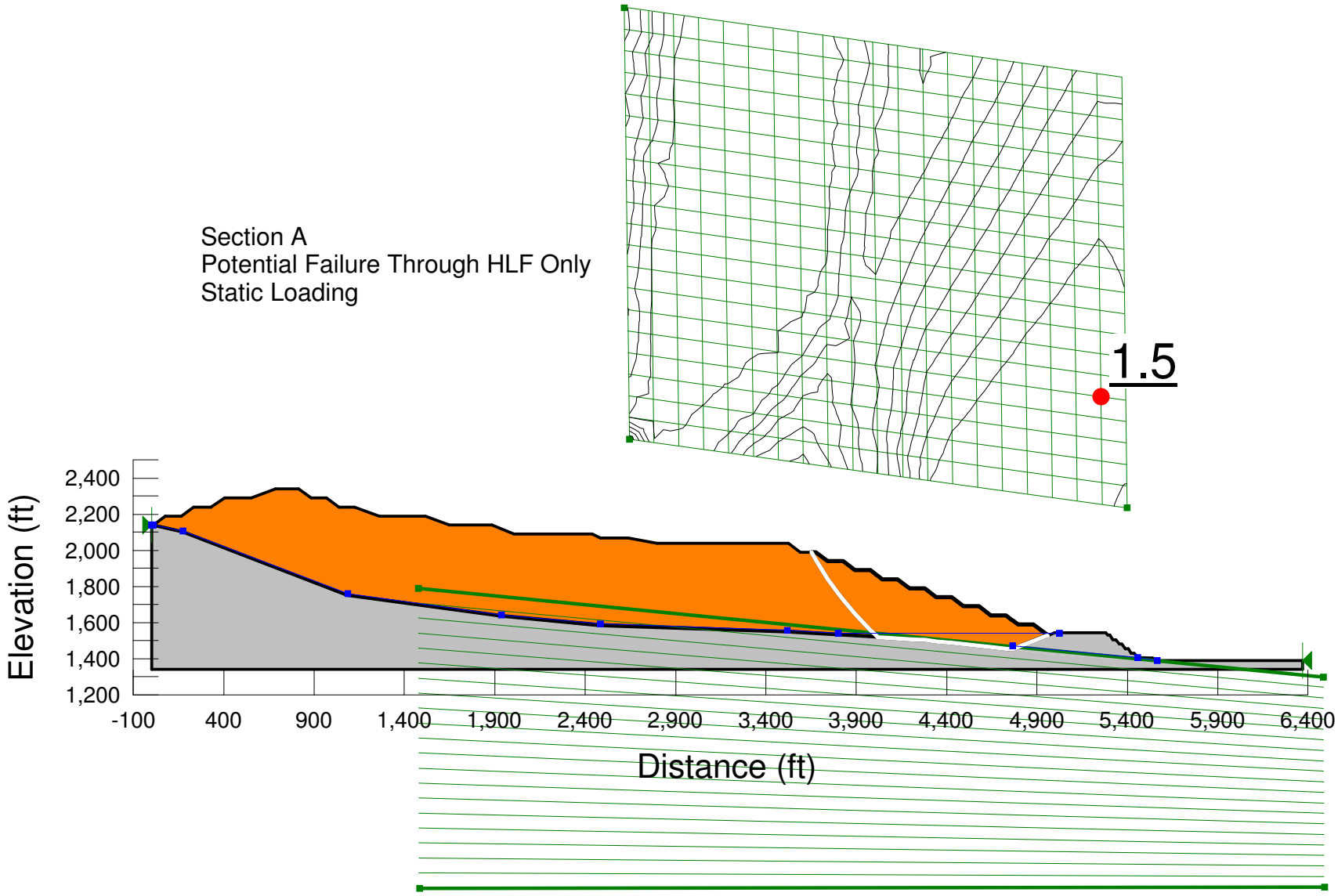
# Appendix C-2

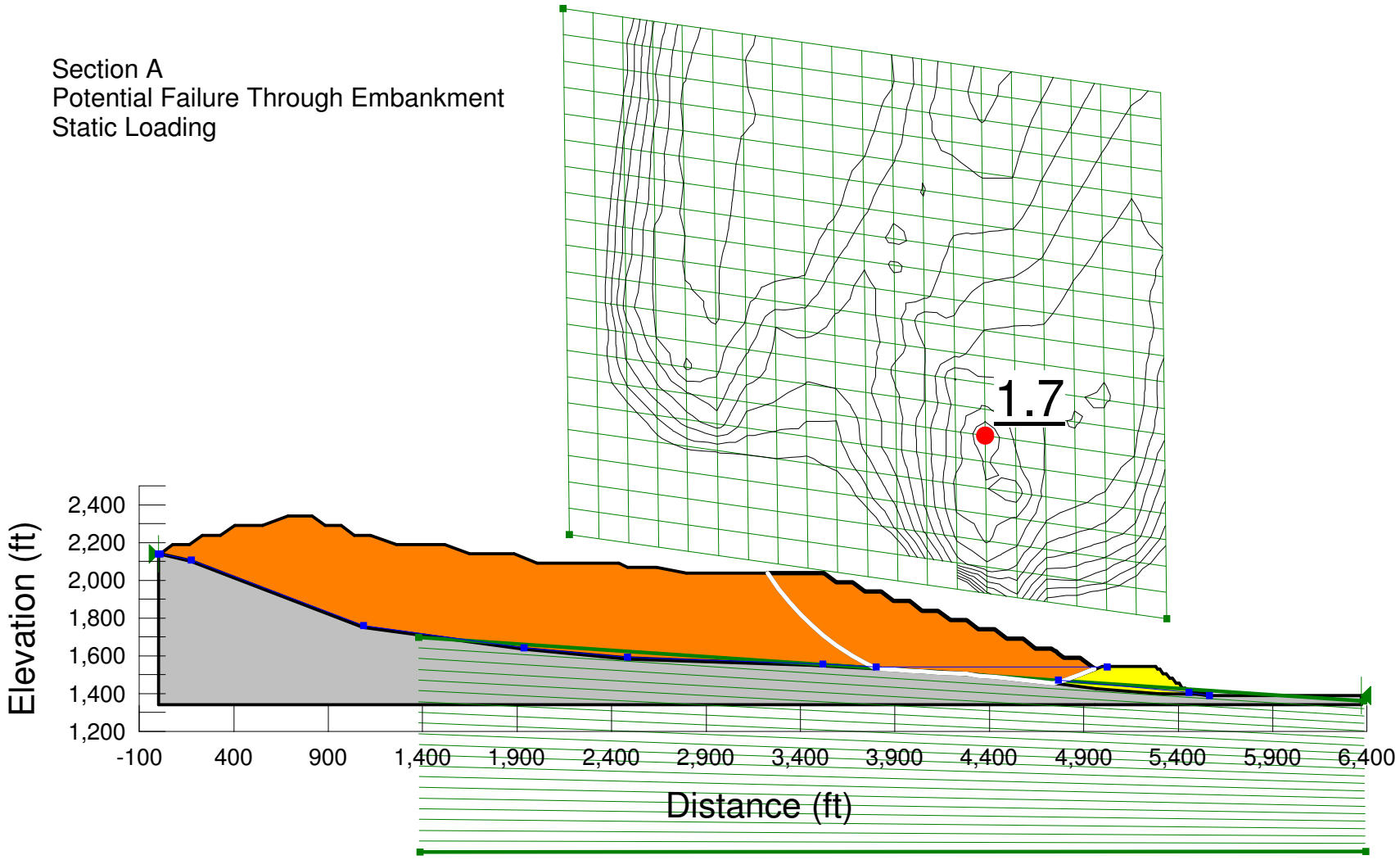
# Stability Analyses Results and Calculations

# Static Slope Stability
# Figures

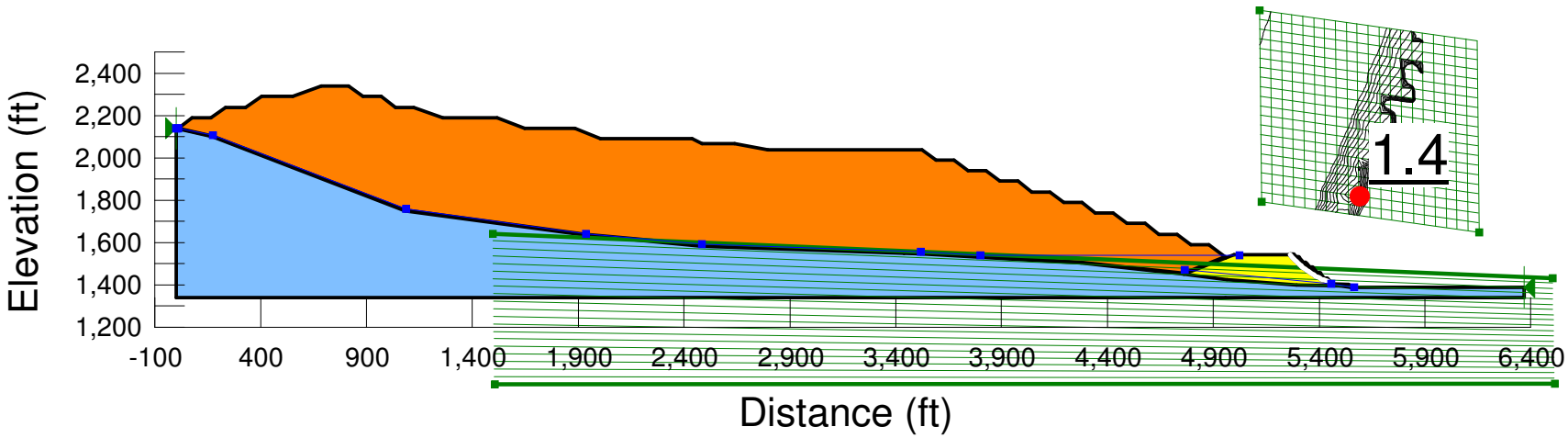
Section A
Potential Failure Through HLF Only
Static Loading

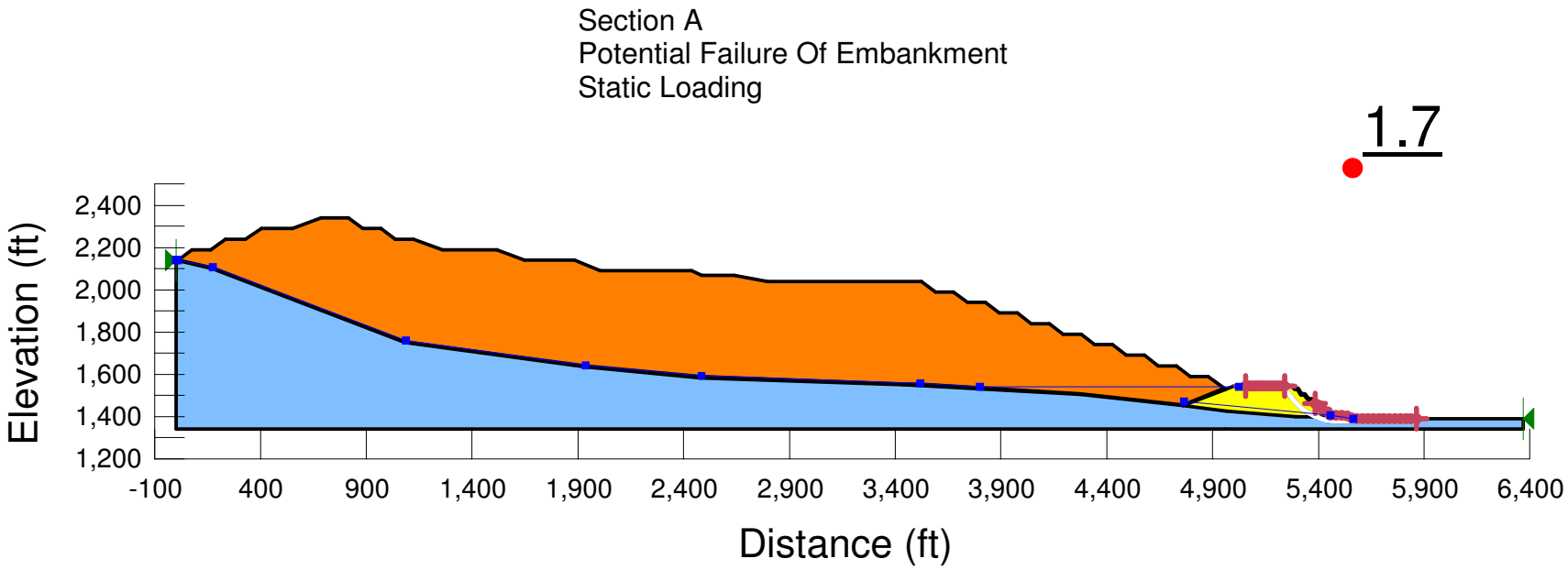


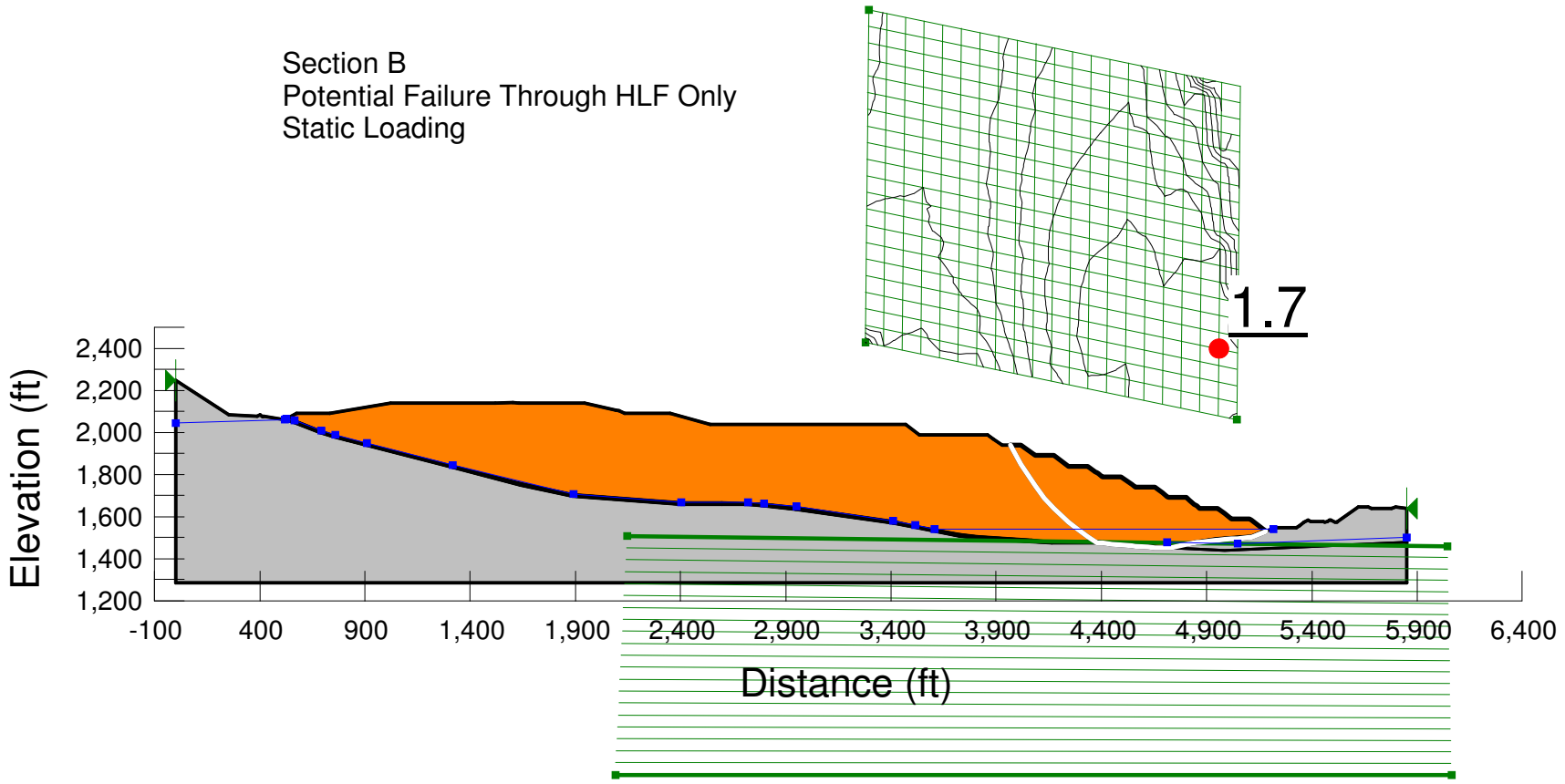
Section A
Potential Failure Through Embankment
Static Loading



Section A
Potential Failure Of Embankment
Static Loading
Surficial Potential Failure




Section A
Potential Failure Of Embankment
Static Loading
1.7
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)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Potential Failure Through HLF Only
Static Loading
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Elevation (ft)
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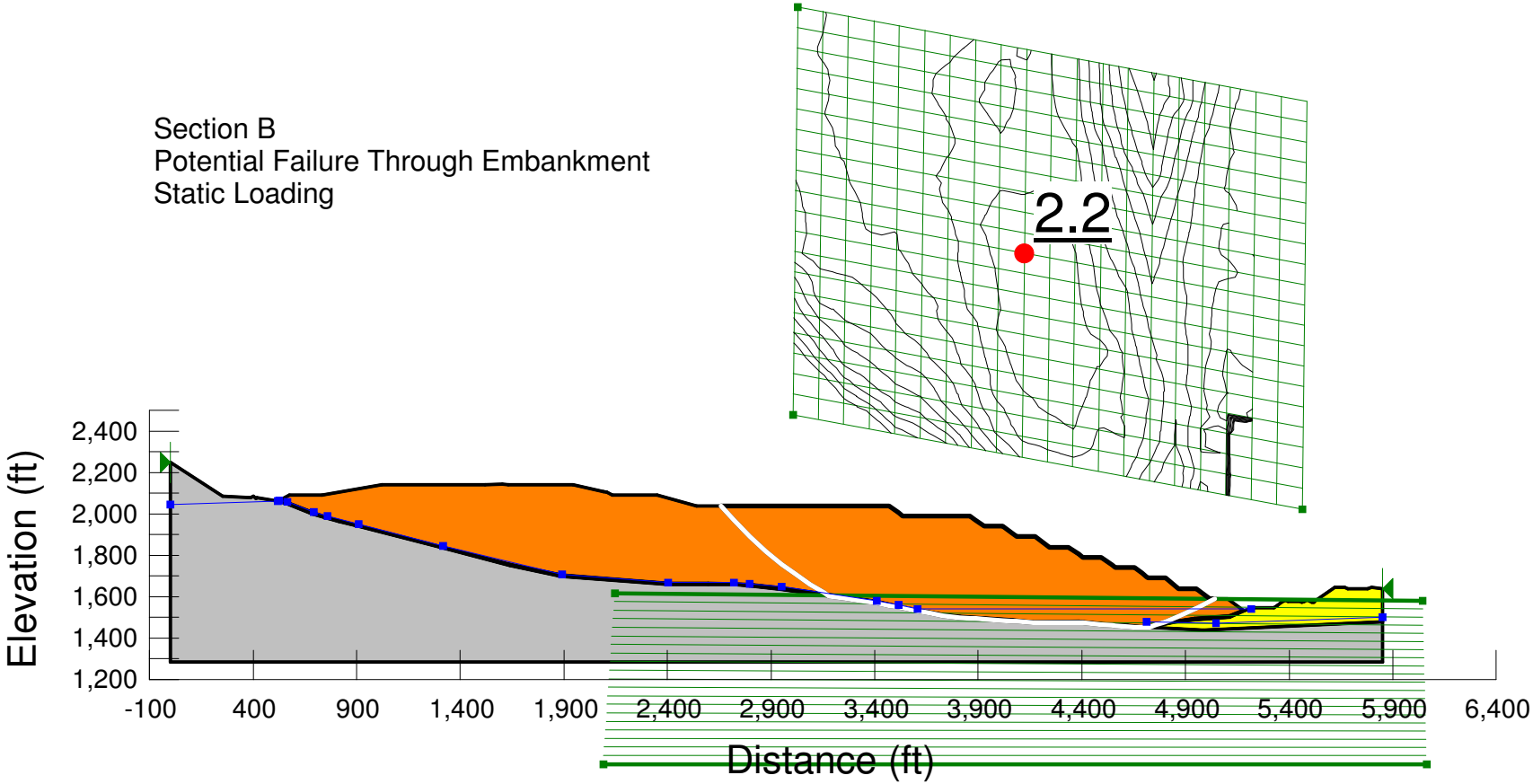
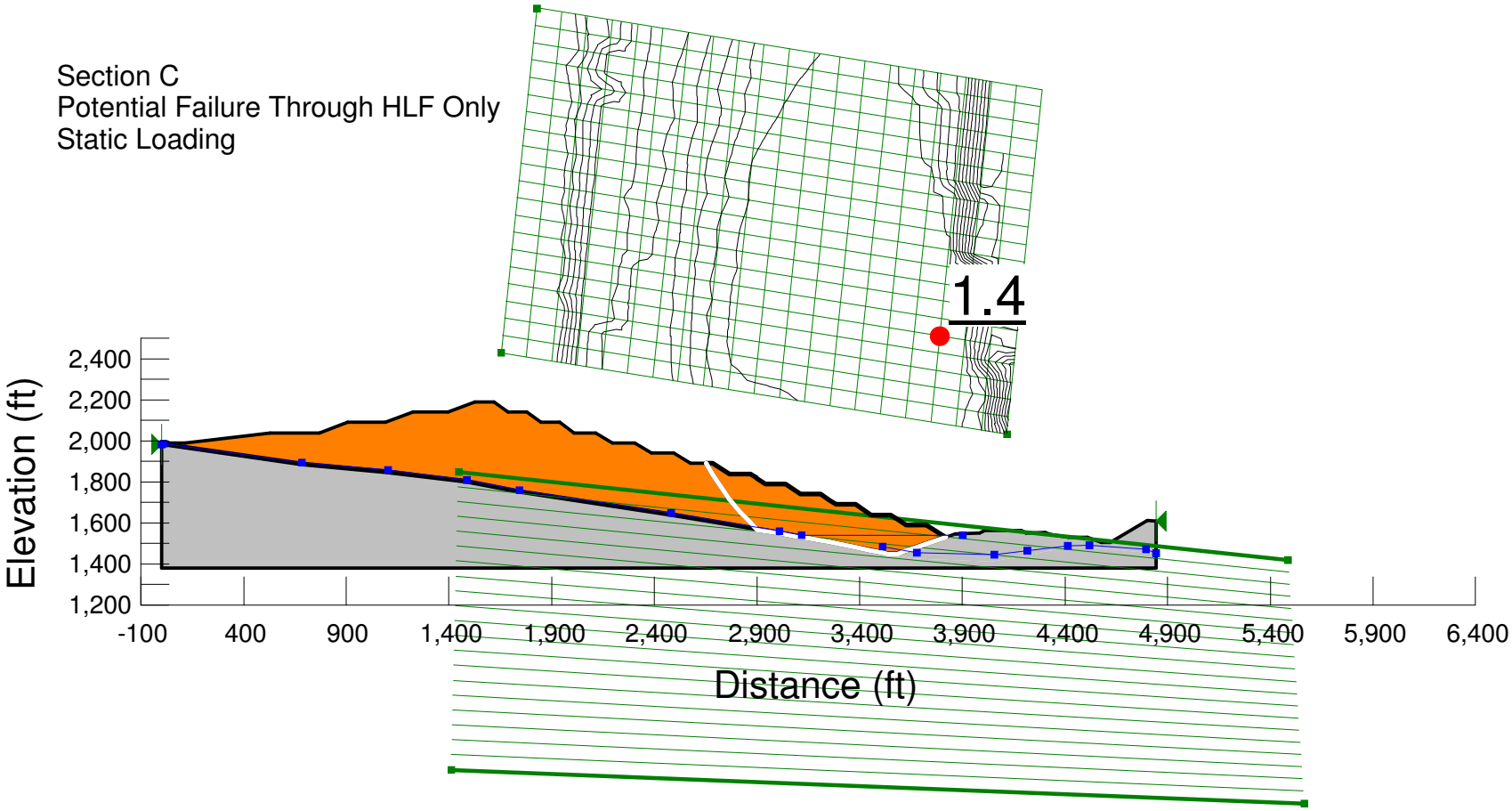
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

Section B
Potential Failure Through Embankment
Static Loading




Section C
Potential Failure Through HLF Only
Static Loading
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Elevation (ft)
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2,000
1,800
1,600
1,400
1,200
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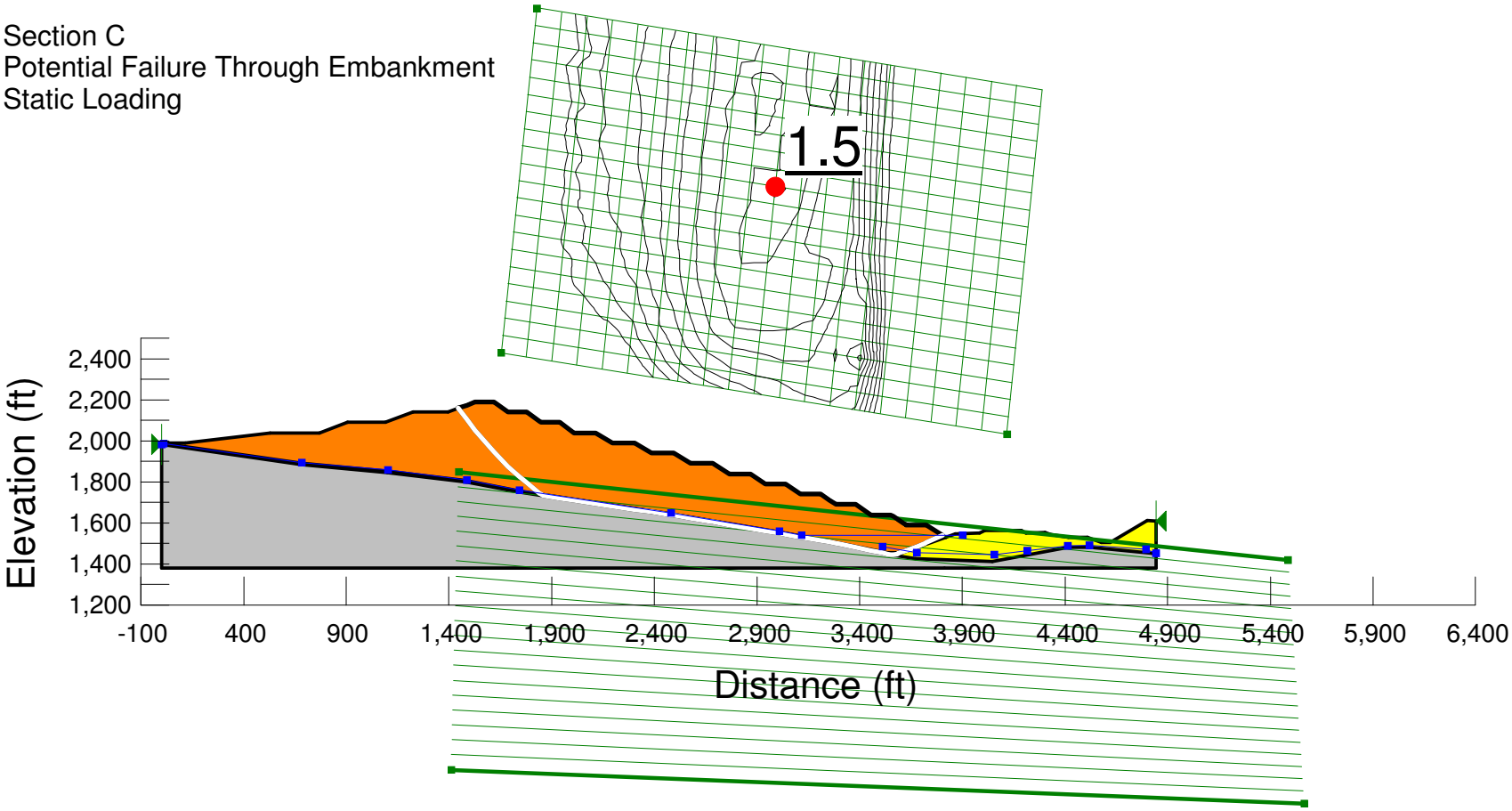
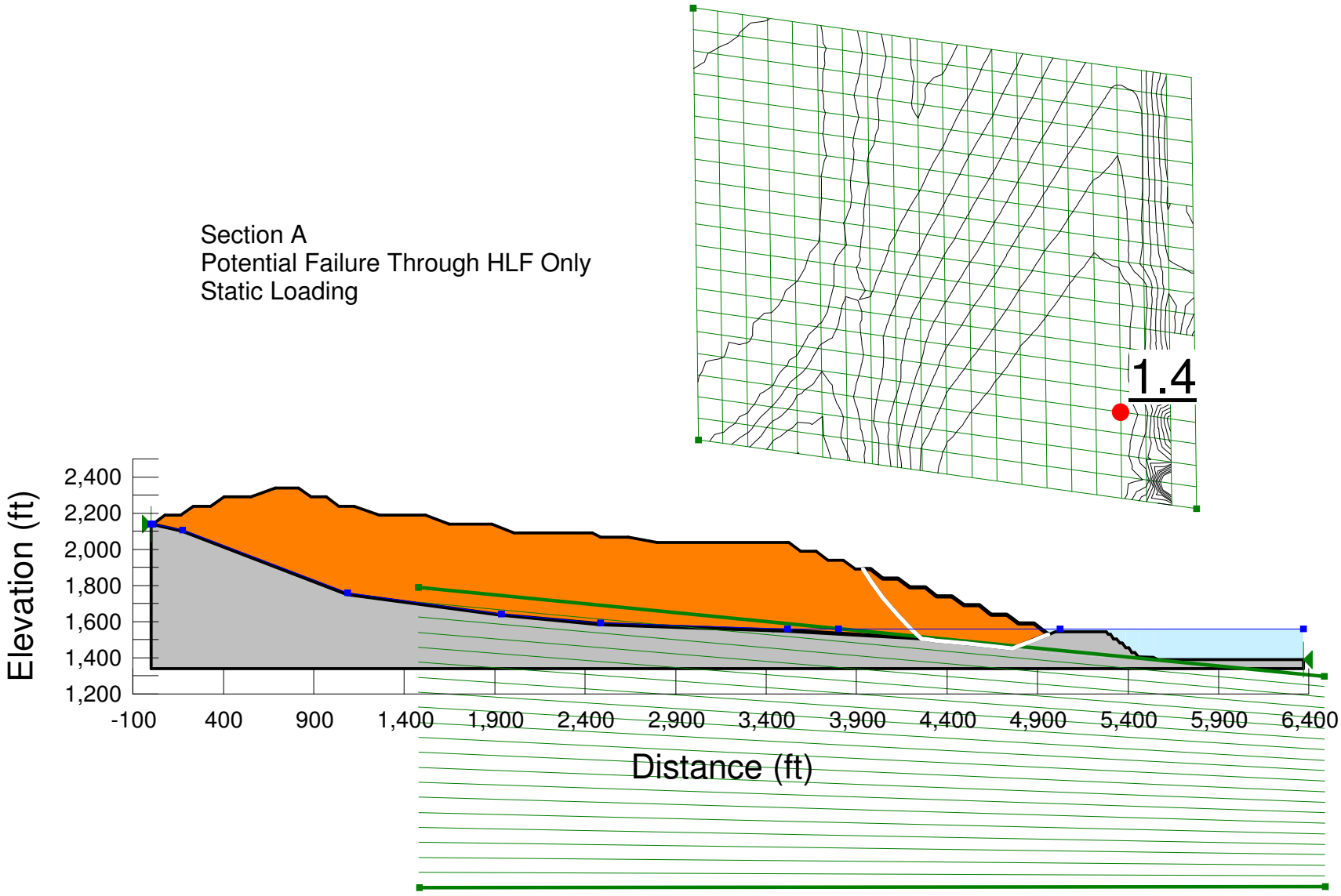
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

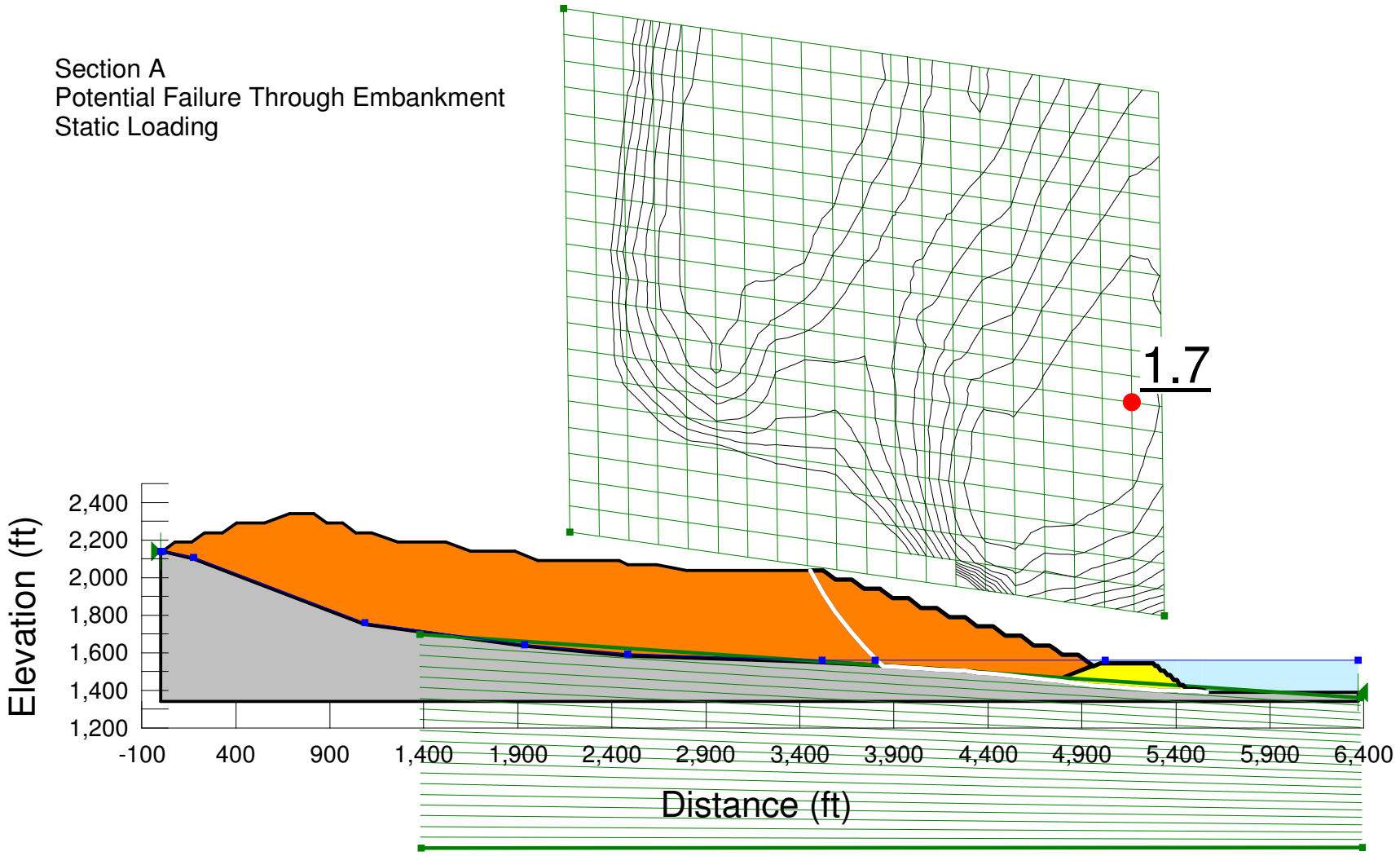
Section C
Potential Failure Through Embankment
Static Loading



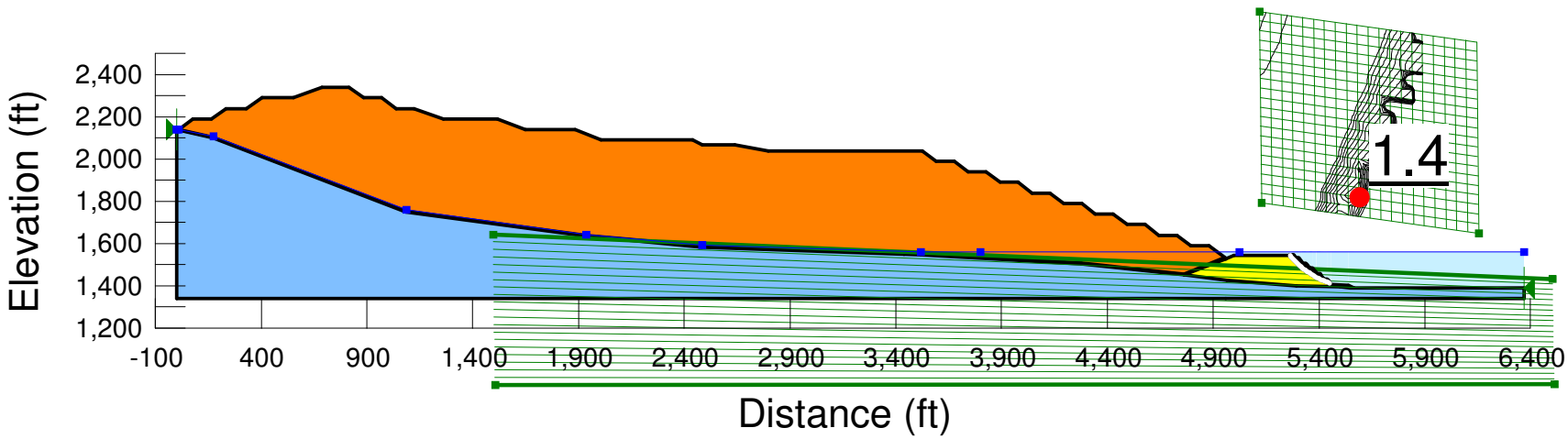
Section A
Potential Failure Through HLF Only
Static Loading

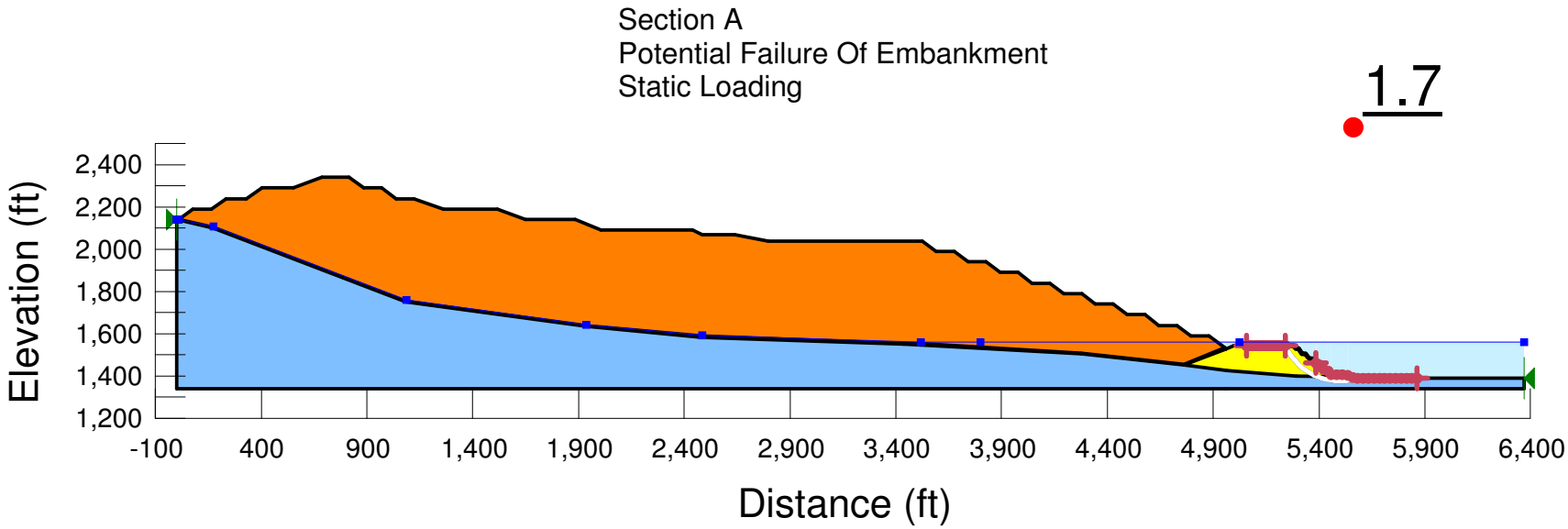


Section A
Potential Failure Through Embankment
Static Loading



Section A
Potential Failure Of Embankment
Static Loading
Surficial Potential Failure




Section A
Potential Failure Of Embankment
Static Loading
1.7
Elevation (ft)
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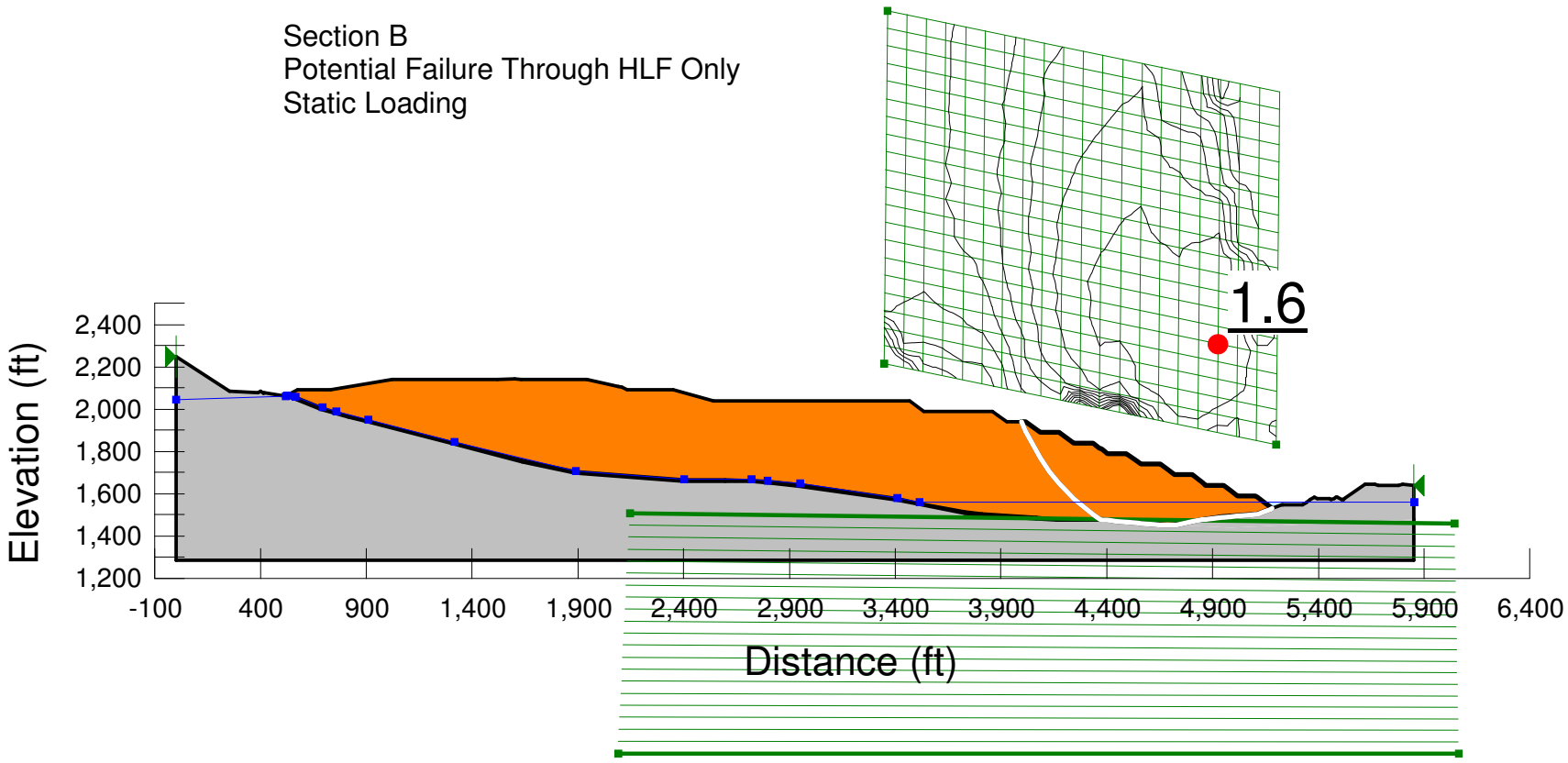
6,400
Distance (ft)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Potential Failure Through HLF Only
Static Loading
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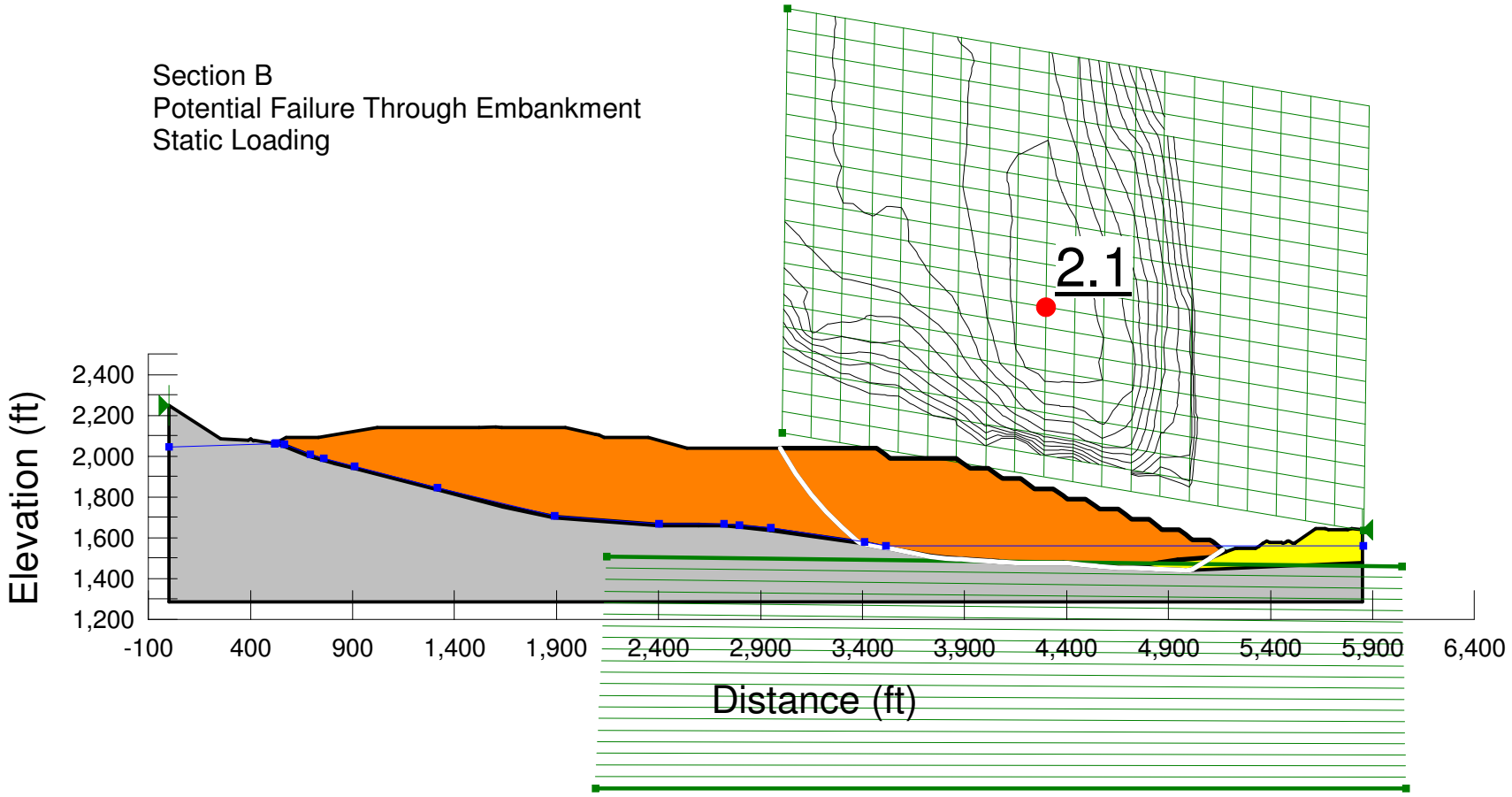
5,400
5,900
6,400
Distance (ft)


Section B
Potential Failure Through Embankment
Static Loading
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Elevation (ft)
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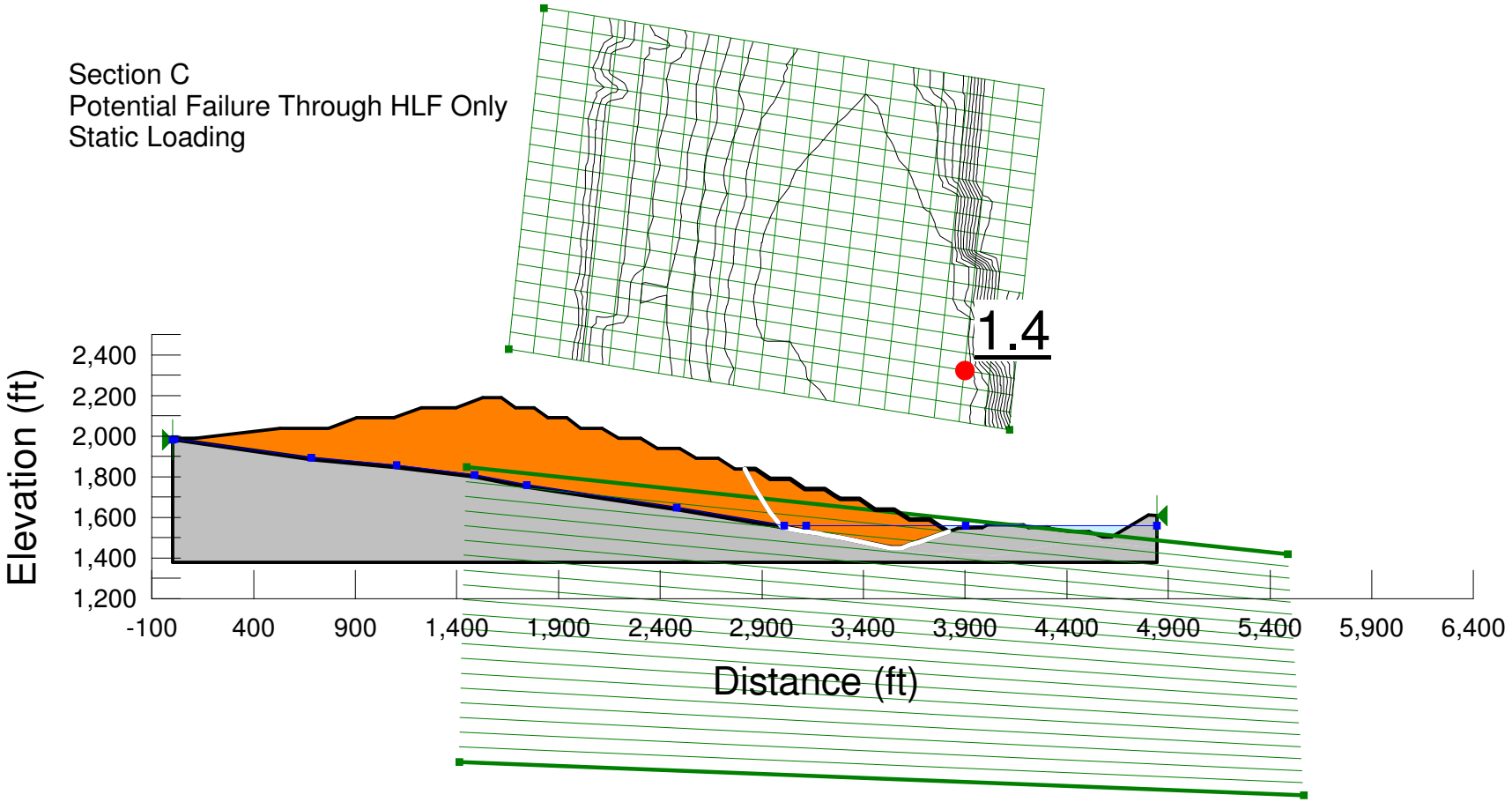
6,400
Distance (ft)


Section C
Potential Failure Through HLF Only
Static Loading
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Elevation (ft)
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1,900
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4,400
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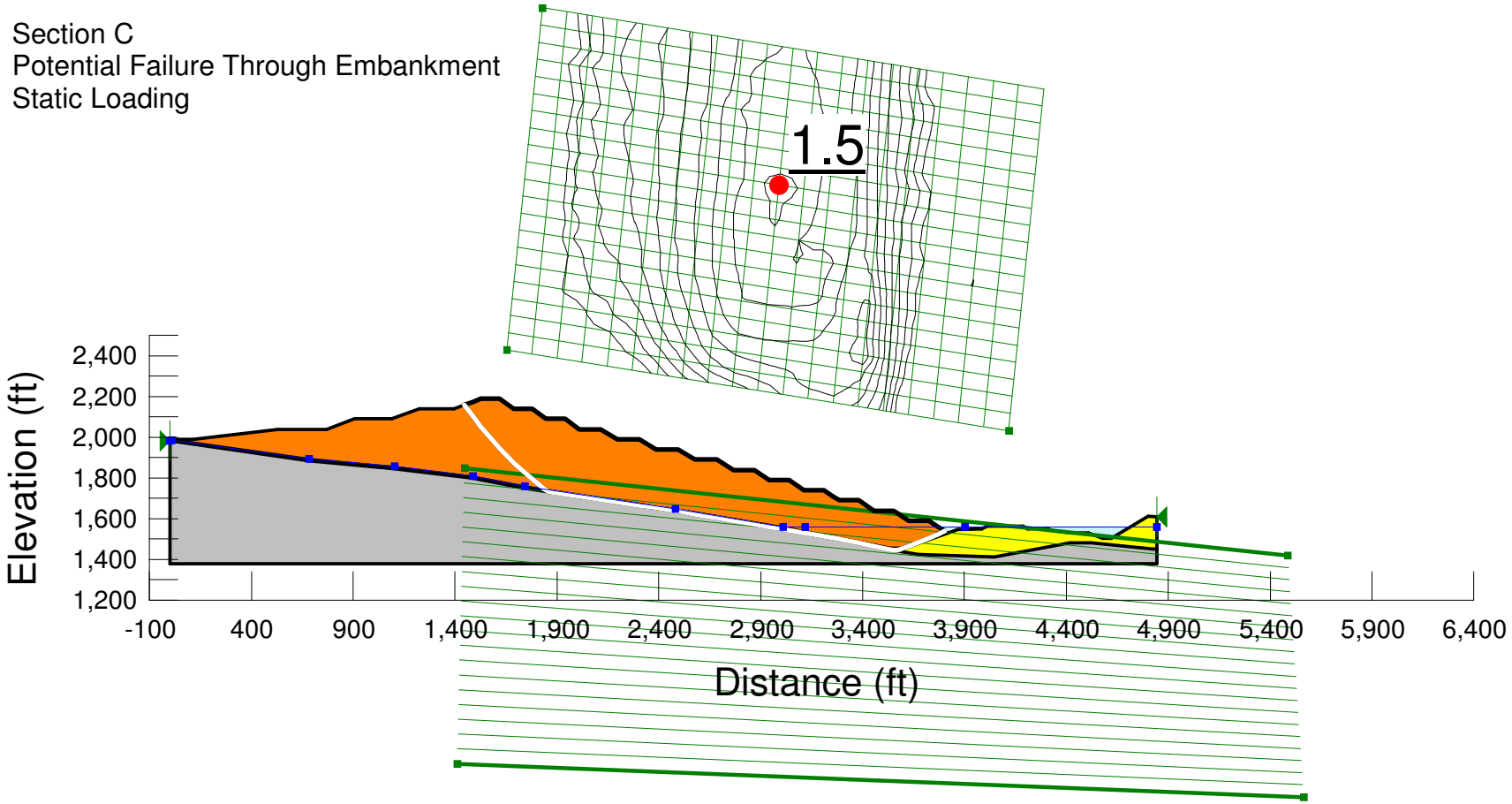
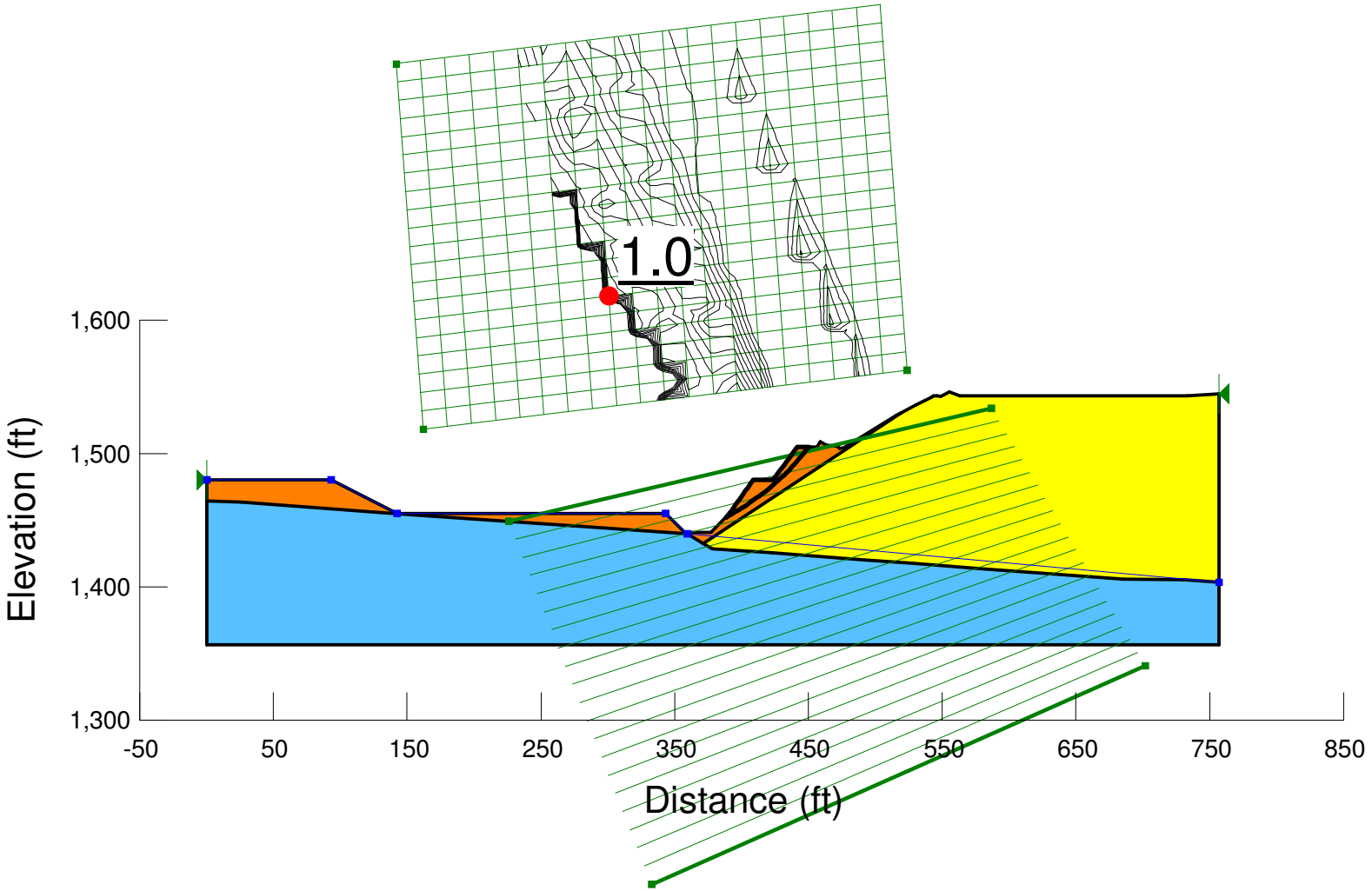
5,400
5,900
6,400
Distance (ft)

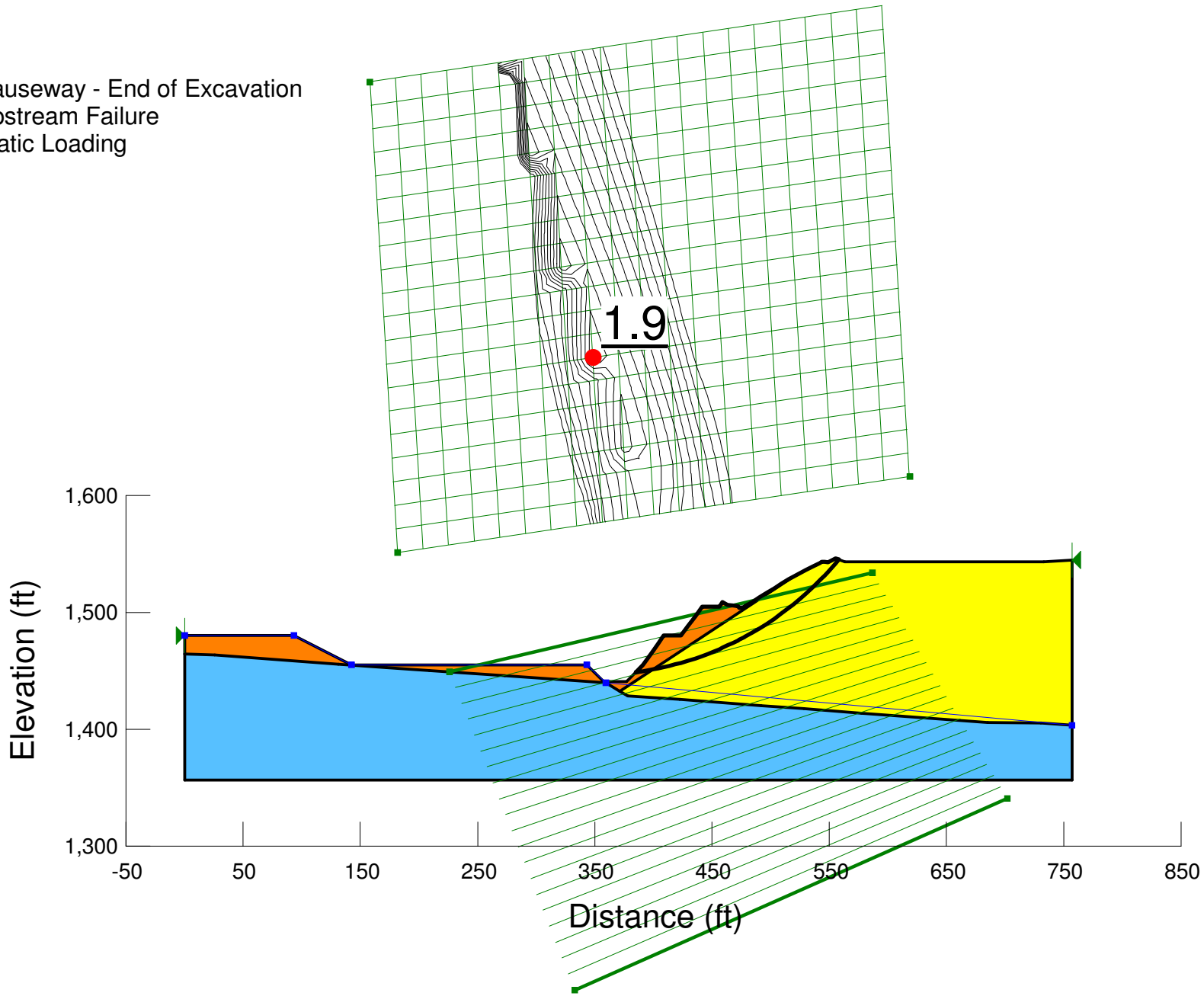
Section C
Potential Failure Through Embankment
Static Loading



Causeway - End of Excavation
Upstream Failure
Static Loading



Causeway - End of Excavation
Upstream Failure
Static Loading



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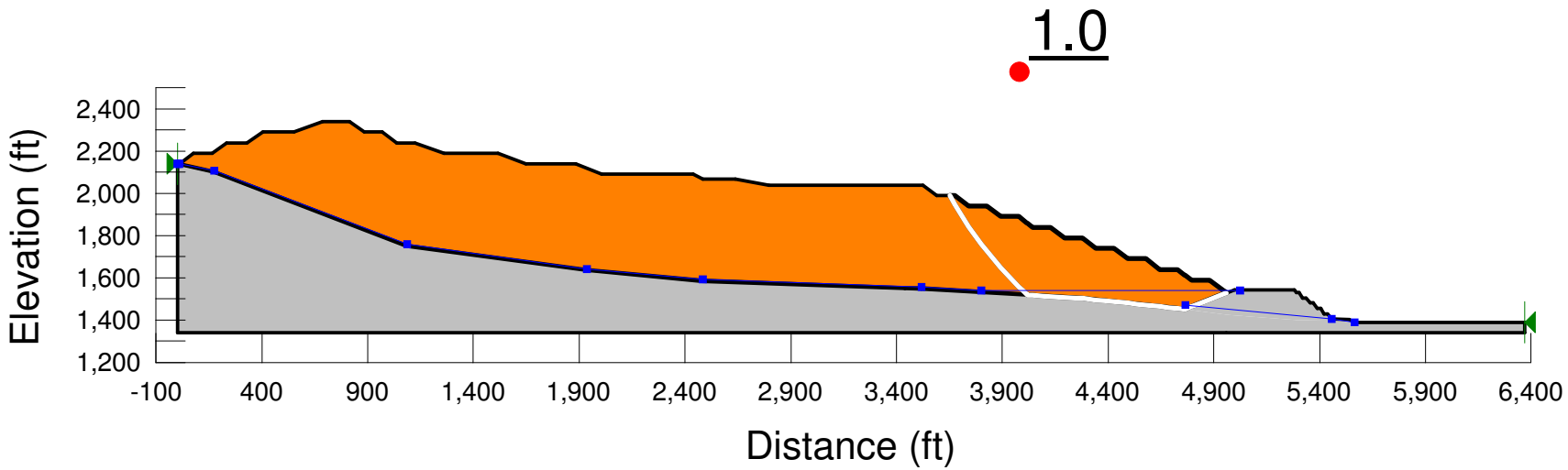
# Appendix C-3

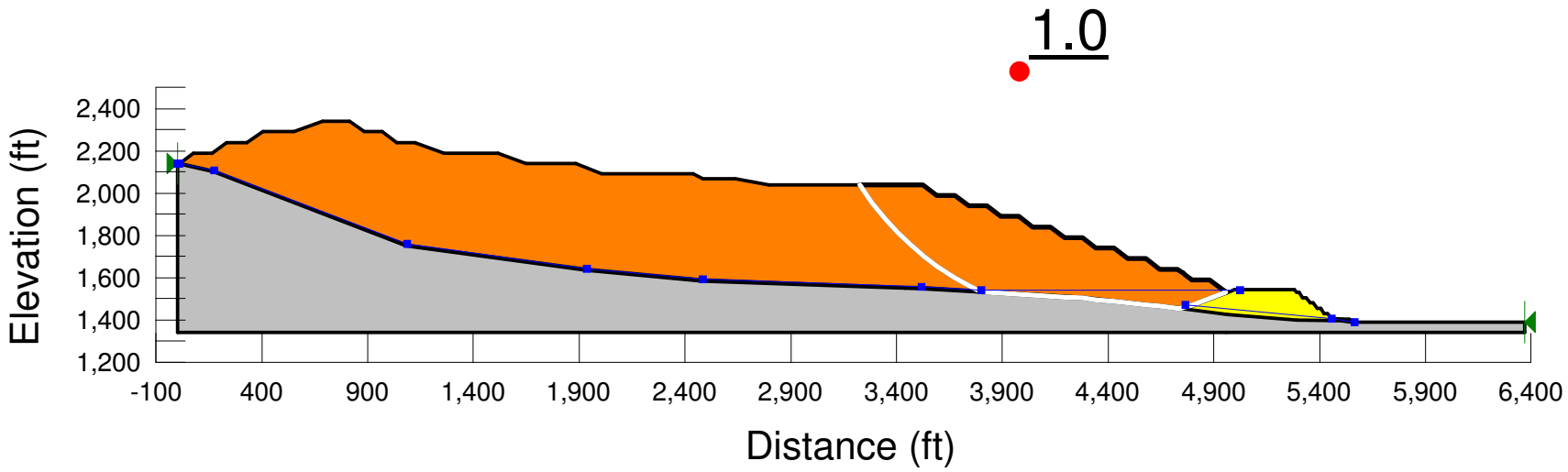
# Stability Analyses Results and Calculations

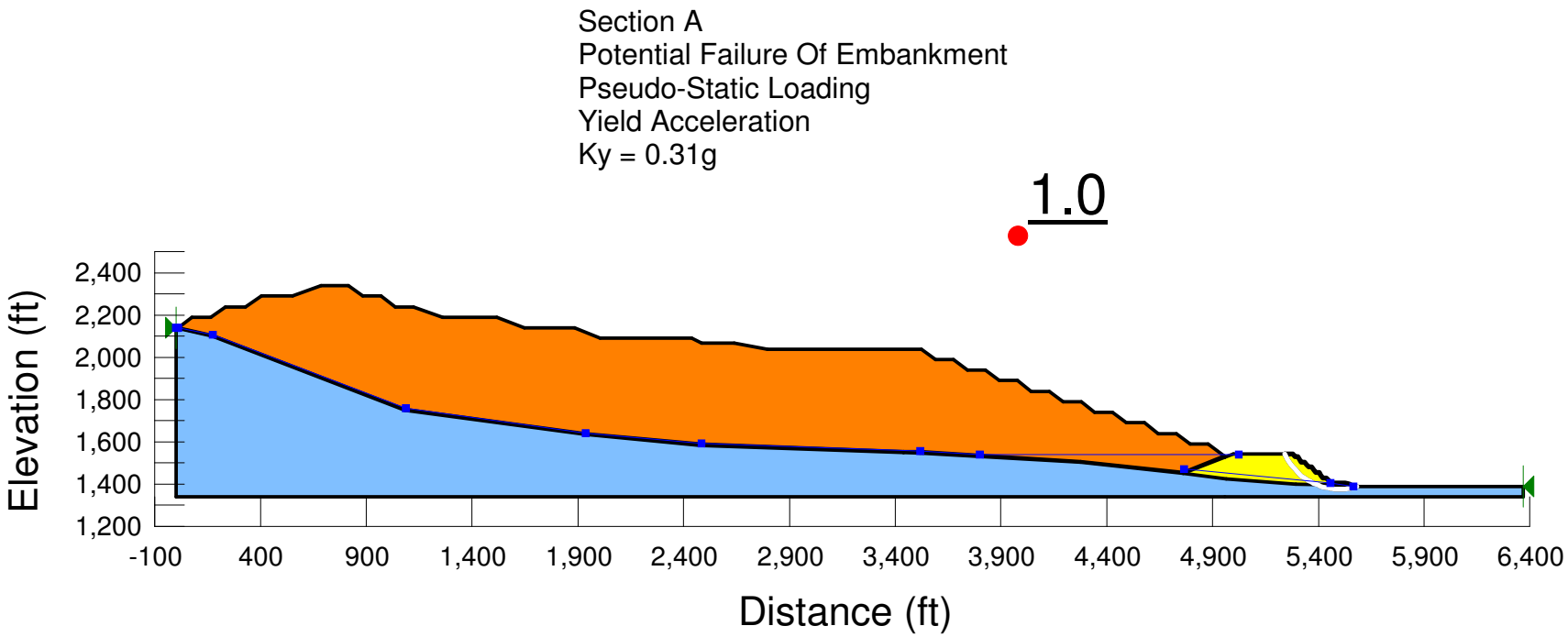
# Estimation of Critical Yield Acceleration (Ky)
# Figures

Section A
Failure Through HLF Only
Pseudo-Static Loading
Yield Acceleration
Ky = 0.12g

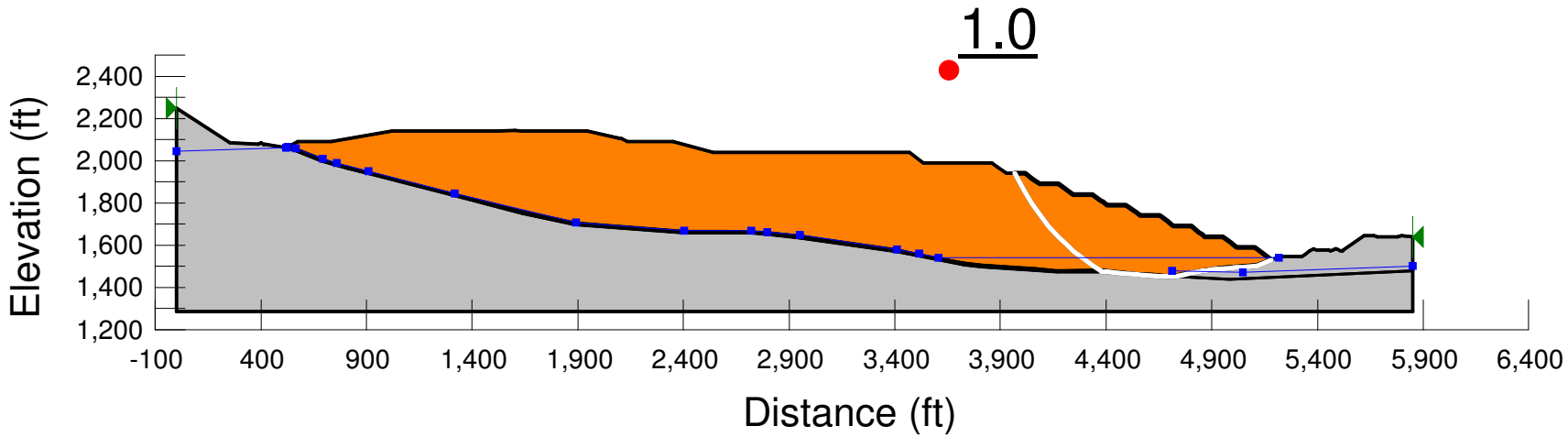


Section A
Failure Through Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.21g

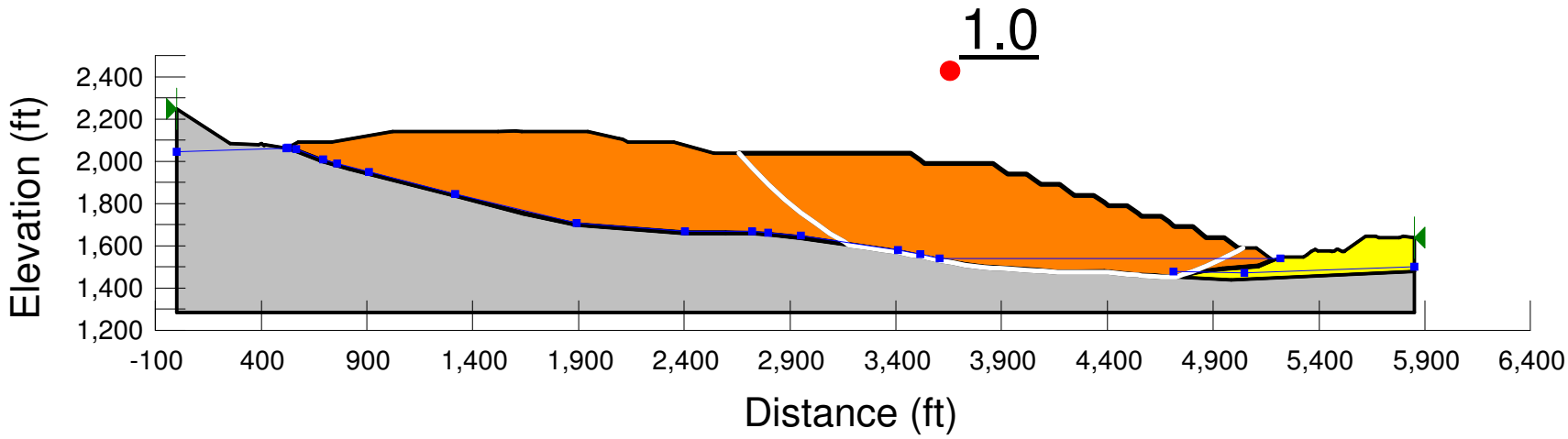



Section A
Potential Failure Of Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.31g
1.0
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

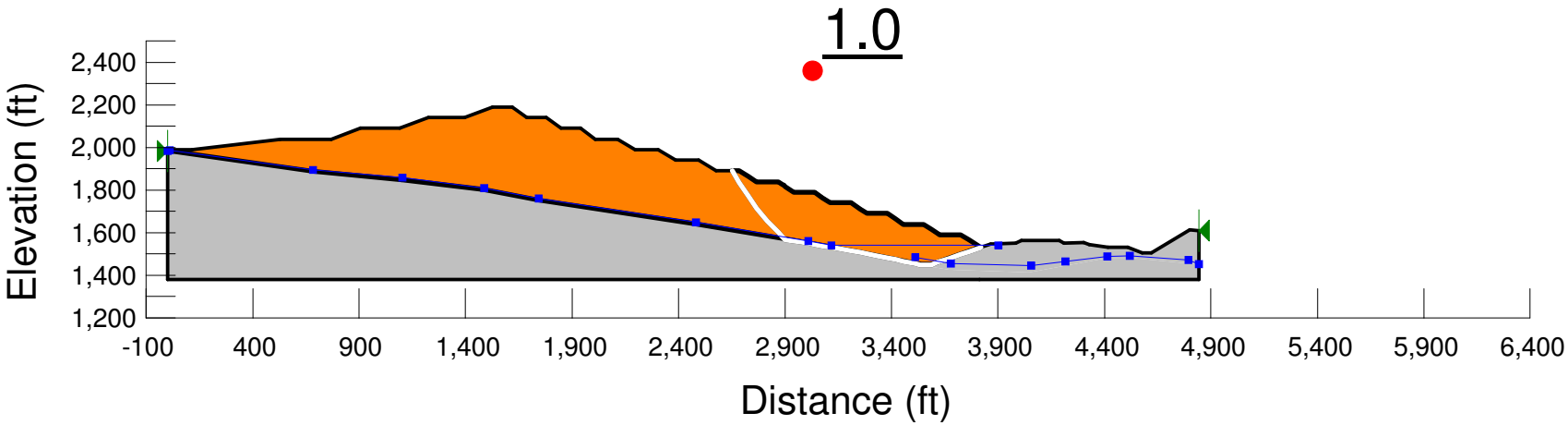
Section B
Failure Through HLF Only
Pseudo-Static Loading
Yield Acceleration
Ky = 0.16g



Section B
Failure Through Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.28g

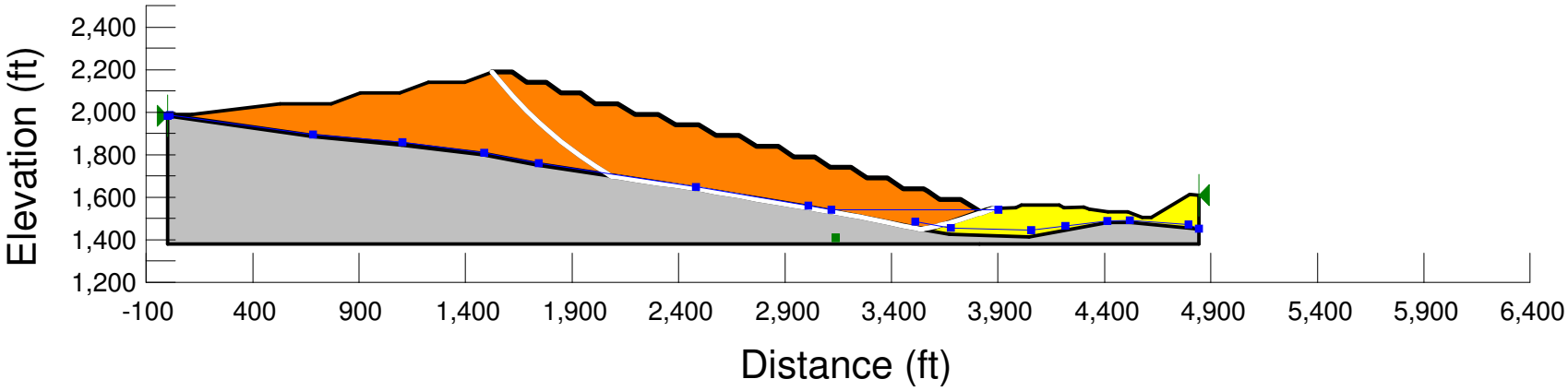


Section C
Failure Through HLF Only
Pseudo-Static Loading
Yield Acceleration
Ky = 0.10g

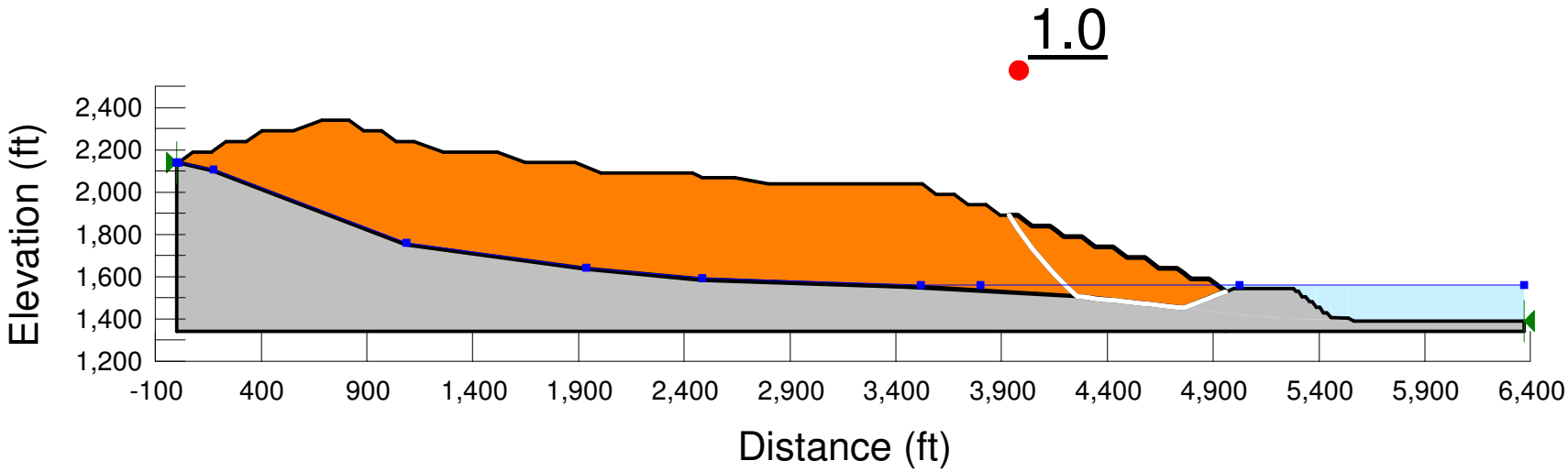


1.0

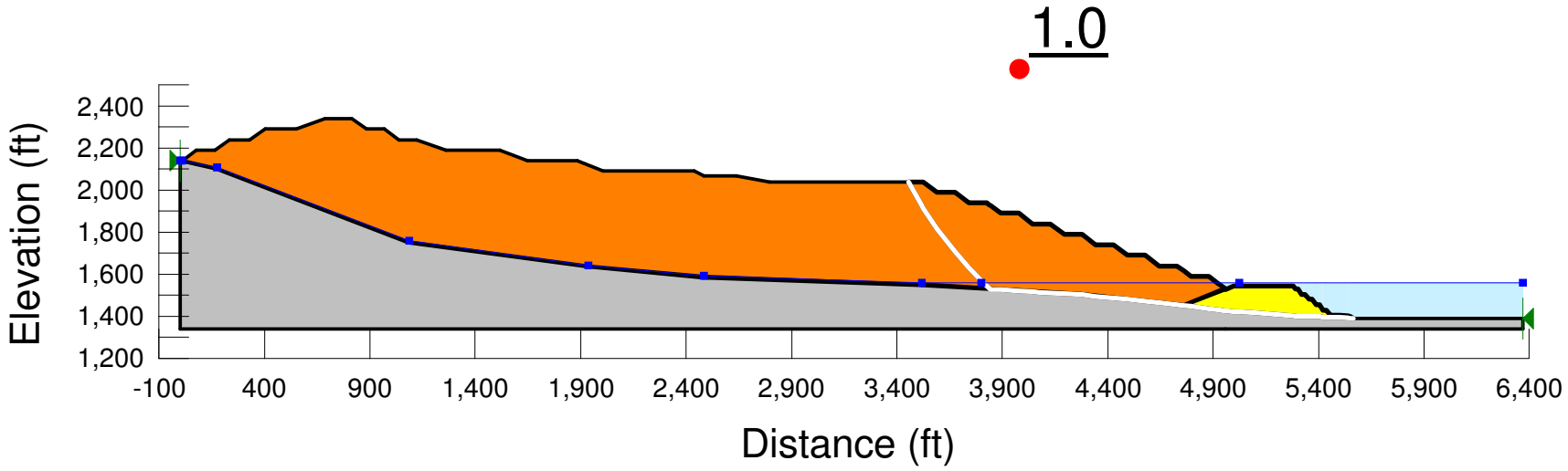
Section C
Failure Through Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.22g

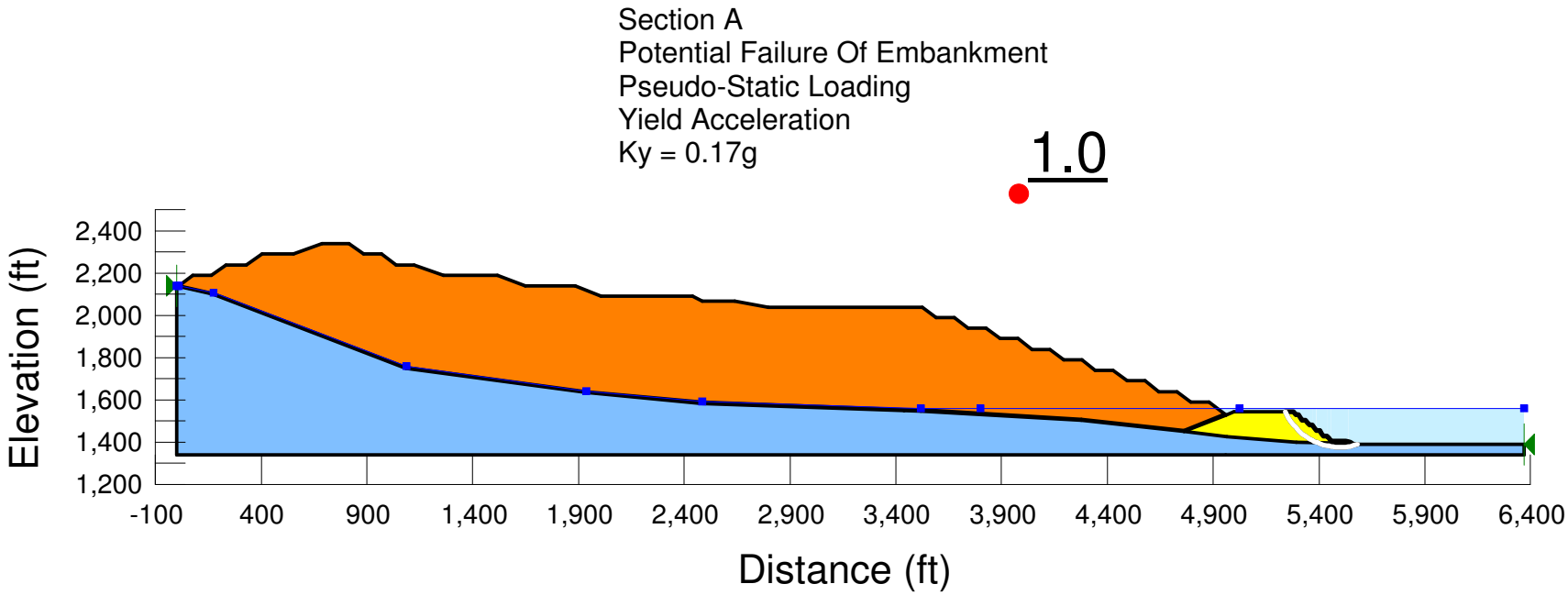


Section A
Failure Through HLF Only
Pseudo-Static Loading
Yield Acceleration
Ky = 0.10g

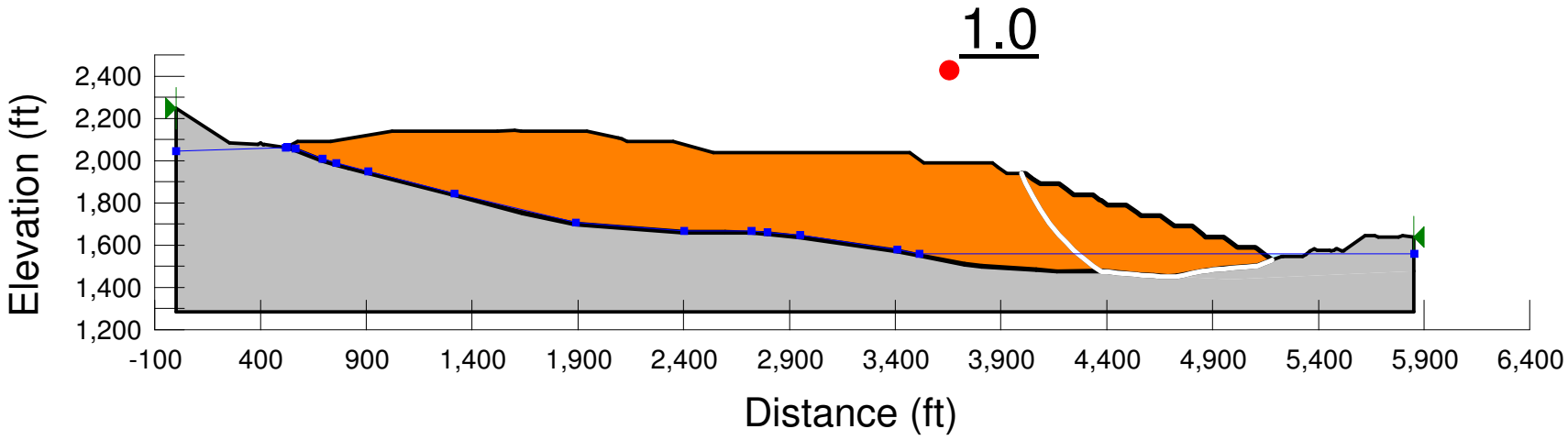


Section A
Failure Through Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.16g

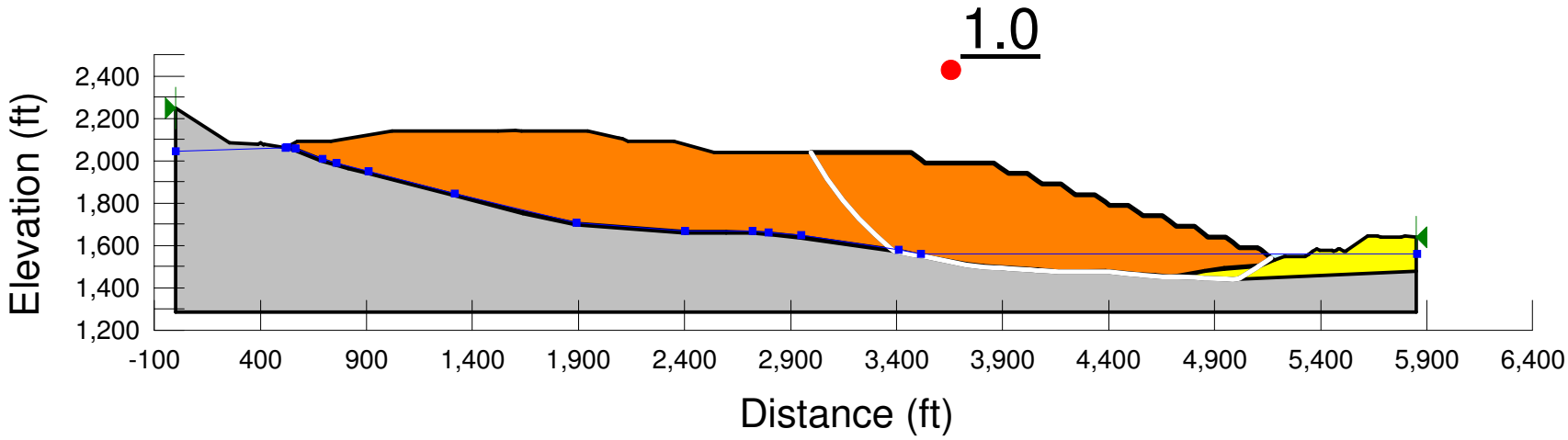



Section A
Potential Failure Of Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.17g
1.0
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

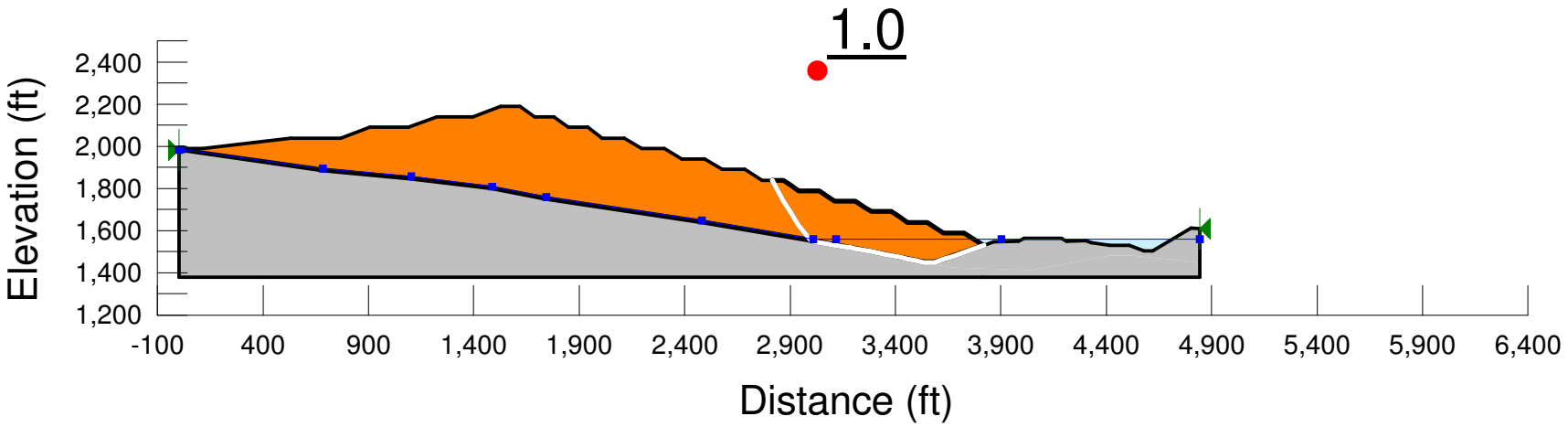
Section B
Failure Through HLF Only
Pseudo-Static Loading
Yield Acceleration
Ky = 0.13g

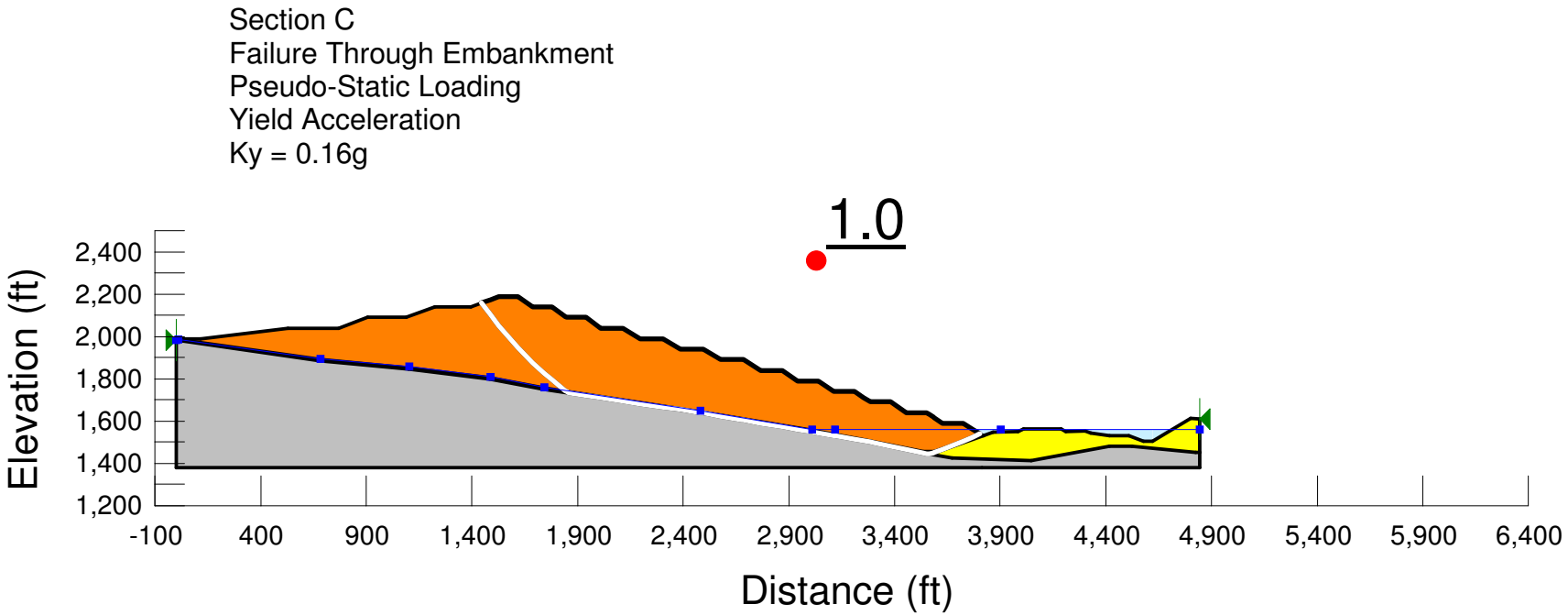


Section B
Failure Through Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.28g



Section C
Failure Through HLF Only
Pseudo-Static Loading
Yield Acceleration
Ky = 0.09g




Section C
Failure Through Embankment
Pseudo-Static Loading
Yield Acceleration
Ky = 0.16g
1.0
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

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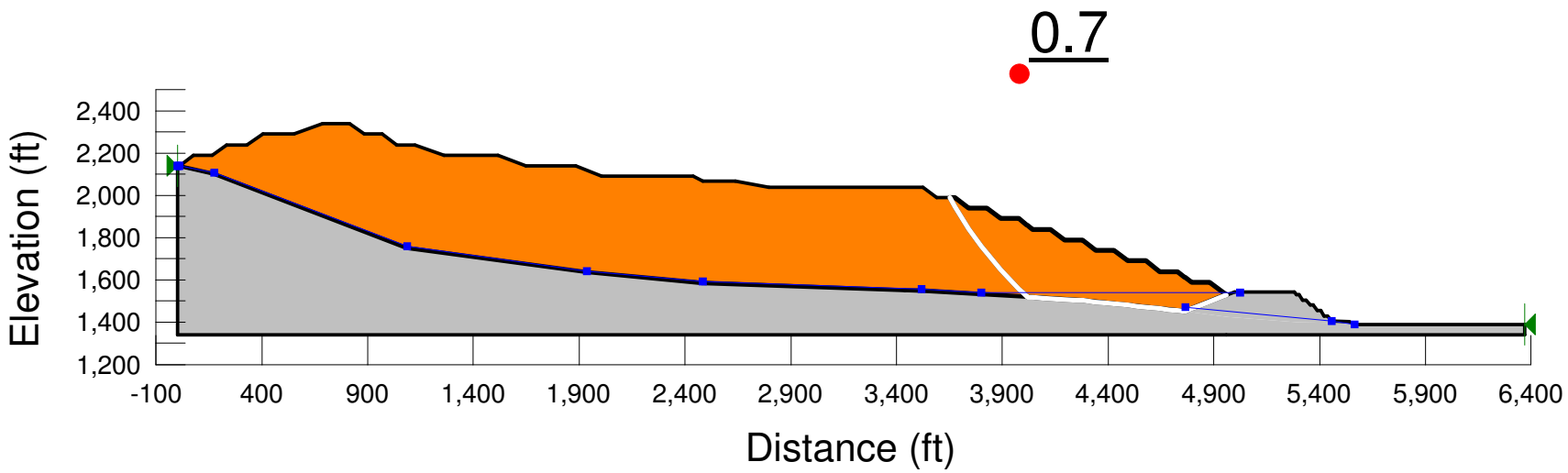
# Appendix C-4

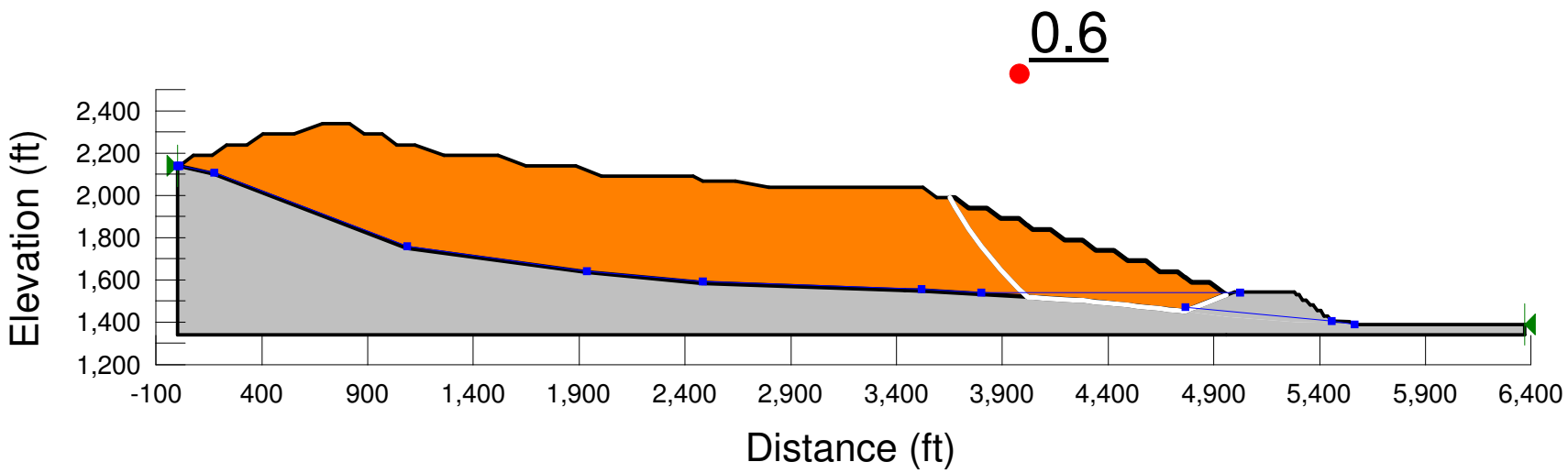
# Stability Analyses Results and Calculations

# Pseudo-Static Slope Stability
# Figures

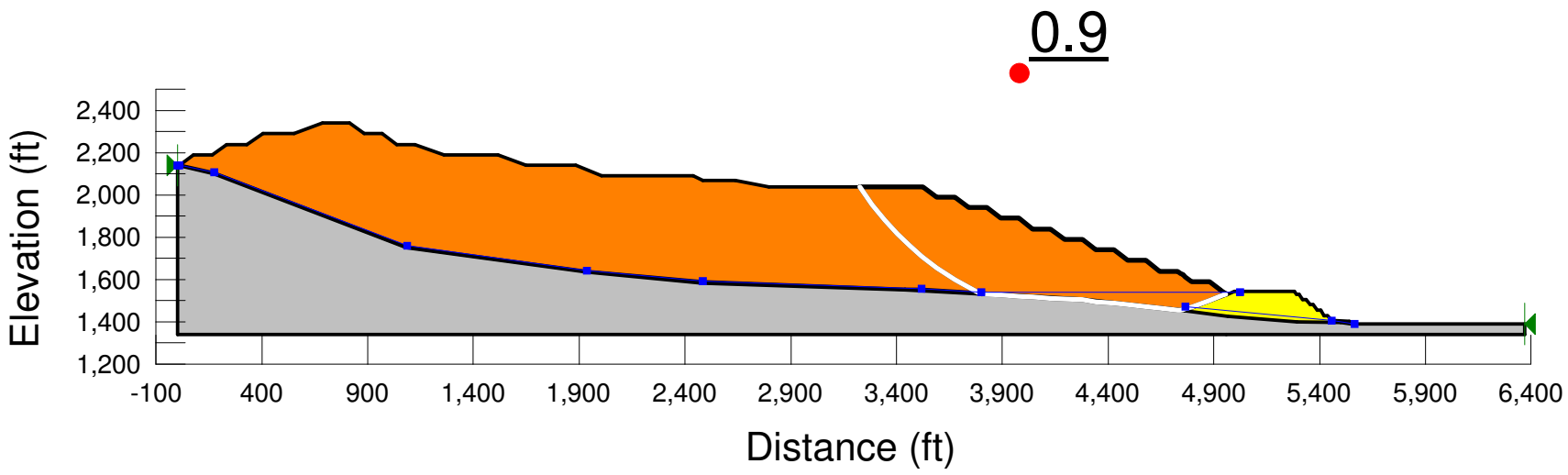
Section A
Failure Through HLF Only
Pseudo-Static Loading
OBE, 0.23g



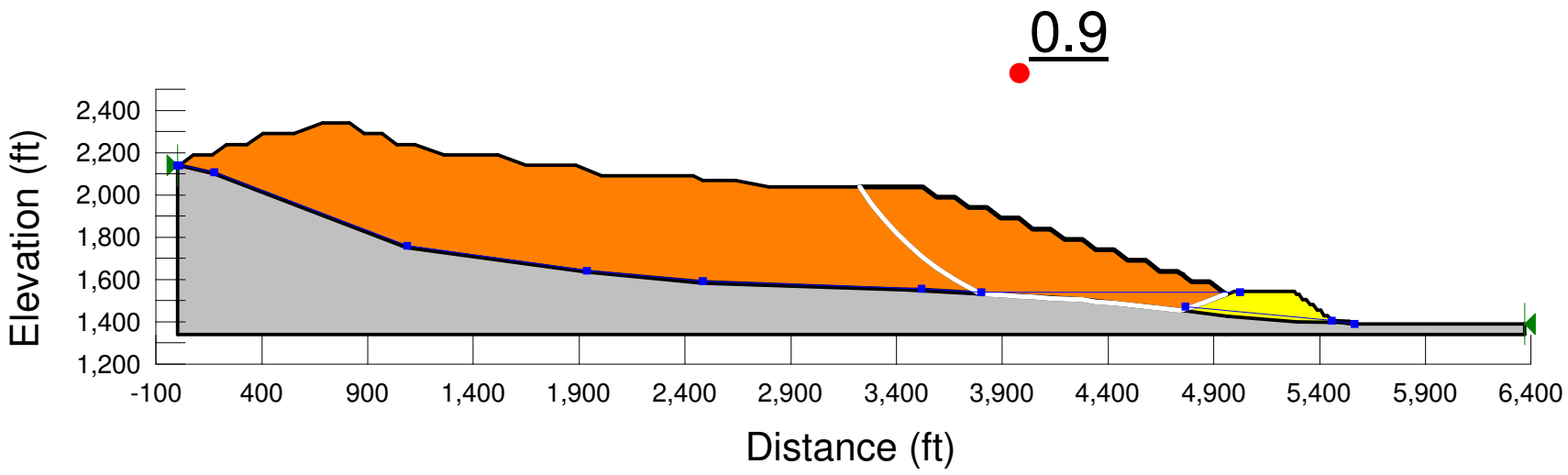
Section A
Failure Through HLF Only
Pseudo-Static Loading
MDE, 0.26g

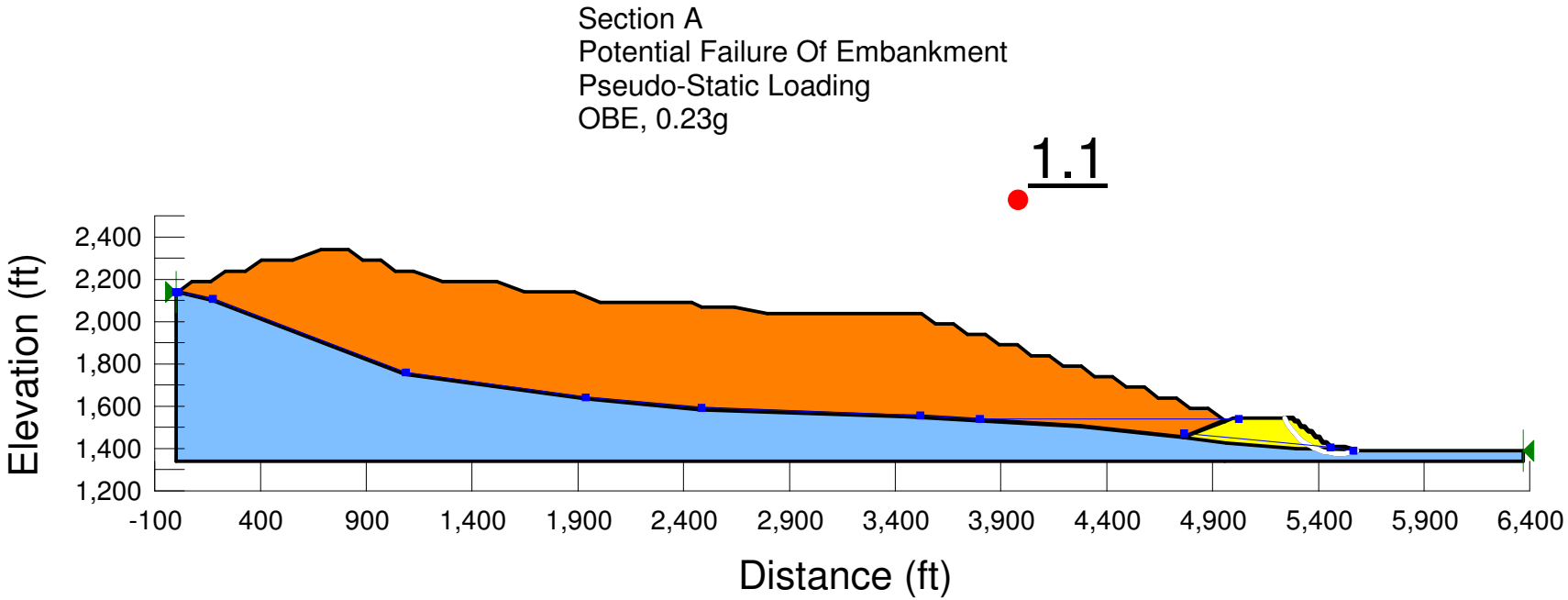


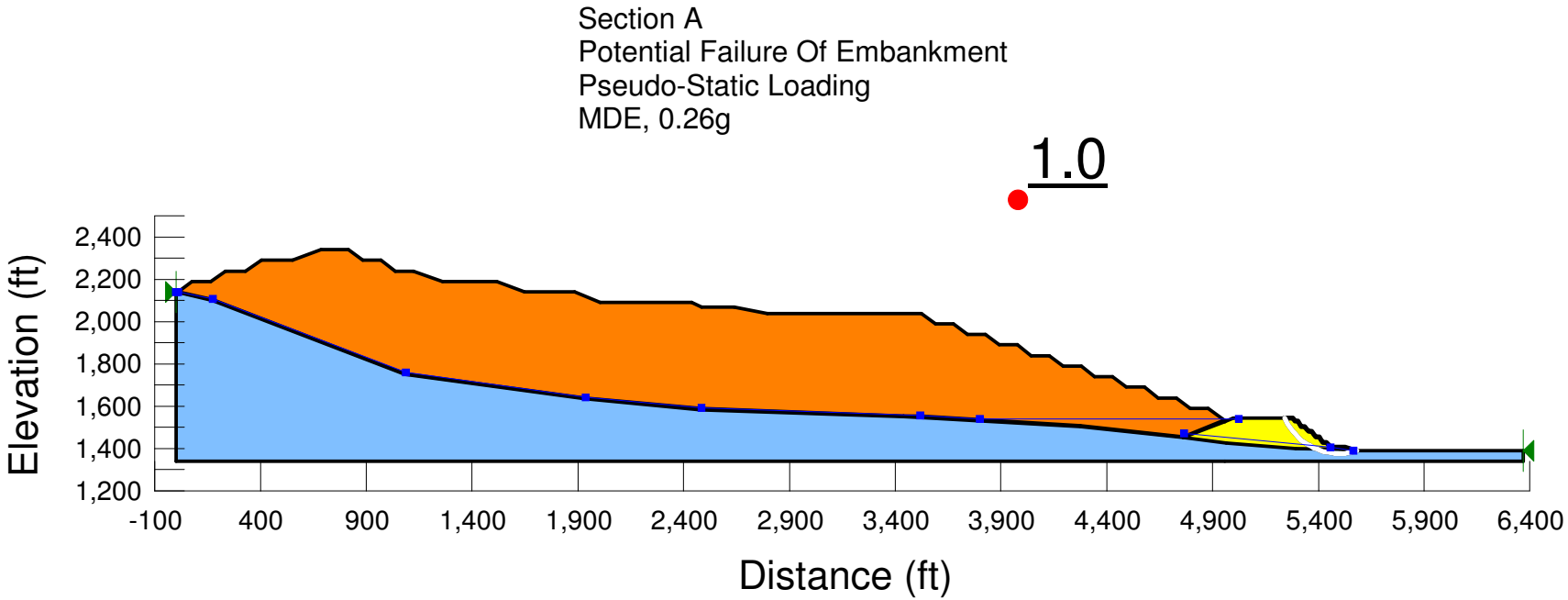
Section A
Failure Through Embankment
Pseudo-Static Loading
OBE, 0.23g



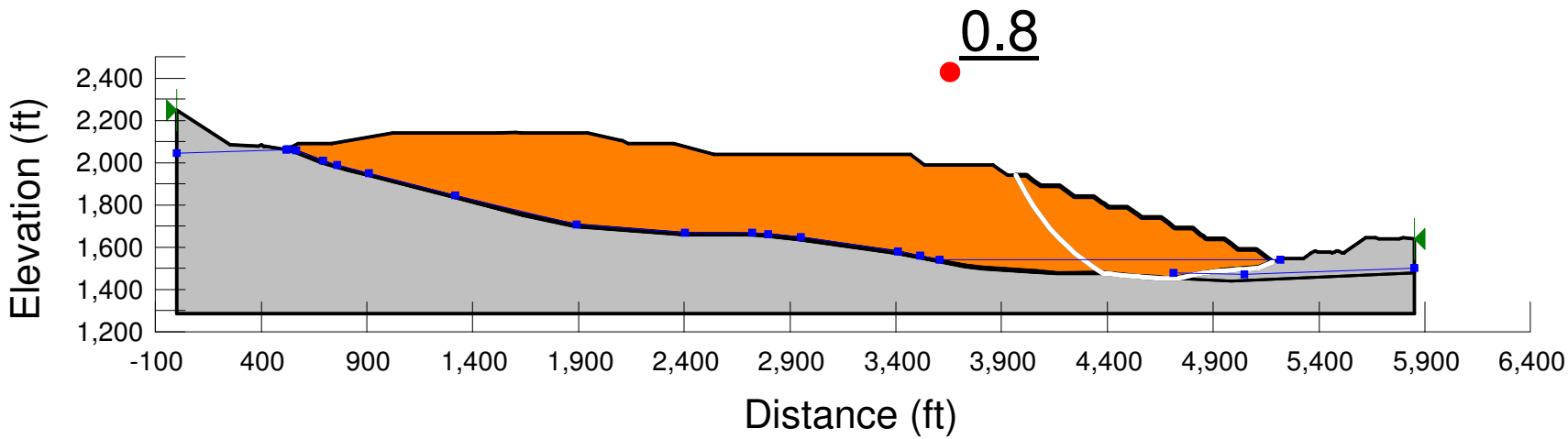
Section A
Failure Through Embankment
Pseudo-Static Loading
MDE, 0.26g



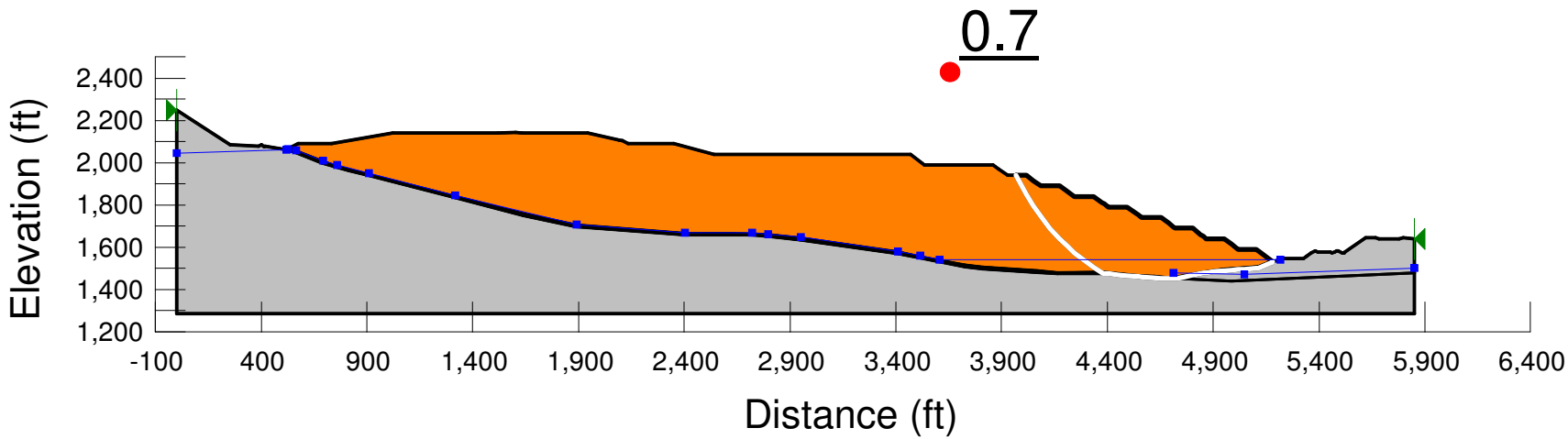

Section A
Potential Failure Of Embankment
Pseudo-Static Loading
OBE, 0.23g
1.1
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)


Section A
Potential Failure Of Embankment
Pseudo-Static Loading
MDE, 0.26g
1.0
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

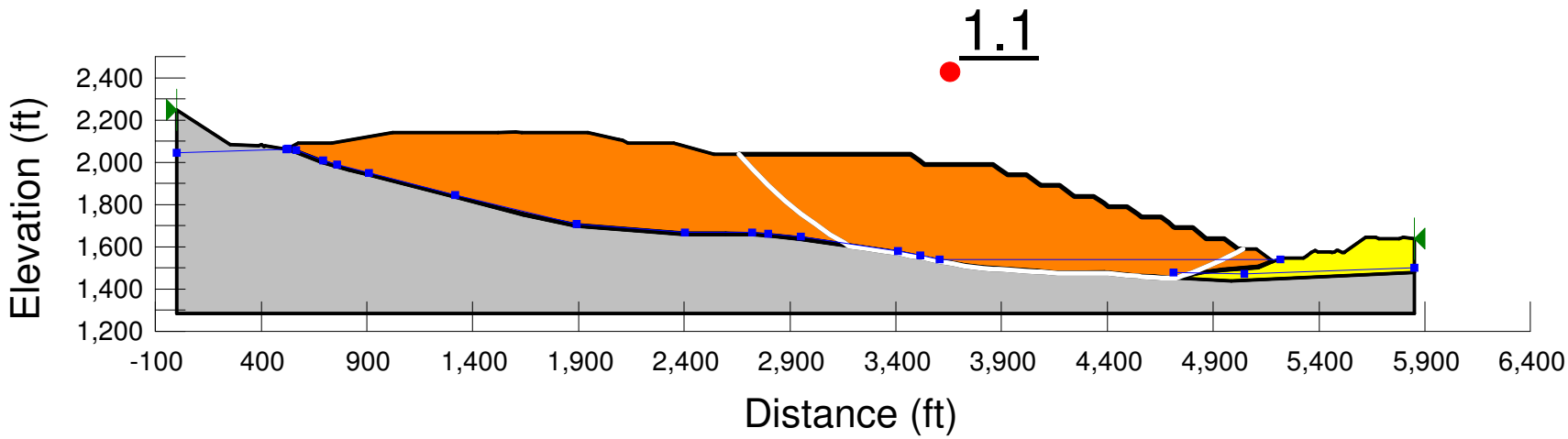
Section B
Failure Through HLF Only
Pseudo-Static Loading
OBE, 0.23g



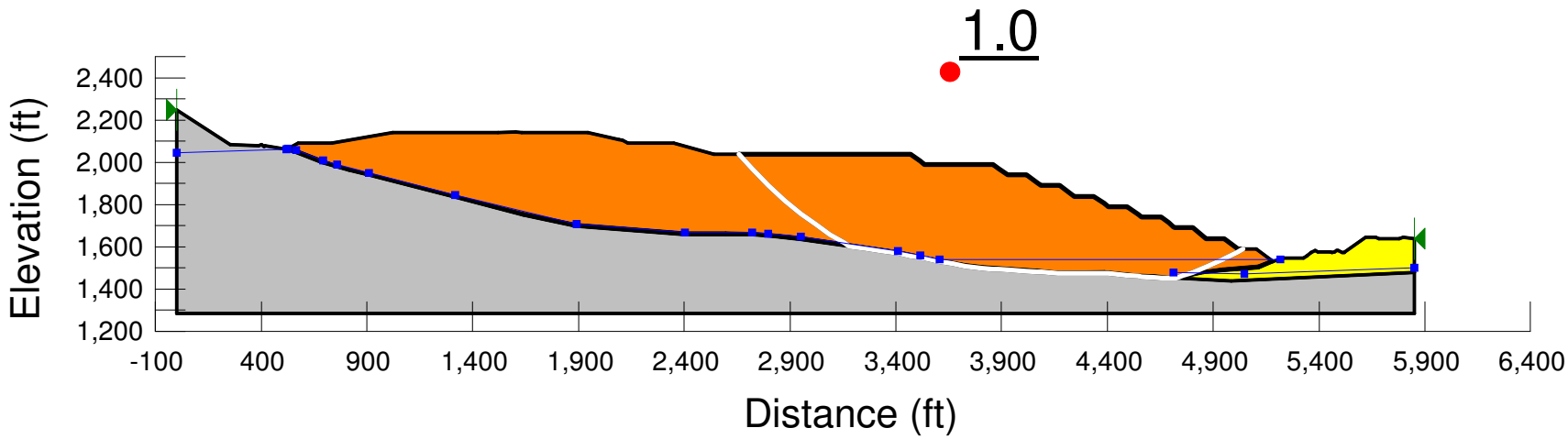
Section B
Failure Through HLF Only
Pseudo-Static Loading
MDE, 0.26g



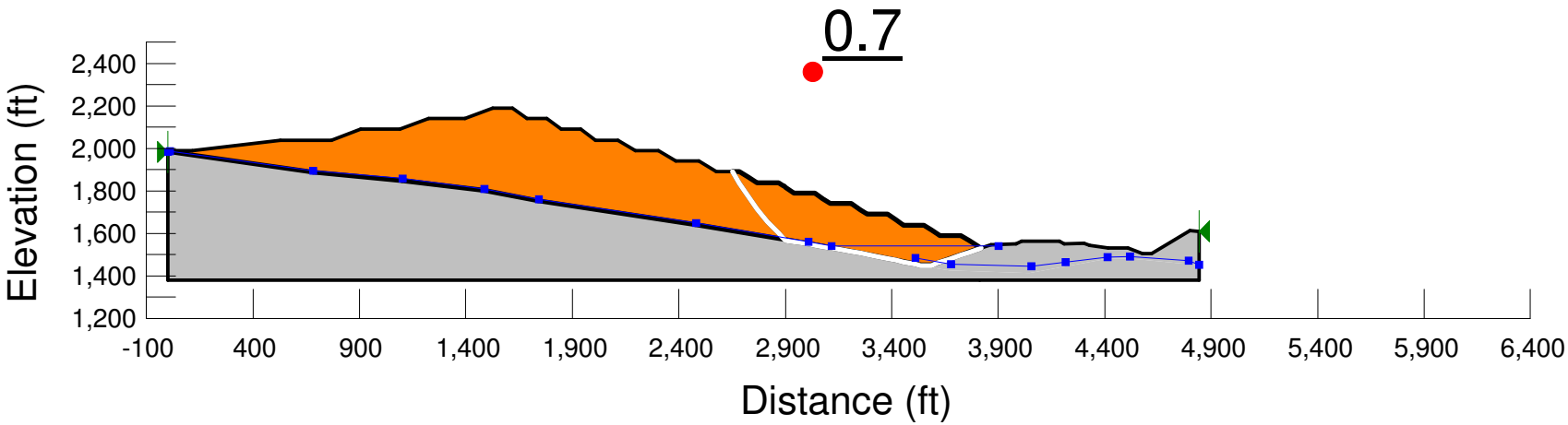
Section B
Failure Through Embankment
Pseudo-Static Loading
OBE, 0.23g



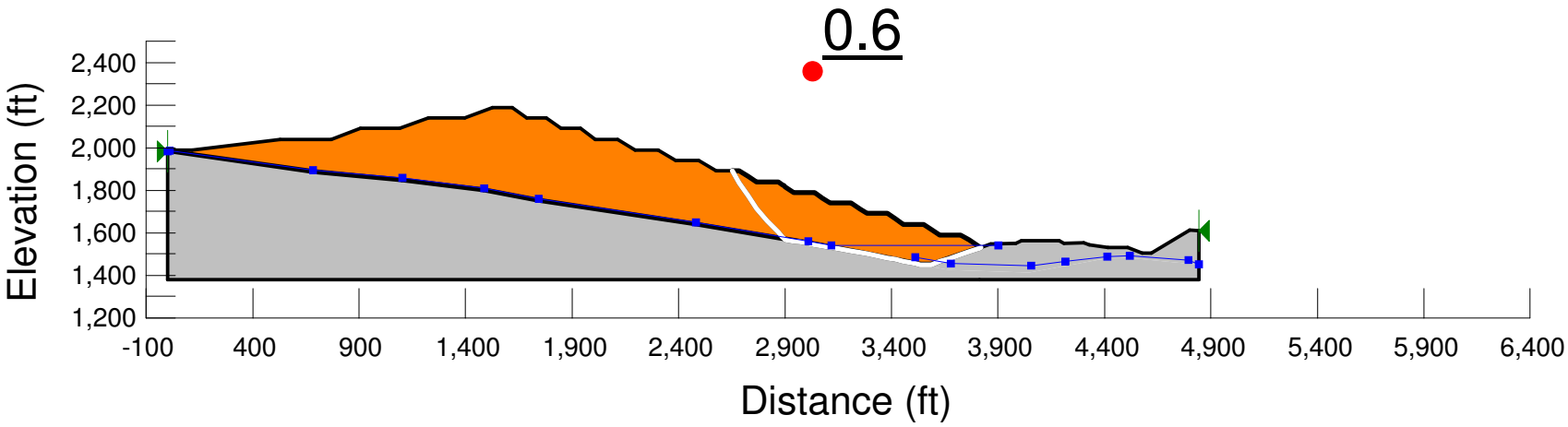
Section B
Failure Through Embankment
Pseudo-Static Loading
MDE, 0.26g

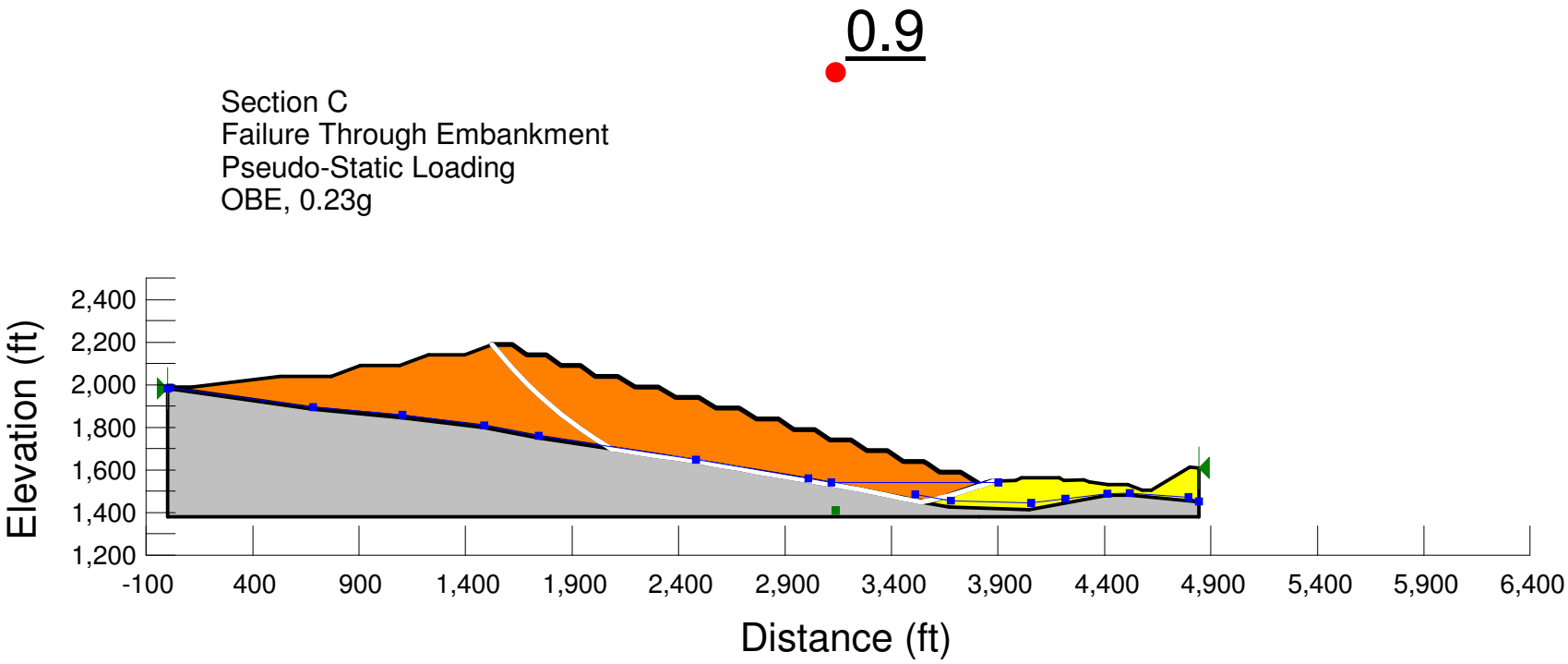


Section C
Failure Through HLF Only
Pseudo-Static Loading
OBE, 0.23g



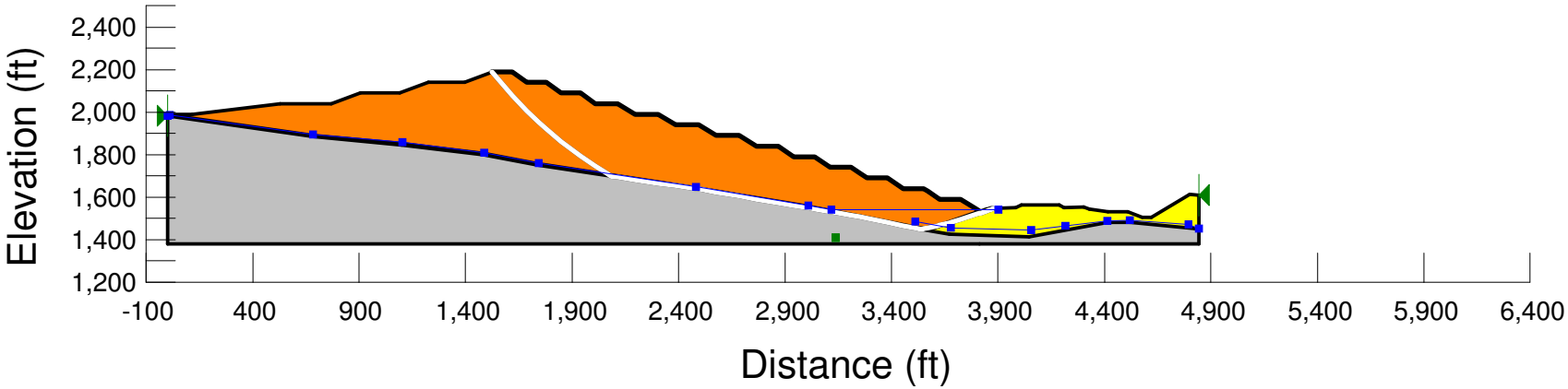
Section C
Failure Through HLF Only
Pseudo-Static Loading
MDE, 0.26g



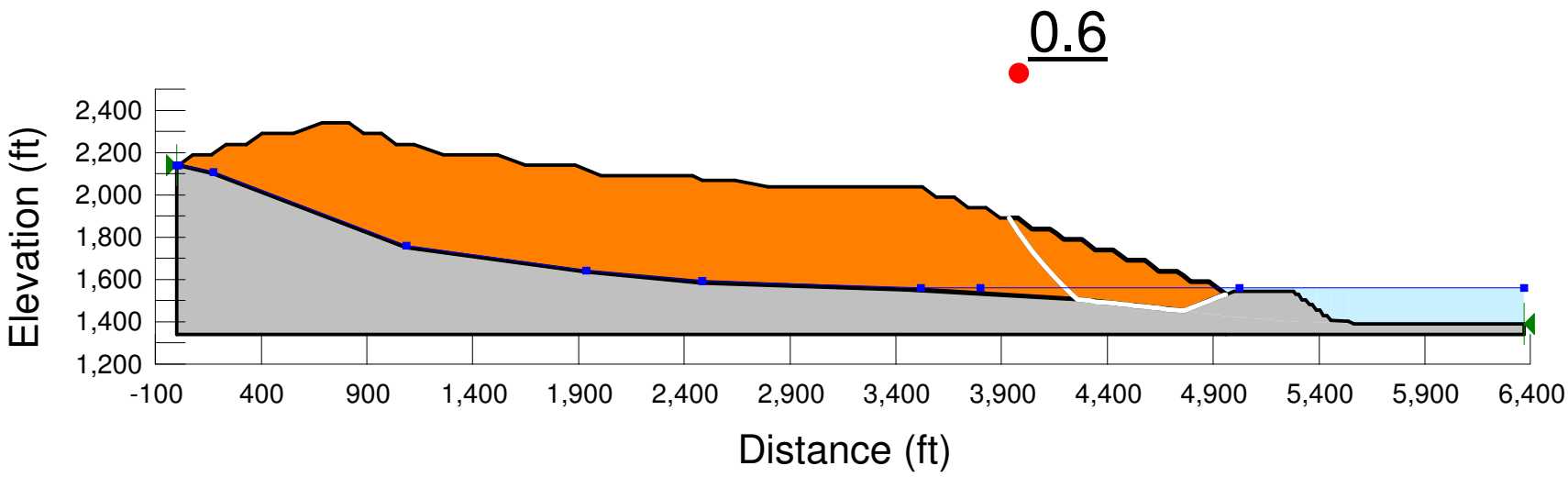

0.9
Section C
Failure Through Embankment
Pseudo-Static Loading
OBE, 0.23g
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

0.9

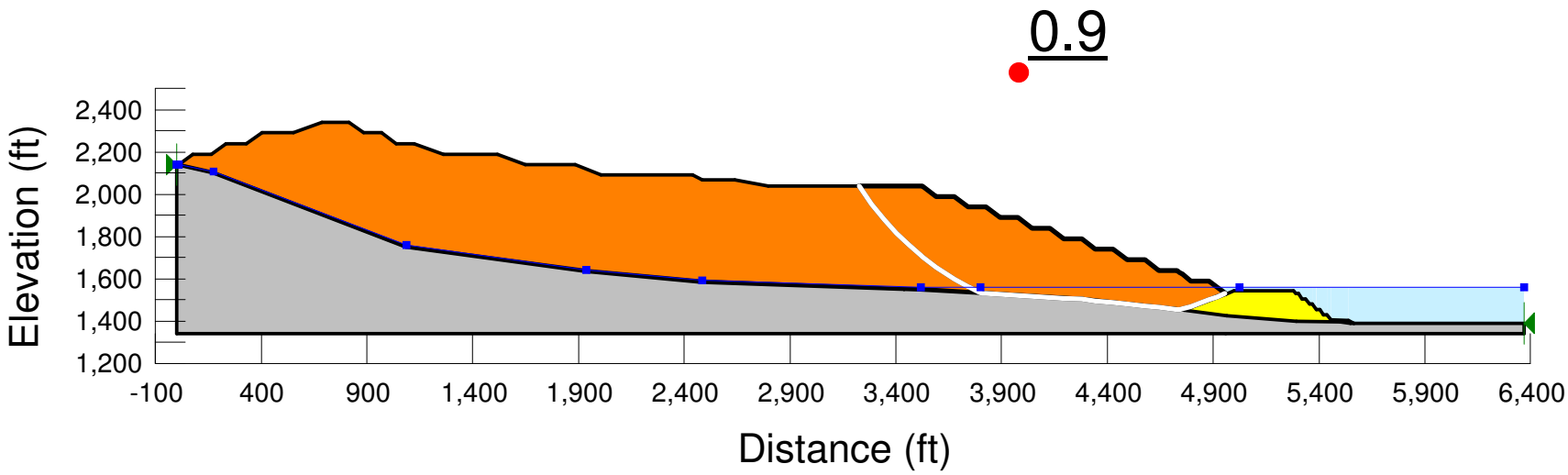
Section C
Failure Through Embankment
Pseudo-Static Loading
MDE, 0.26g

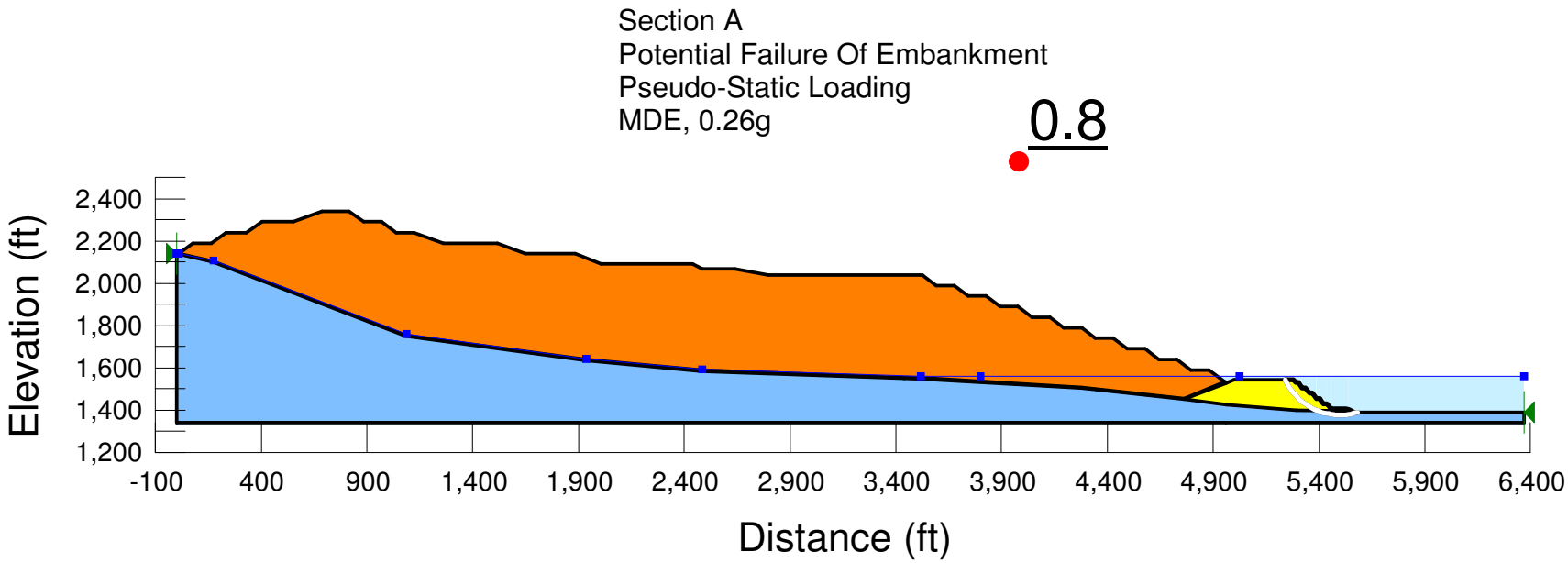


Section A
Failure Through HLF Only
Pseudo-Static Loading
MDE, 0.26g

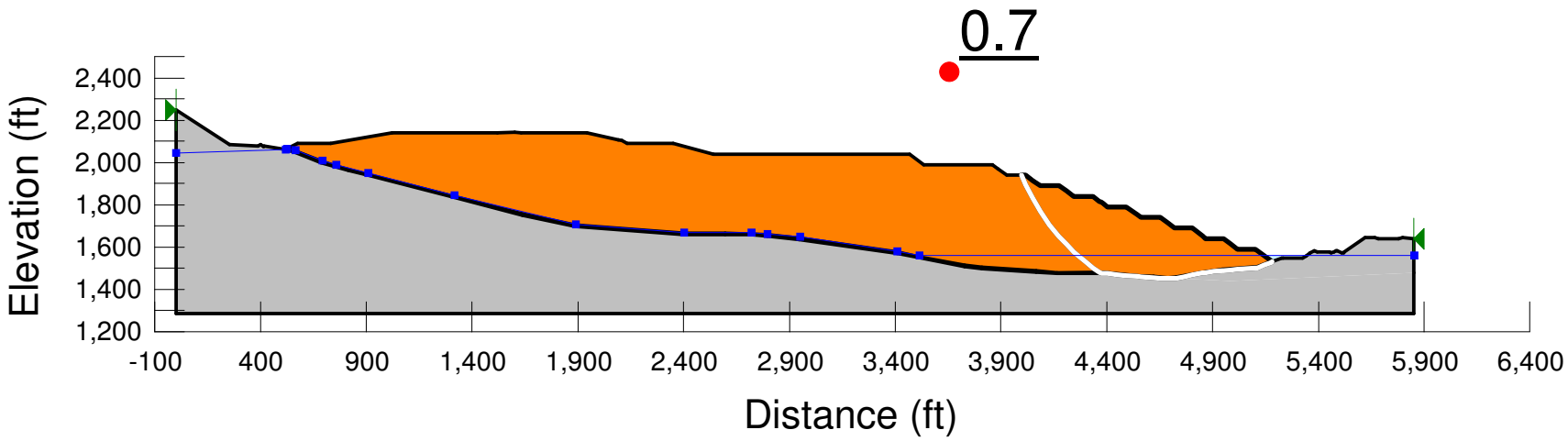


Section A
Failure Through Embankment
Pseudo-Static Loading
MDE, 0.26g

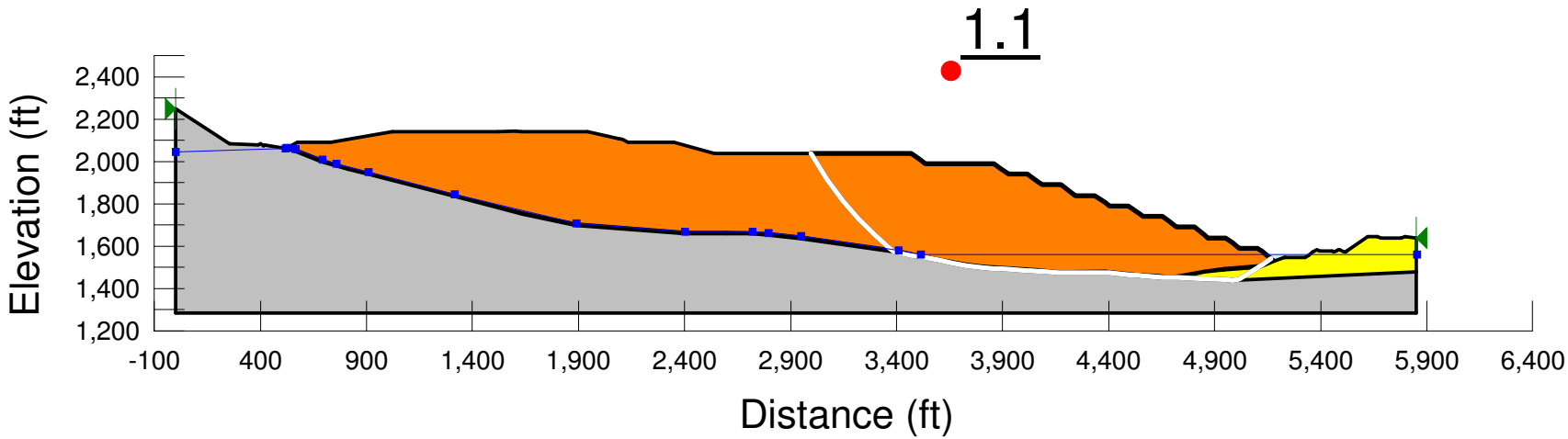



Section A
Potential Failure Of Embankment
Pseudo-Static Loading
MDE, 0.26g
0.8
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

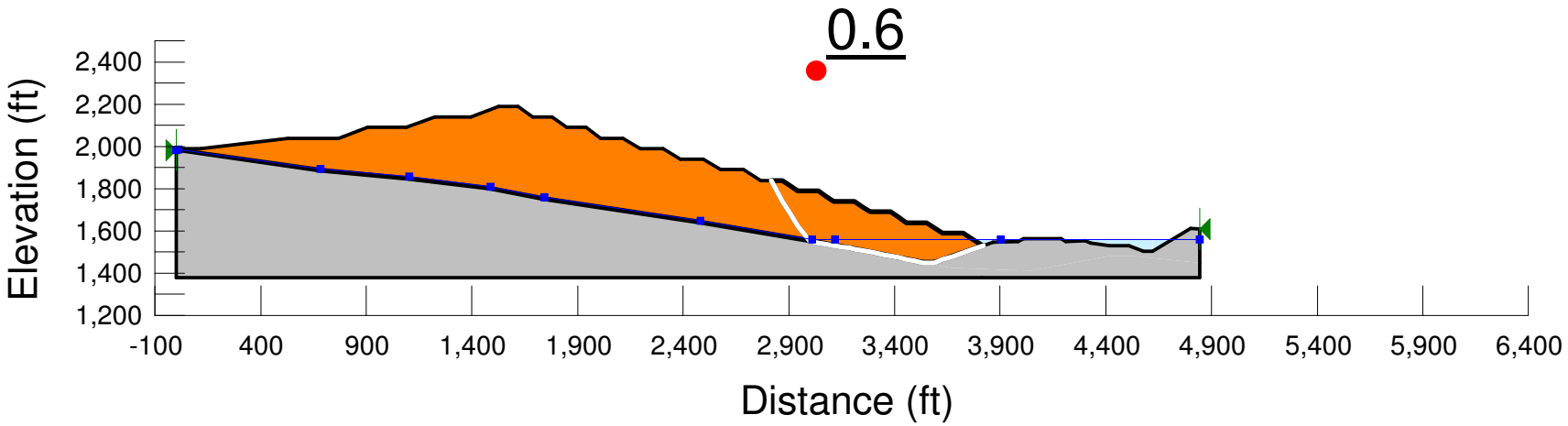
Section B
Failure Through HLF Only
Pseudo-Static Loading
MDE, 0.26g

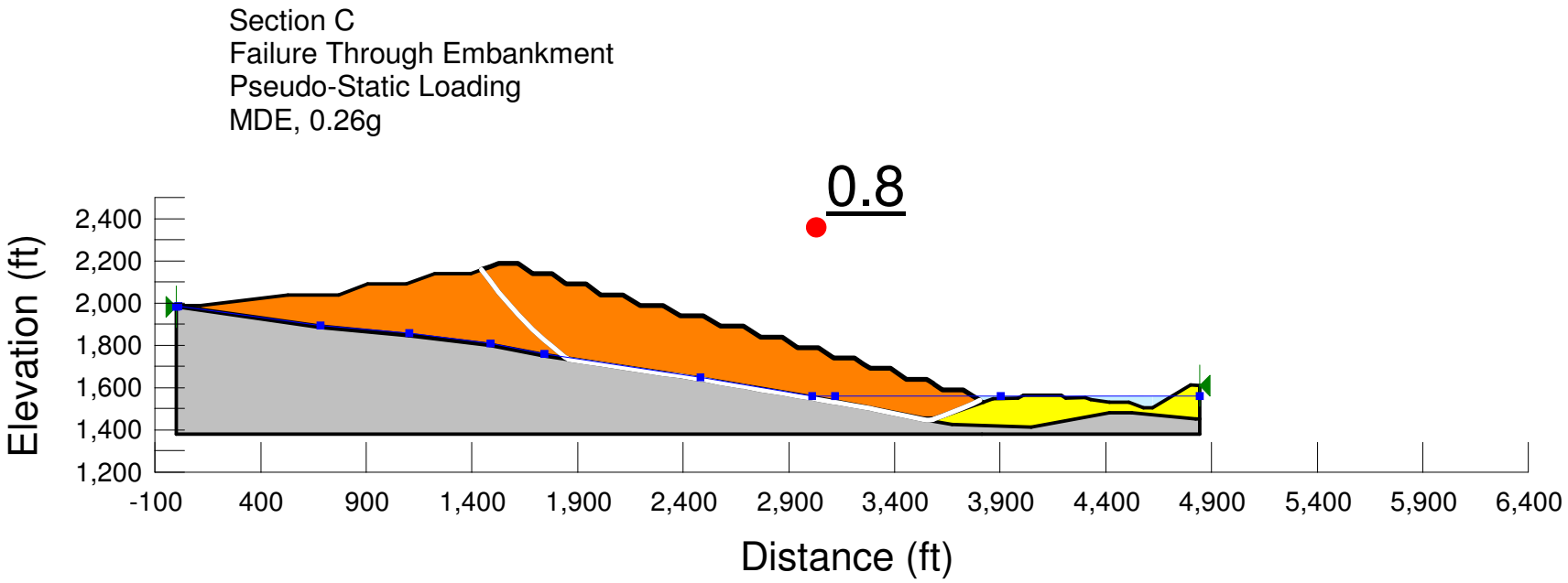


Section B
Failure Through Embankment
Pseudo-Static Loading
MDE, 0.26g



Section C
Failure Through HLF Only
Pseudo-Static Loading
MDE, 0.26g




Section C
Failure Through Embankment
Pseudo-Static Loading
MDE, 0.26g
0.8
Elevation (ft)
2,400
2,200
2,000
1,800
1,600
1,400
1,200
-100
400
900
1,400
1,900
2,400
2,900
3,400
3,900
4,400
4,900
5,400
5,900
6,400
Distance (ft)

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# Appendix C-5

# Stability Analyses Results and Calculations

# Estimation of Representative Peak Horizontal Ground Accelerations (PHGA) for Potential Sliding Mass

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1999 Broadway, Suite 600, Denver, CO 80202
Telephone: (303) 629-8788
Fax: (303) 629-8789

PROJECT Ft Knox - BCHLF PSEUDO-STATIC ANALYSES

P/A DU101-89.71 TASK ..... WI ..... SHEET No 1/3

DESIGNED JC DATE ..... CHECKED ..... DATE .....

DETERMINE REPRESENTATIVE HORIZONTAL ACCELERATIONS FOR PSEUDO-STATIC ANALYSES

PUGA

OPERATIONAL MDE = 0.27g = OBE
CLOSURE MDE = 0.39g = MDE

MAGNITUDE = M 7.5

→ SEISMIC DESIGN DATA FROM PFS REPORT (DU101-89.41)

→ USING MAKDISI & SEED (1978) TO TRANSLATE BASE PUGA TO PUGA OF SLIDING MASS

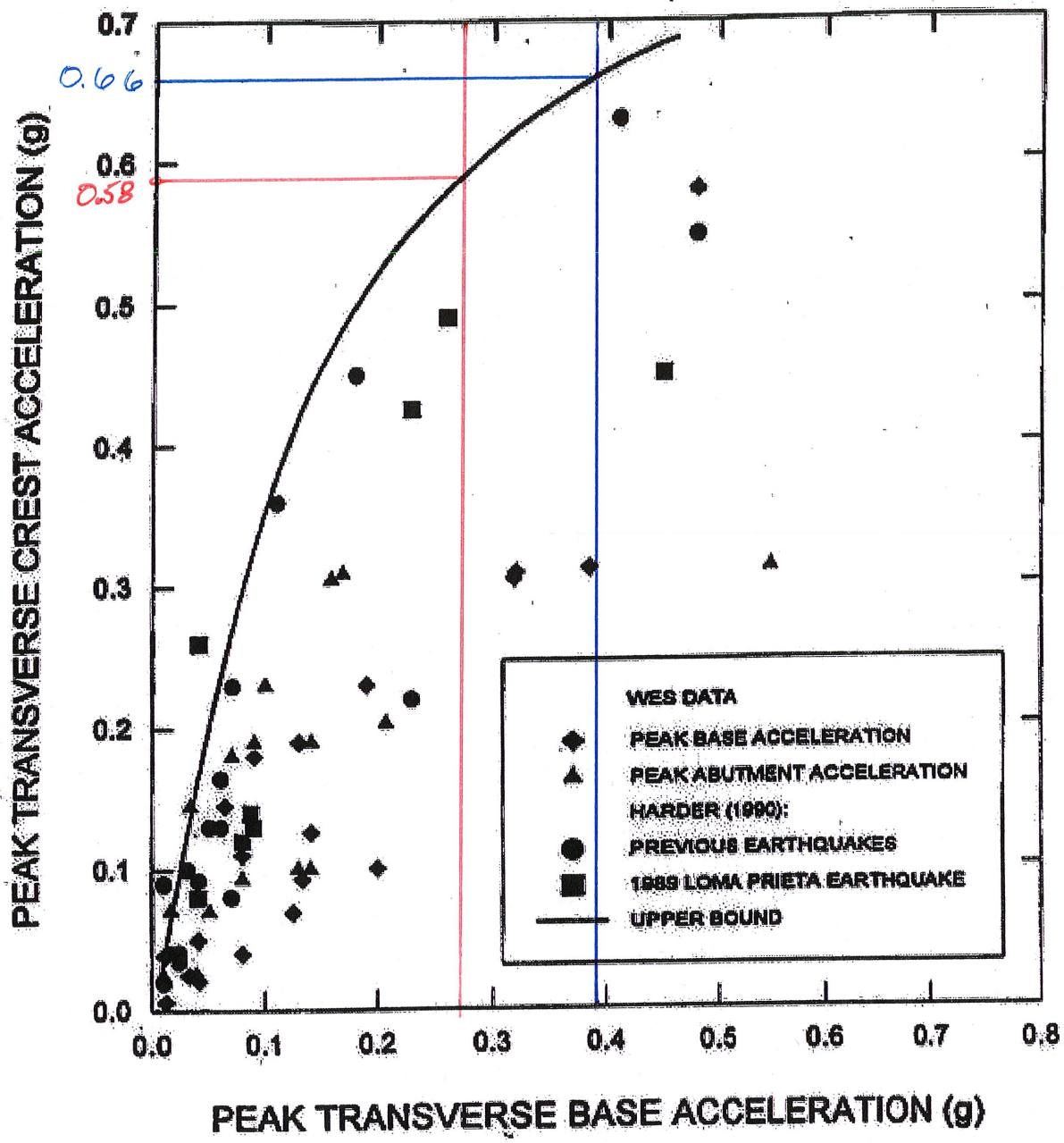
① DETERMINE CREST PUGA ($u_{max}$) (ATTACHED Pg 2/3)

OBE $PUGA_{CREST}$ = 0.58g
MDE $PUGA_{CREST}$ = 0.66g

② DETERMINE SCALING FACTOR ($k_{max}/u_{max}$) TO TRANSLATE CREST PUGA ($u_{max}$) TO SLIDING MASS PUGA ($k_{max}$)
$k_{max}/u_{max}$ = 0.4 FOR FULL HEIGHT FAILURE

③ DETERMINE PUGA OF SLIDING MASS ($k_{max}$) FOR INPUT INTO PSEUDO-STATIC ANALYSES

OBE → $k_{max}$ = 0.23g
MCE → $k_{max}$ = 0.26g

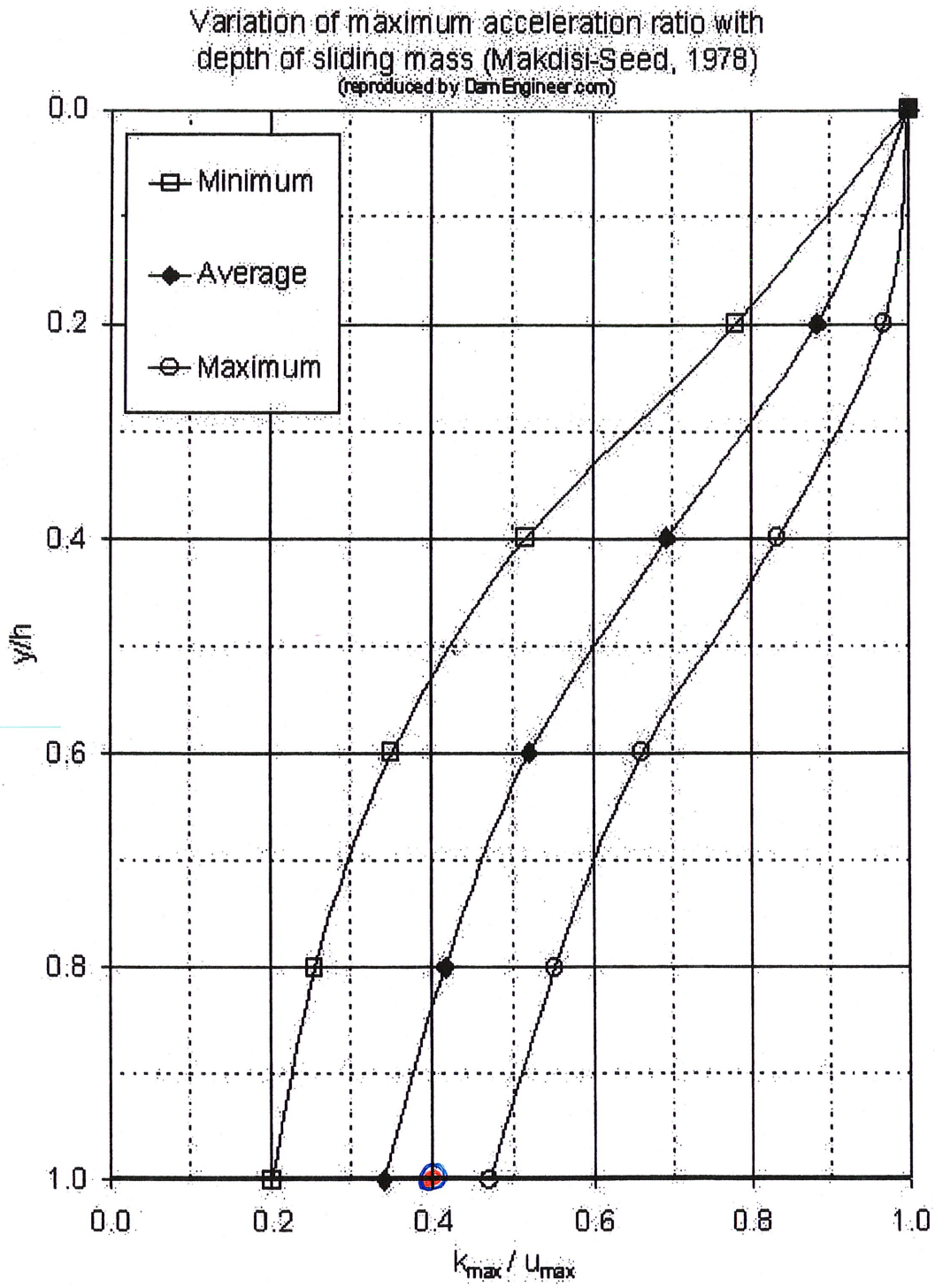



OBE $PHGA_{BASE} = 0.27g$

MCE $PHGA_{BASE} = 0.39g$

http://damengineer.com/embankments/base2crt.jpg

9/8/2008


Variation of maximum acceleration ratio with depth of sliding mass (Makdisi-Seed, 1978)
(reproduced by DamEngineer.com)
Minimum
Average
Maximum
y/h
0.0
0.2
0.4
0.6
0.8
1.0
$k_{max} / u_{max}$

- OBE
- MCE

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# Appendix D

## Settlement Analyses Results and Supporting Information

D-1 Expected Settlement and Elongation Figures
D-2 Elongation Strain at Liner System Calculation
D-3 Biaxial Strain at Liner System Calculation

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# Appendix D-1

## Settlement Analyses Results and Supporting Information

## Expected Settlement and Elongation Figures


Figure 1 BCHLF Section A –
Expected Settlement Contours at Full Build Out

Figure 2 BCHLF Section B –
Expected Settlement Contours at Full Build Out

Figure 3 BCHLF Section C –
Expected Settlement Contours at Full Build Out

Figure 4 BCHLF Section A –
Expected Settlement Along Liner Interface

Figure 5 BCHLF Section B –
Expected Settlement Along Liner Interface

Figure 6 BCHLF Section C –
Expected Settlement Along Liner Interface

Figure 7 BCHLF Section A –
Expected Elongation Strain at Liner Interface

Figure 8 BCHLF Section B –
Expected Elongation Strain at Liner Interface

Figure 9 BCHLF Section C –
Expected Elongation Strain at Liner Interface

Figure 1
BCHLF Section A
Expected Settlement Contours at Full Build Out

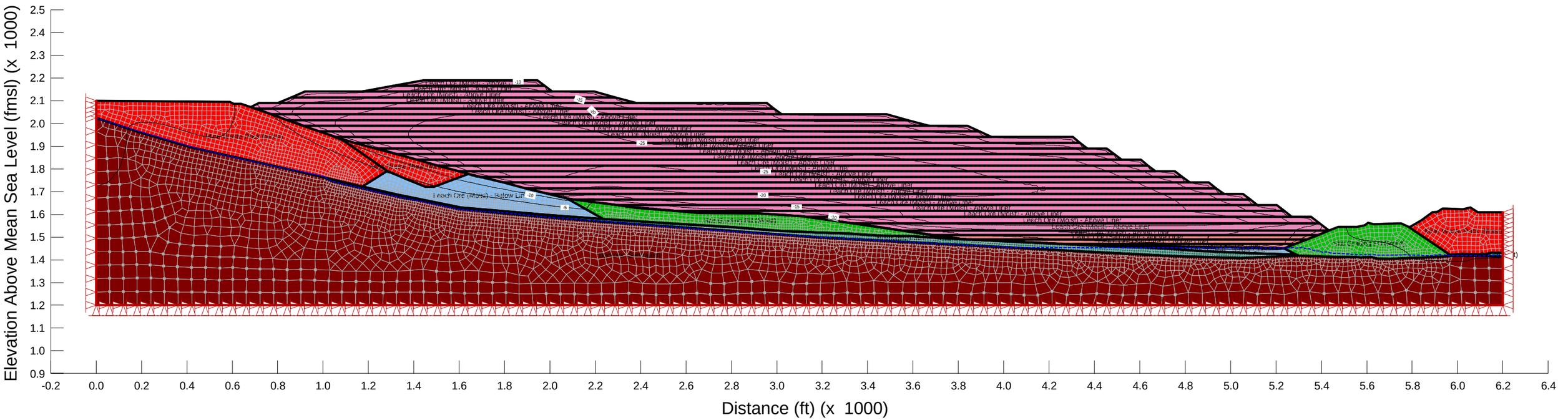


Figure 2
BCHLF Section C
Expected Settlement Contours at Full Build Out

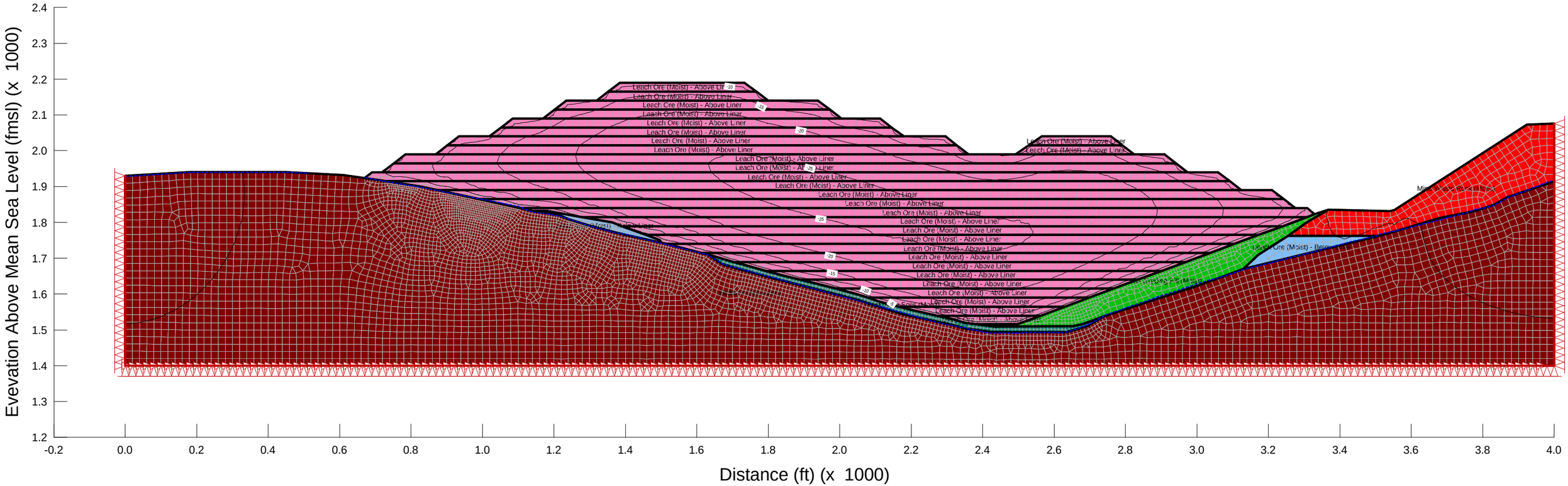

Evevation Above Mean Sea Level (fmsl) (x 1000)
2.4
2.3
2.2
2.1
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
-0.2
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
Distance (ft) (x 1000)
Leach Ore (Moist) - Above Liner
-10
-15
-20
-25
Leach Ore (Moist) - Above Liner
Leach Ore (Moist) - Above Liner
Leach Ore (Moist) - Above Liner

Figure 3
BCHLF Section D
Expected Settlement Contours at Full Build Out

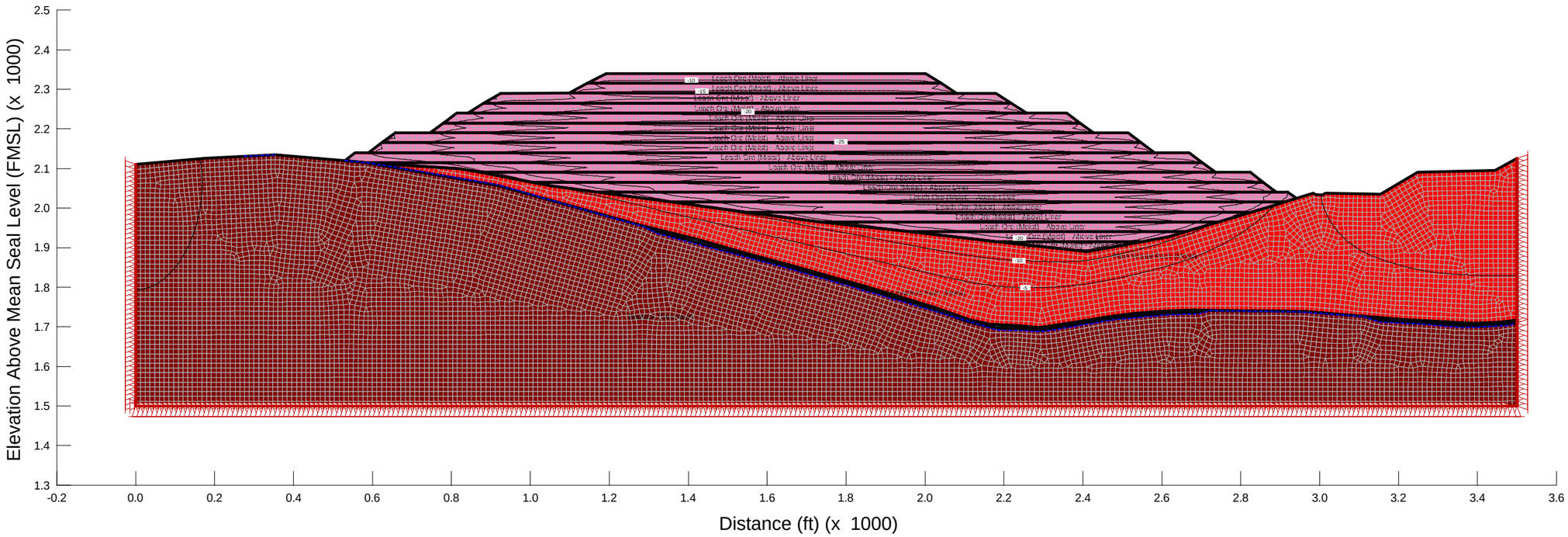


Figure 4

BCHLF Section A - Expected Settlement Along Liner Interface

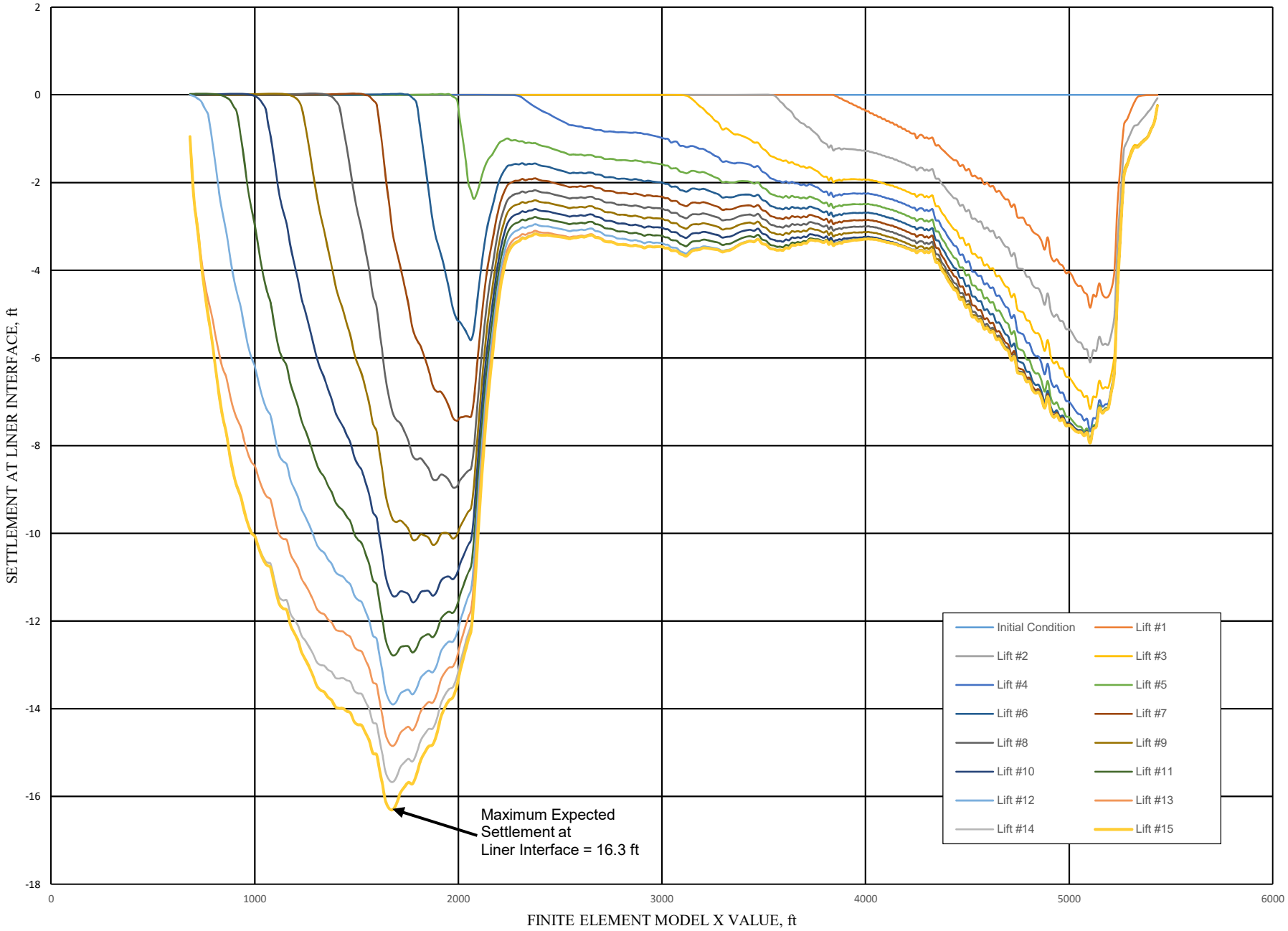


Figure 5
BCHLF Section C - Expected Settlement Along Liner Interface

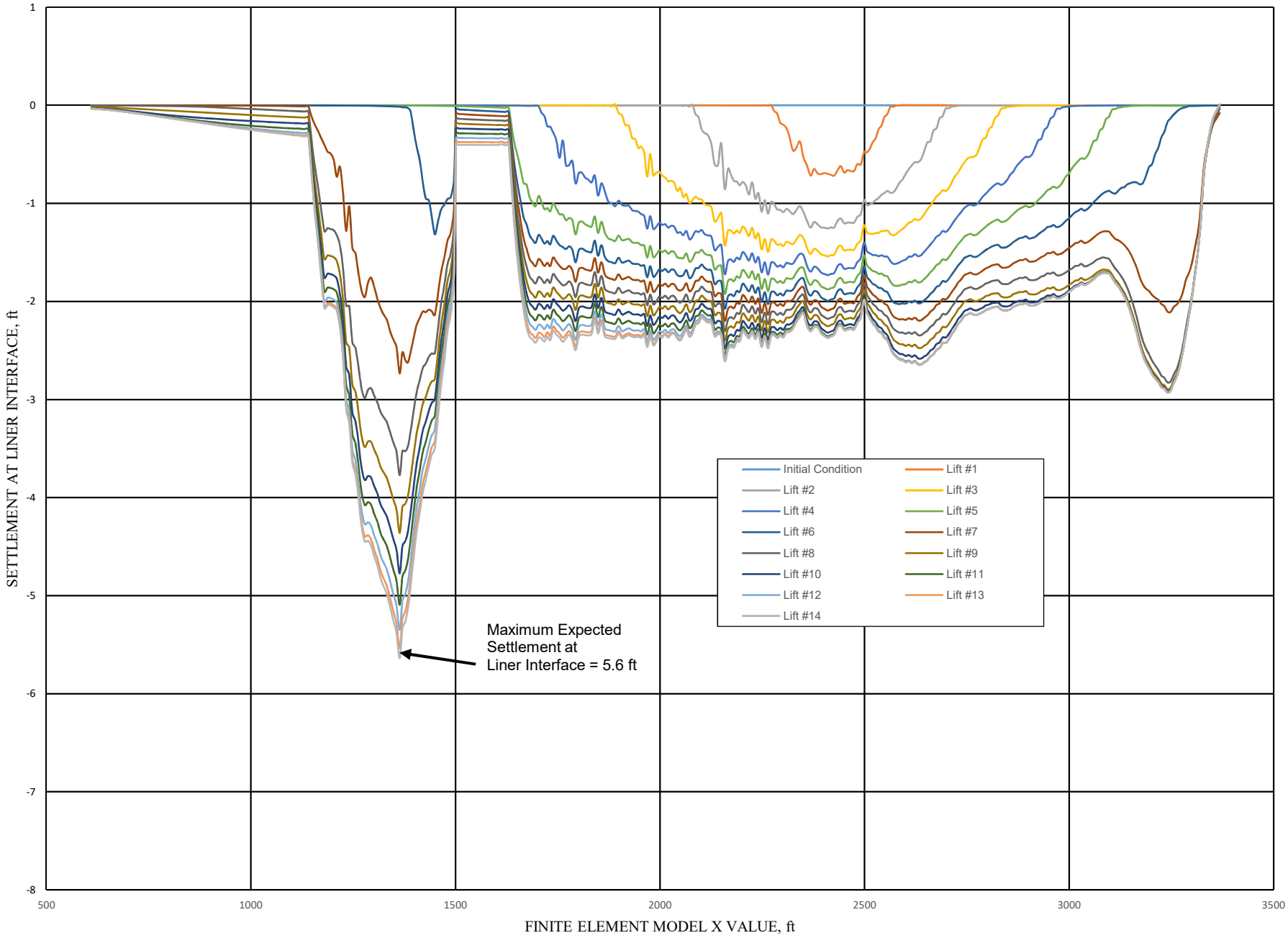


Figure 6
BCHLF Section D - Expected Settlement Along Liner Interface

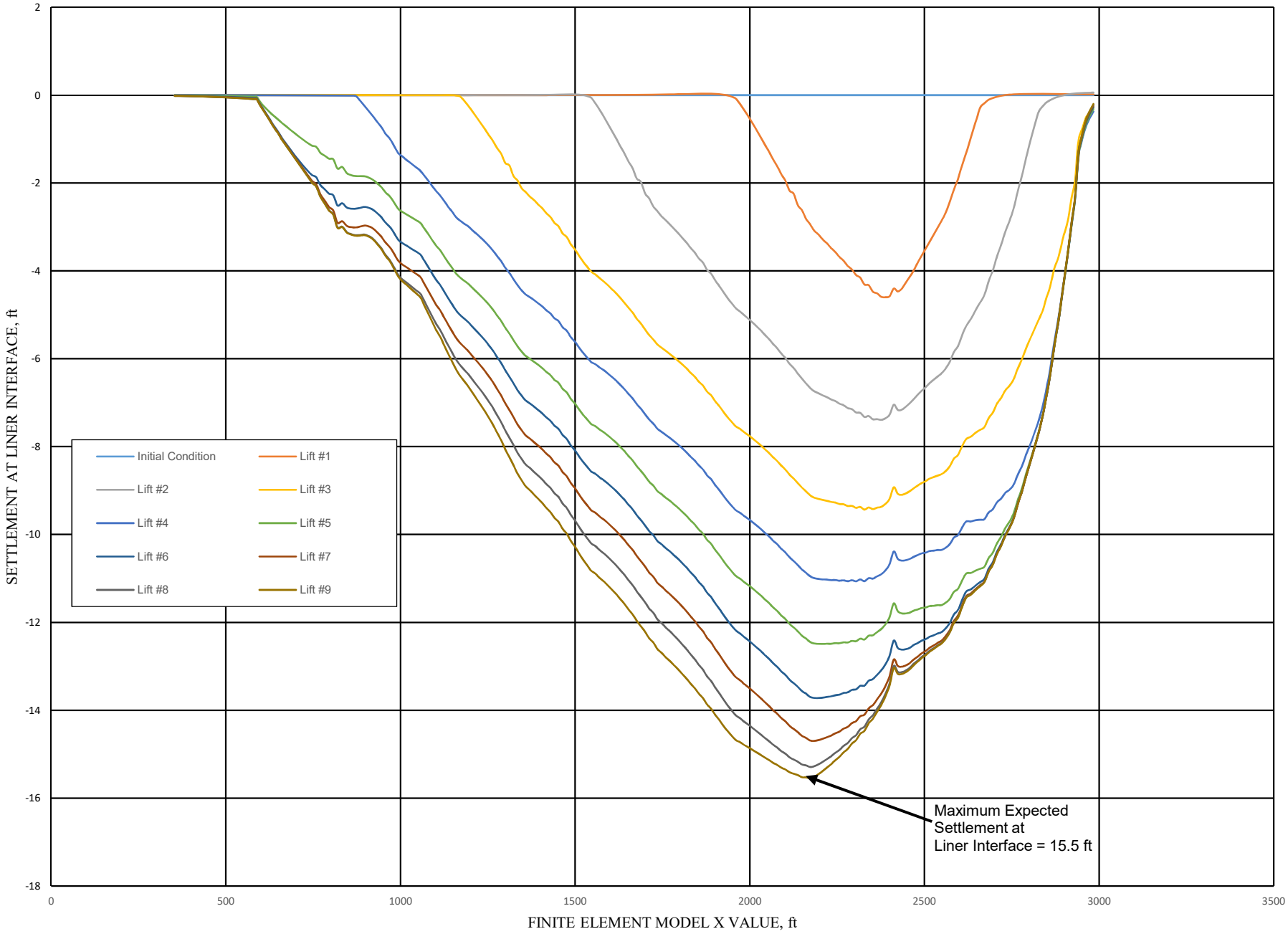


Figure 7
BCHLF Section A - Expected Elongation Strain at Liner Interface

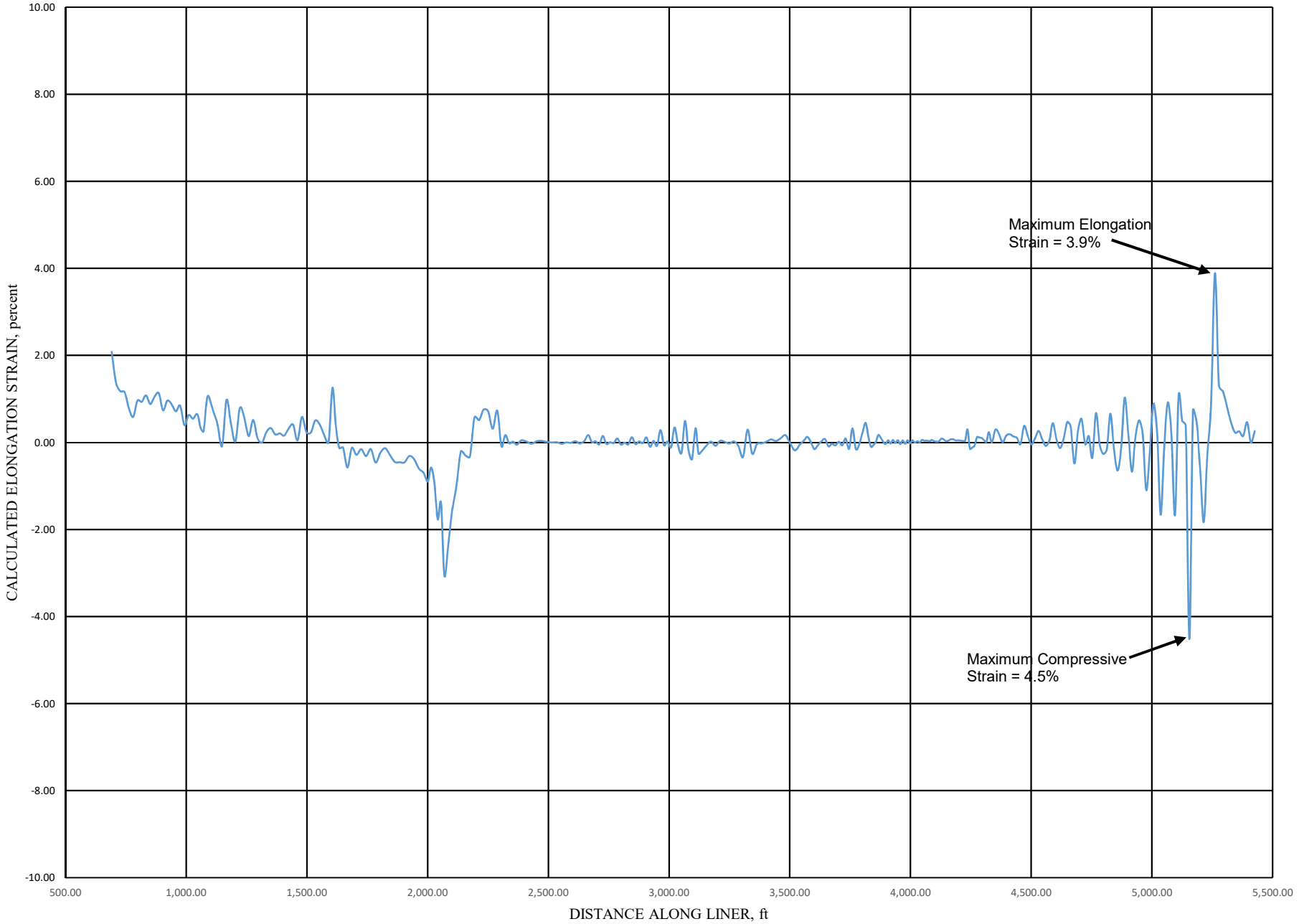


Figure 8
BCHLF Section C - Expected Elongation Strain at Liner Interface

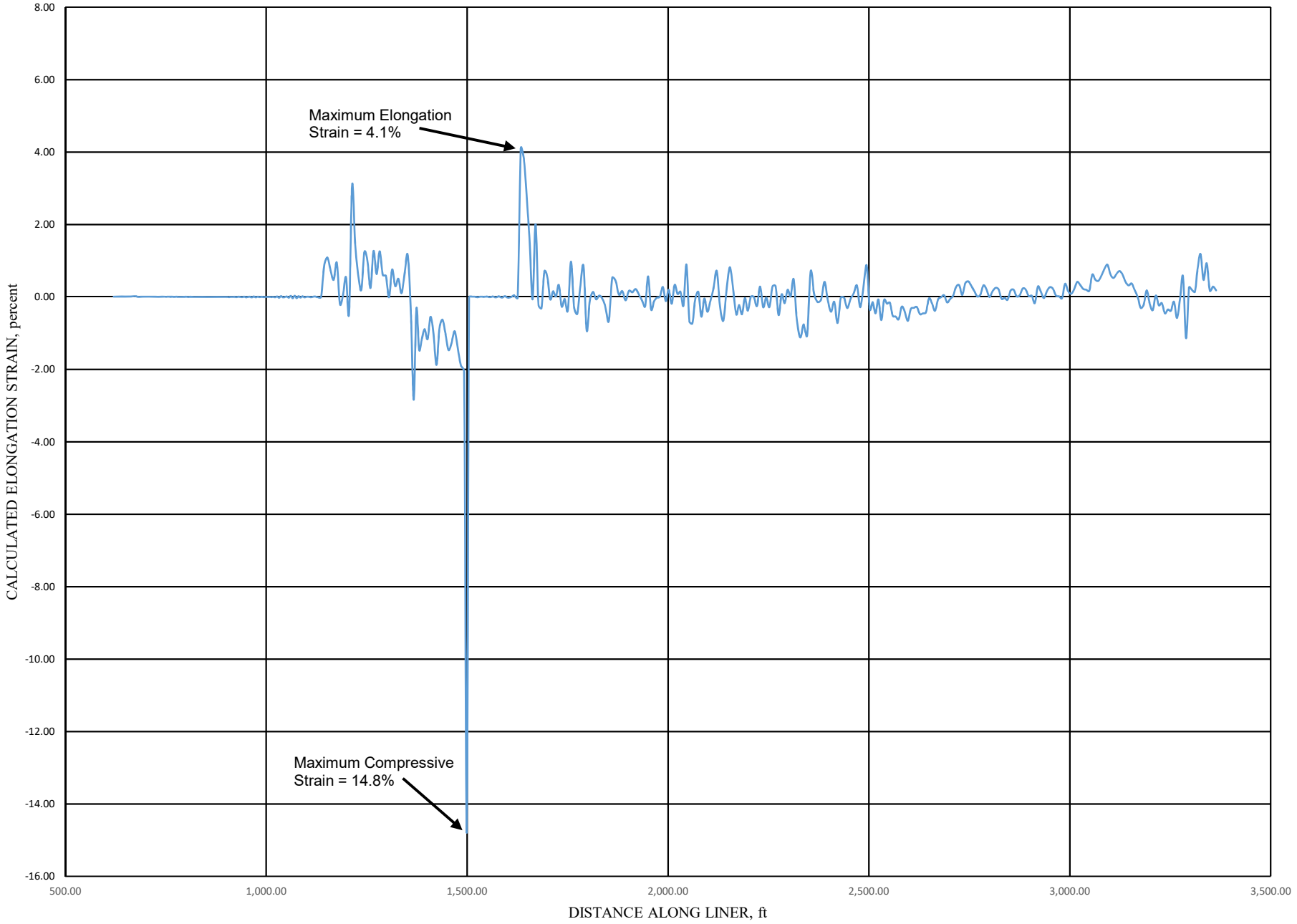
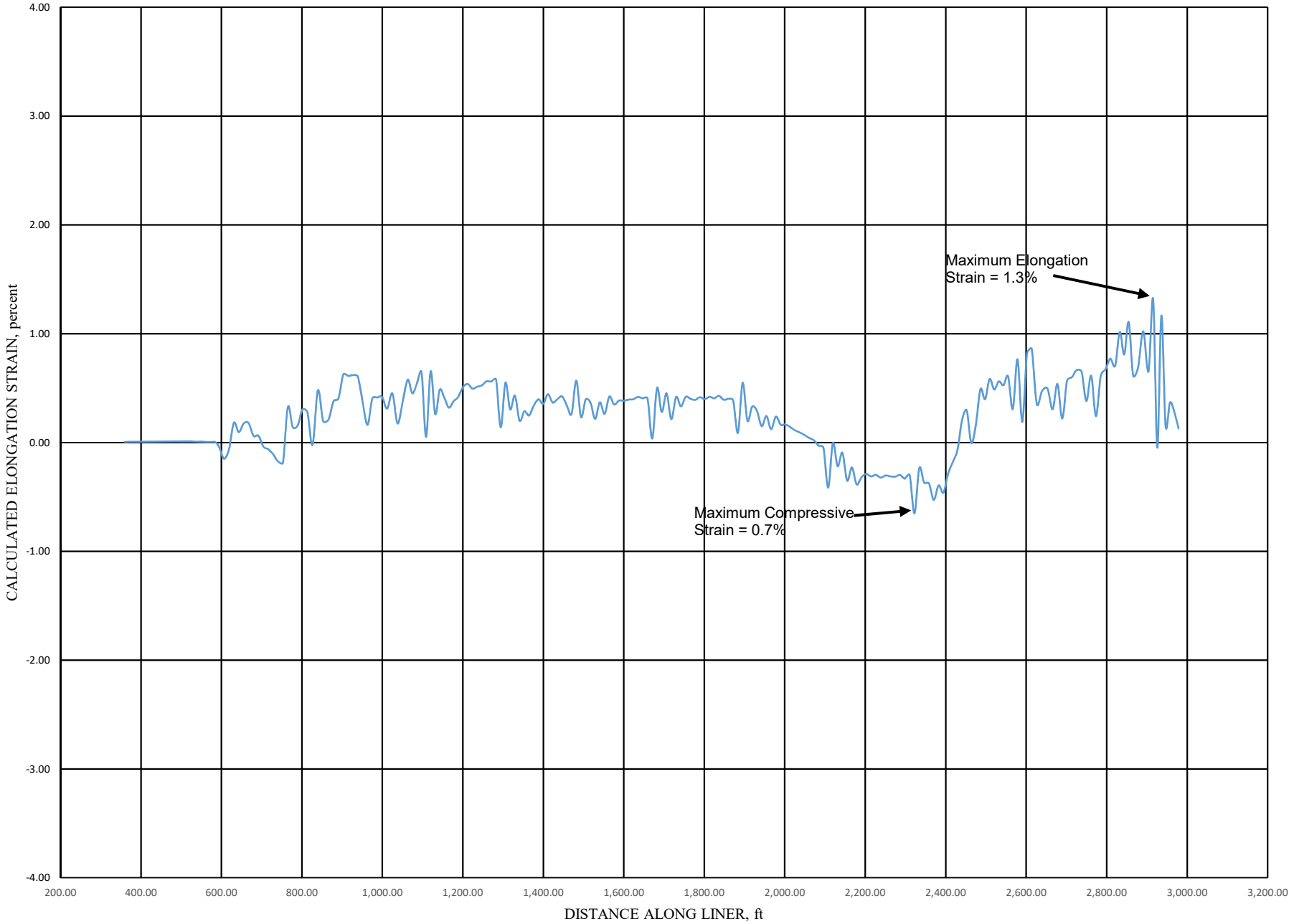


Figure 9
BCHLF Section D - Expected Elongation Strain at Liner Interface



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# Appendix D-2

# Settlement Analyses Results and Supporting Information

# Uniaxial Strain at Liner System Calculation

**Knight Piésold**
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PROJECT.....

P/A ..... TASK ..... WI ..... SHEET No .....

DESIGNED ..... DATE ..... CHECKED ..... DATE .....

# BCHLF - Strain Calculation (Uniaxial)

Given: (from Geostudio output)
- initial 'X' coordinates along liner ⇒ 'Xi'
- initial 'Y' coordinates along liner ⇒ 'yi'
- 'X' displacement at full buildout ⇒ $\Delta x$
- 'y' displacement at full buildout ⇒ $\Delta y$

To calculate strains between two adjacent nodes on the liner interface '1' and '2'

initial distance between nodes 1 and 2 'Li'

$$L_i = \sqrt{(x_{i2} - x_{i1})^2 + (y_{i2} - y_{i1})^2}$$

final distance between nodes 1 and 2 'Lf'

$$L_f = \sqrt{\left((x_{i2} + \Delta x_2) - (x_{i1} + \Delta x_1)\right)^2 + \left((y_{i2} + \Delta y_2) - (y_{i1} + \Delta x_1)\right)^2}$$

$$\text{Uniaxial Strain \%} = \left(\frac{L_f - L_i}{L_i}\right) * 100$$

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# Appendix D-3

# Settlement Analyses Results and Supporting Information

# Biaxial Strain at Liner System Calculation

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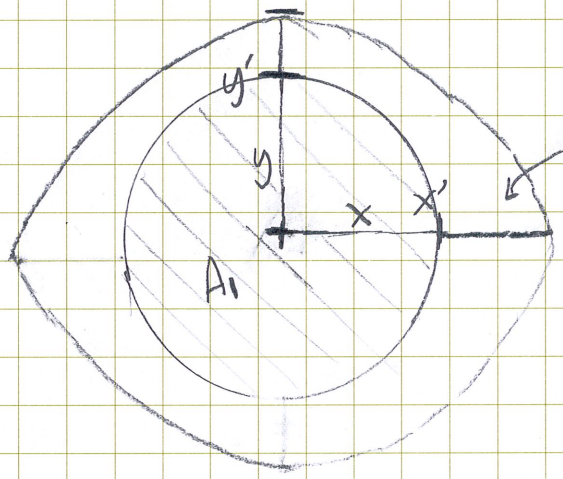
BCHLF

Calculate:

Maximum expected Biaxial Elongation Strain at Liner




$$\varepsilon_{biaxial} = \frac{A' - A}{A}$$

Assumptions:

Maximum elongation Strain along Section A occurs at the same location as max elongation Strain along Sections C and D.

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Initial case: Assume unit circle with $D = 1.0$, $X = 0.5$

① case #1 - assume section A along x axis and section C along y axis

$\varepsilon_{max\ elongation}$ - section A = 3.9%

$\varepsilon_{max\ elongation}$ - section C = 4.1%

$x' = x(1+\varepsilon) = (0.5)(1.039) = 0.5195$

$y' = y(1+\varepsilon) = (0.5)(1.041) = 0.5205$

$A = \pi x^2 = \pi(0.5^2) = 0.785$ $A' = \pi x' y' = \pi(0.5195)(0.5205) = 0.849$

$$\varepsilon_{biaxial} = \frac{A' - A}{A} = 8.2\%$$

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(2) case #2 assume section A along x axis and section D along y axis

$\varepsilon_{max}$ - section A = 3.9% elongation

$\varepsilon_{max}$ - section D = 1.3% elongation

$x' = x(1+\varepsilon) = (0.5)(1.039) = 0.5195$

$y' = y(1+\varepsilon) = (0.5)(1.013) = 0.5065$

$A = \pi x^2 = \pi(0.5^2) = 0.785$ $A' = \pi x' y' = \pi(.5195)(.5065) = 0.8266$

$$\varepsilon_{biaxial} = \frac{A' - A}{A} = 5.3\%$$

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# Appendix E

# Peggs, et al., 2005, Assessment of Maximum Allowable Strains in Polyethylene and Polypropylene Geomembranes

Peggs, I. D., Schmucker, B., & Carey, P. (2005). Assessment of maximum allowable strains in polyethylene and polypropylene geomembranes. *Waste Containment and Remediation* (pp. 1-16).

# Assessment of Maximum Allowable Strains In Polyethylene and Polypropylene Geomembranes

**Ian D. Peggs[1], Bruce Schmucker[2] and Peter Carey[3]**

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## *Abstract*

Geomembranes used in separation liners between old and new waste in vertical expansions of landfills may be subject to differential settlement strains. Therefore, it is important to define a maximum allowable strain (MAS) that any given geomembrane can tolerate without compromising its required service life. In a number of instances some very low values have been proposed – as low as 0.25% and 1.0%for high density polyethylene (HDPE). Such low numbers are probably based on German regulations for HDPE landfill liners that require a maximum allowable global strain of 3% and a limiting local strain due, for example, to protruding drainage stones, of 0.25%. The low allowable strain values adopted by the German regulators have been based on products and practices utilized in the 1980's and do not reflect current conditions, nor do they address membranes other than HDPE. In this paper we present the background and reasoning for updating MAS values.

## *Introduction*

The maximum allowable strain (MAS) in landfill lining systems has become of major importance with the increasing interest in vertical expansion. Landfill liner specifications and regulations have been based on the use of HDPE geomembranes. In other words "HDPE" is synonymous with "geomembrane". This is not a logical situation, since materials such as LLDPE and PP, although of the same polyolefin plastic family as HDPE, have very different mechanical properties that can be used to advantage in a number of applications. Therefore, they are viable candidate geomembrane materials that should be treated quite differently to HDPE.

The major reason for this difference in performance is that HDPE has a semi-crystalline microstructure that makes it susceptible to stress cracking (SC). No other common geomembrane is susceptible to SC in its as-manufactured condition. Hence the reason for limiting geomembrane strains in HDPE geomembranes. However,

recent attempts to limit general strains to 0.25% and 1.0% (whatever the geomembrane material) are too restrictive and cannot be justified on a technical basis.

***HDPE Mechanical Properties***

The chemical resistance and high strength properties that make HDPE so appropriate as a bottom liner are a consequence of its semicrystalline microstructure. Most HDPEs are about 55% crystalline and 45% amorphous. In comparison PP is about 10% crystalline and LLDPE is about 5% crystalline. These different microstructures result in very different short term and long term mechanical performance characteristics. The reasons for the short term differences are most clearly shown in the uniaxial stress-strain curves as shown in Figure 1. HDPE has a unique point of instability – the yield point – which occurs at about 12% strain and at which the tensile specimen thins down locally and elongates like gum. For design purposes HDPE should be, and usually is, considered to have failed at the yield point.

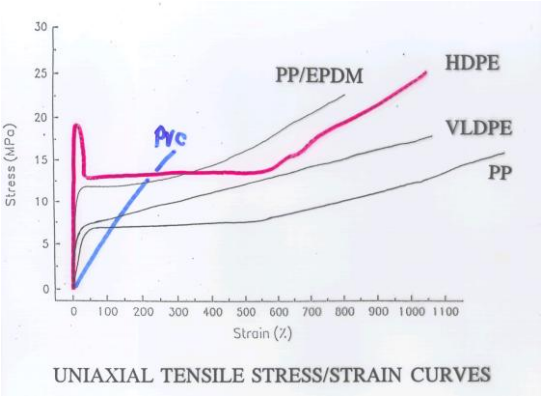


Figure 1. Uniaxial tensile stress strain curves

In comparison the uniaxial stress/strain curves for VLDPE (similar to lower density LLDPEs), PVC, and PP do not have the distinct yield point that HDPE has, so they can be considered far more stable materials. Even so, except for PVC, they do show rapidly increasing strain as stress increases above the steeper predominantly elastic region of the curve.

***Multiaxial tensile properties***

However, the uniaxial stress/strain curves can rarely be used for geomembrane design purposes since uniaxial stressing situations do not occur in the field. An installed geomembrane when predominantly stressed in one direction cannot elongate by drawing material from a direction perpendicular to the applied stress since that material is effectively anchored somewhere. These are typically plane strain conditions. To reproduce these conditions in the laboratory axisymmetric or multiaxial stress strain curves are generated by hydrostatically deforming large circular specimens of geomembrane clamped uniformly around the edges, thereby making the strain in the specimen uniform in all directions in the plane of the

material. Figure 2 shows typically how the strains at break of several different materials are quite different, with HDPE showing the least amount of strain (typically about 30%) and PP the highest. Koch (1987), working on both HDPE pipe and geomembrane, found that biaxial strain amounted to about 25% before the first signs of yielding leading to failure were noted.

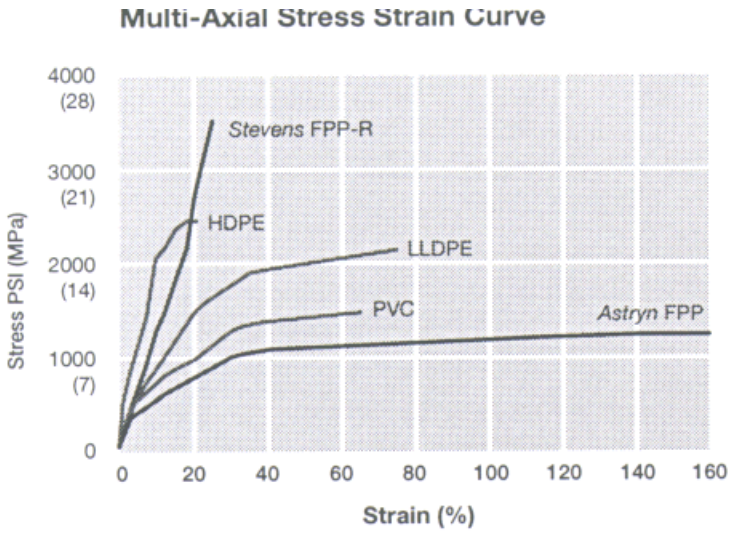


Figure 2. Typical multiaxial tensile stress/strain curves

**Stress Cracking Resistance**

The most significant difference between the materials is the susceptibility to SC demonstrated by HDPE but not by the other materials. This is a consequence of the semicrystalline microstructure that gives HDPE its high strength and broad chemical resistance. The stress cracking resistance (SCR) is not identical in all HDPE geomembranes. It is a function of the HDPE resin used, the molecular weight, the molecular weight distribution, and comonomer used to give each manufacturer's HDPE product the desired mechanical durability.

SC, often erroneously referred to as environmental stress cracking, is a brittle fracture that occurs under a constant stress less than the yield stress or break stress of the material. It is a fundamental property of all HDPEs. Environmental stress cracking is an acceleration of the fundamental SC phenomenon that occurs in chemical environments such as detergents, oxidizing acids, and chlorinated solvents.

In 1992 Hsuan et al. took five field samples of commercially available HDPE geomembranes and measured their SCRs according to the new (then) GRI.GM5 notched constant tensile load test. In this test different weights are added to a dogbone-type specimen in a surface-active agent at an elevated temperature (50ºC) in order to accelerate the break time. Times to break are monitored. To generate the plane strain conditions of the field, the condition necessary to generate a SC type of break, a notch (razor cut) is placed across one face of the specimen. The depth of the

notch is 20% of the thickness of the geomembrane. Plane strain conditions occur at the bottom of the notch. Thus instead of performing a uniaxial tensile test or a creep test, a plane strain stress cracking test is performed. Figure 3 shows the test results.

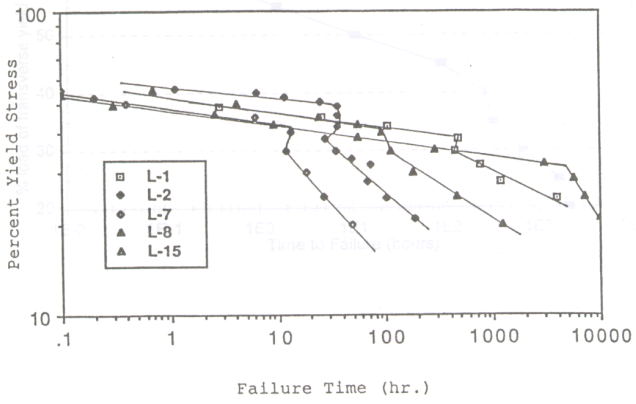


Figure 3. Stress rupture curves on five commercially available geomembranes.

At the higher loads (stresses) breaks occurred in a ductile mode to generate the shallow slope segment of the curve. However, at some critical lower stress, the curve became steeper and breaks occurred without any initial ductility – this is the stress cracking region. Thus the expected long lifetime at lower stress obtained by extrapolating the shallow segment of the curve is cut very short. It is this stress cracking phenomenon that we are trying to avoid by limiting stress, therefore strain, and also by using resins with high SCR. The SCR of the specific geomembrane is represented by the time at which the change in slope (the knee) of the curve occurs. In Figure 4 this knee occurs at times between 10 hr and 5000 hr. There are almost three orders of magnitude difference in the SCRs of the five different geomembranes.

Since then the test has been simplified for QC and QA conformance testing by applying a single stress of 30% of the room temperature yield stress (which will cause a stress cracking break) and requiring break time to exceed 200 hr, recently increased to 300 hr. There are now very few HDPE geomembranes that have SCR times less than about 500 hr and some that have times in excess of 10,000 hr.

SC is also accelerated at any given stress as temperature increases, as shown in Figure 4.

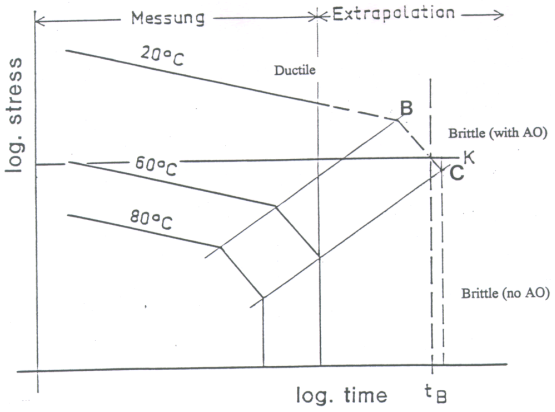


Figure 4. Stress rupture curves as a function of temperature.

It was this susceptibility to stress cracking that prompted German regulators to limit the strains to which HDPE geomembranes could be subjected. This was driven by the need to prevent damaging puncture stresses by drainage stones, and therefore to define the type of test required to assess the protection capabilities of geotextiles and other protection systems. It is important to note the differences between the German and US approaches to geomembrane protection as it affects the limiting strain. The Germans were concerned about a deformation in the geomembrane causing premature failure of the geomembrane by stress cracking some time in the future. The US approach is to assess puncture protection by determining whether complete geomembrane penetration occurs at the time of the test. Hence the German emphasis is on limiting strain, a concern that has not appeared until recently in the US.

***Allowable Strain***

The Germans took two approaches to arrive at a limiting strain. Initially, in 1990, three meetings were held between interested parties calling themselves the "Quo Vadis" group. They identified (Naue Fasertechnik, 1992) the basis of the test method and arbitrarily defined the limiting multiaxial strain at puncture protrusions (not complete penetrations) to be 0.25%. However for a more general strain allowance a maximum of 3% was defined. According to Jones et al. (1998), the Quo Vadis group felt that 6% total strain was the maximum allowable strain for a "satisfactory lifetime performance", which, when applying a safety factor of 2.0 became 3% allowable strain. The reason for the initial selection of 6% strain is not known, but it is comparable to a maximum strain of 5% recommended by Janson (1981) for long term performance of buried HDPE pipe. However, Jones et al. (1998) state:

> *" Clearly more research is required to establish a more rigorous scientific basis for defining this threshold"*

That this threshold is too pessimistic (too low) is also expressed by Sehrbrock (2002), a member of the original Quo Vadis group, where he states that many members of the

group felt that the 3% maximum should be reserved only for settlement of the subgrade. This implies that 3 % should be allowed for differential settlement on top of a more global strain that might occur in the geomembrane.

Sehrbrock (2002) confirms that the 0.25% local strain was a compromise, simply because zero strain (their obvious target) was impossible to measure and confirm. It would also be impractical to achieve zero strain in the field.

The second approach to these limiting strains is described by Seeger and Müller (1996) of the Federal institute of Materials Research and Testing (the BAM Institute – Germany's landfill liner regulatory agency). They also identify a maximum global strain of 3% upon which a maximum additional local strain of 0.25% is allowed due to such things as protrusions by individual drainage stones. This approach is based on the excellent work done at Hoechst Aktiengesellschaft in Germany in the early 1980s on the durability testing of HDPE pipes. Koch at al. (1988) generated creep resistance (stress rupture) curves for pipes at different temperatures and internal pressures (hoop stresses) as shown in Figure 5. These curves are similar to those in Figure 3 subsequently obtained by Hsuan et al. (1992) for geomembranes.

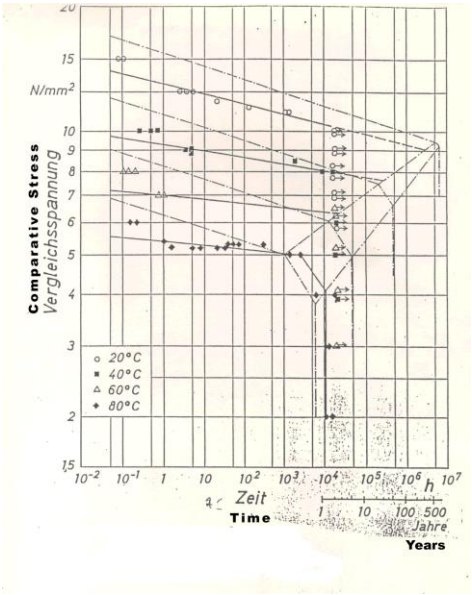


Figure 5. Creep rupture curves for two HDPE pipes made with different resins (Koch et al.)

Unlike in the pipe pressure test and in the notched constant tensile load test where the applied pressure/load is constant with time, Seeger and Müller (1996) recognized that the stress imposed on a confined geomembrane in a lining system would relax with time. Koch et al. show in Figure 6 that the stress relaxation rate is independent of initial strain between 1 and 6% strain and that the maximum stress is a function of initial strain rate; the slower the applied strain the lower the maximum stress achieved – see curves 2a, 2b, and 2c. This will be the situation in the separation liner of the a

vertical expansion, where the strain, and consequently the stress, will build up slowly with time. The stress will not build to the same high level it would have done had the strain been rapidly applied. Recollect that it is the induced stress resulting from the strain, not the strain itself, that defines the material durability. Also note that Koch et al.'s (1988) stress relaxation work was done using uniaxially-stressed specimens. Stress relaxation and uniaxial/biaxial stress factors will be reviewed later.

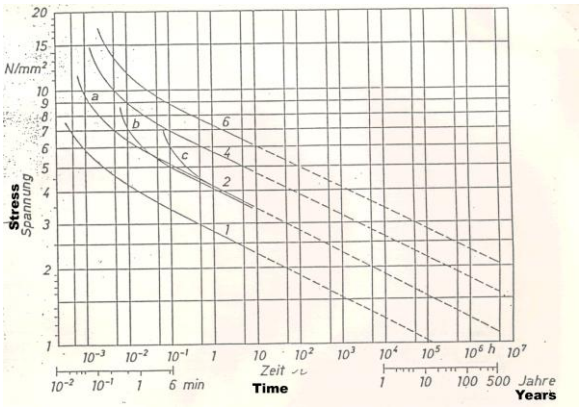


Figure 6. Stress relaxation curves at 1, 2, 4 and 6% strain (Koch et al. 1988)

Figure 7 shows simply the stress at which the stress relaxation history of a specimen initially rapidly strained to 3% strain and maintained at 40°C intersects the steep (SC) segment of the 40°C stress rupture curve for a service lifetime time of 50 years. It intersects the 20°C stress rupture curve at well over 100 yr. This was felt conservatively to be adequate performance, so 3 % was selected as the MAS in the geomembrane as being that strain that would generate a maximum allowable stress for adequate long term performance. However, note that this stress relaxation test was most likely performed by quickly ramping up the pressure/load to achieve the 3% strain, with no allowance for stress relaxation had the pressure/load been applied slowly.

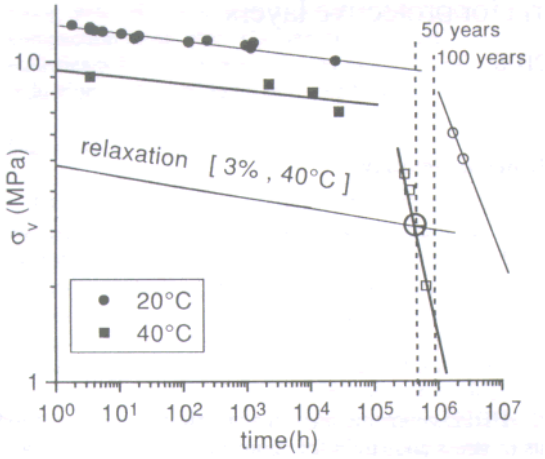


Figure 7. Pipe creep rupture and stress relaxation data (Seeger and Müller 1988)

In Germany 16 to 32 mm drainage stone is used over the geomembrane. It was recognized that individual stones could cause a local puncturing deformation in the top surface of the geomembrane and this would result in bending strains on the lower surface. While this might not result in immediate puncturing there was concern that the stress cracking susceptibility of HDPE might result in premature failure during service from the imposed stresses. In essence, with the minimum specified HDPE thickness of 2.5 mm, a 3% bending strain on the underside of the geomembrane is generated by an indentation that imposes an arch strain in the top surface (due to the indenting stone) of about 0.25%. Hence protection systems were required that would limit the localized multiaxial strain due to a stone indentation to 0.25%.

Thus, a very local strain of 0.25% is not to be exceeded while a global strain of 3% is also not to be exceeded. However, there are a few ameliorating factors that must be taken into account to properly assess the practical performance of a separation geomembrane used in vertical expansions. The two most significant factors are the stress relaxation performance of the geomembrane and the major improvements in the SCR of HDPE geomembranes that have been made since these allowable strains were developed in the mid 1980s. And it is also important to recognize the wide range of SCR values that exist in the different commercially available HDPEs.

As previously described the GRI.GM13 SCR specification for HDPE geomembranes is now a minimum of 300 hr. Until recently many of the European resins on which the 0.25% and 3% allowable strains were based could not meet this specification (as experienced by Peggs, Thomas (TRI/USA), and Jones (Golder/UK in proprietary research and CQA projects), so the US resins have been basically more mechanically durable than the European resins, therefore being able to tolerate higher strains and the consequent stresses. Therefore, Sehrbrock's (2002) statement that 3% general strain is too conservative, particularly for US HDPE resins, is justified.

***Stress Relaxation***

While the benefits of stress relaxation are apparent it is not a topic that has been thoroughly studied for geomembranes. Soong et al. (1994) investigated stress relaxation in a 1.5 mm thick HDPE geomembrane with initial stresses of 40, 50, and 60% of yield stress (at test temperature) and initial strains of 1, 3, and 5%, at temperatures between –10 and 70$^{o}$C. These were quasi-biaxial tensile tests using 4 in. (100 mm) wide by 2 in. (50 mm) gage length "wide width" tensile specimens. Initial loading was done quickly to minimize stress relaxation on loading. Whatever the starting conditions, there was a trend to a very narrow range of final, but still significant stresses, after about 100 days. The relaxation modulus curves (stress/strain as a function of time) for a given starting condition could be superimposed into a master curve for a given relaxation temperature, as shown in Figure 8.

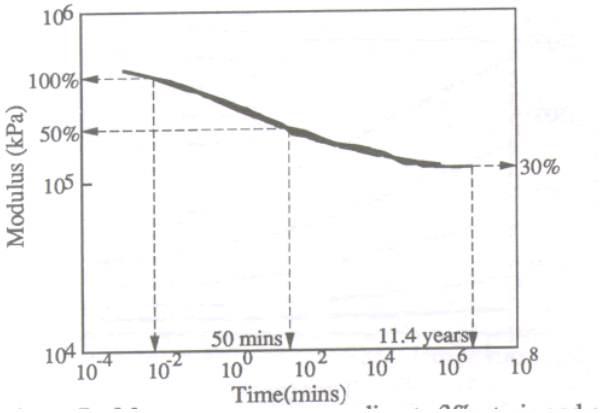


Figure 8. Master stress relaxation curve for 3% strain at 10°C (Soong et al. 1994)

In this case 50% of the applied stress is removed by relaxation after 50 minutes with final equilibrium being achieved at about 30% of applied stress after 11.4 years. At higher temperatures the stress would relax more quickly. The equilibrium residual stress is between 2500 and 4000 kPa, or between about 13 and 21% of the room temperature yield stress. Note that the strain was applied far more quickly than will occur during subgrade settlement, so in a landfill significant stress relaxation will occur during deformation. Soong et al. (1994) stated:

*"Trial tests were performed initially to determine the suitable loading rate. The results suggested a rate of 12.7 mm/min as being appropriate…….. At slower rates a very significant amount of stress relaxation occurred during the loading* process…."

Also, note that Soong et al. (1994) concluded:

> *"…… other HDPE geomembranes will undoubtedly respond differently than the HDPE studied……."*

Thus all HDPE geomembranes are not the same, just as their SCR performances are not the same.

These stress relaxation rates compare well with those generated by Soong and Koerner (1997) for stress relaxation in waves in HDPE geomembranes under a uniform vertical loading. After 1000 hr at temperatures of 23, 42, and 55°C they found stresses relaxed between 60 and 78% leaving residual stresses of between 1% and 22% of the yield stress. However, these tests were done under semi-confined conditions (waves raised off a flat support surface) while the Soong et al (1994) tests were done under unconfined conditions. Under semi-confined conditions the residual stresses were lower than for unconfined specimens, possibly a result of the stress relaxation occurring during loading.

***Creep/Stress Rupture***

Duvall (1993) performed multiaxial creep/stress rupture tests on HDPE geomembranes by clamping a 1.5 mm (60 mil) thick round specimen (density 0.95 g/cm$^3$) in a flanged pipe joint and pressurizing the specimen from one side at up to 60% of the break stress, at temperatures of 23, 40 and 60$^o$C for over 15,000 hr. Stresses and strains in the specimens were determined from the measured deflection of the center of the specimen. After 15,000 hr multiaxial strains of about 20% had been reached without any signs of yielding. Duvall (1993) references work performed by Crissman (1991) who found that at low strain rates yield strains between 20 and 70% occurred in creep tests on "similar" (no explanation) resins at 24$^o$C.

At the higher temperatures both ductile and brittle breaks occurred as shown in Figure 9. Thus these specimens show the same ductile to brittle transition " knee" as the uniaxial specimens in Figure 3. Ductile break strains were in the region of 30% and brittle break strains were around 20%.

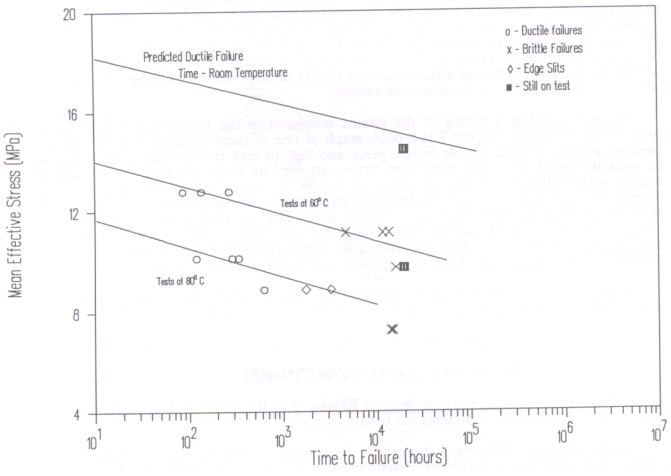


Figure 9. Multiaxial creep curves (Duvall 1993)

Duvall (1993) repeated Soong et al.'s (1994) warning that his data only applied to the test-specific HDPE geomembrane product and that other materials, even those with similar Melt Index and Density, could behave quite differently.

Thus, it is not reasonable to define a single MAS for HDPE geomembranes without unjustifiably penalizing the more mechanically durable higher SCR products.

***DISCUSSION***

The data presented show that, because of its prominent yield point and its susceptibility to SCR, HDPE is unique when compared to LLDPE, PP, and other geomembrane materials in its requirement for a MAS. Not only that, but because of

their wide range of SCR values, different HDPEs will also require different MAS values.

In 1997 Smolkin and Chevrier determined a maximum allowable local strain for HDPE geomembrane lining systems after consideration of both the US and German approaches. They felt that the maximum 0.25% strain at stone indentations was far too conservative. They performed laboratory compression tests using two different protection geotextiles (550 g/m$^2$ and 1200 g/m$^2$) between their drainage stone (19 mm) and the proposed 2.0 mm thick HDPE geomembrane. After 240 hr the maximum indentation arch strains in the geomembrane under the two geotextiles were 1.50% and 0.92% respectively. After 1000 hr they projected that maximum arch strains would be about 2% and 1 to 2% respectively. They then calculated an allowable long term stress that would "prevent rupture" using the approach of Berg and Bonaparte (1993):

$$\sigma_s = (\sigma_r \text{ x } FC \text{x} FW \text{x} FI)/FS$$

where:

- $\sigma_s$ is the allowable tensile stress
- $\sigma_r$ is the rupture stress at service temperature (23$^o$C) and specified design life (100 yr)
- FC is factor of safety for chemical degradation (assumed 1.0)
- FW is factor of safety for seam strength (assumed 0.8)
- FI is factor of safety for installation damage (assumed 1.0)
- FS is the overall factor of safety (assumed 2 to 3)

This generated an allowable tensile stress of 2 to 3 MPa. The data generated by Duvall (1993) were then used to define a 50-year isochronous stress/strain curve which identified MAS values of 1% and 2% for stresses of 2 and 3 MPa respectively. Based on this they chose to use the heavier protection geotextile that would limit local indentation (arch) strains to between 1 and 2 %.

The decision by Smolkin and Chevrier (1997) to seek a higher allowable arch strain complements the thoughts of Saathof and Sehrbrock (1994) when discussing geotextile protection:

> *" It has to be reflected whether the protection effect required in the guidelines with a permissible deformation of the geomembrane of 0.25% is fixed by considering the material properties and whether an increase to 0.5 to 1.0% might be permissible"*

Note that Smolkin and Chevrier (1997) made no allowance for stress relaxation and made no consideration of the fact that in 1997 they were probably working with a more durable HDPE resin than did Duvall in 1993. Saathof and Sehrbrock also made no allowance for the actual performance properties of the geomembrane, but clearly

did recognize that it could have some beneficial effect. Also recollect that Duvall specifically mentions that his test results may not be applicable to geomembranes made with other HDPE resins. And in their paper Berg and Bonaparte (1993) also specifically state:

> *" This simple comparison highlights the importance of identifying the potential for a particular geomembrane to undergo brittle rupture when loaded, and the benefit to be gained by using a geomembrane resin not susceptible to this failure mode during the specified design life. Finally, it is noted that if the geomembrane was able to relax, a larger initial allowable stress would be acceptable."*

This last point is again driven home by Berg and Bonaparte (1993):

> *" The procedure described in this section of the paper is applicable only to stress-controlled boundary conditions; it is not directly applicable to situations where the geomembrane will substantially relax."*

Thus, these ameliorating factors are all recognized but not yet incorporated in design protocols and regulations.

In a typical vertical expansion the geomembrane will not be required to tolerate permanent tensile stress to maintain stability. Tensile stresses will only be generated by uniform and differential settlement of the subgrade waste. Such settlement strains, and the resulting stresses, will occur "extremely slowly" perhaps 5 to 15 years for the initial 60% of the total strain, and 50 to 80 years for the remaining 40%. Therefore stress relaxation will have a significant impact on the actual geomembrane stresses. Recollect that Soong et al (1994) found that significant stress relaxation would occur at strain rates less than 12.7 mm/min in 100 mm gage length specimens.

Typically a geomembrane will be placed on compacted clay or a GCL and will be overlain by a geotextile/geonet composite drain and sand, a cushion geotextile and sand, or simply a drainage sand layer. Sand will typically pass a 0.5 in. sieve and will be sub-angular. Thus there will be no large drainage rock indenting the geomembrane, a more optimum situation than that faced by Smolkin and Chevrier (1997). Thus, there is justification for a more tolerant position on allowable indentation strain.

Separation geomembrane may also be at a service temperature between 20 and 40$^{o}$C. Measurements reported by Carey et al. (1993) on separation liners while in service have shown temperatures of about 30$^{o}$C. However, there is a possibility that sometime in its life the geomembrane might experience temperatures as high as 60$^{o}$C. As the temperature increases, so the stress relaxation will increase. In addition, as the HDPE tries unsuccessfully to expand within the confining soils a compressive stress will be generated within its plane, further reducing the residual tensile stress resulting from any settlement. However, at the same time the SCR resistance of the HDPE

will decrease somewhat, thus there will be a balancing of effects. Even so, note that whatever the SCR of the geomembrane, if there is no stress there will be no break. Thus, the change in performance of the geomembrane under elevated temperatures is purely a function of the performance of the confining materials, the increased stress relaxation rate, and the reduction in SCR which is a function of the SCR of the basic material.

HDPE geomembranes have operated for many years without failure when exposed to tropical sunshine in applications such as leachate and evaporation pond liners. In such applications they regularly reach temperatures of over 80ºC. Fortunately there are no microstructural changes in ultimate break characteristics until the HDPE reaches over 90ºC, therefore laboratory tests at elevated temperatures are confidently used to accelerate in-service kinetic processes.

A simplistic assessment of mechanical changes as temperature increases from 20 to 60ºC indicates that the SCR might decrease by about two orders of magnitude. However, this change in temperature will occur slowly such that stress relaxation will occur at a faster rate than it would have occurred at 20$^{o}$C. At the same time, any tensile stress in the geomembrane will be reduced by about 6% of the yield stress (using an expansion coefficient of $1.7 \times 10^{-4}$/ºC for HDPE), as compressive stresses increase due to constrained thermal expansion. A balancing increase of two orders of magnitude in SCR occurs if the tensile stress in the geomembrane is decreased by about 20% of the yield stress, or is reduced to about 35% of the stress at 20ºC. Since Figure 8 shows that approximately 50% stress relaxation occurs in 50 minutes – it appears that practical increases in in-situ geomembrane temperatures will not significantly affect the durability of the primary liner.

In summary, for a confined separation liner, we know:

- HDPE must be treated differently to other materials due to its susceptibility to stress cracking.
- HDPE's susceptibility to stress cracking is one of the major reasons for the concern about a limiting strain.
- Available HDPE geomembranes have a wide range of SCRs that can be used to advantage.
- Stress relaxation must be taken into account.
- Confinement is beneficial.
- Biaxial stress states rather than uniaxial stress states must be considered.
- There is a general feeling among design engineers that 3% general strain and 0.25% local strain limitations are too conservative. The practical performance of lining systems in the USA, where heavy puncture protection geotextiles are not used, appears to support this position.
- An increase in geomembrane temperature has no adverse effect on its durability.

For the definition of a meaningful MAS it is necessary to know the SCR of the HDPE geomembrane proposed for use when measured according to ASTM D5397 (single point). Materials with an SCR above 1500 hr will be treated differently to those below 1500 hr. This is admittedly an arbitrary specification but is felt to be quite practical in relation to available HDPE geomembranes. A minimum SCR of 400 hr is recommended. While this exceeds the GRI.GM13 specification, it has already been proposed by some as being an appropriate upgrade, and it was used by one HDPE geomembrane manufacturer in the 2001 Geotechnical Fabrics Report Specifier's Guide.

Clearly, the most appropriate experimental data for multiaxial creep testing of HDPE geomembranes has been generated by Duvall (1993). Both Smolkin and Chevrier (1997) and Berg and Bonaparte (1993) used Duvall's data to generate a maximum allowable long term stress at 23$^{o}$C for design lives of 50 and 100 years, respectively. Both identified a rupture stress of about 7 MPa. From this Smolkin and Chevrier (1997) identified a maximum allowable liner strain and therefore were able to define the required geotextile to use for puncture protection. However, as previously stated, we should not be seeking a safe operating stress and deducing an allowable strain. Since deformation is a consequence of settlement we need to identify the MAS that will not result in damaging stresses – the strain comes first, the stress second.

Nevertheless, Smolkin and Chevrier (1997)identified a maximum allowable indentation arch strain of 1 to 2% associated with a long term rupture strength after appropriate factors of safety had been applied. Following the BAM model that defined a bending strain of 3% on the underside of an arch strain of 0.25% in a 2.5 mm thick geomembrane, the bending strain developed by a top surface arch strain of 1.5% on the underside of a 2mm thick geomembrane is approximately 10%. This is approximately 40% of the biaxial stress at which local yield failures occur in HDPE.

In recent correspondence Müller states:

> *" ..... A strain limit of about 3% is an extremely conservative estimate from pipe pressure data for environmental conditions (base lining) with temperatures up to 40$^{o}$C. At room temperature (cap lining) a limit of up to 6% seems to be acceptable."*

Hence, the German BAM Institute now allows a maximum general strain of 6% in cap HDPE geomembranes.

The allowable general strain figure of 3% was generated from pipe tests in which the stress was maintained constant (stress relaxation does not occur) and Janson's (1981) recommendation of a maximum 5% strain is in a service situation where applied stresses (internal gas pressure) are active and constant. And in none of these situations is the material intimately confined between two masses holding the material "together" – and controlling its strain history. For instance, the confining soil will prevent the local ballooning of a thin spot that would otherwise occur if the

geomembrane were pressurized on one side and unsupported on the other side. Under equilibrium settlement conditions the confining soils will not allow a geomembrane to further deform to allow a crack to open up. Therefore, if an active strain of 3% is felt sufficient to induce a potential critical stress in an HDPE geomembrane, but loading is very slow so that stress relaxation (by a factor of 2) can occur, the critical stress will actually be achieved at a strain of 6% or more, by the time stress relaxation has accumulated.

Therefore, on consideration of the various test data, the opinions expressed by those involved with the regulations and specifications, the contributions of confining pressure, the occurrence of stress relaxation, and the nature of the soils on each side of the liner, it is our calculated opinion that maximum allowable biaxial strains be conservatively set at 6% and 8% for HDPE geomembranes with 400 hr<SCR<1500 hr and SCR>1500 hr, respectively.

## *FRICTION ENHANCED GEOMEMBRANES*

The influence of surface profiles on the MAS requires some interpolation from basic HDPE data since no meaningful creep, stress relaxation, or stress cracking tests have been performed on textured geomembranes. The significant factor is the influence that the profiling has on the SCR of the basic sheet. Therefore one cannot test a notched specimen that transfers the measurement point into the center of the geomembrane – simply another test on the equivalent smooth geomembrane. Tests should be performed on unnotched specimens (Thomas 1993), which in Germany are required to survive beyond 700 hr.

No special consideration will probably be necessary for the structured profiles generated by calendering since these profiles are built on top of a uniform thickness of geomembrane and the structures undergo the same thermal history as the bulk of the geomembrane. In the early days of such products there were significant stress concentrations at the base of conical profiles, but profiles are now smaller and profiling techniques have been improved.

While the post-extrusion thermally-bonded particulate textures are also added to the surface of the basic geomembrane and have little effect on the uniaxial tensile properties of the geomembrane, the point welding process has been seen to initiate microcracks in the weld around the base of the particle. Hence, the compromise that has to be made with bond strength: an increase in bond strength causes a decrease in SCR. For this reason it is necessary to be more conservative in the MAS values for the randomly textured products.

The textures generated by the nitrogen-injection round-die process in three layer co-extruded geomembranes reduce the uniaxial tensile break strength and elongation of the geomembrane but may have very little effect on the SCR of the geomembrane since the texture is subjected to essentially the same thermal history as the core of the

geomembrane. The outer layers are often made using lower density resins (even LLDPE is possible) than the core, resulting in lower comparative SCR values.

The multi-axial stress/strain curve of one nitrogen-injection randomly textured geomembrane is shown in Figure 10. It shows a somewhat lower break strain than for the two smooth materials – approximately 22% compared to 29% and 48%. However, this may also be a function of the higher density of the textured geomembrane – note that break strain increases as density decreases, as would be expected. On the other hand, the strain at ultimate (maximum) strength of both smooth materials is about 26%, still somewhat, but not much, higher than that for the textured product.

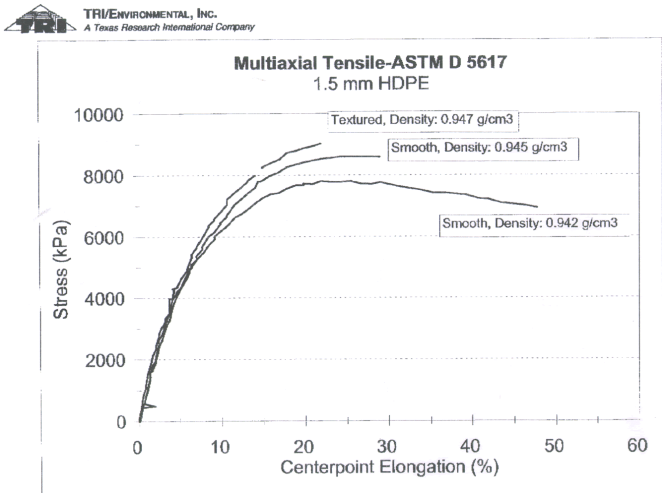


Figure 10. Multiaxial tensile stress/strain curves for textured and smooth 1.5 mm HDPE geomembranes (TRI 2002).

From these curves the relative positions of the yield points cannot be determined, but past experience has shown that yield points are relatively unaffected by surface profiles. This is confirmed by the GRI.GM13 specification for smooth and textured HDPE in which there is no difference in yield parameters but where the break parameters for textured materials are considerably lower than those for smooth materials. Thus ultimate and break strengths are the more appropriate indicators of the relative long term performance of the material. Thus, there is some indication that the textured geomembrane should have a more conservative MAS.

As a consequence of the changes and potential changes in mechanical performance characteristics of structured and textured materials it is proposed that MAS for calendered structures be set at 6% and at 4% for the randomly textured products, both regardless of geomembrane/resin SCR values. As an added safety factor it is assumed that all profiled HDPE geomembranes are made from the lower SCR commodity type of resin. These are again considered to be conservative MAS values.

## *LLDPE AND PP GEOMEMBRANES*

As previously described it is possible and desirable to treat LLDPE and PP quite differently to HDPE because of their microstructures and resulting different mechanical performance characteristics. A recognition of these differences should be incorporated in designs. Since these materials are in the same polyolefin polymer family as HDPE, and since they are less crystalline than HDPE, they will have higher stress relaxation rates than HDPE. It is also well-established that they are not susceptible to stress cracking in the as-manufactured condition – the GRI.GM17 specification for LLDPE does not include SCR, nor did the GRI.GM18 specification for PP prior to its provisional withdrawal.

Multiaxial stress/strain curves for PP and LLDPEs of different densities are shown in Figure 11. They all show "break" strains above 50%, all much higher than HDPE. They show ultimate strength strains of about 23%, not much different to those shown for HDPE. However, these materials do not have yielding-type failures as does HDPE – they uniformly deform up to final break. Therefore, the break strain is a valid measure of their relative long-term performance.

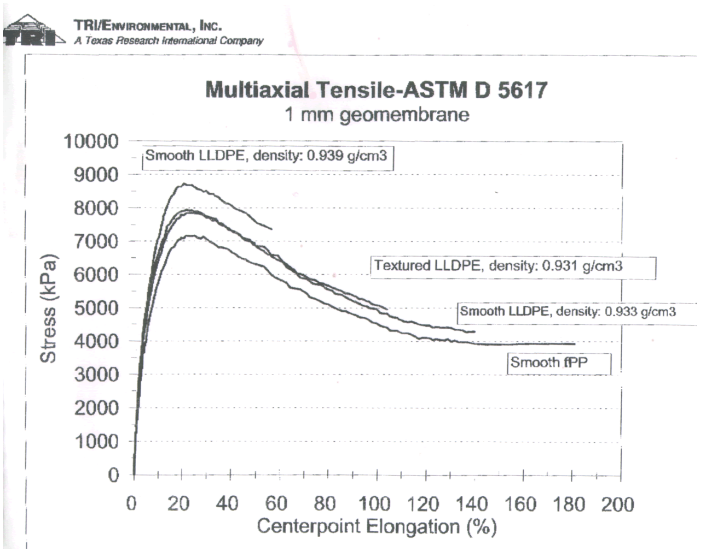


Figure 11. Multiaxial tensile stress/strain curves for 1 mm LLDPE and PP geomembranes (TRI 2002).

Since all available information leads us to recommend 6 and 8% allowable strain for HDPE with a yield/ultimate strain in the multiaxial stress/strain curve at about 22%, it is reasonable to recommend an allowable strain of about 12% (a factor of only 1.5 to 2 higher) in these similar materials that do not yield, that have break strains higher than the ultimate strength strains in HDPE by a factor of 2 to 8 (50 – 180% strain), that are not susceptible to stress cracking, and that have a higher stress relaxation rate.

Note that the curves for smooth LLDPE, as for HDPE, show a significant dependence of break strain on density – as density decreases from 0.939 g/cm$^3$ to 0.933 g/cm$^3$ the multiaxial break strain increases from about 50% to 140%. Hence the lower density products, 0.935 g/cm$^3$ and below, could have a higher MAS than the higher density products. We propose 12% compared to 10% for the higher density products. Such a difference between LLDPE products was apparent when GRI was developing the GRI. GM17 specification – there were considerable discussions on whether there should have been different specifications for two classes of LLDPE product – the higher modulus lower ductility type and the lower modulus higher ductility type.

It is noted that the textured LLDPE shown in Figure 11 has a lower break strain than a smooth product of slightly higher density – the opposite of the expected density effect. Thus, the textured surface appears to cause a reduction in break strain. This may simply be a result of the reduced cross-sectional area at deep valleys on the surface. Since LLDPE is not susceptible to stress cracking there is not the need for such a significant reduction in MAS as for HDPE, therefore it is proposed that structured profiles be limited to 10%, independent of density, and randomly textured profiles be limited to 8% also independent of density.

Since PP has a larger elongation at break (~180%) than does LLDPE (< 140%) its maximum allowable strain will be set at 15%. There is very little variation in the conventional short term mechanical performance of PP geomembranes since all in North America are made from one resin made by one manufacturer.

## *CONCLUSIONS*

The maximum allowable multiaxial strains proposed for candidate geomembranes for separation liners in vertical landfill expansions are:

- Smooth HDPE (SCR <1500 hr) 6%
- Smooth HDPE (SCR >1500 hr) 8%
- Structured HDPE 6%
- Textured HDPE 4%
- LLDPE (Density >0.935 g/cm$^3$) 10%
  (Density <0.935 g/cm$^3$) 12%
- Structured LLDPE 10%
- Textured LLDPE 8%
- PP 15%

These are proposed as conservative MAS values.

## *ACKNOWLEDGEMENTS*

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Allen for their technical support and permission to reference personal communications, and to Felix Jaecklin for technical translation support.

## *CAVEAT*

It is essential to understand that these MAS values are only applicable to geomembrane strained slowly between confining layers. They do not apply to free geomembrane with a constant active load applied.

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# Appendix F

## Operational Period Process Water Balance Figures

Figure 1 BCHLF Pad and Heap Footprint Areas
Figure 2 BCHLF Barren Solution Flows
Figure 3 BCHLF Ore Thicknesses for Under Leach Areas
Figure 4 BCHLF Makeup Flows from TSF
Figure 5 BCHLF Cumulative Makeup Requirements
Figure 6 BCHLF In-Heap Pond Pumping to CIC (Pregnant Solution)
Figure 7 BCHLF In-Heap Pond Volumes (Probabilistic Approach)
Figure 8 BCHLF Average In-Heap Pond + 100-yr/24-hr Storm Event + 24-hr Draindown Volumes (Deterministic Approach)
Figure 9 BCHLF CIC Pumping to Neutralization Treatment and Discharge to TSF
Figure 10 BCHLF Cumulative Neutralization Treatment Requirements and Discharge to TSF
Figure 11 BCHLF In-Heap Pond Operational Period Trigger Elevations for Pumping to Neutralization

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**Figure 1**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF Pad and Heap Footprint Areas**

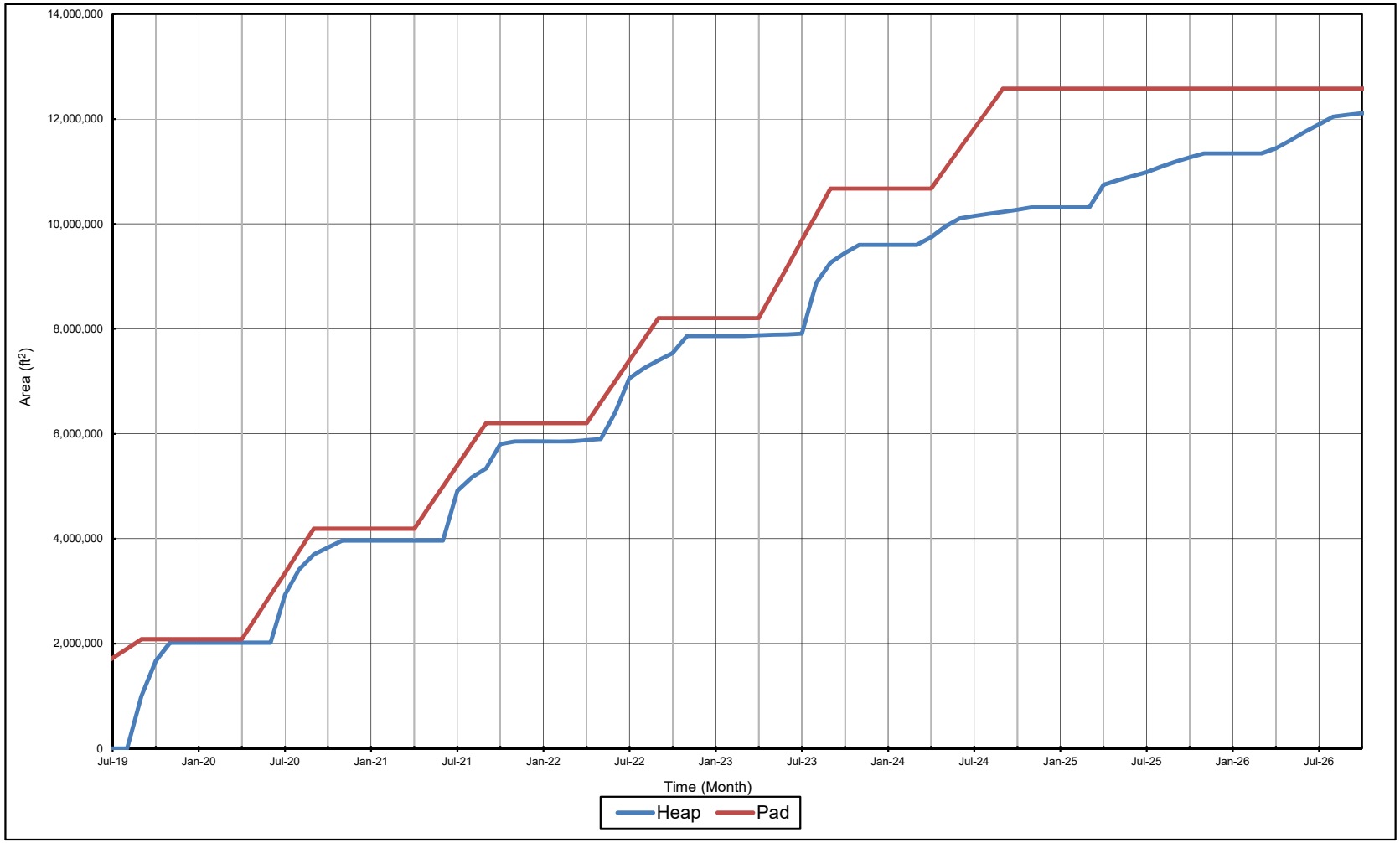

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**Figure 2**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF Barren Solution Flows**

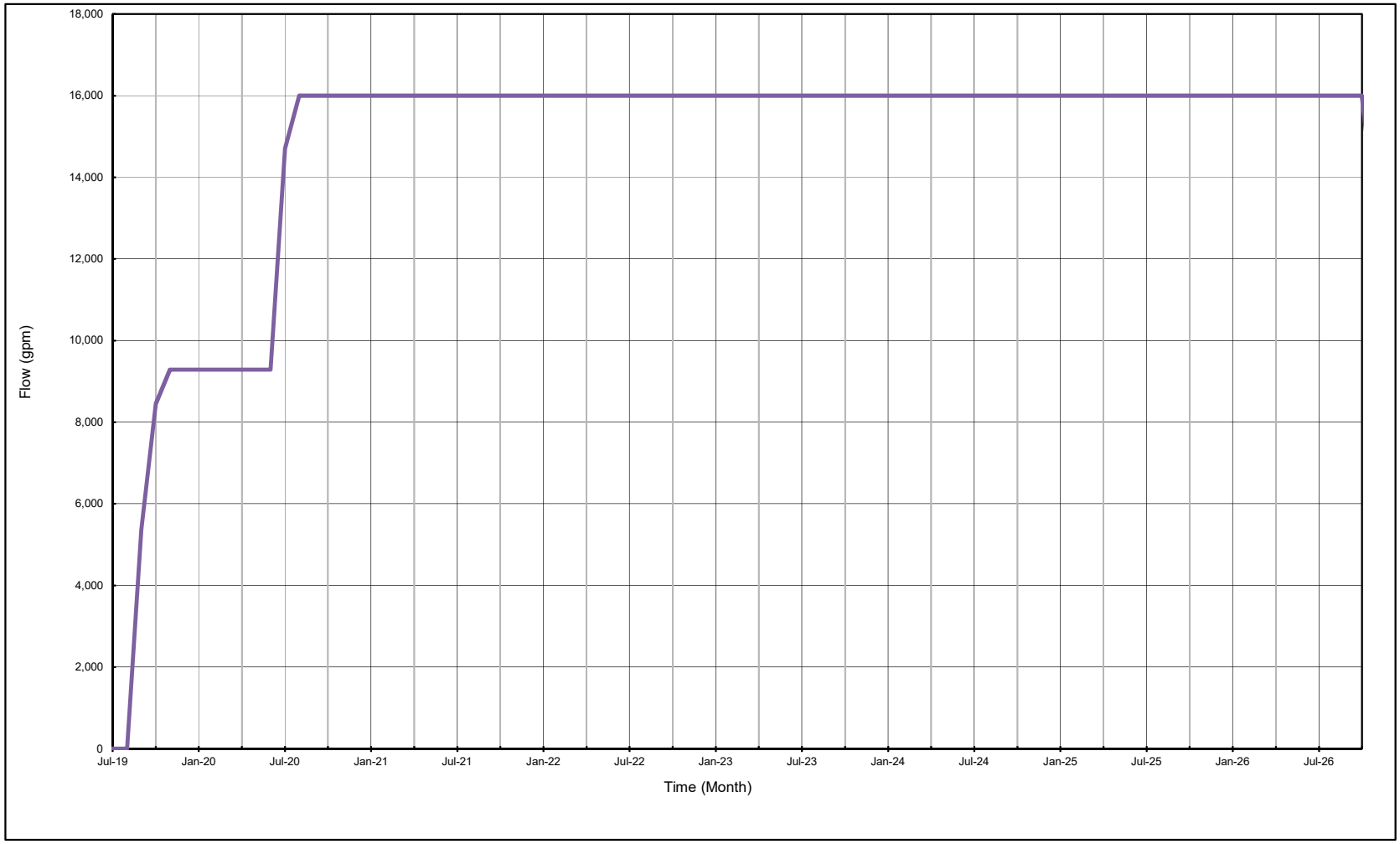

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**Figure 3**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF Ore Thicknesses for Under Leach Areas**

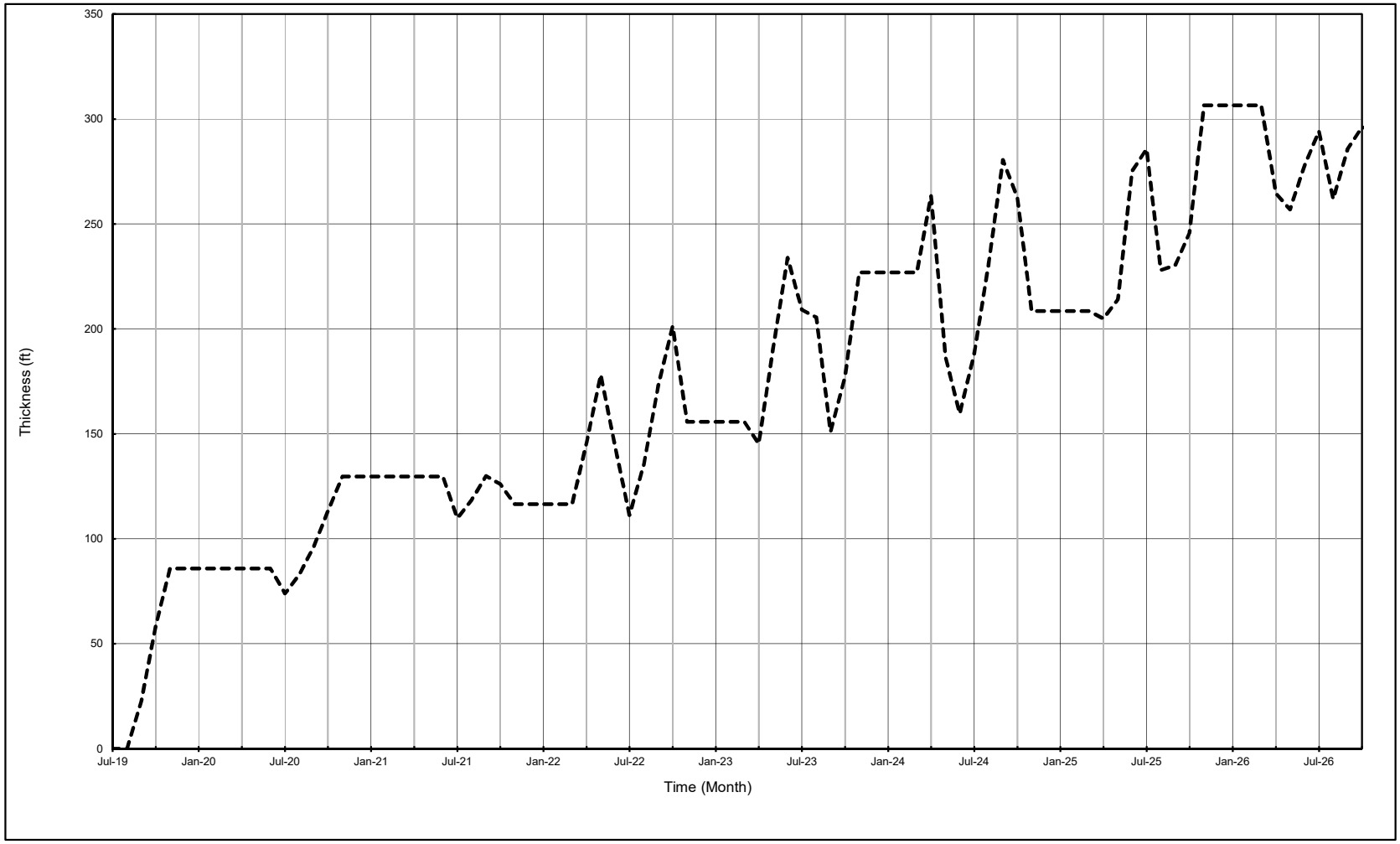

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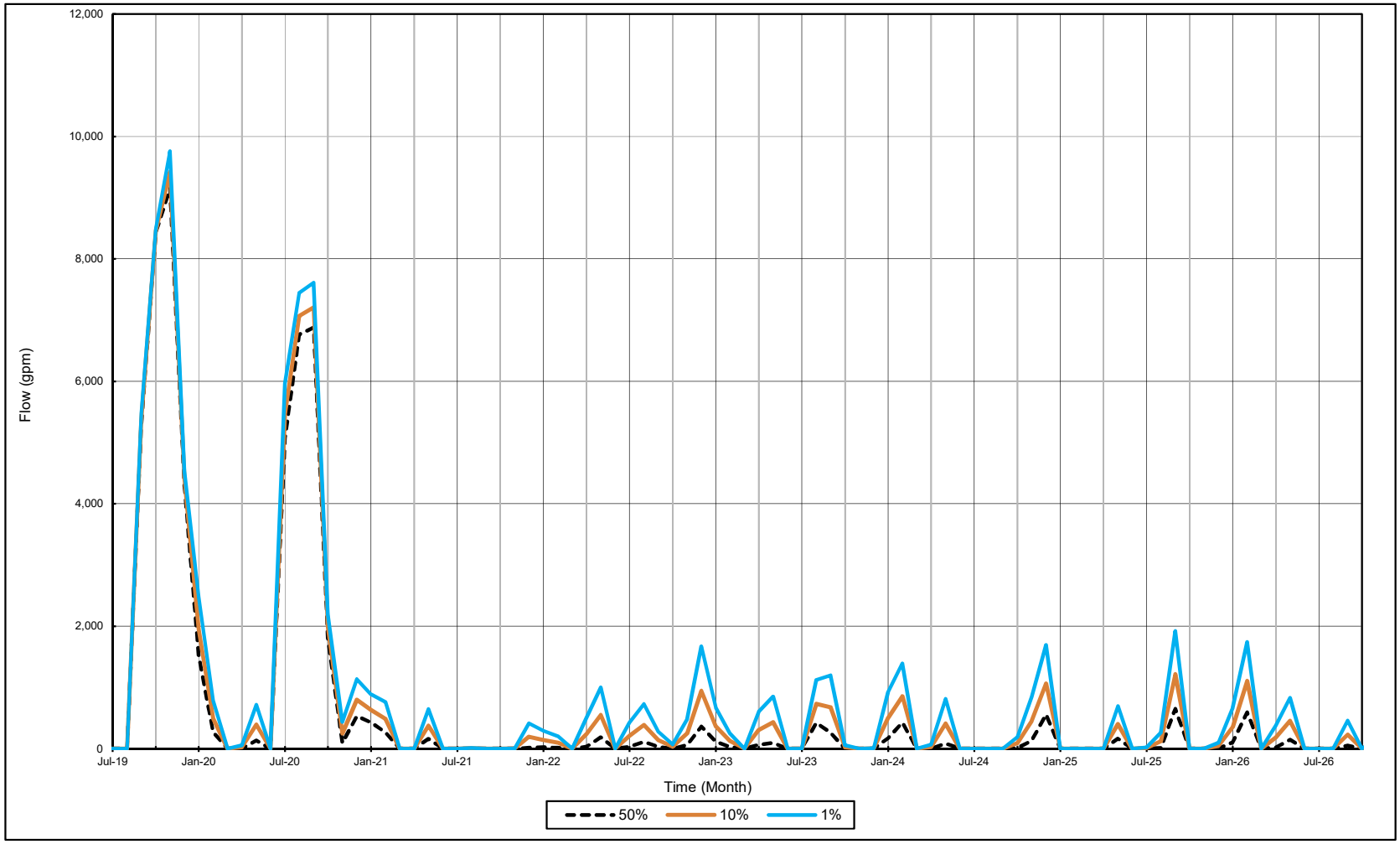
**Figure 4**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF Makeup Flows from TSF**




Notes:

1. The probabilistic values of the parameters are those that would occur for a given percent chance of exceedance. For example, if a 1.0 percent chance was estimated to be 300 in a given month, there would be a 1.0 percent chance of equaling or exceeding that value.

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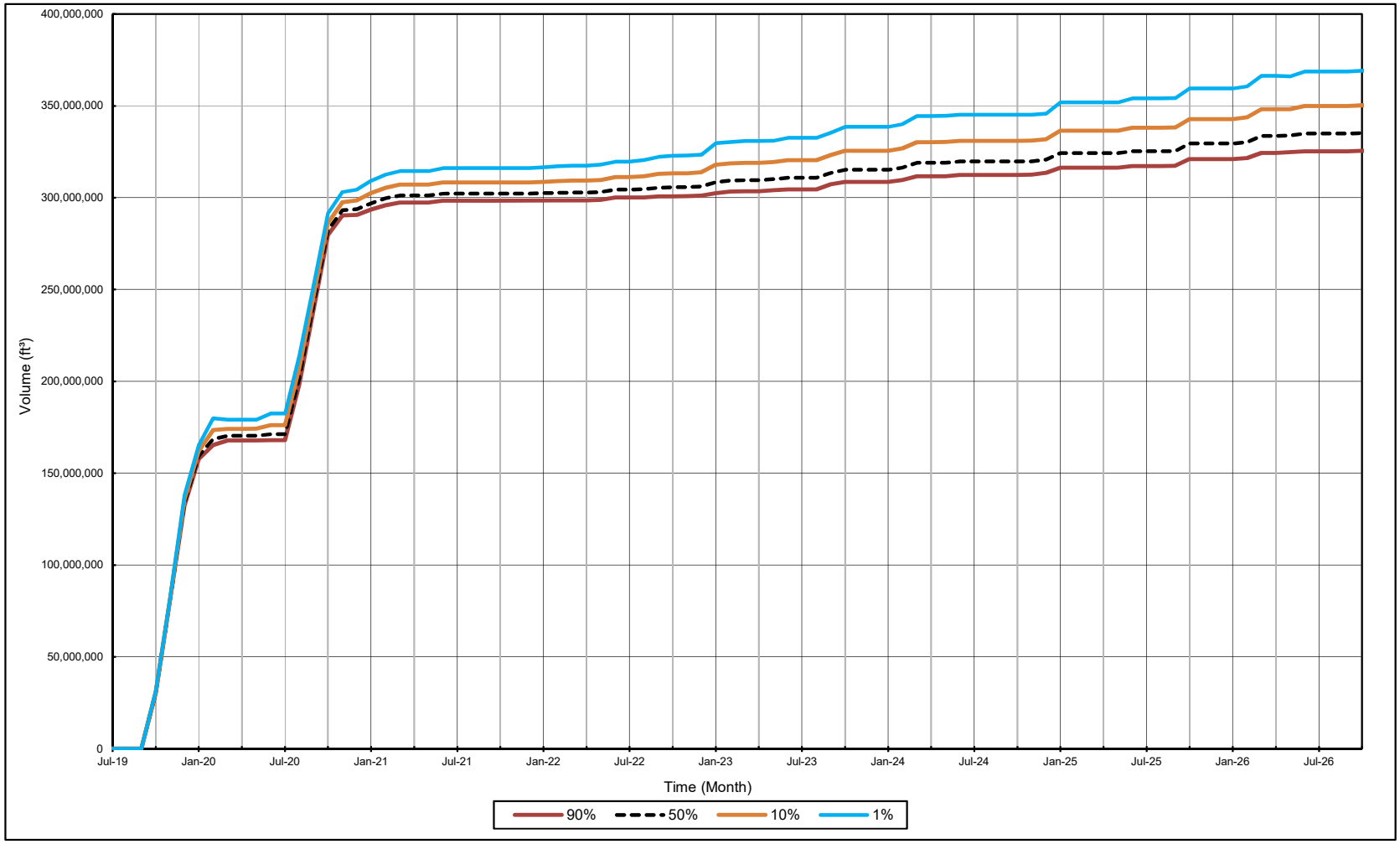
**Figure 5**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF Cumulative Makeup Requirements**




Notes:

1. The probabilistic values of the parameters are those that would occur for a given percent chance of exceedance. For example, if a 1.0 percent chance was estimated to be 300 in a given month, there would be a 1.0 percent chance of equaling or exceeding that value.

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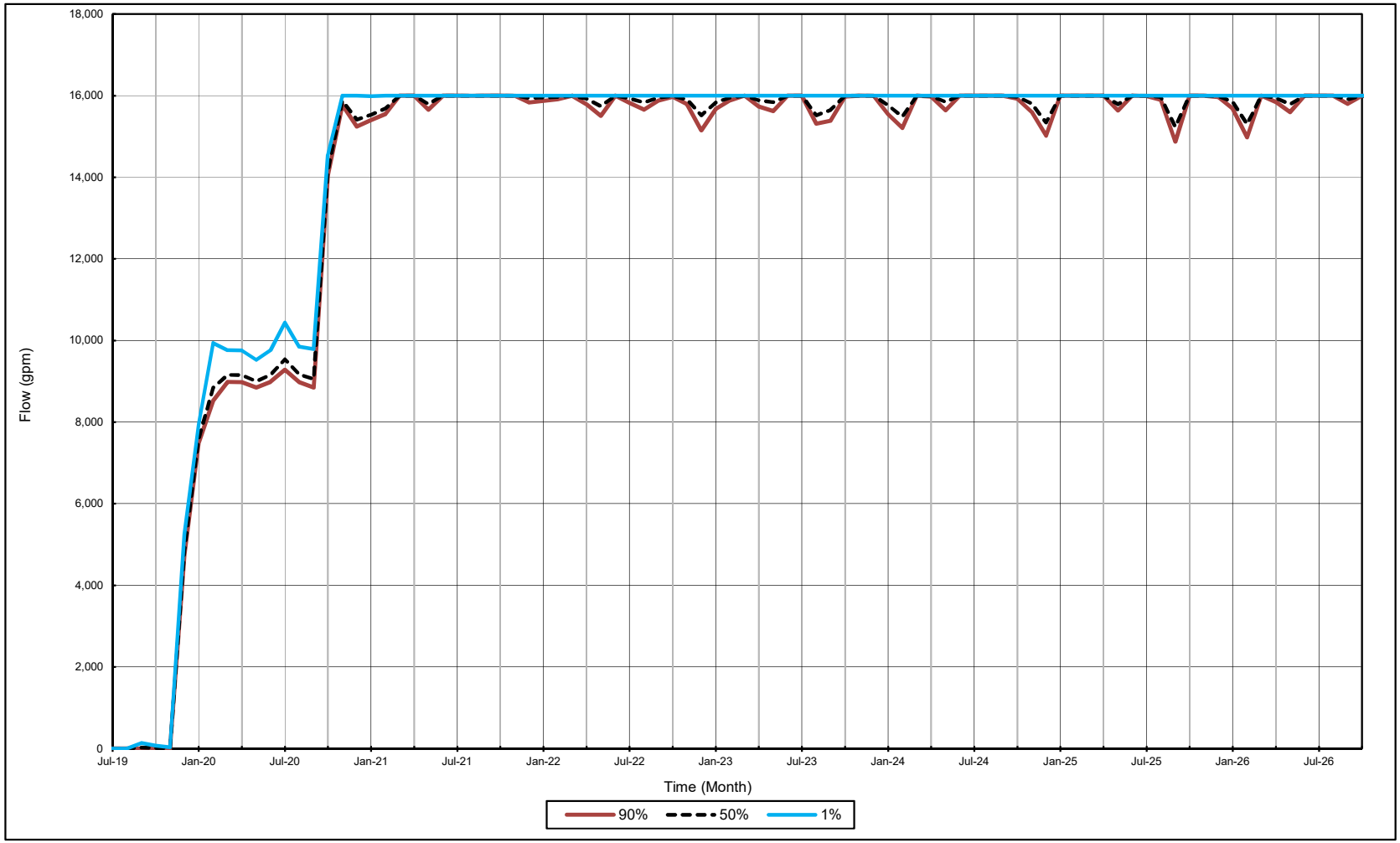
**Figure 6**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF In-Heap Pond Pumping to CIC (Pregnant Solution)**




Notes:

1. The probabilistic values of the parameters are those that would occur for a given percent chance of exceedance. For example, if a 1.0 percent chance was estimated to be 300 in a given month, there would be a 1.0 percent chance of equaling or exceeding that value.

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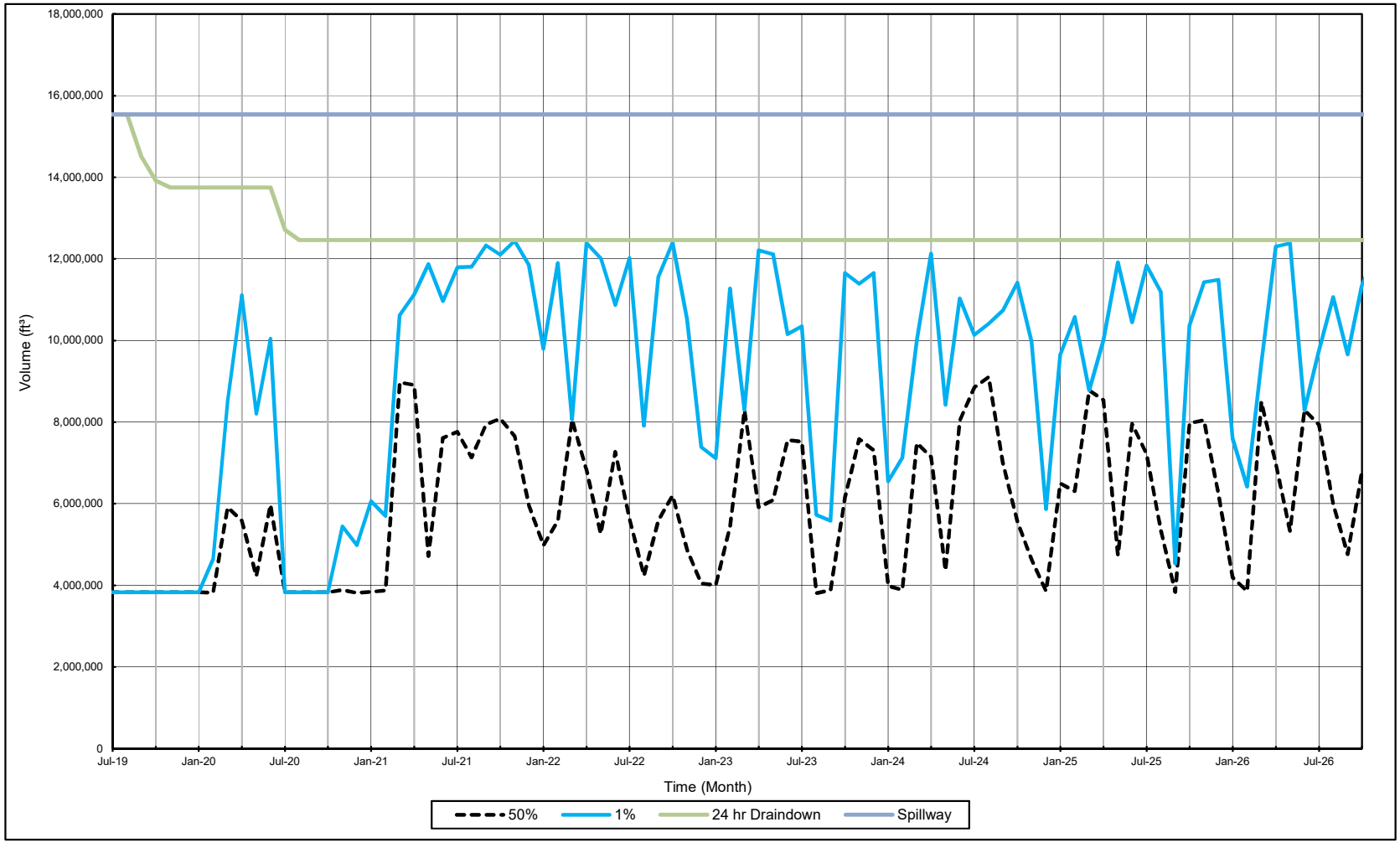
**Figure 7**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF In-Heap Pond Volumes**




Notes:

1. The probabilistic values of the parameters are those that would occur for a given percent chance of exceedance. For example, if a 1.0 percent chance was estimated to be 300 in a given month, there would be a 1.0 percent chance of equaling or exceeding that value.

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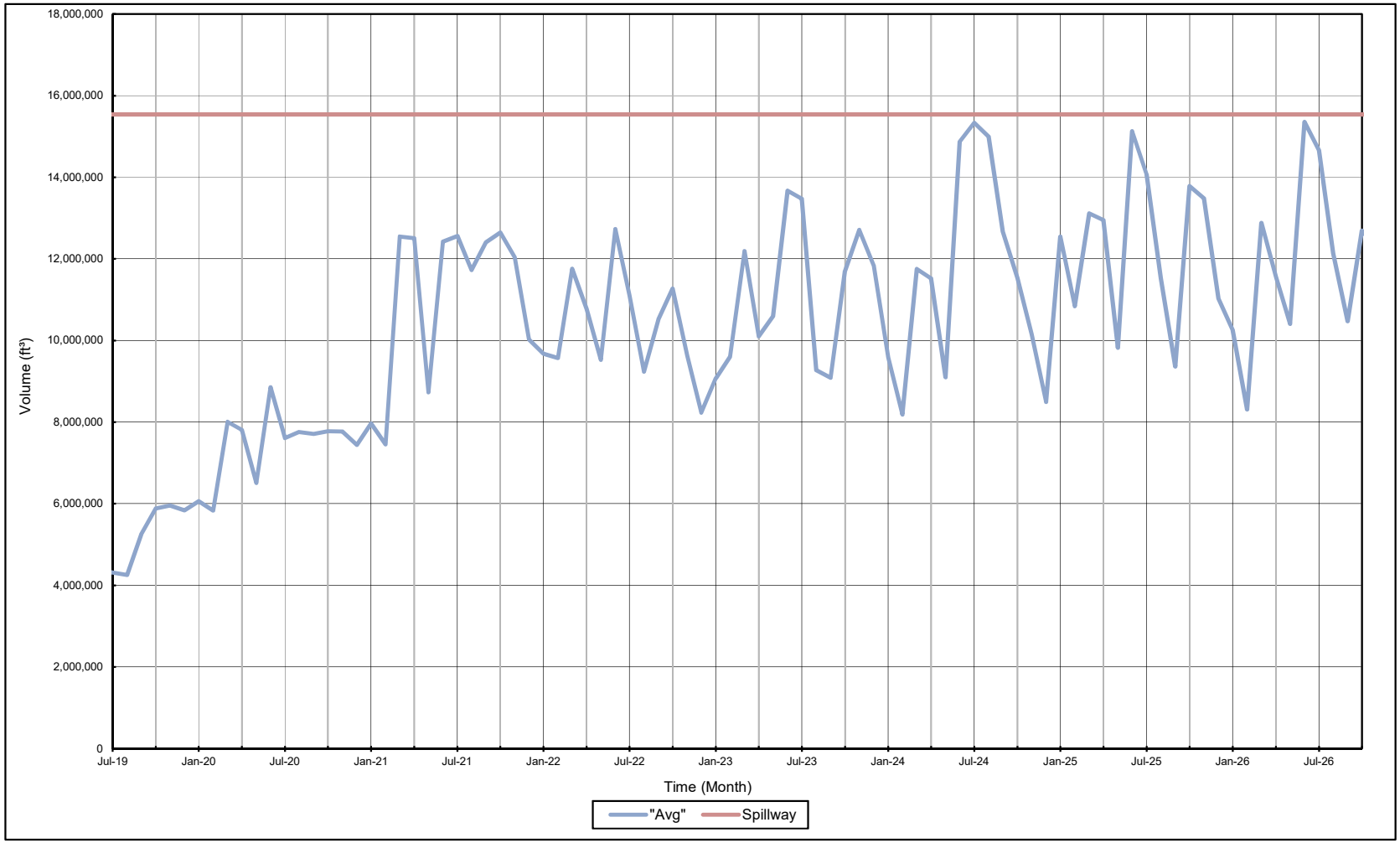
**Figure 8**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF Average In-Heap Pond + 100-yr/24-hr Storm Event + 24-hr Draindown Volumes**




Notes:

1. The deterministic values of the parameters are those that would occur based on the historical climatic/hydrologic records (maximum, minimum, and average).

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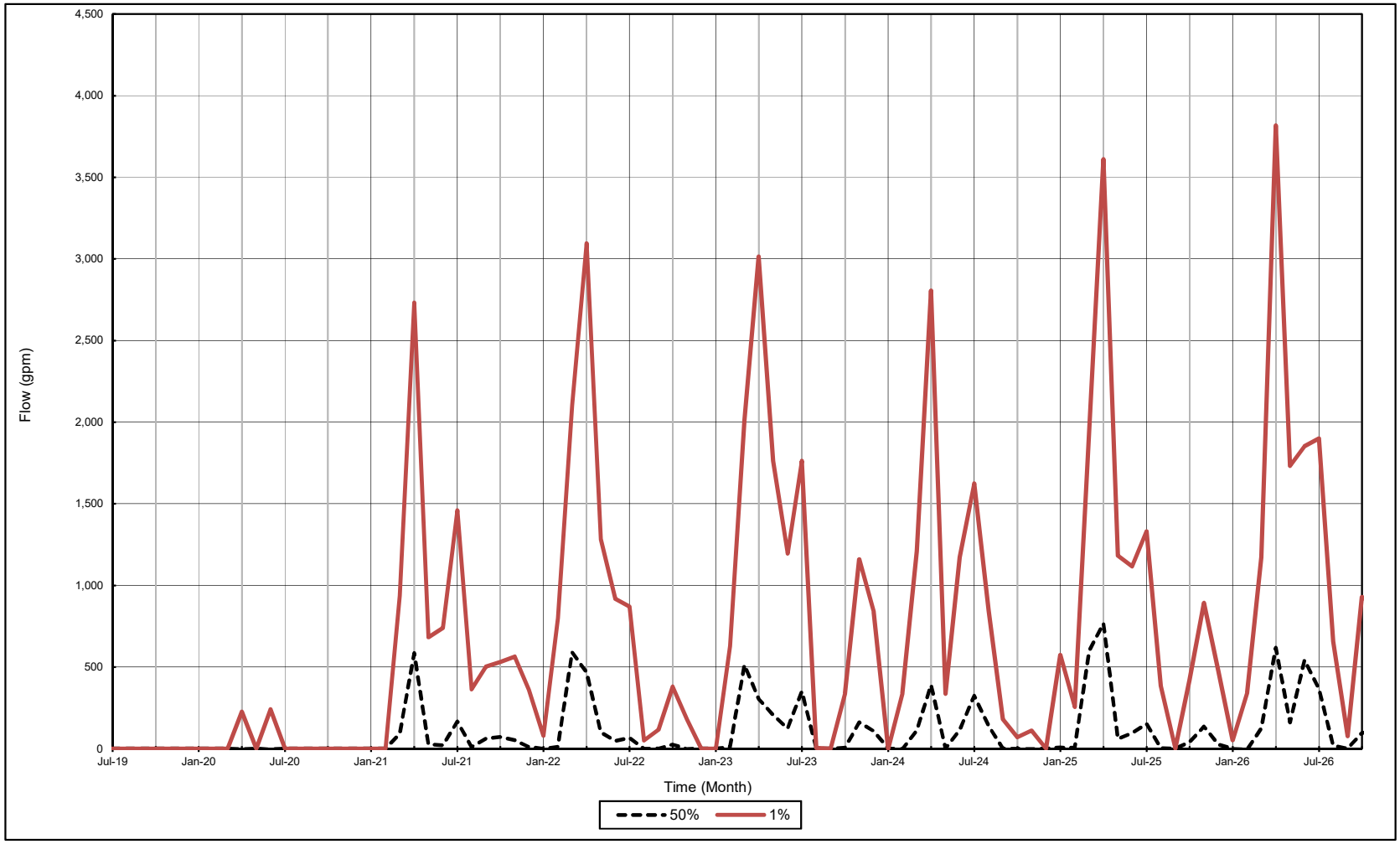
**Figure 9**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF CIC Pumping to Neutralization Treatment and Discharge to TSF**




Notes:

1. The probabilistic values of the parameters are those that would occur for a given percent chance of exceedance. For example, if a 1.0 percent chance was estimated to be 300 in a given month, there would be a 1.0 percent chance of equaling or exceeding that value.

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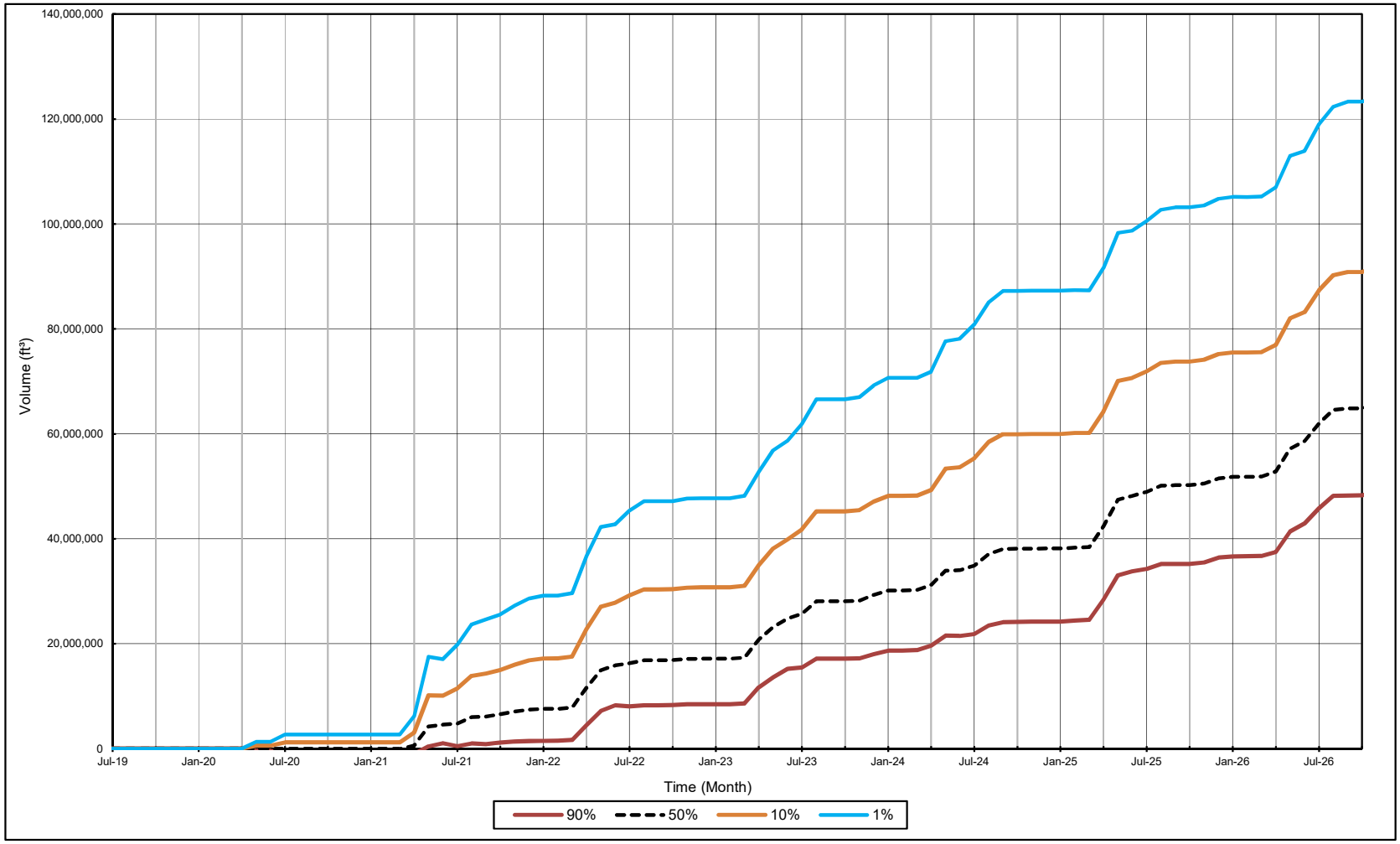
**Figure 10**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF Cumulative Water Neutralization Treament Requirements and Discharge to TSF**




Notes:

1. The probabilistic values of the parameters are those that would occur for a given percent chance of exceedance. For example, if a 1.0 percent chance was estimated to be 300 in a given month, there would be a 1.0 percent chance of equaling or exceeding that value.

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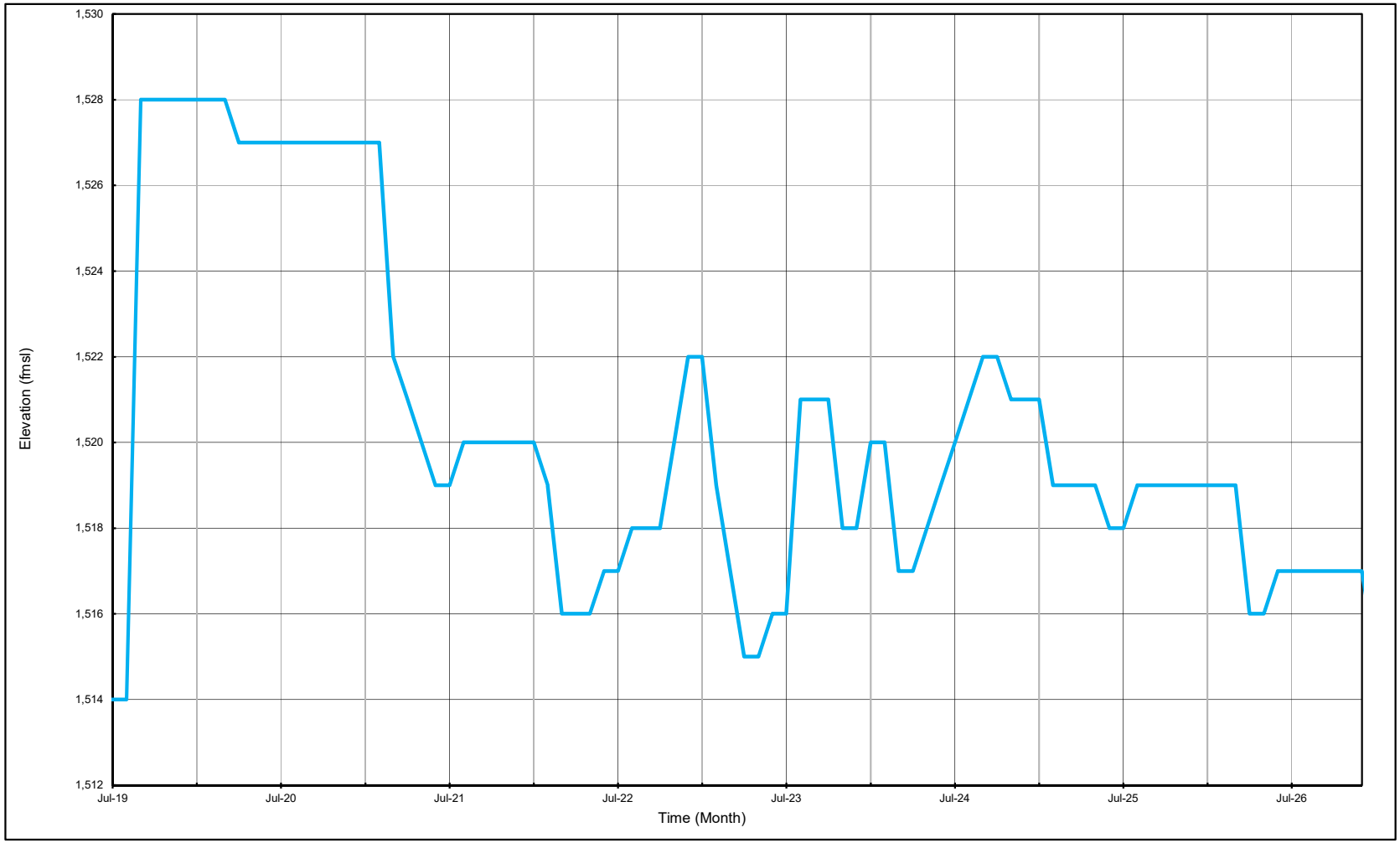
**Figure 11**
**Fairbanks Gold Mining, Inc.**
**Fort Knox Project - BCHLF Final Design**
**Operational Period Process Water Balance**

**BCHLF In-Heap Pond Operational Period Trigger Elevations for Pumping to Neutralization**