Appendix 11-B

Numerical Groundwater Modelling

HARPER CREEK PROJECT

Application for an Environmental Assessment Certificate / Environmental Impact Statement

HARPER CREEK MINING CORP.
HARPER CREEK PROJECT

NUMERICAL GROUNDWATER MODELLING

PREPARED FOR:

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VA101-458/14-2
Rev 1
October 23, 2014

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ISO 9001 - FS 64925
ISO 14001 - EMS 550121
OHSAS 18001 - OHS 550122

HARPER CREEK MINING CORP.
HARPER CREEK PROJECT

NUMERICAL GROUNDWATER MODELLING
VA101-458/14-2

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EXECUTIVE SUMMARY

Knight Piésold Ltd. (KP) was retained by Harper Creek Mining Corporation (HCMC) to provide a representation of baseline groundwater conditions and to evaluate potential effects of the Harper Creek Project (the Project) on hydrogeological conditions. A three-dimensional steady-state, regional-scale numerical groundwater model was developed to achieve this objective using MODFLOW-SURFACT to simulate baseline hydrogeological conditions at the Project site. The baseline model was then modified to include proposed mine facilities in order to assess hydrogeological conditions during mine operations and the post-closure period. The results of baseline and predictive numerical groundwater models were used to inform environmental effects assessment as part of the EIS submission.

Baseline Model and Calibration

The baseline model was calibrated to measured groundwater elevations from 21 on-site groundwater monitoring wells and to synthetic baseflow estimates for five hydrometric stations within the study area. Baseflow estimates were obtained from the results of a baseline watershed model developed for the Project. The baseline model was calibrated by iteratively adjusting hydraulic conductivity and groundwater recharge values until a suitable match between observed and simulated conditions was achieved. Recharge applied to the calibrated baseline model varied according to ground surface elevation with an area weighted average recharge rate of 114 mm/year. The normalized root mean squared error (NRSME) for hydraulic head targets in the calibrated baseline model was approximately 1%.

The simulated baseline water table generally mimics the surface topography with groundwater elevations ranging from 2,000 masl in the high elevation region to the south of the mine site to 650 masl and 500 masl at the downstream extents of Harper Creek and Baker/Jones Creeks, respectively. Groundwater recharge occurs along topographic highs within the active model domain and flows to groundwater discharge zones located within the valleys of Harper (including P-Creek and T-Creek), Baker and Jones Creeks and the Barriere and North Thompson Rivers.

Predictive Models and Potential Effects of the Project during Operations and Post-Closure

Two numerical models were developed using the calibrated model as a basis. The models were used to assess potential effects of proposed mine development on baseline hydrogeological conditions. The models were developed for the Operations and Post-Closure project phases.

The objectives of the predictive modelling were to:

1. Characterize hydrogeological conditions during the Operations and Post-Closure project phases.
2. Quantify the potential effects of the Project on baseflow in the study area.
3. Estimate groundwater inflow to the Open Pit at the end of Operations I (Year 24).
4. Estimate the groundwater capture zone and extent of drawdown surrounding the Open Pit.
5. Estimate the seepage rate from the Pit Lake during Post-Closure.
6. Characterize potential groundwater flow pathways for seepage originating from major mine facilities, including estimates of groundwater travel times and seepage rates to downstream discharge locations.

Major mine facilities were simulated in the predictive models, and included the Tailings Management Facility (TMF), Open Pit, Closure Pit Lake, Non-PAG Waste Rock Stockpile and Low-Grade Ore (LGO) Stockpiles. The results of the modelling are summarized below.

Open Pit and Pit Lake Simulation Results

Model results indicate that groundwater elevations surrounding the Open Pit are expected to decrease by up to 350 m while the pit is actively dewatered. Groundwater inflow rates are expected to reach a maximum of approximately 16 L/s during Year 24 when the extent of the Open Pit is largest. The capture zone of the Open Pit will extend into the P-Creek and Baker Creek watersheds.

Upon closure, the Open Pit will be flooded to maintain a Pit Lake. Groundwater elevations directly surrounding the Pit Lake are expected to recover to the elevation of the Pit Lake water surface (1,530 masl). Seepage from the Pit Lake is expected to occur at a rate of approximately 8 L/s and groundwater inflow at about 4 L/s. Model results indicate that seepage from the Pit Lake is expected to feed into the upper Baker Creek groundwater system. MODPATH particle tracking indicates that seepage from the Pit Lake will discharge within the upper Baker Creek valley, approximately 3 km upslope of the confluence of Baker Creek with the North Thompson River.

Seepage Pathway Delineation and Travel Time Estimates

MODPATH particle tracking and endpoint analysis were conducted to characterize potential seepage pathways from proposed key mine infrastructure. The analyses were used to delineate pathways and estimate seepage travel times from the TMF, Non-PAG Waste Rock Stockpile, LGO Stockpiles and Pit Lake. MODPATH analysis was completed using both the Operations and Post-Closure models. Approximate groundwater travel time along the seepage pathways only considered advective travel and disregarded the effects of dispersion and diffusion. Results of the particle trace analysis indicate that:

- The majority of seepage from the Non-PAG Waste Rock Stockpile is predicted to discharge to the water management pond with only a small portion (approximately 1-2% of particle traces) reporting to P-Creek watershed.
- Seepage from the Pit Lake is expected to discharge to Baker Creek watershed with zero discharge to P-Creek watershed or to the existing domestic water well in Baker Creek watershed.
- Seepage from the TMF is expected to occur at a rate of approximately 13 L/s. A small amount of seepage is expected to bypass the collection ponds at the Main and North TMF embankments.

Domestic Water Well MODPATH Particle Tracking:

MODPATH reverse particle tracking analysis was conducted to provide a conceptual understanding of the flow of groundwater to four domestic groundwater wells in the Baker and Jones Creek catchments. This analysis indicates that Wells 00084 and 39609 source groundwater from valley areas within 1 km of the North Thompson River (approx. 5 km downslope of the Pit Lake). Wells 97740 and 97736 receive groundwater from topographic highs to the west and east of the Pit Lake, respectively. Analysis of advective groundwater travel times indicates that groundwater takes approximately 40 years to travel from its source to reach Wells 97736 and 97740 and about 90 days to reach Wells 39609 and 00084.

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ABBREVIATIONS

BC .... British Columbia
EIS .... Environmental Impact Statement
HCMC .... Harper Creek Mining Corporation
the Project .... Harper Creek Project
GIS .... geographic information systems
HSU .... hydrostratigraphic unit
KP .... Knight Piésold Ltd.
MAE .... mean absolute error
masl .... meters above sea level
mbgs .... meters below ground surface
m/s .... meters per second
Non-PAG .... non-potentially acid generating
NRSME .... normalized root mean square error
PAG .... potentially acid generating
RIV .... river boundary conditions
RMSE .... root mean square error
RPD .... relative percent difference
TMF .... Tailings Management Facility
TPD .... tonnes per day

1 – INTRODUCTION

1.1 PROJECT DESCRIPTION

Harper Creek Mining Corporation (HCMC) proposes to construct and operate the Harper Creek Project, an open pit copper mine near Vavenby, British Columbia (BC). The Project has an estimated 28-year mine life based on a process plant throughput of 70,000 tonnes per day (25 million tonnes per year). Ore will be processed on site through a conventional crushing, grinding and flotation process to produce a copper concentrate, with gold and silver by-products, which will be trucked from the Project site along approximately 24 km of existing access roads to a rail load-out facility located at Vavenby. The concentrate will be transported via the existing Canadian National Railway network to the existing Vancouver Wharves storage, handling and loading facilities located at the Port of Vancouver, for shipment to overseas smelters.

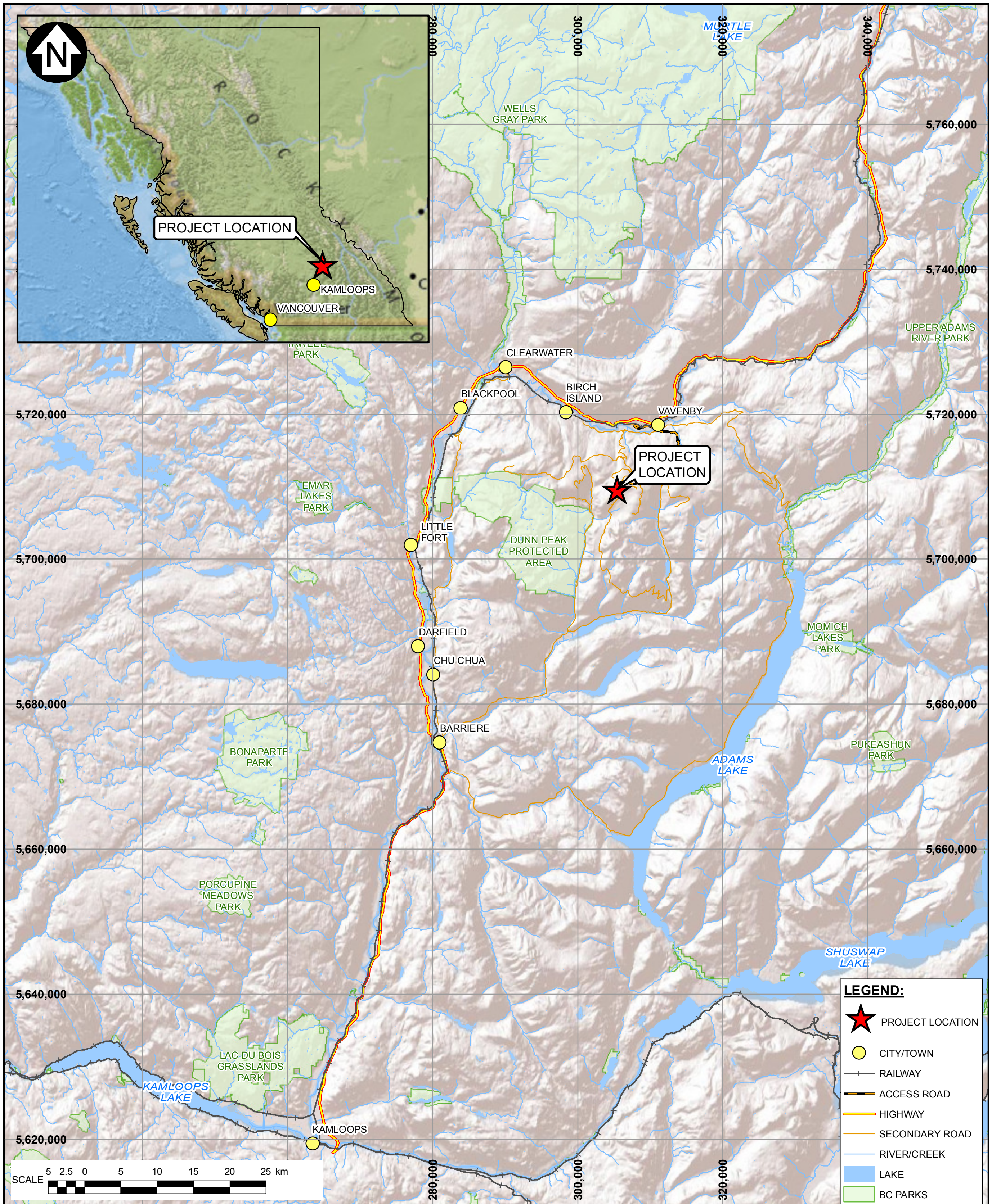
The Project consists of an open pit mine, on-site processing facility, tailings management facility (TMF) (for tailings solids, subaqueous storage of potentially acid generating (PAG) waste rock, and recycling of water for processing), waste rock stockpiles, low grade and overburden stockpiles, a temporary construction camp, ancillary facilities, mine haul roads, sewage and waste management facilities, a 24 km access road between the Project site and a rail load-out facility located on private land owned by HCMC in Vavenby, and a 12 km power line connecting the Project site to the BC Hydro transmission line corridor in Vavenby. Proposed mine facilities will be located primarily in the Harper Creek watershed and the headwaters of the Baker and Jones Creek Watersheds.

1.2 PROJECT LOCATION

The Project is located in the Thompson-Nicola area of BC, approximately 150 km northeast of Kamloops along Yellowhead Highway #5, approximately 10 km southwest of the unincorporated municipality of Vavenby, BC. The Project is located within National Topographic System (NTS) map sheets 82M/5 and 82M/12, is geographically centred at 51º30’N latitude and 119º48’W longitude, and is situated at approximately 1800 m above sea level (masl). The mineral claims comprising the Project cover an area of 42,640 ha. The Project location is shown on Figure 1.1.

1.3 PROJECT PROPONENT

The Proponent of the Project is HCMC, a wholly owned subsidiary of Yellowhead Mining Inc. (YMI). YMI was formed in 2005 as a private BC company specifically to acquire, explore and, if feasible, develop the Project. YMI is now a publicly owned BC based mineral development company trading on the Toronto Stock Exchange in Canada. HCMC’s strategy is to engineer, permit, finance, construct, and operate the Project.



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NOTES:

1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
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HARPER CREEK PROJECT

PROJECT LOCATION

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FIGURE 1.1 | REV 0

1.4 SCOPE OF WORK

Knight Piésold Ltd. (KP) completed engineering studies in support of an Environmental Impact Statement (EIS) and a Feasibility Study. Part of these studies included development of a numerical groundwater model to support baseline hydrogeologic characterization of the project area and to evaluate potential effects of proposed mine facilities on baseline hydrogeological conditions. A three-dimensional steady-state, regional-scale numerical groundwater model was developed to achieve this objective using MODFLOW-SURFACT to simulate baseline hydrogeological conditions at the Project site. The baseline model was then modified to include proposed mine facilities in order to assess hydrogeological conditions during mine operations and the post-closure period. A general arrangement of the Project showing proposed mine facilities is included as Figure 1.2. The results of baseline and predictive numerical groundwater models were used to inform environmental effects assessment as part of the EIS submission.

1.5 NUMERICAL MODELLING OBJECTIVES

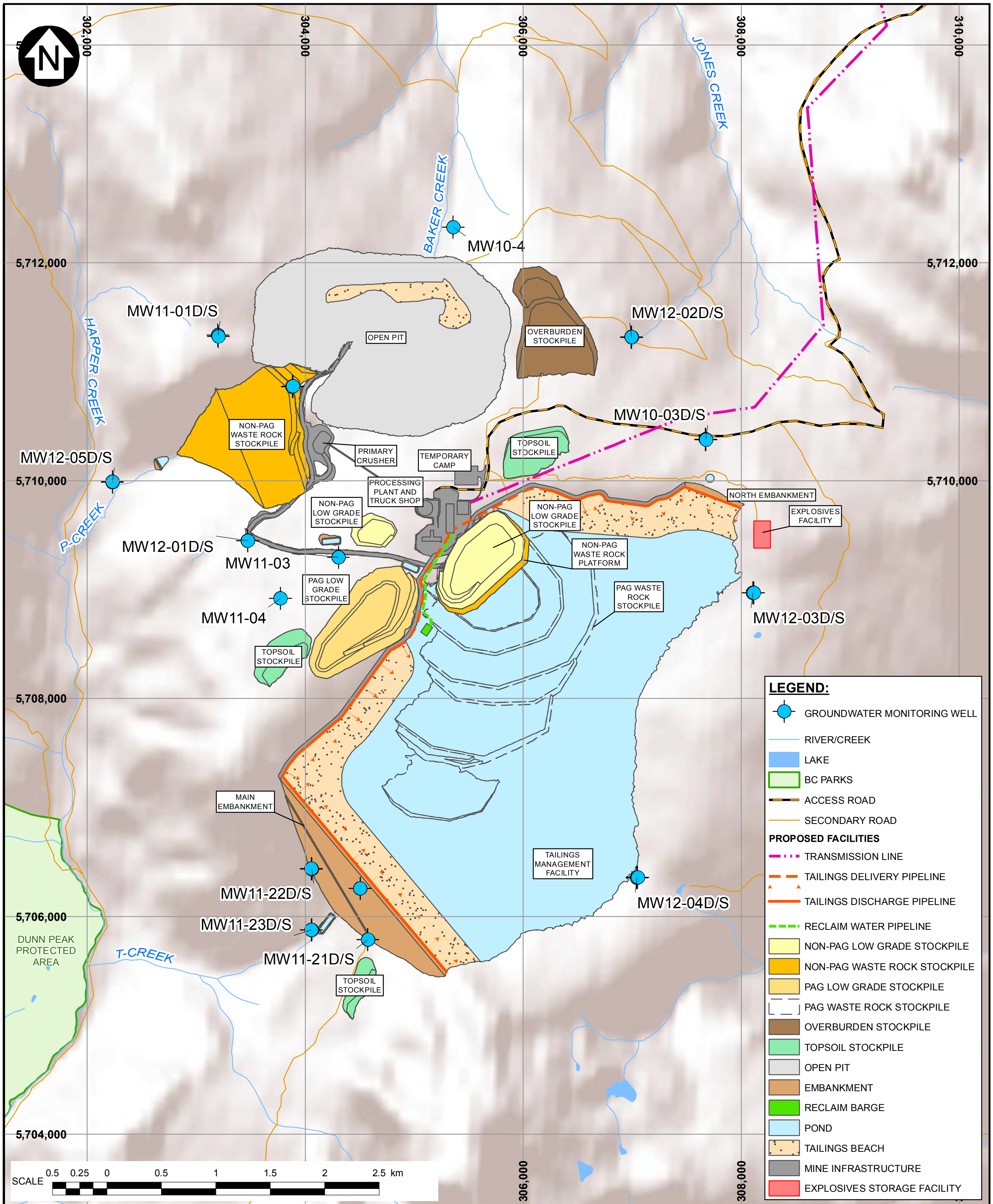
The objectives of the numerical groundwater modelling were to:

1. Develop a conceptual understanding of the pre-development groundwater system based on the available hydrogeological and hydrologic data.
2. Develop and calibrate a baseline numerical groundwater model to simulate pre-development hydrogeological conditions including groundwater flow directions, distribution of hydraulic head, and discharge of groundwater to creeks within the study area.
3. Predict potential effects of the proposed mine development and operations on pre-development hydrogeological conditions in the project area.
4. Characterize potential groundwater flow pathways for seepage originating from major mine facilities, including estimates of groundwater travel times and seepage rates to downstream discharge locations.

A steady-state, regional-scale numerical groundwater model was developed using MODFLOW-SURFACT to simulate baseline hydrogeological conditions at the Project site. Two predictive model scenarios were developed to assess potential effects of proposed mine development on baseline hydrogeological conditions. The two ‘predictive models’ represent mine development and infrastructure during the following key phases of the Project:

- Operations I: A steady-state model representing the operational period during which the open pit will be actively dewatered. Mine infrastructure including the Open Pit, Tailings Management Facility (TMF) and low-grade ore and waste rock stockpiles were simulated at their maximum build-out extents.
- Post-Closure: A steady state model representing Post-Closure conditions during which the pit lake and TMF have reached their maximum water storage volumes and are discharging excess water, and Low-Grade Ore Stockpiles have been removed and reclaimed.

Results of the numerical groundwater models were used to support water quality predictions completed for the project.



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GENERAL ARRANGEMENT (YEAR 24)

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FIGURE 1.2 | REV 0

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1.6 REFERENCE REPORTS

Baseline characterization for the Harper Creek Project relies on hydrometeorological, geological, geomorphological, and hydrogeological data previously presented within the following reports:

- **Project Description –** KP report *Mine Waste and Water Management Design Report*, Ref No. VA101-458/11-1 Rev 0.
- **2011 Site Investigation** – KP report *2011 Geotechnical Site Investigation Factual Report*, Ref. No. VA101-458/3-1 Rev 0.
- **2012 Site Investigation** – KP report *2012 Geotechnical Site Investigation Factual Report*, Ref. No. VA101-458/7-1 Rev 0.
- **Terrain Mapping** – KP report *Reconnaissance Terrain Mapping*, Ref. No. VA101-458/4-4 Rev 0.
- **Surface Hydrology** – KP report *Surface Hydrology Baseline*, Ref No. VA101-458/15-1 Rev 0.
- **Baseline and Predictive Watershed Modelling**– KP report *Watershed Modelling*, Ref. No. VA101-458/14-1 dated October 9, 2014.
- **Seepage Modelling –** KP letter report, *Harper Creek Project – Seepage and Stability Modelling*, Ref No. VA14-00865.
- **Baseline Hydrogeology–** ERM Rescan. 2014. *Harper Creek Project: Hydrogeology Baseline Report.* Prepared for Harper Creek Mining Corporation by ERM Consultants Canada Ltd.: Vancouver, British Columbia.

2 – HYDROGEOLOGICAL CONCEPTUAL MODEL

2.1 PHYSIOGRAPHY, CLIMATE AND DRAINAGE

The Project is located in the Shuswap Highlands, which are characterized by gently sloping plateau areas dissected by deep valleys. The topographic relief in the region is steep to moderate with elevations ranging from 450 m in the North Thompson River valley to highs of 1900 m on the ridges surrounding the mine site area, as shown on Figure 2.1.

The mine site is situated on gently sloping upland ridges flanked by steepened valley slopes. These valleys include the Harper Creek valley to the west and the Barriére River valley to the east, with the moderately sloped Thompson River valley to the north. The ground surface elevation of the deposit area ranges from 1575 m to 1800 m, and the plant site is situated at an elevation of 1840 m. The elevation of the TMF area ranges from 1600 m to 1900 m. The area was glaciated and mountain tops are typically rounded. The mine area is covered mostly by thick coniferous forest with heavy underbrush, however, in some places there are open logged patches. Much of the Harper Creek area has been logged and at higher elevations there are small marshy alpine meadows.

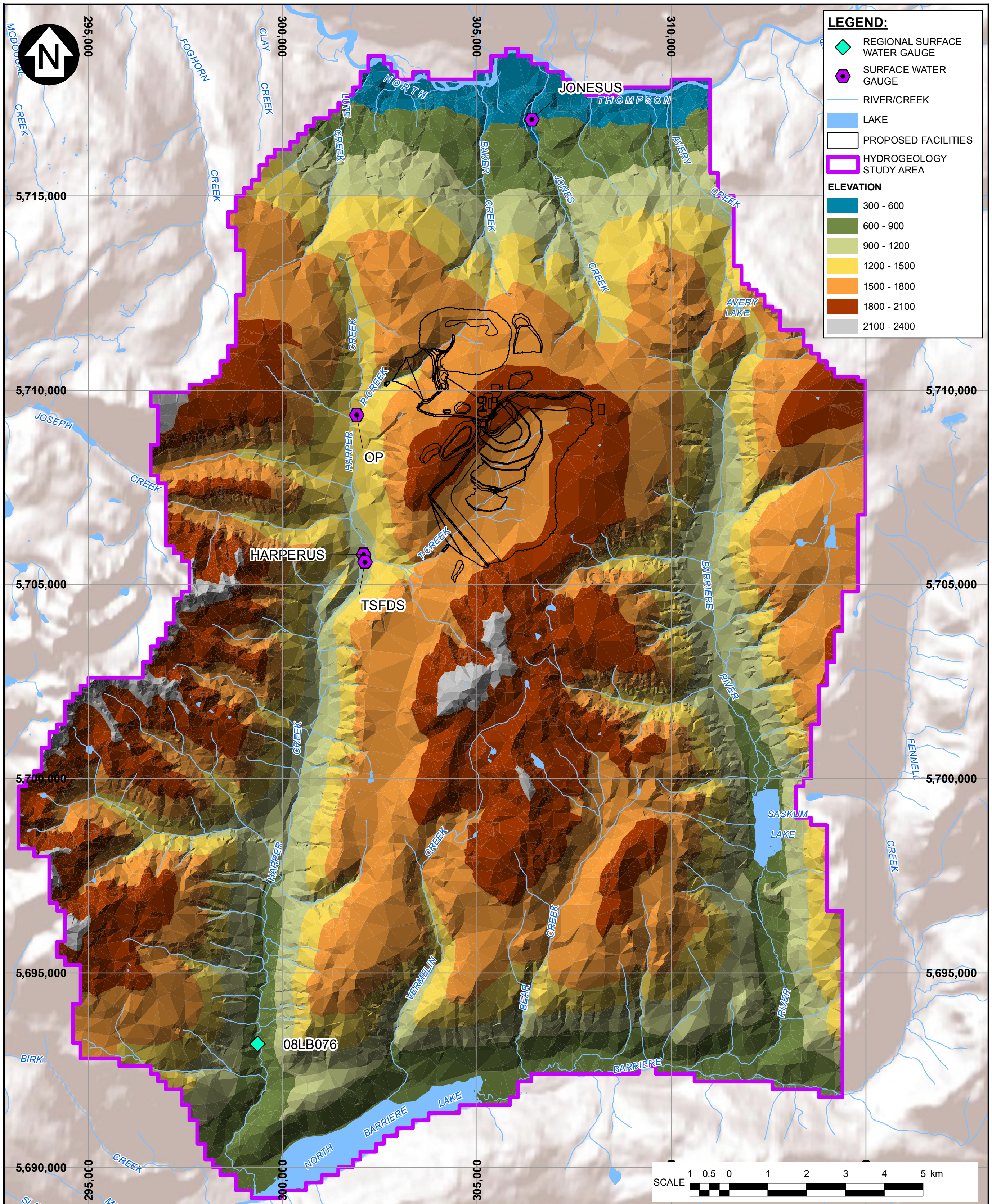
The TMF is located within a broad, shallow valley, which drains southward into a steep bedrock canyon and into Harper Creek at an elevation of 1100 m. The side slopes of the TMF basin are gentle to moderately sloped, and the centre of the basin features hummocky terrain with swampy, poorly drained areas.

The Project is situated on the watershed divide between Harper Creek and the North Thompson River. Harper Creek flows south from the Project site and discharges into the western end of North Barriére Lake, just upstream of the lake outlet. Barriére River flows out of the lake, flowing in a southwesterly direction for approximately 25 km before meeting the North Thompson River 58 km north-northeast of Kamloops. Jones and Baker Creek both drain north facing watersheds in the mine site area and flow approximately 5 km from their headwaters to the North Thompson River. The major drainages in the study area are presented on Figure 2.1.

2.2 HYDROMETEOROLOGY

Climate at the Harper Creek property is typical of British Columbia’s interior plateau characterized by warm summers and cold winters. Meteorological parameters estimated for the Project have been estimated using data collected at two climate stations in the immediate project area and correlated with data from regional climate stations, and estimates based on watershed modelling conducted for the Project (KP 2014c). Mean monthly temperatures range from -9.4°C in December to 10.7°C in July at the project elevation of 1,680 masl. Watershed modelling results indicate the average annual precipitation calculated from 1998 through 2012 is 1025 mm at the project site elevation (1680 masl). Watershed model results also estimate the mean annual potential evapotranspiration (PET) at 450 mm and actual evapotranspiration (AET) at 280 mm.

Average annual groundwater recharge across the modelled areas was estimated as 13% of total annual precipitation (an equivalent area weighted average of 114 mm) based on the results of the watershed model (KP, 2013c).



NOTES:
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HARPER CREEK PROJECT

STUDY AREA BOUNDARY AND GROUND SURFACE ELEVATION

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FIGURE 2.1 | REV 0

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2.3 GEOLOGICAL MODEL

The geological model for Harper Creek is summarized below. Detailed descriptions of the study area geomorphology, surficial geology, and bedrock geology are provided in the Mine Waste and Water Management Design Report (KP 2014d).

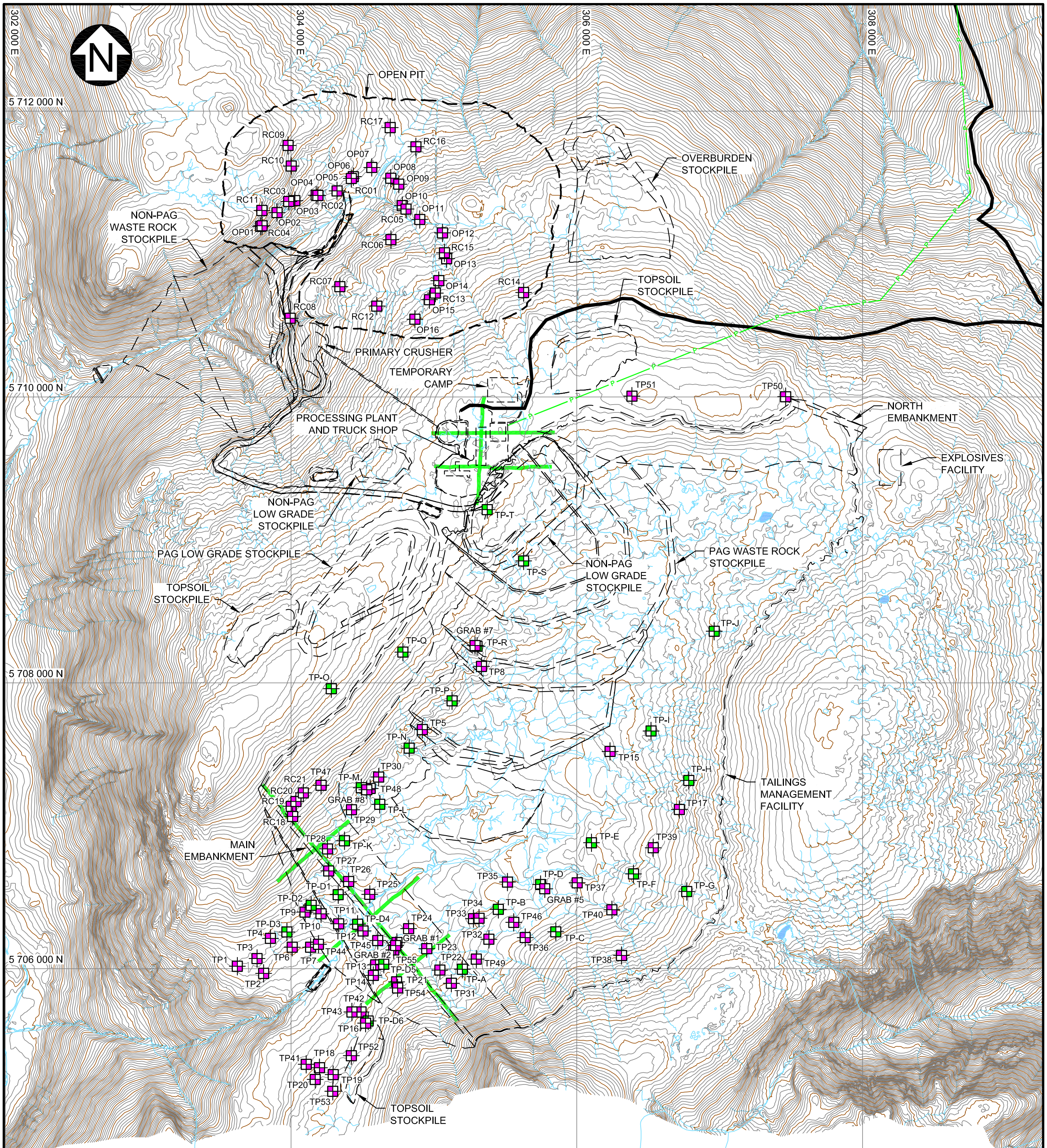
Data collected during site investigation programs completed in 2011 and 2012 were used to characterize the geology, hydrogeology, and geotechnical conditions at the site. Very little pre-existing geotechnical or hydrogeological information was available prior to 2011. The factual data from the 2011 and 2012 site investigation programs were reported on previously in the following documents:

- **2011 Site Investigation** – KP report *2011 Geotechnical Site Investigation Factual Report*, Ref. No. VA101-458/3-1 dated February 29, 2012.
- **2012 Site Investigation** – KP report *2012 Geotechnical Site Investigation Factual Report*, Ref. No. VA101-458/7-1 dated July 25, 2013.
- **Terrain Mapping** – KP report *Reconnaissance Terrain Mapping*, Ref. No. VA101-458/4-4 dated November 28, 2012.

The site investigation programs included the following:

- Excavation of 71 test pits and logging of 21 pre-existing road cuts.
- 32 geotechnical drillholes in and around the TMF, waste dump, and plant site areas.
- 28 overburden drillholes around the TMF and open pit, terminating in shallow bedrock.
- 7 geomechanical (oriented core) drillholes in the open pit.
- Installation of 20 long-term monitoring wells at 11 locations across the project area.
- Installation of 31 standpipe piezometers in geotechnical and geomechanical drillholes.
- In-situ packer testing conducted in bedrock in all geotechnical and geomechanical drillholes.
- Response testing conducted in all standpipe piezometers and monitoring wells.
- Laboratory rock mass strength and direct shear testing of bedrock.
- Laboratory index testing of overburden material.
- Seismic refraction surveys along the TMF main embankment and plant site areas.

The simplified project layout including the test pits and drillholes from all investigations at the site are illustrated on Figure 2.2 and Figure 2.3, respectively. Additional details on field data collection methods and findings can be found in the reference reports listed above (Knight Piésold Ltd., 2012a, 2012b, and 2013).



LEGEND:

- TEST PIT (KP)
- TEST PIT (KLOHN CRIPPEN BERGER, 2008)
- MINE ACCESS ROAD
- SEISMIC REFRACTION LINE

NOTES:

1. COORDINATE GRID IS UTM NAD83 ZONE 11.
2. PLAN BASED ON INFORMATION PROVIDED BY HARPER CREEK MINING CORP. DATED MARCH 11, 2014.
3. CONTOUR INTERVAL IS 5 METRES.
4. DIMENSIONS ARE IN MILLIMETRES AND ELEVATIONS ARE IN METRES, UNLESS NOTED OTHERWISE.

SCALE A 400 200 0 400 800 1200 1600 2000 m

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TEST PIT AND SEISMIC REFRACTION PLAN

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FIGURE 2.2	REV 0

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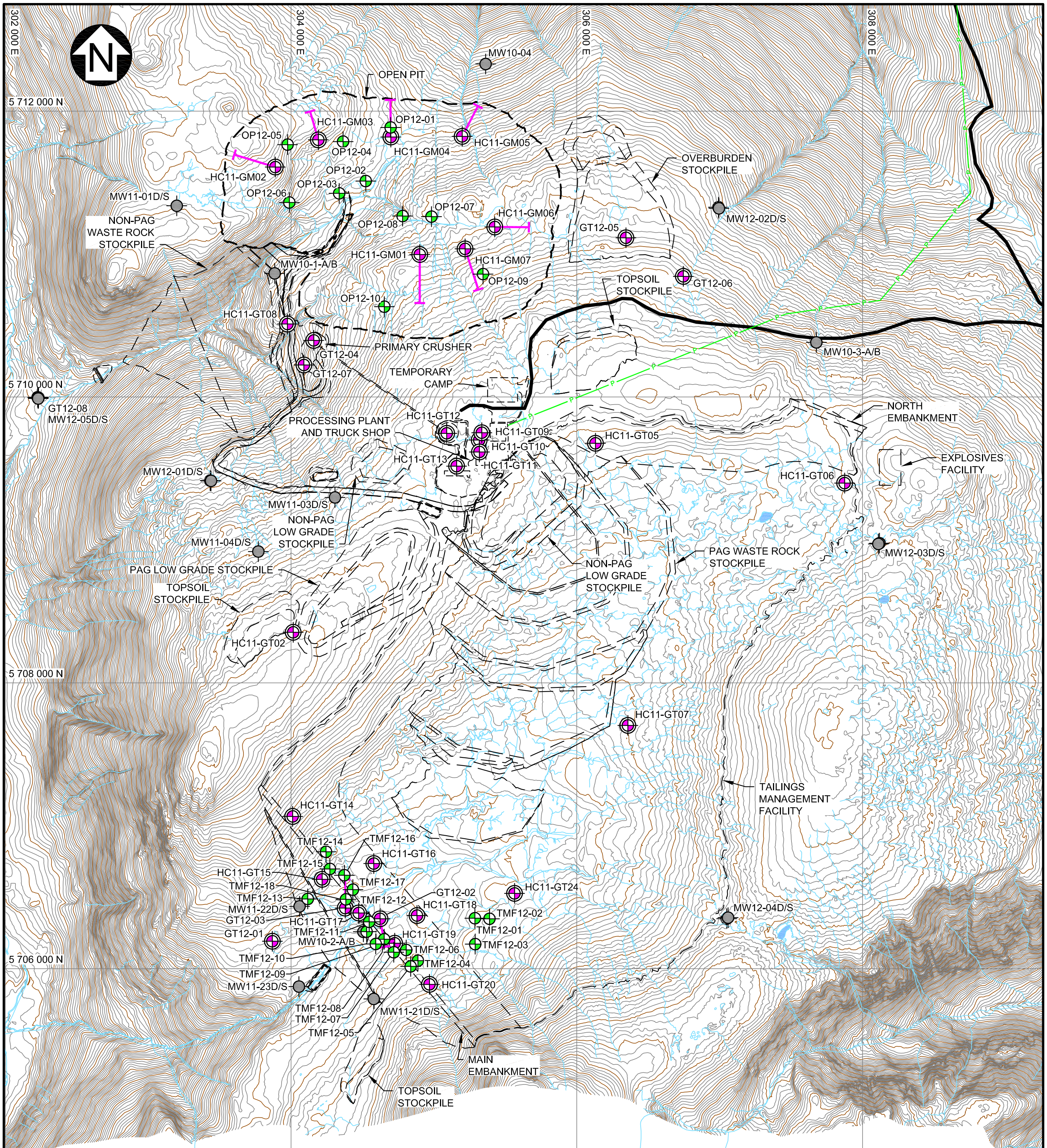

LEGEND:

- GEOTECHNICAL DRILLHOLE WITH PIEZOMETER
- MONITORING WELL
- GEOTECHNICAL DRILLHOLE
- MINE ACCESS ROAD

NOTES:

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SCALE A 400 200 0 400 800 1200 1600 2000 m

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DRILLHOLE AND MONITORING WELL PLAN

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FIGURE 2.3 REV 0

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2.3.1 GEOMORPHOLOGY

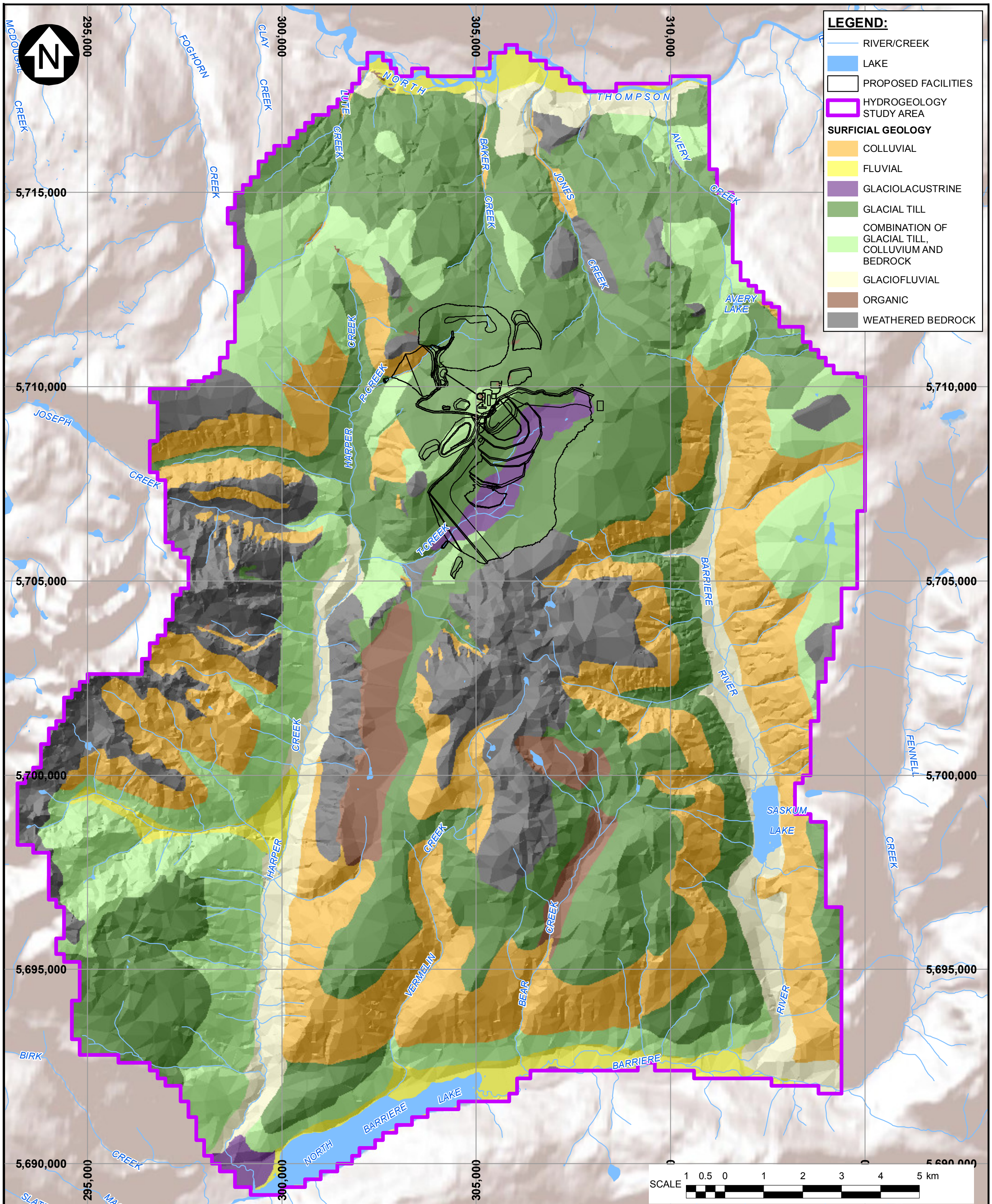
Surficial deposits and landforms in the project area are primarily associated with the Fraser Glaciation, the last period of continental ice sheet glaciation in British Columbia. Based on the rounded nature of the mountain tops, that most of the Study Area was glaciated with a large thickness of ice. Glacial till was deposited at the base of the ice sheet. Glacial lakes developed locally on the flat mountain-top areas as the ice retreated. Fine sediments, comprising silts and fine sands, accumulated in the lakes and coarser beach deposits, comprising gravelly sands, accumulated along the shorelines. With the continuing retreat of the ice sheet, the ice dams breached and the lakes dissipated, giving way to swamps. Organic soils accumulated in the swamps as a result of the decomposition of vegetation. Extensive kames, comprising hummocky terrain and terraces, accumulated at the toe of the North Thompson River Valley. Glacial outwash deposits accumulated as the ice retreated further. These deposits were subsequently incised by the North Thompson River, resulting in the formation of glaciofluvial terraces. Colluvium has developed locally on the steeper side slopes of the valley as a result of soil creep and landslides. Fluvial terraces have developed over time as the river eroded to a lower level. The North Thompson River is actively depositing coarse alluvium within its channel and finer sediments on its floodplain.

2.3.2 SURFICIAL GEOLOGY

The surficial deposits encountered in the Project area are as follows:

- **Colluvial Deposits** – thin layers of colluvium, typically boulder gravel with some silt and sand, are found along the base of some steeper slopes developed on the steeper valley side slopes as a result of soil creep and landslides.
- **Glaciofluvial and Fluvial Deposits** - comprised of sand, silt and gravel deposited along valleys as outwash from ablation of glacial ice and along the North Thompson and Barriere River Valleys.
- **Glacial Till and Glaciolacustrine Deposits** – Glacial till is identified as coarse grained soils with gravels and fines deposited over much of the project site. Glaciolacustrine deposits are found usually overlying glacial till and are classified as fine grained soils silts and clays deposited during a period of de-glaciation as a result of glacial meltwater detention.
- **Organic Deposits** – a thin veneer of topsoil that accumulated in poorly drained areas as swamps as a result of the decomposition of vegetation.
- **Weathered Bedrock** – Deformed and metamorphosed, sedimentary and volcanic rocks.

The distribution of the surficial materials within the study area is shown on Figure 2.4 and descriptions of the material properties of each stratigraphic unit are provided in the sections that follow.



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SURFICIAL GEOLOGY

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FIGURE 2.4

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Organic soils accumulated in swamps as a result of the decomposition of vegetation with the retreat of the ice sheet. A thin veneer of topsoil ranging from 0.1 to 0.5 m deep covers much of the project area. Thicker layers of organics are present within the poorly drained areas of the property, particularly in the centre of the TMF basin, and consist of brown and block spongy fibrous peat to organic silt wet fibric to mesic plant material in various stages of decomposition (KP, 2012a).

Colluvium has developed locally on the stepper valley side slopes as a result of soil creep and landslides. A surface veener of colluvium is expected in the steeper areas of terrain and weathered bedrock colluvium is expected to be more prevalent on the moderately steep, south-facing slopes in the project area. The colluvium is comprised of silty sand, gravel and cobbles and the consistency of this material is expected to vary locally (KP, 2012a). Colluvium was only encountered in one area of the project footprint – the south facing slope of the P-creek valley. This material is also expected in the lower reaches of Harper, Jones and Baker Creeks.

Glaciolacustrine deposits developed from glacial lakes locally on the flat mountain-top areas as the ice retreated. This type of deposit is found in upper T-Creek valley within the study area by the proposed TMF. Fine sediments accumulated in the glacial lakes varying from silt with some fine sand to fine to coarse sand. These deposits when encountered in the TMF basin area were generally up to 2 m thick and underlain by glacial till.

Glacial till deposits are present in the valleys of the project area and in a discontinuous blanket on mountain crests and slopes. Glacial till was deposited at the base of the ice sheet and is found thickest in the valley bottoms and thinner on valley side slopes and discontinuous over the bedrock on topographic highs. Glacial till thickness within the TMF area ranges from 1 m to 12 m, and typically is greater than 4 m thick (KP, 2012a). Glacial till generally comprised fine to coarse gravel with trace to some sand and silt and trace cobbles. The site investigation programs indicated that the glacial till on the east side of the valley contains a slightly higher proportion of fines than that on the west side.

2.3.3 BEDROCK GEOLOGY

The Project is located within structurally-complex, low-grade metamorphic rocks of the Eagle Bay Assemblage (EBA) and Fennell Formations. A bedrock geology map for the Project site was presented in the Hydrogeology Baseline Report (ERM Rescan, 2014: Figure 1.5-8). Bedrock in the region includes quartzites, quartz-mica schists and metavolcanics that were folded and metamorphosed to greenschist during the Jurassic-Cretaceous Columbian Orogeny (Höy, 1999). The rocks of the Eagle Bay Assemblage are overlain by metavolcanics and granitic orthogniess with intrusions of Mid-Cretaceous granodiorite and quartz monzonite of the Bayonne plutonic belt.

The regional structure consists typically of east-west striking, low to moderately dipping stratigraphy. The EBA is divided by four northwest-dipping thrust faults which disrupt the stratigraphic sequence by positioning Cambrian rocks overtop of younger Paleozoic strata. One of these faults, the Harper Creek fault, bisects the proposed open pit area, running sub-vertically along a southwest-northeast trend.

The Harper Creek deposit is an extensive volcanogenic sulphide system, with a mineralized zone spanning 2000 m along strike, 2000 m down dip and lies within a 1000 m thickness of volcano-sedimentary stratigraphy. The deposit is hosted in the Eagle Bay Assemblage, specifically within the

Lower Paleozoic and Greenstone Belts. The deposit is interpreted to be a polymetallic volcanogenic sulphide deposit comprised of lenses of disseminated, banded and fracture-filling iron and copper sulphides. The mineralization consists of chalcopyrite with accessory pyrite, magnetite and pyrrhotite. There are significant amounts of Au and Ag present within the mineralized zone. The mineralization is tabular and strikes east-west, dipping at 15° to 25°, with sulphide lenses up to tens of metres thick. This tabular mineralization comprises the central and west zones of the pit. There is a broad lower-grade zone of Cu with Au/Ag that is linked to multi-phased stringer or feeder zones within the eastern zone of the pit area (KP, 2013).

Bedrock outcrop exposure is rare and generally restricted to higher elevations in the area, and it is typically overlain by 1 to 15 m of overburden. Bedrock within and surrounding the immediate project area consisted of intrusives, orthogneiss, fault zones, phyllites, schists, quartz eye schists and silica altered host rocks (KP, 2013).

A cumulative summary of the rock mass properties grouped by lithology is presented in Table 2.1.

Table 2.1 Summary of Rock Mass Quality

Lithology [1]	**RQD (%)**				**RMR^{89}**				
	# of Runs	**Mean**	**Median**	**St. Dev.** [2]	**# of Discontinuities**	**Mean**	**Median**	**St. Dev.**	**Description**
Intrusives	151	72	79	25	831	69	68	11	GOOD
Orthogneiss	580	74	85	27	3182	67	67	10	GOOD
Fault Zone	42	60	69	36	144	57	57	11	FAIR
Phyllite	394	64	75	33	2117	65	64	10	GOOD
Schist	436	77	88	26	898	63	63	10	GOOD
Schist (w/Quartz Eyes)	859	75	85	27	2236	63	63	9	GOOD
Silica Altered Zone	110	74	85	28	258	66	67	8	GOOD

NOTES:
1. ADOPTED FROM TABLE 4.1 OF THE 2012 GEOTECHNICAL SITE INVESTIGATION FACTUAL REPORT (KP, 2013).
2. ST. DEV. STANDS FOR STANDARD DEVIATION.

2.4 HYDROGEOLOGY

Hydrogeological baseline data were collected during geotechnical, geomechanical and hydrogeological site investigations conducted by KP and others. Hydraulic conductivity testing was completed as part of these investigations in monitoring wells, piezometers, geotechnical drillholes and geomechanical drillholes. In addition, long-term continuous groundwater elevation data were collected from groundwater monitoring wells within the study area.

2.4.1 Hydrostratigraphic Units

The geology in the area has been simplified into five hydrostratigraphic units that provide paths for groundwater flow:

- Glacial till and glaciolacustrine deposits
- Glaciofluvial and fluvial deposits
- Shallow weathered bedrock
- Deeper fractured bedrock, and
- Fault zones.

Further detail on each hydrostratigraphic unit includes:

- **Glaciofluvial and fluvial deposits**: The lower reaches of Harper, Jones and Baker Creeks are characterized by glaciofluvial deposits composed of silty sand, gravel and trace cobbles. Fluvial sand and gravel deposits are present adjacent and underlying the North Thompson River to the North of the Project site. These materials provide a preferential flow path for groundwater movement along creek and river valleys.
- **Glacial till and glaciolacustrine deposits:** Glacial till deposits are present in the low lying valleys of the project area and in a discontinuous blanket on mountain crests and slopes. The till is comprised of varying composition of sand, silt and gravel. These materials vary in thickness from 0 to 12 m throughout the site and provide a pathway for movement of groundwater downslope. Glaciolacustrine deposits are found in the T-Creek valley within the footprint of the TMF. These deposits are thin and likely underlain by glacial till.
- **Shallow, weathered bedrock:** Weathered bedrock is present in many locations across the site in areas were bedrock is shallow or where outcrops exits. Weathering provides a preferential flow path for groundwater flow in bedrock.
- **Fractured bedrock:** Much of the bedrock in the Project area is fractured, where preferential flow directions are likely oriented along fault planes or fractures. Lugeon test data indicates hydraulic conductivity values decrease with increasing depth.
- **Fault zones:** Faults provide both conduits and barriers to groundwater flow. Conduits are provided through fractured ground adjacent to the fault allowing flow along the fault. Barriers are created due to fault gouge within the fault itself inhibiting the flow of groundwater across the fault. Hydraulic conductivity testing has not identified elevated hydraulic conductivities near faults. The presence of geologic structures is not expected to affect groundwater elevation or flow on a project site scale. As such, faults are not included as a hydrostratigraphic unit in the numerical models.

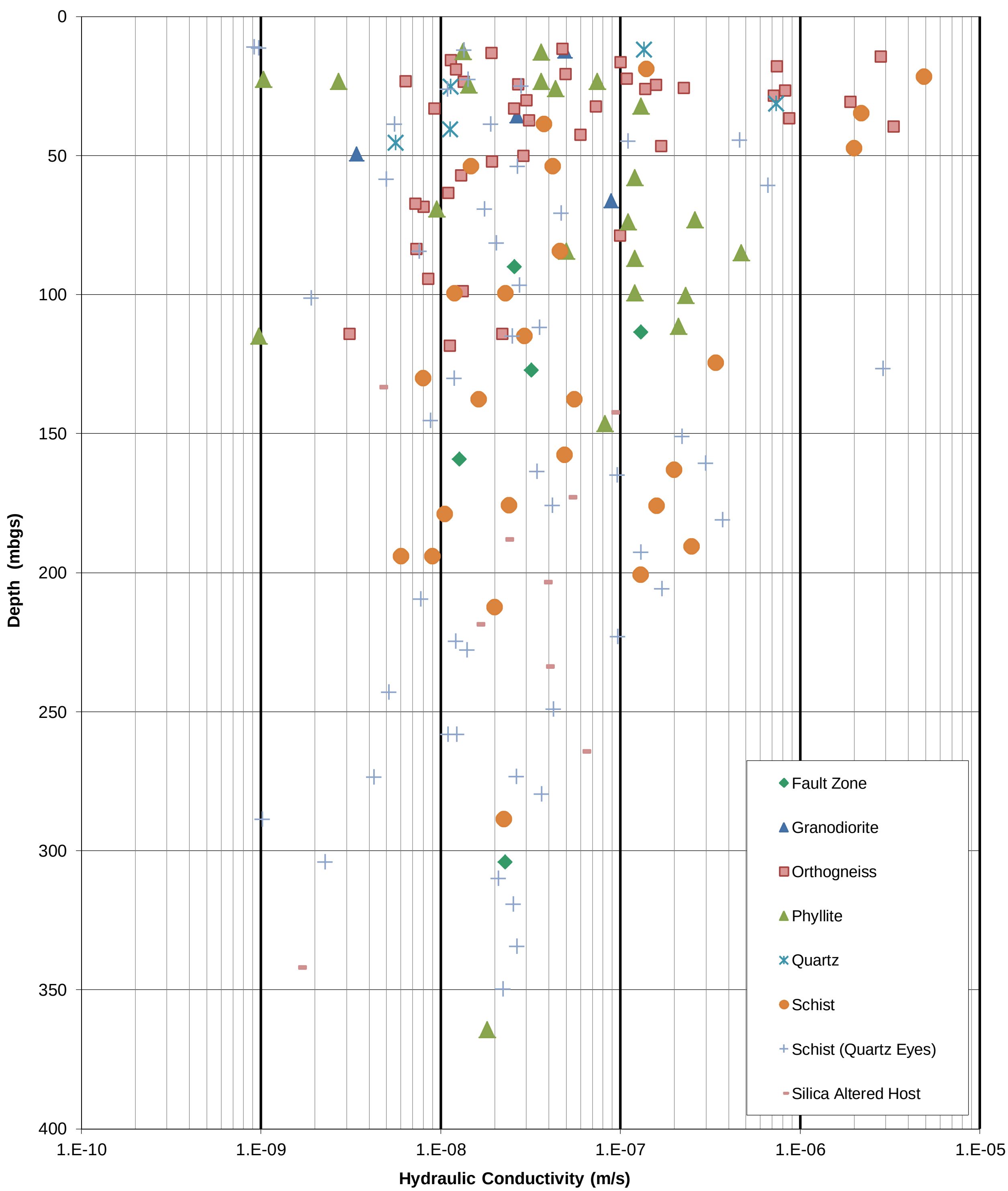
2.4.2 Hydraulic Conductivity

Hydrogeological testing was completed during the site investigation programs in order to estimate the in situ hydraulic conductivity of the rock mass in the project area and to develop an understanding of the variability of rock mass permeability by rock lithology and depth.

Response tests were carried out in geotechnical and geomechanical drill holes during drilling to measure the hydraulic conductivity of discrete intervals within the bedrock as well as in monitoring wells and piezometers to measure hydraulic conductivity in an isolated completion zone. A total of nineteen response tests were conducted in groundwater monitoring wells with test interval geology including orthogneiss, phyllite, schist, or quartz monzonite intrusion and at depths ranging from approximately 10 mbgs to 45 mbgs. Hydraulic conductivity values from these tests range from approximately $1x10^{-9}$ m/s to $1x10^{-5}$ m/s with a geometric mean value of $1x10^{-7}$ m/sec (KP 2012b, KP 2013). Forty-four response tests were completed in standpipe piezometers installed in geotechnical and geomechanical drillholes in the vicinity of the proposed open pit, tailings management facility, low-grade ore stockpiles, and waste rock stockpiles. All tests were conducted in piezometers completed in bedrock with test interval depths ranging from approximately 10 mbgs to 130 mbgs. Resulting hydraulic conductivity values ranged from less than $7x10^{-10}$ m/sec to approximately $5x10^{-6}$ m/s with a geometric mean value of approximately $1x10^{-8}$ m/s.

Packer tests (Lugeon tests) were conducted during geotechnical and geomechanical drilling in 2011 and 2012 (KP 2012b, KP 2013) to measure the hydraulic conductivity of discrete depth intervals within the bedrock. A total of 139 packer tests were completed in 34 drillholes. Hydraulic conductivities were measured to depths of up to 350 m but were mostly in the upper 200 m. The values ranged from less than $1x10^{-9}$ m/s to about $1x10^{-6}$ m/s with a geometric mean of $2x10^{-8}$ m/s.

A plot of hydraulic conductivities from Packer and Response testing vs. test interval depth, separated by rock type is shown on Figure 2.5. Hydraulic conductivity values typically decreased with depth. This trend is attributed to increasing stress and decreasing bedrock weathering as depths increase. There does not appear to be a relationship between hydraulic conductivity values and rock type.



NOTES:

1. DEPTH OF THE TEST INTERVAL IS REPORTED IN METERS BELOW GROUND SURFACE (mbgs).

2. AVAILABLE HYDRAULIC CONDUCTIVITY TEST DATA IS LIMITED TO TEST INTERVALS WITHIN 375 mbgs. AS MODEL LAYERS 5 THROUGH 8 ARE DEEPER THAN 375 mbgs, EACH SUBSEQUENT LAYER WAS ASSIGNED A DRCREASING HYDRAULIC CONDUCTIVITY VALUE BASED ON THE EXTRAPOLATED TREND SHOWN ABOVE.

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OBSERVED HYDRAULIC CONDUCTIVITY VS. DEPTH BY ROCK TYPE

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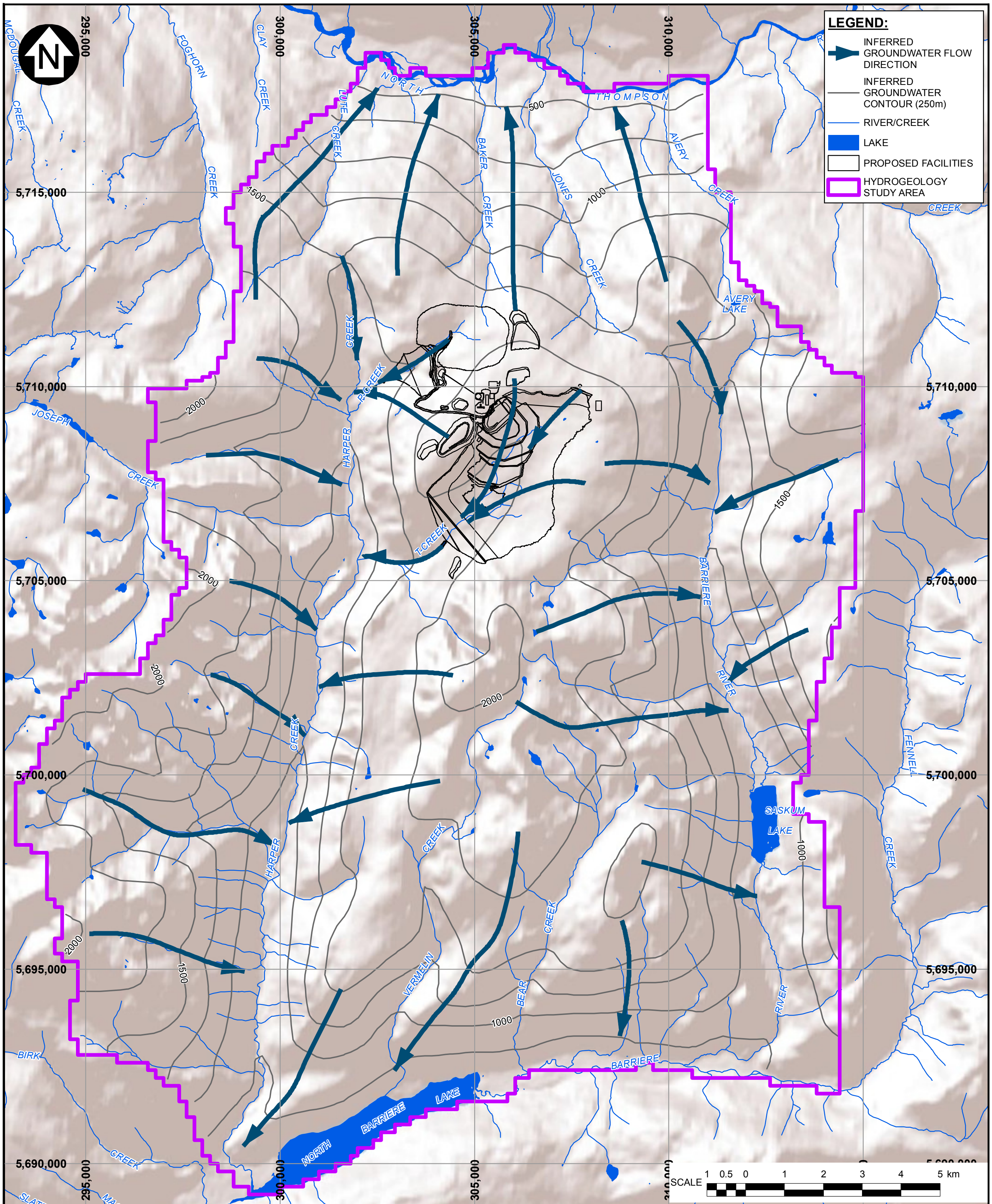
FIGURE 2.5 REV 0

2.4.3 Groundwater Elevation, Flow Direction and Gradients

The high level of topographic relief at this site indicates that local flow systems dominate groundwater movement. Groundwater at the site flows from recharge zones located in topographic highs, such as in the vicinity of the proposed open pit, towards discharge zones located in the Harper Creek (including P-Creek and T-Creek), Baker Creek and Jones Creek valleys. Groundwater discharge to streams provides baseflow that sustains streamflow within major drainages during the winter and early spring months. Groundwater divides are present near surface watershed boundaries between the Harper, Baker and Jones Creek watersheds. Conceptual local groundwater flow direction and water table elevation within the study area are shown on Figure 2.6.

The rate of groundwater recharge was estimated as 13% of total annual precipitation (an equivalent area weighted average of 115 mm) based on the results of the watershed model (KP 2013d). Local recharge may be derived from the following areas:

- Weathered bedrock on hilltops and hillsides: This recharge zone provides groundwater for deeper bedrock materials and also flow downslope, within the top of rock.
- Overburden in valley bottoms: The valleys along the tributaries to Harper Creek, North Barrieré River and the North Thompson River, such as P-Creek and T-Creek, include overburden deposits of glaciofluvial and colluvial materials. These deposits typically exhibit higher hydraulic conductivity and provide a preferential flow path for groundwater movement. Generally these areas are more likely to be discharge areas driven by groundwater flow from regions of higher elevation. Artesian conditions observed in the deposit area and near the TMF reflect discharging conditions.
- Groundwater levels in the study are expected to be a subdued reflection of ground surface topography. Generally, depth to water is greatest in regions of high elevation, such as the topographic highs along the northwest of the proposed open pit and along upland watershed boundaries of the P-Creek and T-Creek. A shallow groundwater table is expected in the Harper Creek, Baker Creek, Jones Creek, P-Creek and T-Creek valleys where groundwater discharge to surface drainages occurs.
- In general, downward vertical hydraulic gradients are expected in regions of high elevation where groundwater recharge occurs. Upward vertical gradients occur in low-lying valleys where groundwater discharge to the surface water system occurs. Horizontal groundwater flow gradients in the project are estimated to range from about 0.1 m/m to 0.3 m/m on valley slopes such as in the P-Creek, T-Creek, Jones Creek and Baker Creek watersheds. Horizontal gradients along valley bottoms are estimated to be approximately 0.03 m/m in the Harper Creek watershed and up to 0.3 m/m in the P-Creek, T-Creek, Jones Creek and Baker Creek watersheds.
- The average groundwater flow velocity within the overburden was estimated as 0.35 m/day based on the estimated horizontal gradient (0.03 m/m), a hydraulic conductivity value of 2x10-5 m/s and an assumed porosity of 0.15 (Freeze and Cherry, 1979). This value is applicable to groundwater flow in river and creek valleys such as the Harper Creek valley where alluvial and glacial till materials are the primary pathway for groundwater flow. The average groundwater flow velocity within the bedrock is about 0.2 m/day based of the estimated horizontal gradient of 0.1 m/m a hydraulic conductivity value of 2x10-8 m/s and a porosity of 0.001 (Freeze and Cherry, 1979).



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NOTES:
1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:140,000 FOR 8.5x11 (LETTER) PAPER.

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CONCEPTUAL MODEL GROUNDWATER ELEVATION AND FLOW DIRECTION

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FIGURE 2.6 | REV 0

3 – BASELINE NUMERICAL MODEL

3.1 OVERVIEW

A steady-state, regional-scale numerical groundwater model was developed to simulate baseline hydrogeological conditions and to provide the basis required to assess potential effects of the Project on the local groundwater system. The model was developed using the MODFLOW-SURFACT computer code run in the Groundwater Vistas (version 6.20; ESI, 2011) graphical user interface. MODFLOW-SURFACT is a three-dimensional finite-difference flow model developed by the U.S. Geological Survey and HGL Software Systems that has become an industry standard for groundwater modelling applications (Hydrogeologic Inc., 1996).

Model boundary conditions and input parameters (i.e., groundwater recharge and hydraulic conductivity) govern the flow of groundwater within the model and control the addition or removal of water from the model domain. The baseline model was calibrated to measured groundwater elevation data collected from on-site groundwater monitoring wells and to baseflow estimates for hydrology stations located on the major surface water drainages within the study area.

The results of the baseline model are representative of the pre-development hydrogeological conditions including groundwater flow directions, distribution of hydraulic head and groundwater/surface water interaction on a project-site scale. Baseline model development, calibration and results are discussed in the sections that follow.

3.2 BASELINE MODEL GEOMETRY AND GRID

The baseline model domain encompasses an area of 458 km^2 with the Harper Creek Project site located at its center, as shown on Figure 2.1. The model domain extends south to include the Harper Creek and Barriere River watersheds to their point of confluence downstream of North Barriere Lake and north to include the Jones and Baker Creek watersheds extending to their point of confluence with the North Thompson River. The perimeter of the active model domain was defined by the watershed boundaries of Harper, Jones and Baker Creeks. Groundwater flow divides were inferred to be coincident with watershed boundaries.

The model has a rectangular grid of 328 rows by 220 columns covering an area of approximately 22 km (East-West) by 33 km (North-South). The model was divided into 8 layers in the vertical dimension for a total of 577,280 cells, approximately 450,550 of which are active. Cell size was 50 m by 50 m within the mine site and expands to 200 m by 200 m at the edges of the model. The grid was refined in the vicinity of the mine site in order to provide a higher resolution over that portion of the model domain. A maximum grid expansion factor of 1.5 was used to increase dimensions of adjacent cells. The finite-difference grid is shown on Figure 3.1.

Ground surface elevation was defined in Layer 1 of the model using a GIS-based contour shapefile of surface topography. Elevation within the active model domain ranges from approximately 650 masl at the downstream extent of Harper Creek and 450 masl at the confluence of Jones and Baker Creeks with the North Thompson River up to 2,000 masl in the mountainous terrain near the proposed mine site.

The finite-difference grid was discretized into eight layers of increasing thickness with depth:

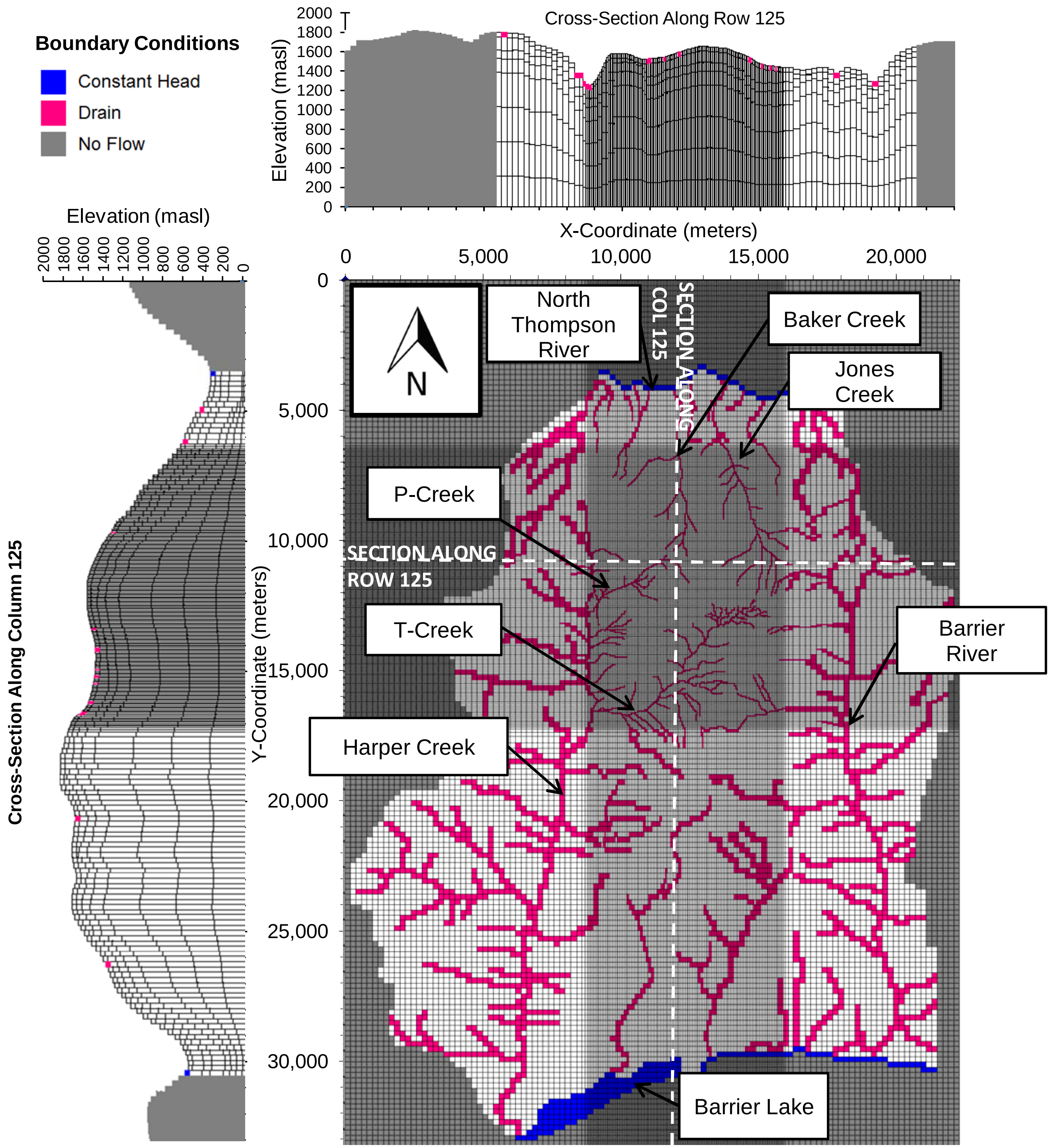
- Layer 1 is generally 50 m thick (top elevation defined by GIS contour shapefile)
- Layer 2 is 75 m thick
- Layer 3 is 100 m thick
- Layer 4 is150 m-thick
- Layers 5 through 7 are of variable thickness, evenly spaced between the bottom of Layer 4 and the top of Layer 8, and
- Layer 8 is of variable thickness, with a base elevation equal to mean sea level.

Due to steep surface topography, some modification of Layer 1 was required to ensure that adjoining model cells shared a minimum 5-meter overlap along the vertical dimension. A uniform thickness was assigned to Layers 2 through 4 based on characterization of the hydrostratigraphic units represented by each layer. Layers 5 through 7 are of variable thickness, evenly spaced between the bottom of Layer 4 and the top of Layer 8. The bottom of the model domain (Layer 8) was set to a uniform elevation of 0 masl. All eight layers were modelled as convertible layers (MODFLOW Layer Type 3).

3.3 HYDROSTRATIGRAPHIC UNITS AND MODEL LAYERS

The model layers represent four hydrostratigraphic units based on the conceptual model presented in Section 2. The hydrostratigraphic units represented in the numerical model include:

- **Glaciofluvial and fluvial deposits:** glaciofluvial deposits are represented in Layer 1 by grid cells adjacent to and underlying Harper Creek and the Barriere River. Fluvial sand and gravel deposits are represented adjacent to and underlying the North Thompson River by grid cells in Layer 1.
- **Glacial till and glaciolacustrine deposits:** Glaciolacustrine and till deposits were included in model layer 1 in a localized zone within the footprint of the TMF supernatant pond.
- **Shallow, weathered bedrock:** weathered bedrock is represented by all grid cells of Layer 1 with the exception of those that represent the overburden deposits along the North Thompson River, Barriere River and Harper Creek valleys.
- **Fractured bedrock:** fractured bedrock is represented by model Layers 2 through 8. The fractured bedrock unit was subdivided into seven layers to allow hydraulic conductivity values to decrease with depth. Even though several types of bedrock are present at the site, hydraulic testing did not indicate a significant relationship between hydraulic conductivity values and rock type. As such, bedrock within the model is assumed to be a homogeneous unit which was sufficient for the purpose of this hydrogeological assessment.



NOTES:

1. THE BOUNDARY CONDITIONS SHOWN IN PLAN VIEW ABOVE ARE FROM MODEL LAYER 1.

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BASELINE MODEL
NUMERICAL GRID, MODEL LAYERS
AND BOUNDARY CONDITIONS

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FIGURE 3.1 REV 0

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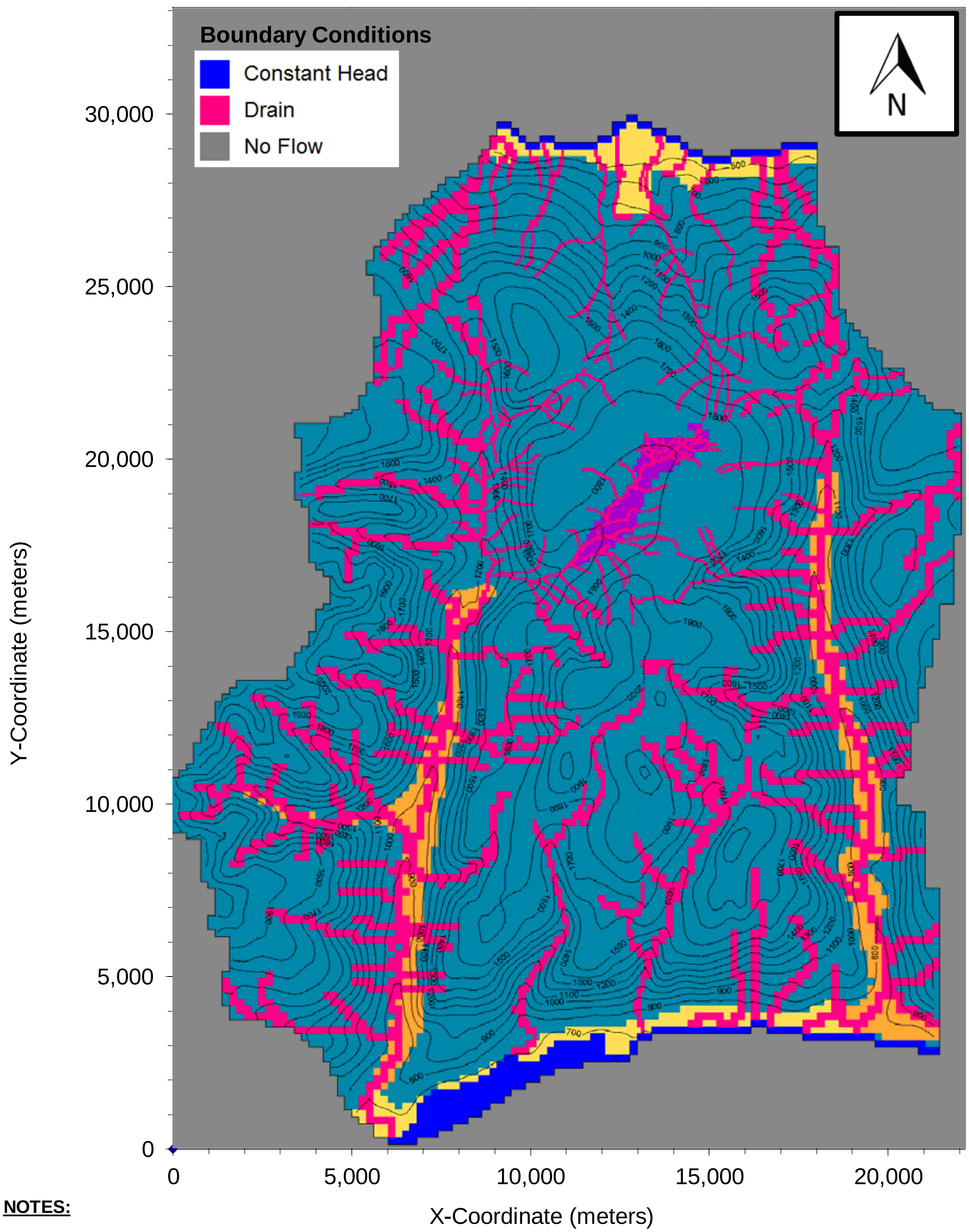
3.4 HYDRAULIC CONDUCTIVITY

Initial values of hydraulic conductivity were assigned to the hydrostratigraphic units/model layers based on available hydraulic test data (Section 2.4.2). Initial hydraulic conductivity values assigned to the model were varied within the range of observed and expected values during calibration of the baseline model. The calibrated hydraulic conductivity values assigned to each model layer were assumed to be isotropic ($K_x = K_y = K_z$) and are summarized in Table 3.1.

The cells in Layer 1 were subdivided into three hydraulic conductivity zones in order to differentiate between the three hydrostratigraphic units modelled in this layer. The first zone represents the glaciofluvial and colluvial deposits along Harper Creek and Barriere River valleys. The second zone is used to simulate the fluvial sand and gravel deposits adjacent to and underlying the North Thompson River. The third zone represents the weathered bedrock unit (including a thin till/colluvium cover) across the remainder of the study area. Plan and section views of the spatial distribution of the three property zones are presented on Table 3.1.

Table 3.1 Baseline Model Hydraulic Conductivity and Porosity Values

Hydrostratigraphic Unit	MODFLOW Layer	MODFLOW Color	Hydraulic Conductivity	Effective Porosity
Glaciofluvial and Colluvial Deposits	Layer 1		1.0E-04	0.15
Fluvial Sand and Gravel Deposits	Layer 1		1.0E-04	0.15
Glacioflacustrine and Glacial Till Deposits	Layer 1		5.0E-07	0.15
Weathered, Fractured Bedrock	Layer 1		5.0E-07	0.001
Fractured Bedrock	Layer 2		9.0E-08	0.001
Fractured Bedrock	Layer 3		3.5E-08	0.001
Fractured Bedrock	Layer 4		8.5E-09	0.0001
Fractured Bedrock	Layer 5		2.1E-09	0.0001
Fractured Bedrock	Layer 6		9.5E-10	0.0001
Fractured Bedrock	Layer 7		4.5E-10	0.0001
Fractured Bedrock	Layer 8		2.3E-10	0.0001



NOTES:

1. THE HYDRAULIC CONDUCTIVITY ZONES SHOWN ABOVE ARE FROM MODEL LAYER 1.

Zone	Color	Material Type
Zone 1		Weathered Fractured Bedrock
Zone 6		Glaciofluvial Deposits
Zone 7		Glacial Lake Deposits
Zone 8		Fluvial Deposits

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BASELINE MODEL
HYDRAULIC CONDUCTIVITY ZONES

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FIGURE 3.2

REV 0

Model Layers 2 through 8 were assigned hydraulic conductivity values representative of unweathered, fractured bedrock. Figure 3.3 shows measured hydraulic conductivity decreasing with depth. Calibrated hydraulic conductivity for each model layer are shown for comparison alongside the measured data. The unweathered bedrock unit was divided into seven separate model layers and hydraulic conductivity values were assigned to decrease in each subsequent layer to reflect this decreasing trend in the model. The decrease in hydraulic conductivity from Layers 5 through 8 was estimated following the approach defined by Wei et al. (1995).

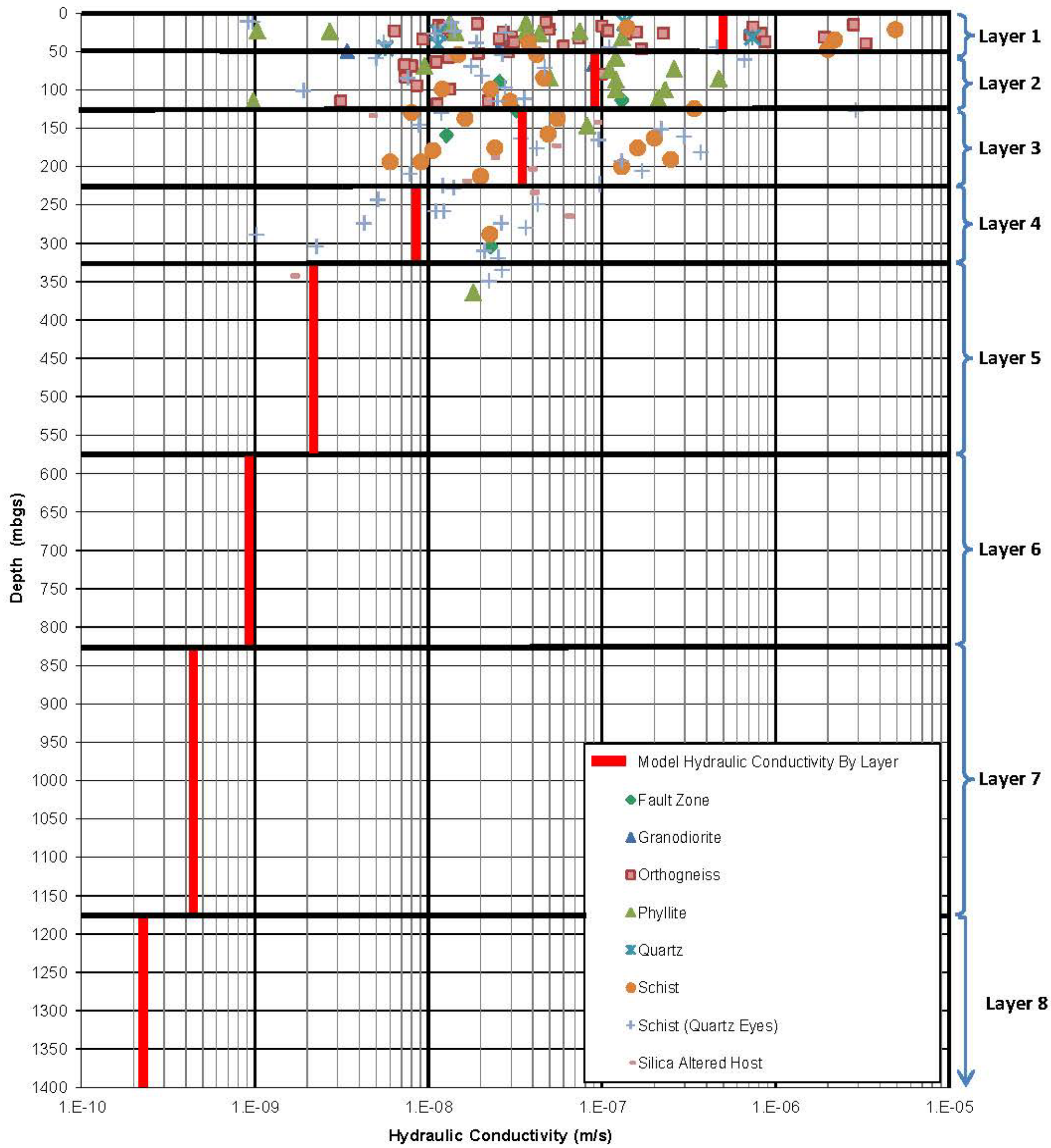


Figure 3.3 Comparison of Modeled and Observed Hydraulic Conductivity vs. Depth

3.5 BASELINE MODEL BOUNDARY CONDITIONS

Boundary conditions are used to specify groundwater sources and sinks in the model domain. The boundary conditions used to define the active model domain are shown on Figure 3.1 and include:

1. No-flow boundaries
2. Constant head boundaries
3. Drain cells to represent creeks, and
4. Meteoric recharge.

3.5.1 No Flow Boundary Conditions

Most of the perimeter of the active model domain is defined by no-flow boundary conditions that correspond to the inferred groundwater divides at the Harper Creek, Jones Creek, Baker Creek, and the Barrier River watershed boundaries. Of the total 577,280 grid cells, approximately 127,000 are no-flow boundary cells. No-flow cells are specified as inactive and are excluded from the groundwater flow calculations within the model. The locations of the no-flow cells are shown on Figure 3.1 for Layer 1 and are the same in all layers of the model.

3.5.2 Constant Head Boundary Conditions

Constant head boundary conditions were specified along the northern and southern boundaries of the model domain to represent the North Thompson River and North Barriere River/Barriere Lake, respectively. The stage assigned to a constant head cells used to represent the North Thompson and Barriere Rivers was set equal to the ground surface elevation for a given cell based on the top elevation of model Layer 1. Constant head cells representing North Barrier Lake were assigned a stage of 630 masl. The constant head boundaries shown for Layer 1 on Figure 3.1 are only in layer 1 of the model.

3.5.3 Drain Boundary Conditions

Harper, Jones and Baker Creeks, the Barriere River and their tributaries were modelled using drain boundary cells. Drain cells allow groundwater to be removed from the model surface where the simulated piezometric head is higher than a user defined drain stage. Drain cells were specified in the model using a GIS shapefile of TRIM river data. Drain stages were set equal to 1 m below the ground surface elevation along the stream channels in Layer 1.

The rate of flux into a drain cell is dependent on a conductance coefficient. Conductance values were estimated using the following formula:

$$C = L * w * K / t$$

Where:
C = conductance of streambed
L = length of stream in cell
w = width of streambed
K = hydraulic conductivity of streambed
t = thickness of streambed

Conductance values were calculated using the estimated hydraulic conductivity of the underlying materials and the drain cell dimensions and were varied slightly during model calibration in order to obtain a best fit to measured streamflow and groundwater elevation data. Conductance values ranged from 10 m^2/day to 50 m^2/day.

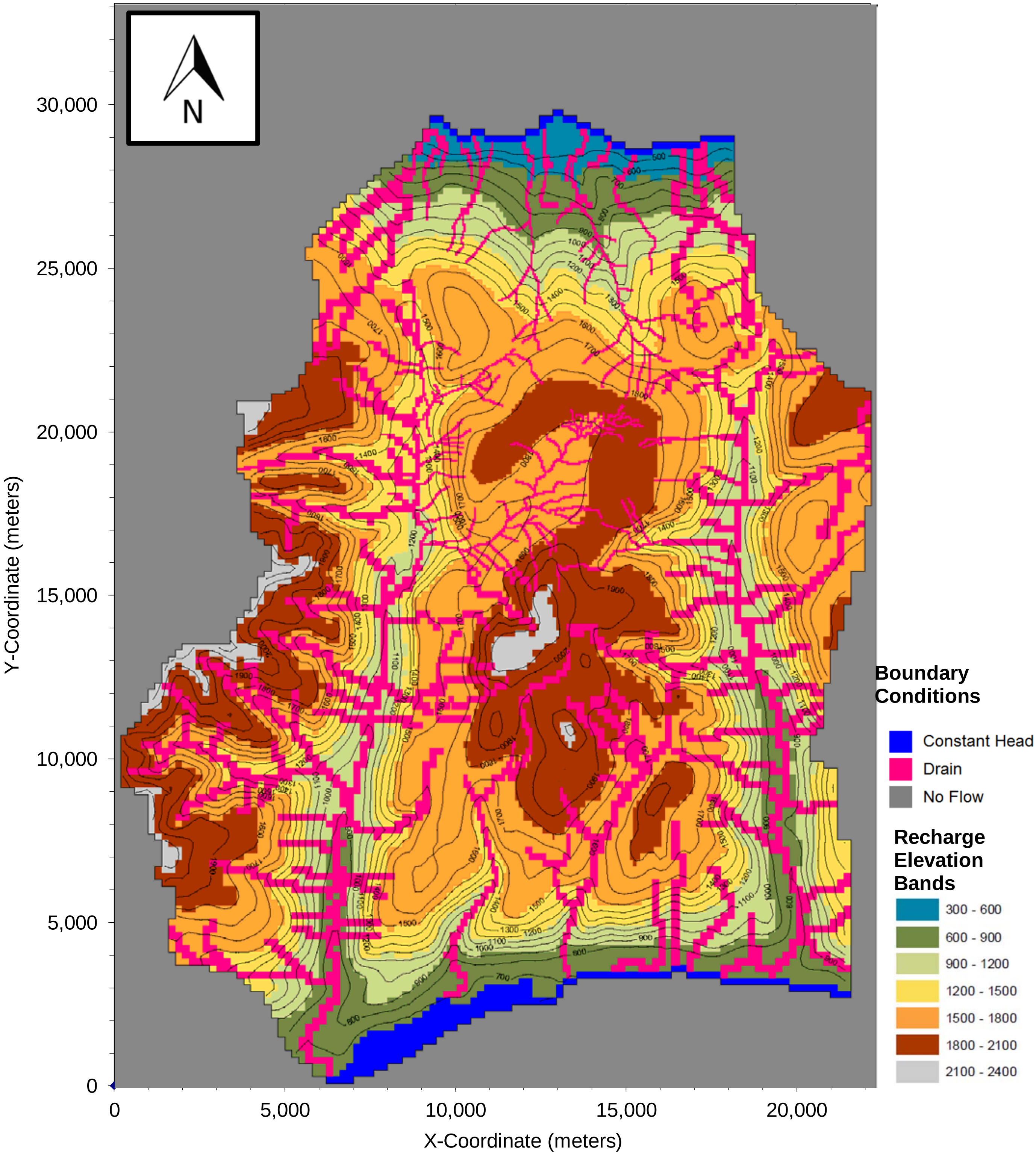
The use of drains to simulate streams does not allow for recharge from stream to aquifer to be simulated. There may be local areas of stream loss to the groundwater system in the study area, but these are expected to be localized and would not have a noticeable effect on the areas around the mine facilities.

3.5.4 Meteoric Recharge

Meteoric recharge was applied to the water table and was specified using seven recharge zones discretized based on ground surface elevation. The model domain was divided into seven 300 m elevation bands covering a range from 300 masl to 2400 masl. A groundwater recharge rate was calculated for the mid-point of each elevation band based on watershed modelling completed for the project (KP, 2014c). These values were applied to the model such that groundwater recharge increases as ground surface elevation increases in each subsequent elevation band. This approach considers orographic effects on groundwater recharge by applying a higher recharge rate to higher elevation regions of the models where the net precipitation is greater and lower recharge in low-lying regions where the precipitation is less. The elevation-based groundwater recharge zones and the recharge rates applied to each zone are presented on Figure 3.4 and in Table 3.2, respectively. The average groundwater recharge rate applied to the model domain was approximately 114 mm/year or 13% of the MAP.

Table 3.2 Groundwater Recharge Rate by Elevation Band

Recharge Elevation Band (masl)	Mean Annual Precipitation (mm/yr)	Groundwater Recharge (mm/year)	Groundwater Recharge (% of MAP)
300 - 600	562	64	11%
600 - 900	651	79	12%
900 - 1200	753	95	13%
1200 - 1500	872	112	13%
1500 - 1800	1009	127	13%
1800 - 2100	1168	150	13%
2100 - 2400	1353	175	13%
Area Weighted Average	**910**	**114**	**13%**



NOTES:
1. THE CONTOURS SHOWN ABOVE ARE OF APPROXIMATE GROUND SURFACE ELEVATION (masl)

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BASELINE MODEL
OROGRAPHIC GROUNDWATER
RECHARGE ZONES

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FIGURE 3.4	REV 0

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3.6 BASELINE MODEL CALIBRATION

The baseline model was calibrated using an iterative trial-and-error method in order to refine the match between model predictions and observed pre-development conditions at the site. Hydraulic conductivity and groundwater recharge rates were the primary calibration parameters varied during the calibration process. These parameters were systematically varied to achieve the best match to the calibration targets, average hydraulic head measurements in monitoring wells and estimates of average annual baseflow conditions at the hydrology stations within the study area. The locations of the groundwater elevation targets and hydrometric stations are shown on Figure 1.2 and Figure 2.1, respectively.

The PCG-5 solver was used to solve the groundwater flow equations in MODFLOW-SURFACT, with the following solver parameters:

- Number of outer iterations: 300
- Number of inner iterations:600
- Maximum orthogonalizations: 10, and
- Maximum head change criterion: 0.001 meters.

The baseline model converged in less than 50 outer iterations with an overall mass balance error of 4E-4 %.

3.6.1 Hydraulic Head Targets

The baseline model was calibrated to hydraulic head measurements recorded by continuous data loggers at 21 monitoring well locations across the Project area, as shown on Figure 1.2. These are the locations of the hydraulic head targets used in the model. The measured groundwater levels used as calibration targets in the model area were extracted from the continuous water-level data series and were recorded on February 25, 2013 at 00:00 (midnight). These measurements are believed to be representative of groundwater elevations under low-flow (baseflow) conditions, as discussed in Section 3.6.2. Furthermore, this period corresponds to a site visit where many manual water-level measurements were collected, and were available to check the data logger records for accuracy. A summary of the measured and simulated hydraulic heads at the model calibration targets is provided in Table 3.3 and on Figure 3.5. The primary calibration criterion was to achieve a normalized root mean squared error (NRMSE) of 5% (0.05) or less for hydraulic head targets. After calibration, a NRMSE of 1.1% (0.011) was achieved, satisfying the calibration criterion. All of the simulated hydraulic heads are within 10 m of the observed value. The mean absolute error (MAE) for all hydraulic head targets is 4.8 meters.

Table 3.3 Observed and Simulated Hydraulic Heads

Well I.D.	Measured Groundwater Elevation (masl)	Simulated Groundwater Elevation (masl)	Residual Head (m)
MW12-01D	1,701	1,698	3.4
MW12-01S	1,701	1,696	5.3
MW12-02D	1,670	1,674	-3.9
MW12-02S	1,668	1,673	-5.2
MW12-03D	1,830	1,828	1.9
MW12-03S	1,829	1,828	1.1
MW12-04D	1,826	1,827	-1.4
MW12-04S	1,827	1,827	-0.5
MW12-05D	1,341	1,337	4.0
MW12-05S	1,328	1,337	-8.6
MW10-01D	1,584	1,592	-8.0
MW10-01S	1,587	1,585	2.0
MW10-03	1,755	1,748	7.2
MW10-04	1,500	1,505	-4.8
MW11-21D	1,675	1,684	-8.8
MW11-21S	1,674	1,680	-6.0
MW11-22D	1,676	1,670	5.5
MW11-22S	1,679	1,675	3.9
MW11-23D	1,635	1,629	6.1
MW11-23S	1,634	1,627	6.7
		MAE (m)	**4.7**
		RMSE (m)	**5.3**
		NRMSE	**1%**

NOTES:

1. THE VALUES LISTED AS MEASURED GROUNDWATER ELEVATIONS EXTRACTED FROM THE CONTINUOUS WATER-LEVEL DATA LOGGER RECORDS FOR FEBRUARY 25, 2013 AT 00:00 (HH:MM).

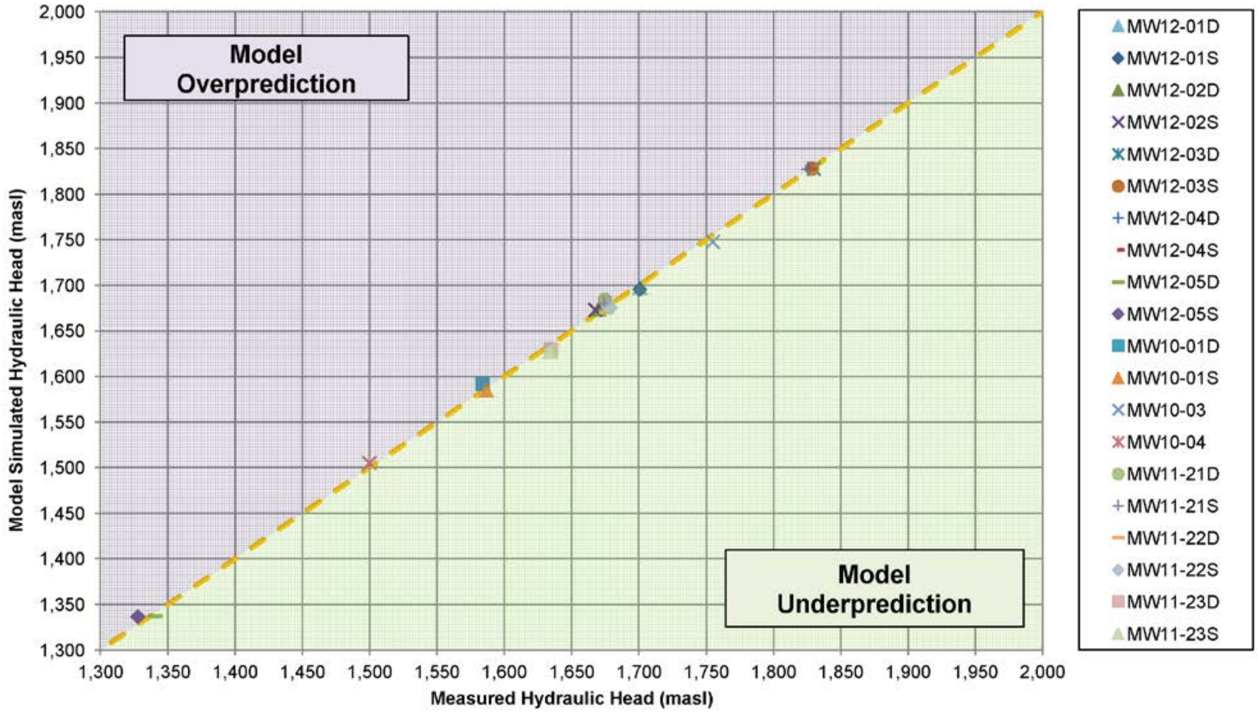


Figure 3.5 Observed vs. Simulated Hydraulic Head

3.6.2 Baseflow Targets

Groundwater discharge to streams provides baseflow that sustains streamflow within major drainages during the winter and early spring months. The baseline model was calibrated to mean monthly streamflow discharge from February of 2013 at five hydrometric stations located within the Harper, Baker and Jones Creek watersheds. Mean monthly streamflow discharge was extracted from synthetic streamflow records developed for the Project (KP, 2014a). February discharge was selected to represent baseflow conditions before the contribution of spring snowmelt occurs (freshet). The baseflow targets used in the model are summarized in Table 3.4.

Mean monthly streamflow rates from the synthetic series for February 2013 were compared with the simulated groundwater discharge to drain boundary cells representing creeks. Drains cells were grouped into reaches corresponding to channel segments draining to one of the five hydrology stations for which synthetic data are available.

Table 3.4 provides a summary of the calibration results for baseflow at the hydrometric stations and Figure 3.6 presents a plot of synthetic versus simulated baseflows. All model simulated baseflows are less than 9% of the target streamflow value and the MAE for all baseflow targets is approximately 10%.

Table 3.4 Baseflow Calibration Targets and Results

Hydrometric Station (Gauge I.D.)	February Synthetic Baseflow Estimate (m^3/day)	MODFLOW Simulated Baseflow (m^3/day)	Residual Baseflow (m^3/day)	Relative Percent Difference (%)
P-Creek Above Harper Creek (OP Gauge)	1,272	1,228	44	3%
Harper Creek Above T-Creek (HARPERUS Gauge)	23,328	20,267	3,061	14%
T-Creek Above Harper Creek (TSFDS Gauge)	10,079	9,103	976	10%
Harper Creek at WSC Station (08LB076 Gauge)	59,616	62,266	-2,650	-4%
Jones Creek Above N. Thompson (JONESUS Gauge)	6,480	5,333	1,147	19%
			Average RPD (%)	**9%**

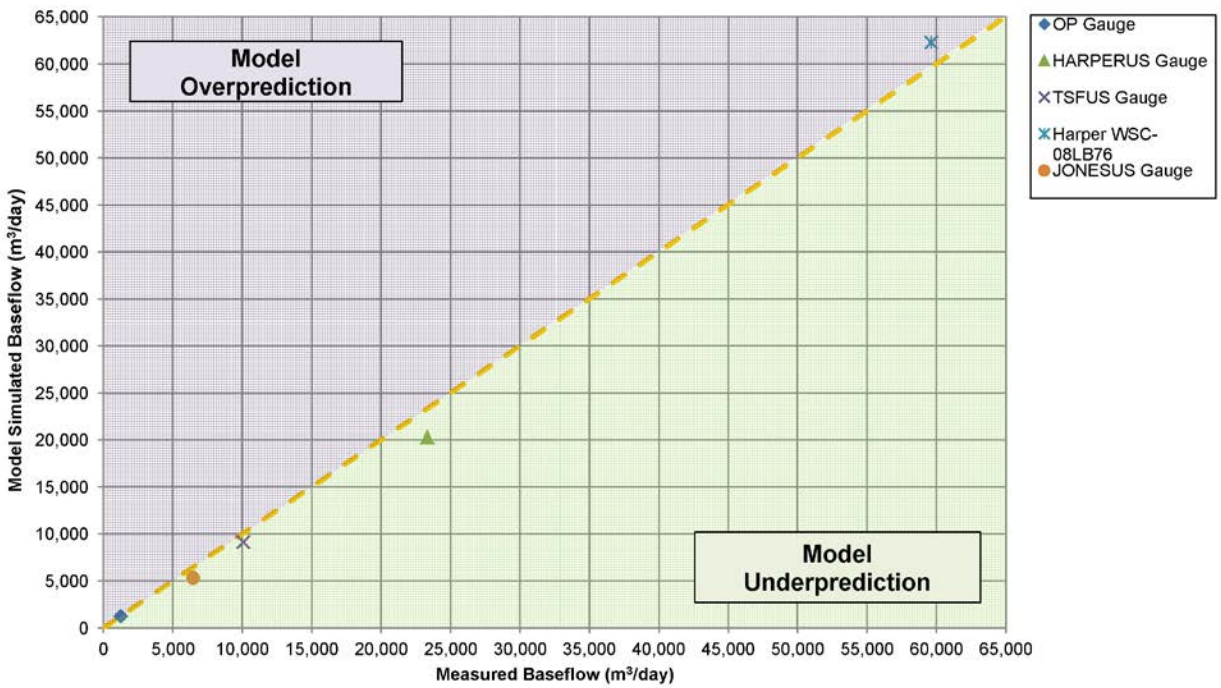


Figure 3.6 Synthetic vs. Simulated Baseflow

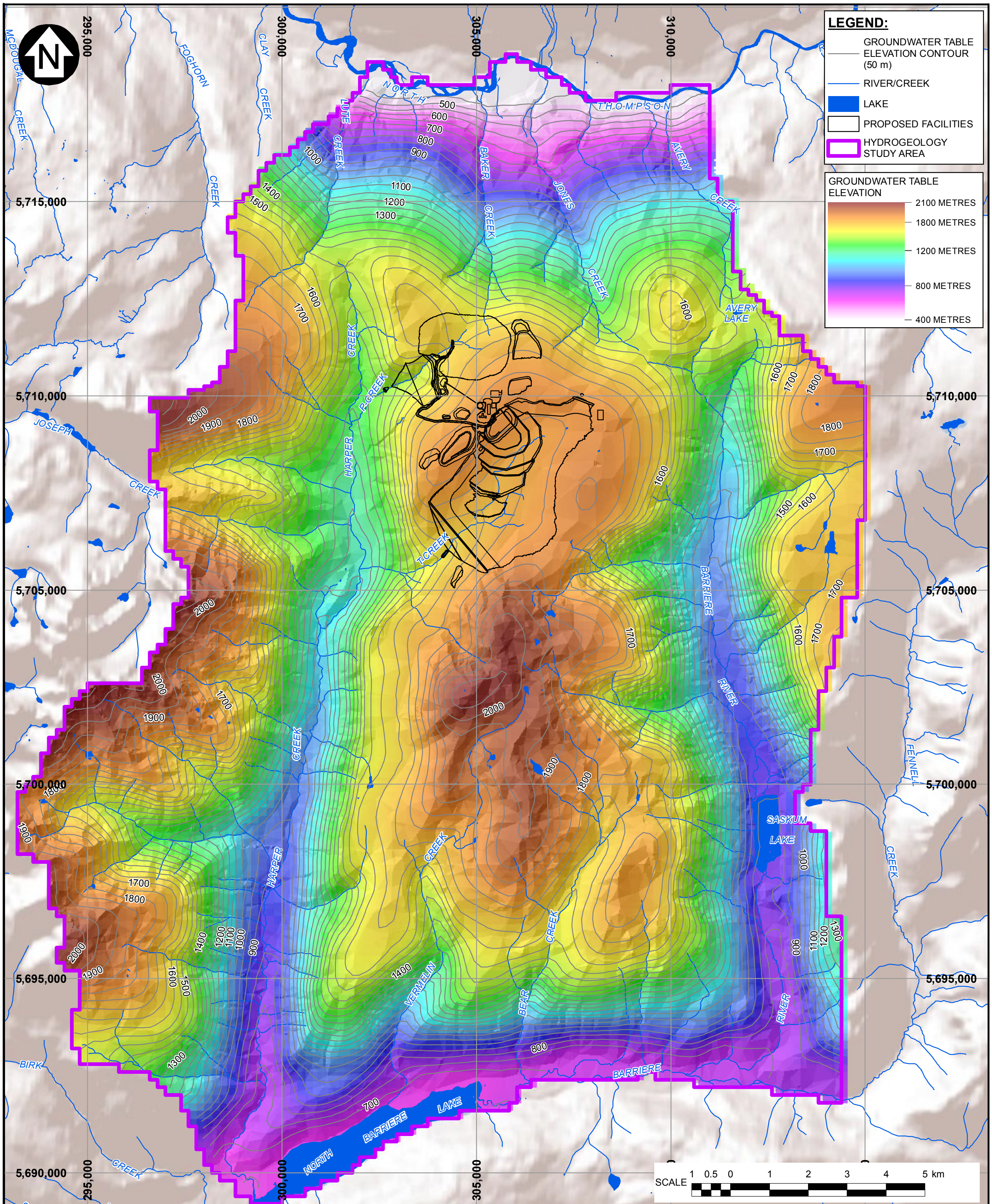
3.7 BASELINE MODEL RESULTS

A water table contour map for the calibrated baseline model is presented in Figure 3.7. The figure shows a continuous water table surface plotted in model Layer 1 that was generated by merging head data sets from multiple model layers. Head data from lower layers was used in regions were the simulated water table resides below Layer 1. This approach provides a top-down look at the water table for a multi-layer water table simulation.

The simulated water table generally mimics the surface topography with groundwater elevations ranging from 2,000 masl in the high elevation region to the south of the mine site to 650 masl and 500 masl at the downstream extents of Harper Creek and Baker/Jones Creeks, respectively.

A plan view of groundwater flow directions in model Layer 1 is presented on Figure 3.8. Red arrows on Figure 3.8 indicate where groundwater flow has a predominantly downward vertical component of flow (recharge) and blue arrows indicate where there is a predominantly upward vertical component of flow (discharge). The figure illustrates that groundwater recharge occurs within topographic highs and groundwater flows downslope to discharge zones in creek valleys. The predicted water table and flow directions from Figures 3.7 and 3.8 were combined on Figure 3.9 to provide a larger-scale view of groundwater flow and elevation within the Project area.

Cross-sections depicting simulated groundwater flow directions along with surface water drainages are presented on Figure 3.10.



NOTES:

1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:140,000 FOR 8.5x11 (LETTER) PAPER.

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BASELINE MODEL REGIONAL GROUNDWATER TABLE ELEVATION

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FIGURE 3.7 REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	CHK'D	APP'D
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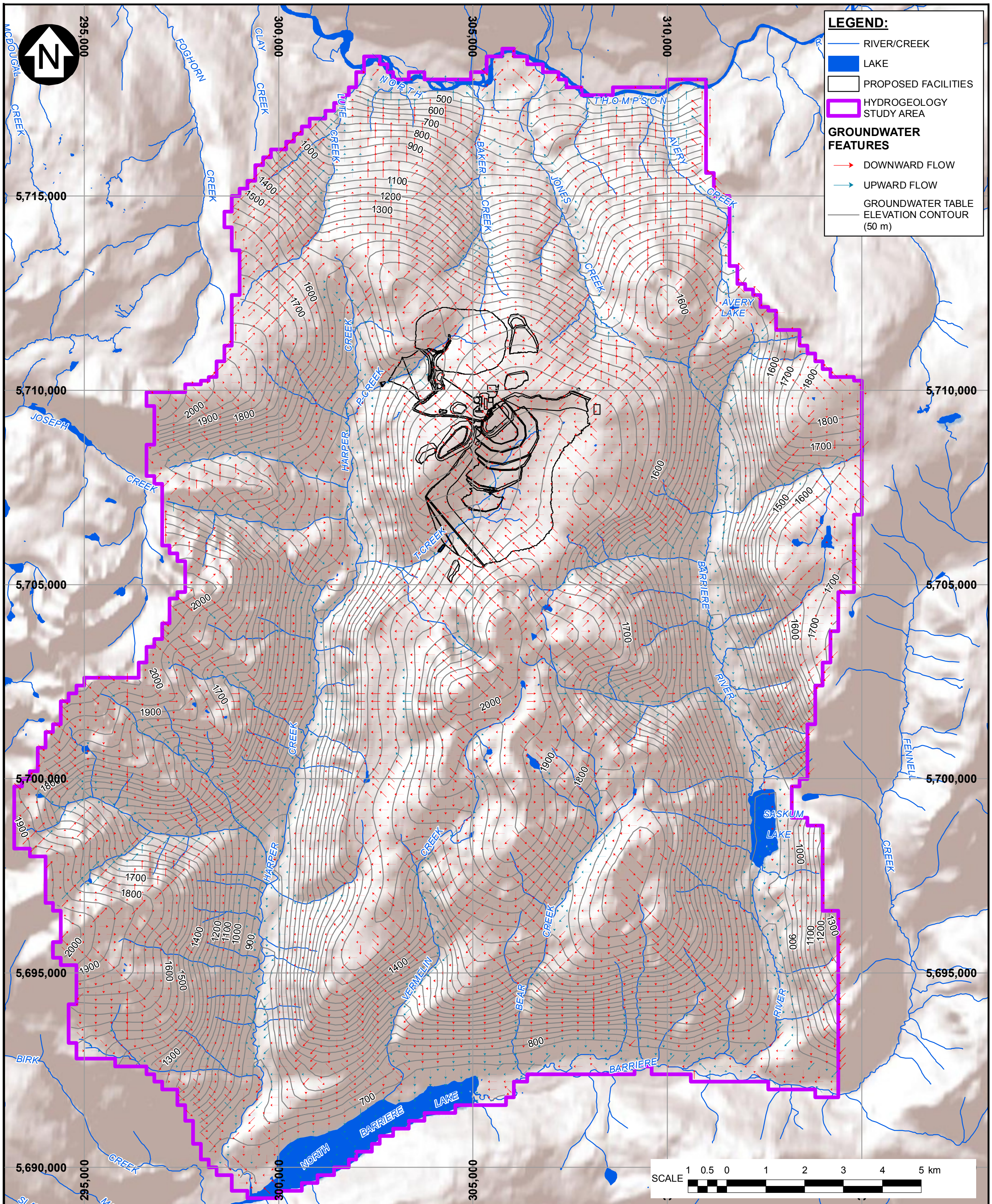

NOTES:

1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:140,000 FOR 8.5x11 (LETTER) PAPER.
4. THE GROUNDWATER FLOW DIRECTION VECTORS SHOWN WERE GENERATED IN GIS SOFTWARE USING CELL DATA EXPORTED FROM THE MODEL.

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	CHK'D	APP'D
0	10OCT'14	ISSUED WITH REPORT	KTD	CAC	RS	KJB

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BASELINE MODEL GROUNDWATER FLOW DIRECTION (PLAN VIEW)

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FIGURE 3.8 REV 0

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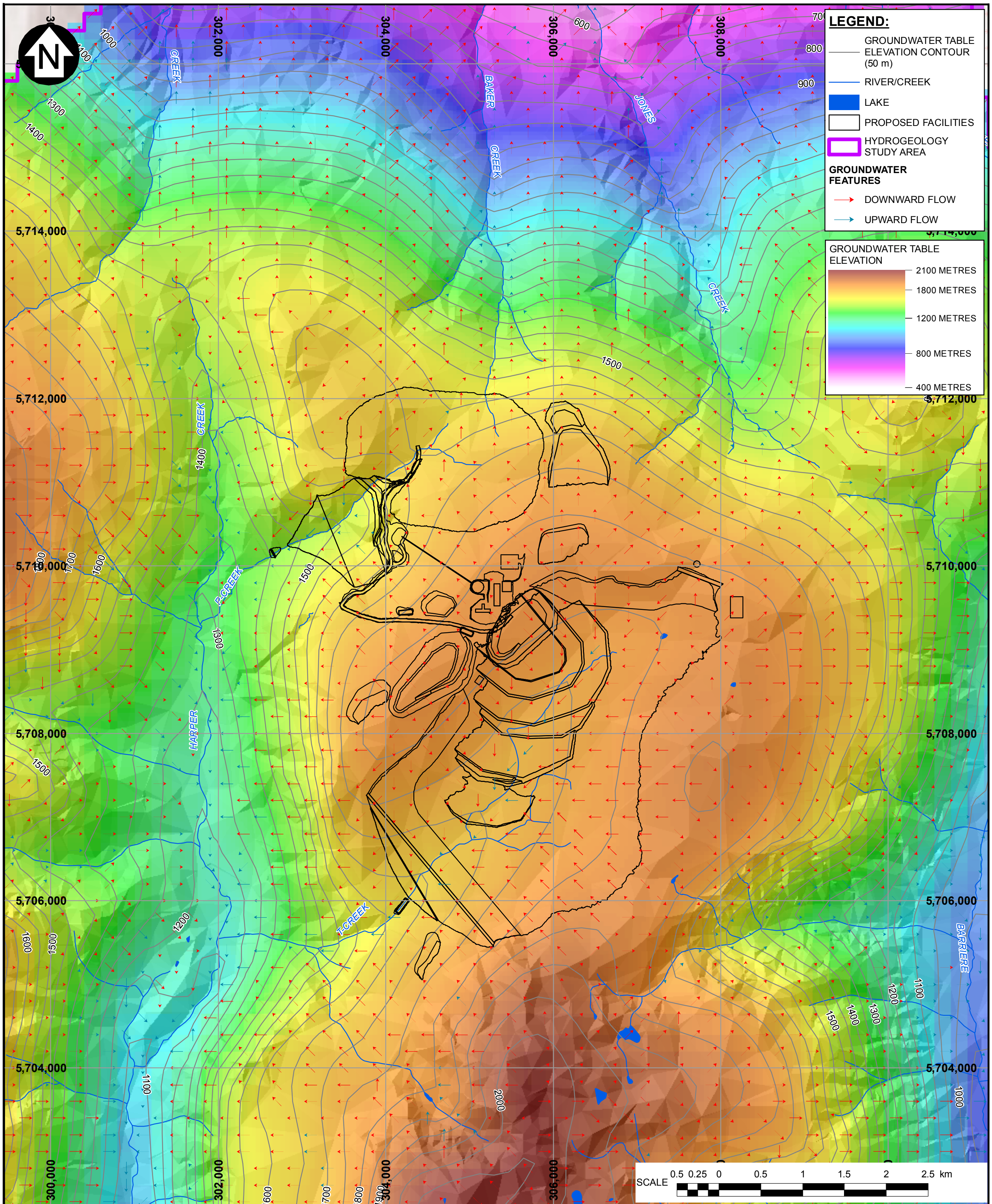

NOTES:
1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:65,000 FOR 8.5x11 (LETTER) PAPER.

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BASELINE MODEL PROJECT SITE
GROUNDWATER TABLE ELEVATION

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REF NO. 2
FIGURE 3.9
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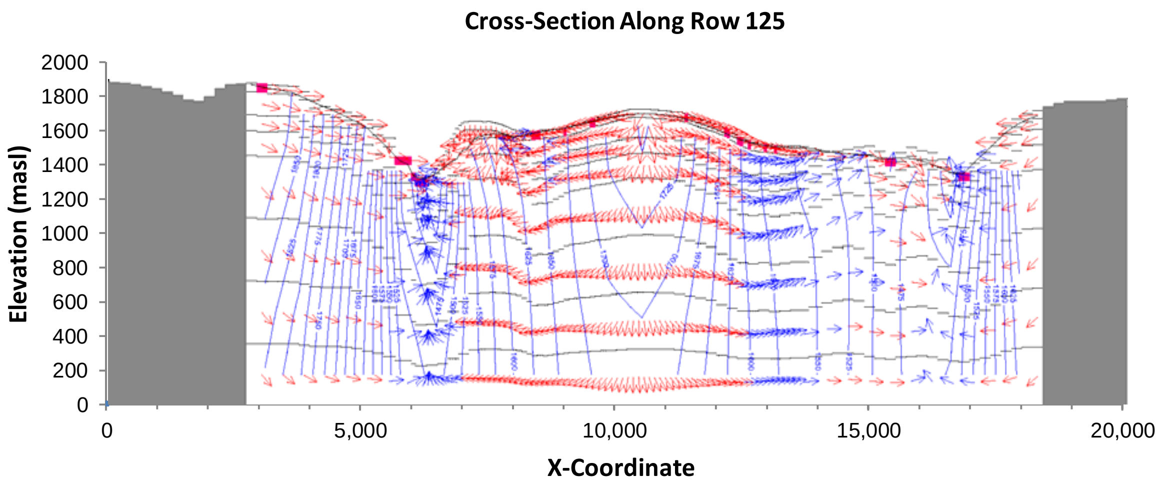

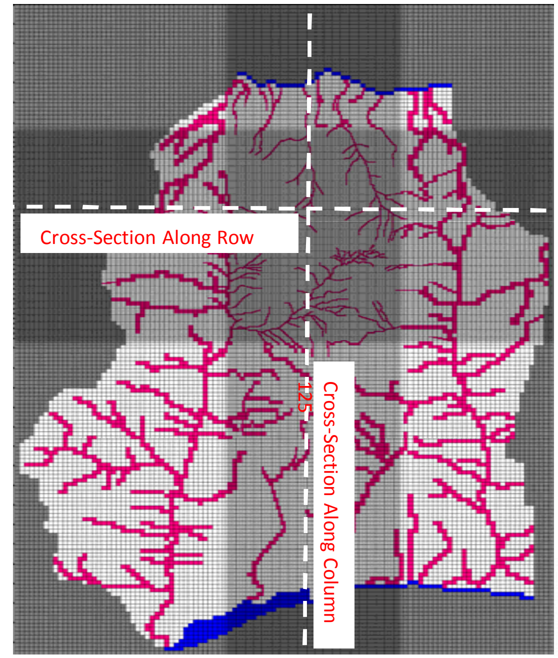

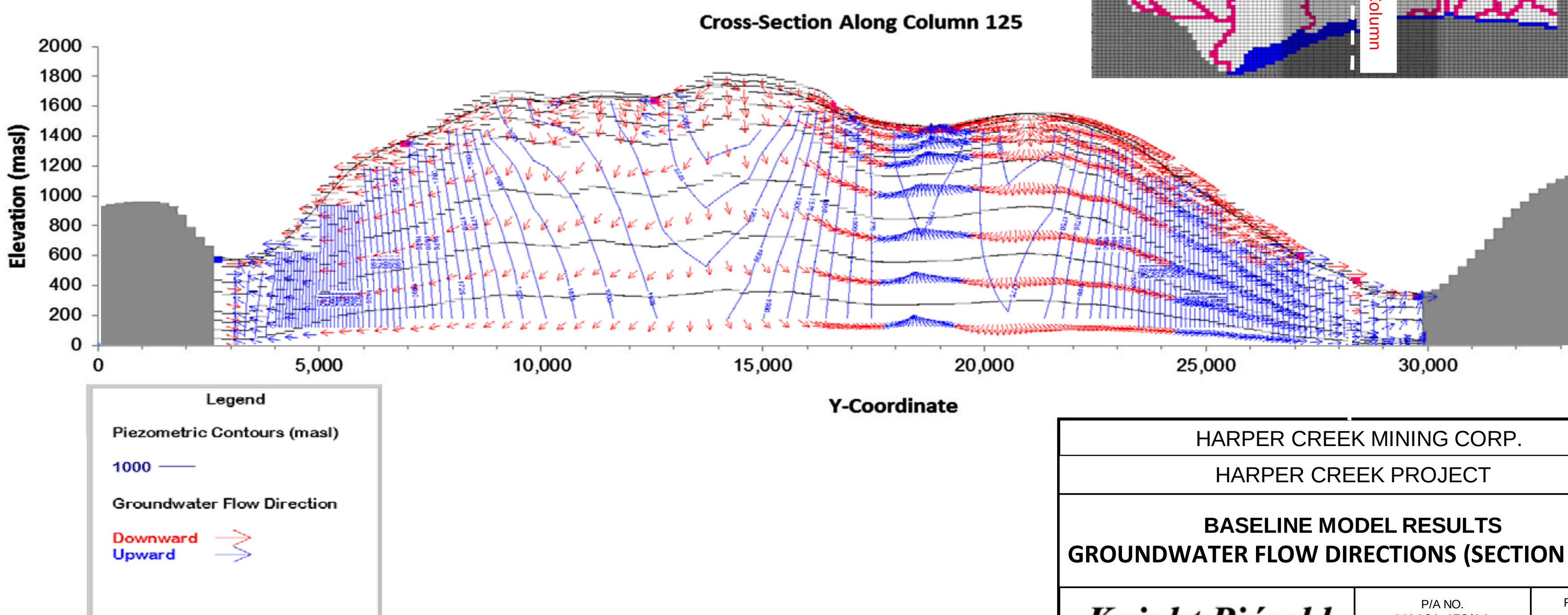

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BASELINE MODEL RESULTS
GROUNDWATER FLOW DIRECTIONS (SECTION VIEW)

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FIGURE 3.10

REV 0

4 – MINE OPERATIONS SIMULATION

4.1 OVERVIEW

A steady-state Operations Model was completed to simulate potential effects of the Harper Creek Project on pre-development hydrogeological conditions. The Operations Model was developed from the calibrated baseline groundwater model using MODFLOW-SURFACT and Groundwater Vistas.

The main objectives of the model were to:

- Characterize potential effects of mine facilities on baseline hydrogeology
- Estimate groundwater inflow rate to the Open Pit at ultimate pit extents, and
- Characterize potential seepage pathways from key mine facilities.

The results of the Operations Model along with the methodology and assumptions used to develop the model are presented in the sections that follow.

4.2 MODEL CONSTRUCTION

4.2.1 Operations Model Geometry, Layering and Grid

The model geometry, layering and numerical grid remained unchanged from the Baseline Model. The model grid and model layers for the Operations Model are shown on Figure 4.1. Hydraulic heads from the calibrated Baseline Model were set as the initial heads for the Operations Model.

4.2.2 Hydraulic Conductivity

The hydraulic conductivity values assigned to Baseline Layers 1 through 5 remained unchanged from the baseline model. An additional hydraulic conductivity zone was assigned to the Operations Model to define the TMF embankment in model Layer 1. A hydraulic conductivity value of $1x10^{-7}$ m/s was assigned to cells representing the low permeability core zone for the TMF embankments and the rockfill shell of the embankment was assigned a hydraulic conductivity of $1x10^{-5}$ m/s. No hydraulic conductivity zone was assigned to tailings and waste rock within the TMF. Hydraulic conductivity of these materials is controlled by the river boundary conditions used to simulate the TMF supernatant pond, as discussed in Section 4.2.3.

4.2.3 Operations Model Boundary Conditions

Boundary conditions assigned to the Operations Model remained unchanged from the Baseline Model except where mine facilities are proposed. Details on the boundary conditions assigned to each mine facility are provided below. Boundary conditions assigned to Layer 1 of the Operations Model are shown on Figure 4.1.

4.2.3.1 Tailings Management Facility (TMF)

The TMF Pond was represented using river boundary (RIV) cells defined in model Layer 1 within the pond footprint. River boundaries allow inflow to or outflow from the model domain based on the difference between simulated hydraulic head and a user defined stage elevation. A stage elevation of 1,834 masl was assigned to the RIV cells of the TMF, which is the maximum design elevation of the supernatant pond.

M:\1\01\00458\14\A\Report\2 - Numerical Groundwater Modelling\Rev 0\Figures\[Rev A Excel Figures.xls]FIGURE 4.1

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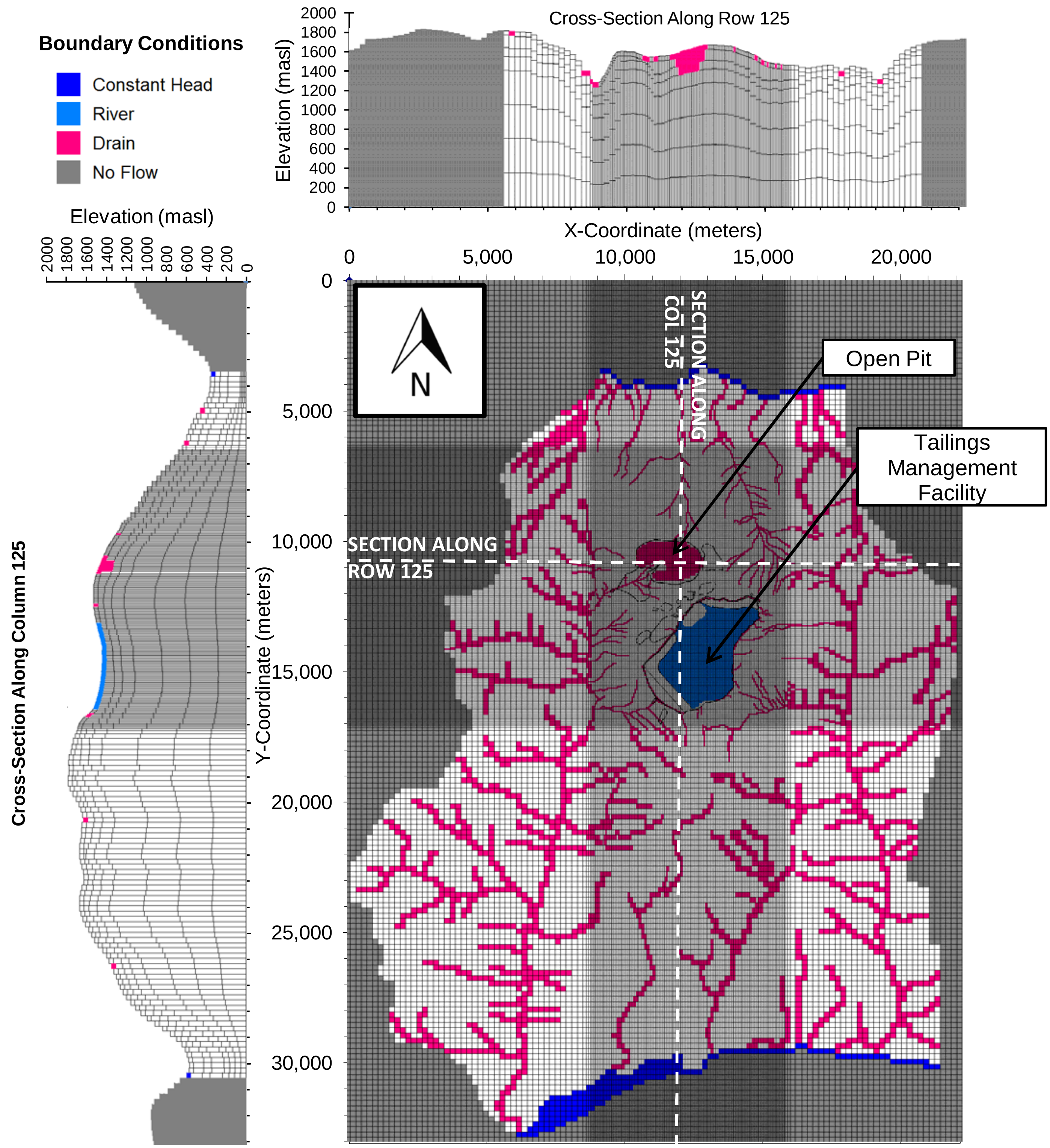

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HARPER CREEK PROJECT

OPERATIONS MODEL NUMERICAL GRID, MODEL LAYERS AND BOUNDARY CONDITIONS

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FIGURE 4.1

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The rate of flux into or out of a river cell is dependent on a conductance coefficient. Conductance values were estimated using the following formula:

$C = L * w * K / t$

Where:

C = conductance of underlying tailings or waste rock

L = length dimension of cell

w = width dimension of cell

K = hydraulic conductivity of underlying tailings or waste rock

t = thickness of underlying tailings or waste rock (tailings or waste rock elevation minus the natural ground surface elevation for a given cell location)

A hydraulic conductivity of 1E-7 m/s was assigned to river cells overlying tailings material within the TMF, and 1E-4 m/s was assigned to the river cells overlying the submerged PAG Waste Rock Stockpile.

Simulated storage of water and tailings in the TMF increases the piezometric pressures under the pond. This naturally leads to discharge of groundwater both under the pond and to the undisturbed upgradient areas adjacent to the pond. In the model, however, this creates flooded cells (simulated groundwater elevation greater than ground surface elevation) up-gradient from the TMF. Conceptually, this discharge would increase surface water run-on into the TMF supernatant pond. Drain boundary conditions were added along the up-gradient shores of the TMF pond in order to simulate discharge to the adjacent shore areas and to eliminate flooding up-gradient from the TMF. Drain stages were defined at ground surface elevation and a high conductance value was assigned to allow water to drain freely.

4.2.3.2 Open Pit

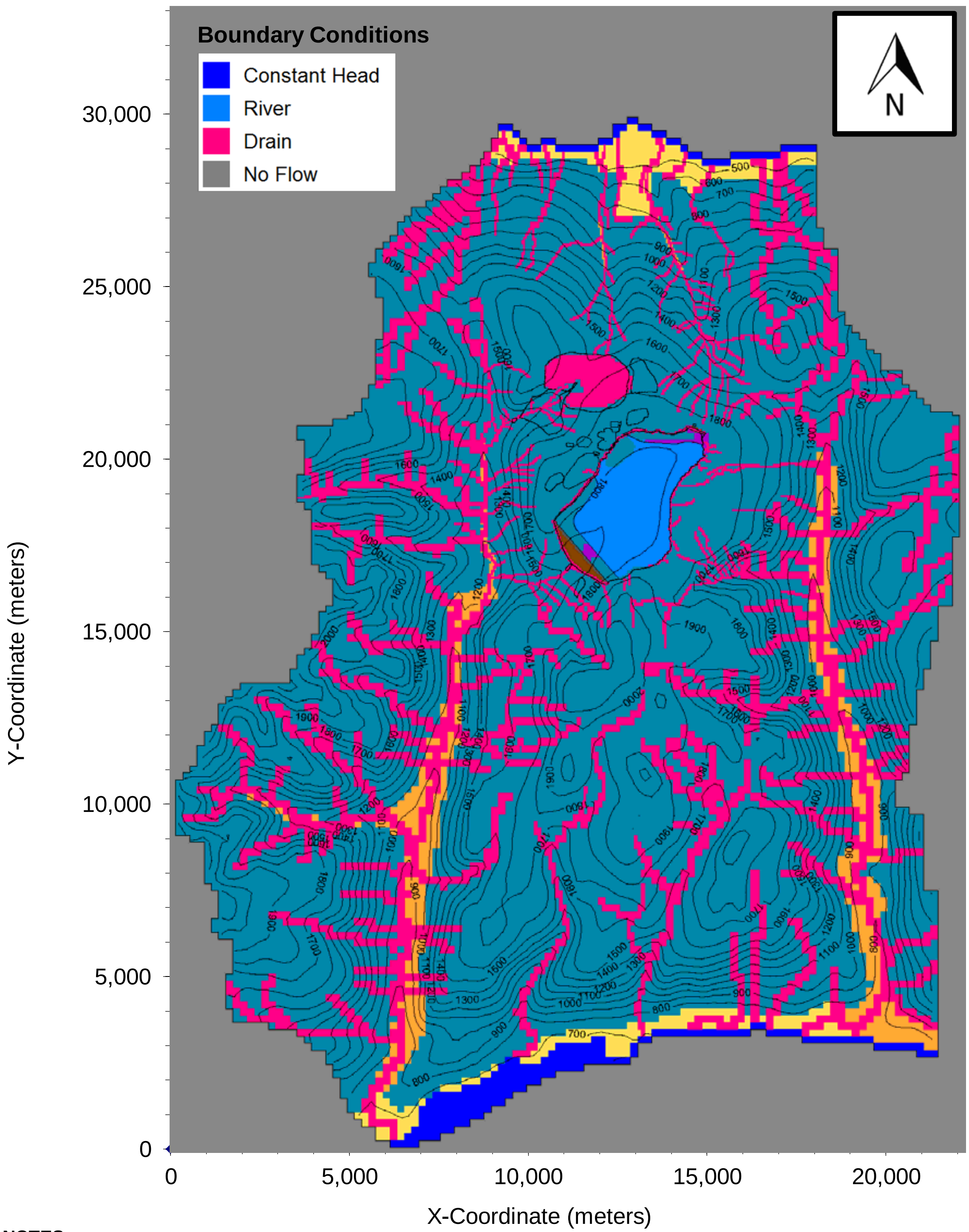
Drain cells were specified in the open pit area of the model to simulate operational dewatering during active mining. An operational pit shell for the maximum extents of the open pit (Year 24) was used to assign drain cells within model Layers 1 through 4 of the model. Drain stage elevations were set equal to the elevation of the pit shell at a given cell location. Drain conductance was assigned a value high enough to allow water to drain freely into the open pit while still minimizing mass balance error and preventing convergence issues (10 m^2/day).

4.2.3.3 Groundwater Recharge

The spatial distribution and rate of groundwater recharge for areas undisturbed by proposed mine facilities remained unchanged from the baseline model. Changes to the specified groundwater recharge boundary condition in the model were made for the following mine components:

- Non-PAG Waste Rock Stockpile
- Low-Grade Ore Stockpiles
- tailings beach and embankments, and
- tailings pond.

A recharge rate of 175 mm/year was applied to the tailings beach and embankment in the Mine Operations model. A recharge rate of 150 mm/yr was assigned to the Low-Grade Ore Stockpiles and the Non-PAG Waste Rock Stockpile. No recharge was assigned to the TMF pond footprint as recharge to the pond area is controlled by the RIV cells of the supernatant pond.



NOTES:

1. THE HYDRAULIC CONDUCTIVITY ZONES SHOWN ABOVE ARE FROM MODEL LAYER 1.

Zone	Color	Material Type
Zone 1		Weathered Fractured Bedrock
Zone 6		Glaciofluvial Deposits
Zone 7		Glacial Lake Deposits
Zone 8		Fluvial Deposits

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OPERATIONS MODEL
HYDRAULIC CONDUCTIVITY ZONES

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REF. NO. 2

FIGURE 4.2

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M:\1\01\00458\14\A\Report\2 - Numerical Groundwater Modelling\Rev 0\Figures\[Rev A Excel Figures.xls]FIGURE 4.3
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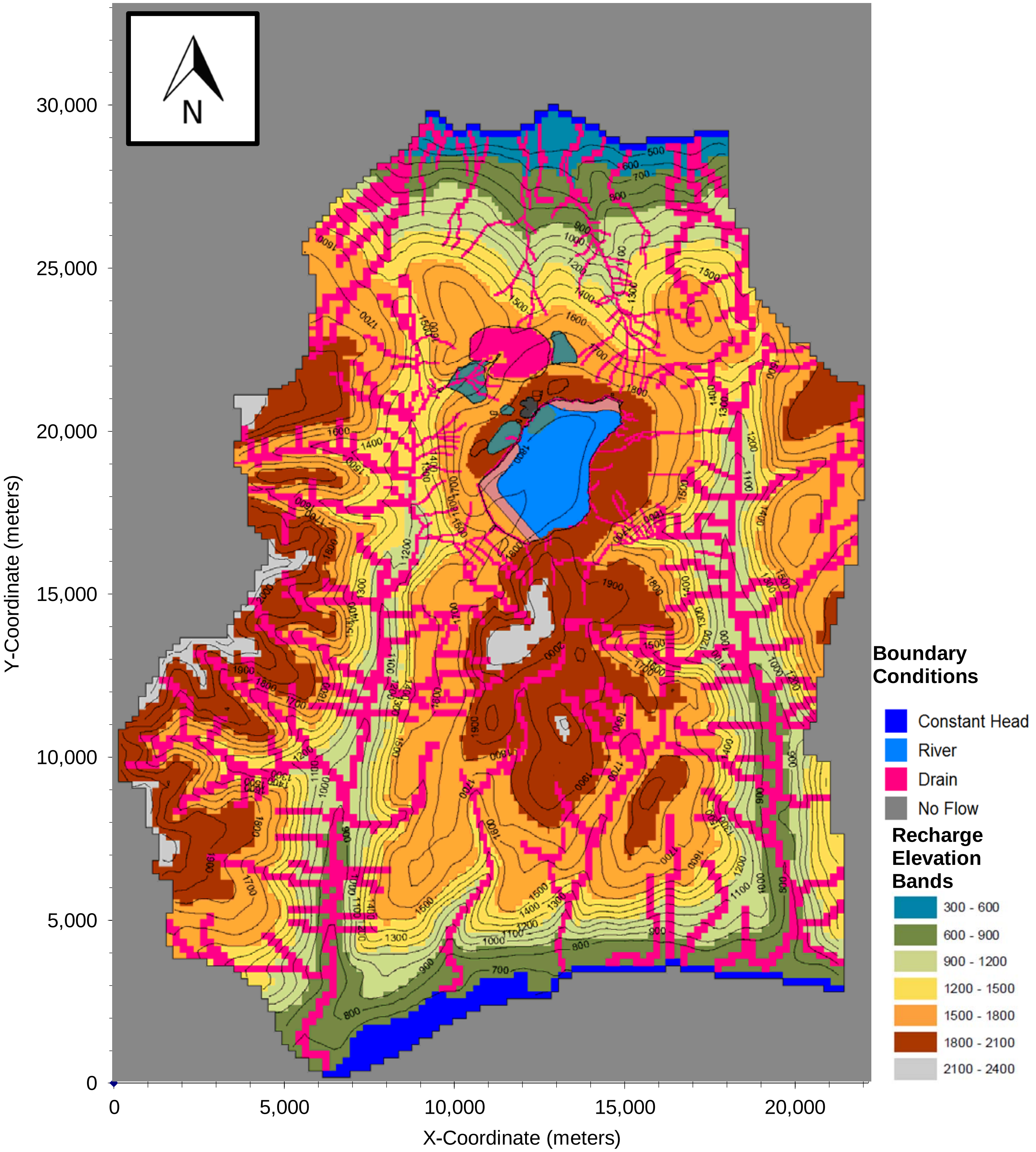

NOTES:
1. THE CONTOURS SHOWN ABOVE ARE OF APPROXIMATE GROUND SURFACE ELEVATION (masl)

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OPERATIONS MODEL
OROGRAPHIC GROUNDWATER
RECHARGE ZONES

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FIGURE 4.3

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0	10OCT'14	ISSUED WITH REPORT	KTD	DDF	KJB

4.3 OPERATIONS MODEL RESULTS

The major mine components of the proposed Harper Creek Project are expected to have localized effects on groundwater elevation and flow direction within the Project area. A simulated water table contour map representing the predicted water table corresponding to the end of active mine dewatering (Year 24) is presented on Figure 4.4. Groundwater flow directions for Layer 1 in the study area are shown in plan view on Figure 4.5. Comparison of water table elevations representing baseline conditions (Figure 3.7) indicates that groundwater elevations directly surrounding the TMF supernatant pond are predicted to increase to an operational maximum of 1834 masl in Year 24, as supernatant water elevations in the TMF increase.

The predicted water table and flow directions from Figures 4.4 and 4.5 were combined on Figure 4.6 to provide a larger-scale view of groundwater flow and elevation within the Project area.

Hydraulic gradients and groundwater flow directions in proximity to the Main Embankment as shown on Figure 4.6 indicate the potential for seepage loss through the Main Embankment and foundation materials. To the north of the TMF, flow directions are indicitive of the potential for seepage discharge through the North Embankment. A mass balance analysis of the TMF supernatant pond indicates that approximately 13 L/s seepage is expected in Year 24 of Operations. This agrees with the results of a 2-dimensional SEEP/W seepage analysis completed for the project (KP, 2014b), which is provided in Appendix B. Additional analysis of potential seepage pathways from the TMF is provided in Section 6.2.2.1.

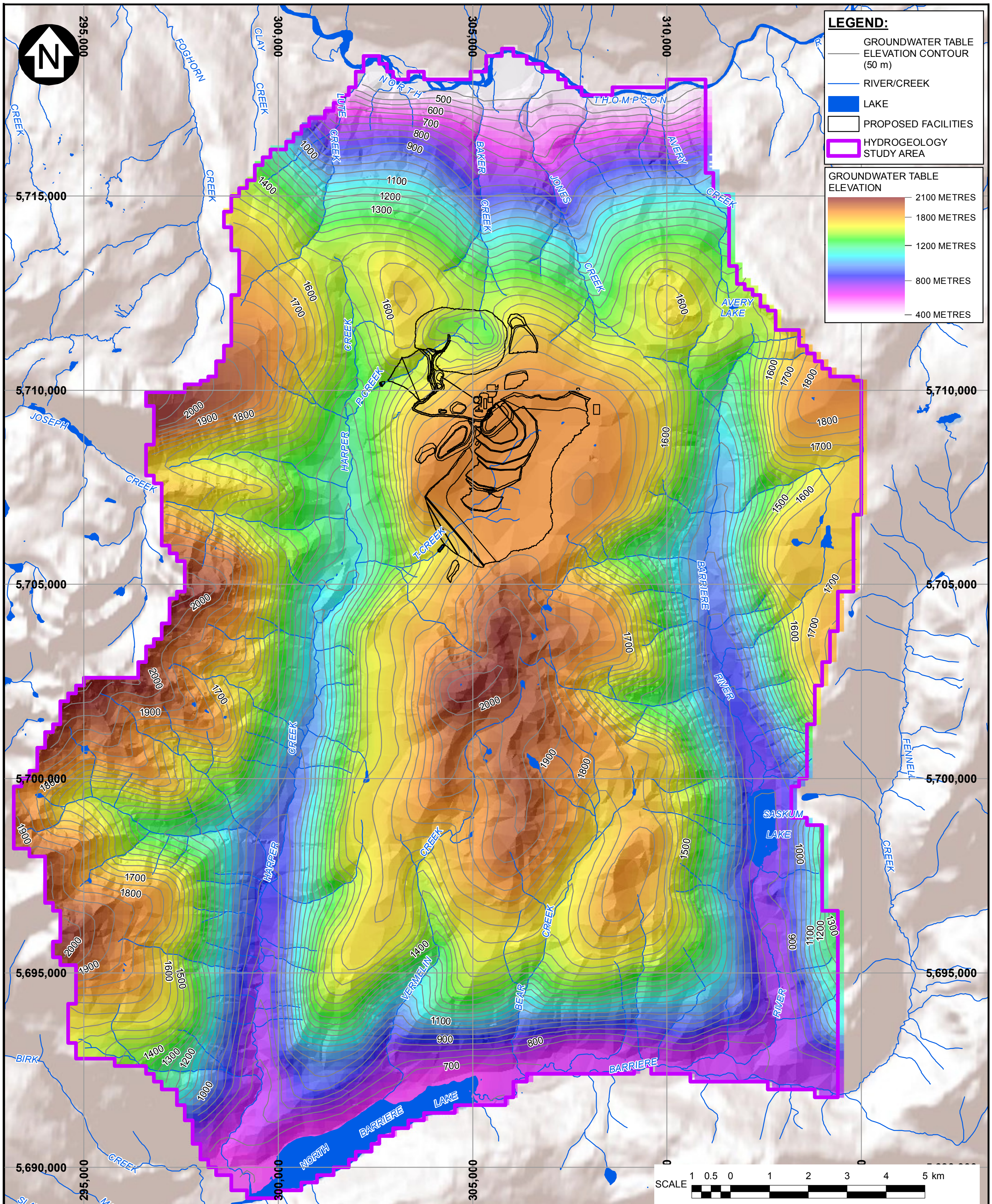
The Operations Model was used to assess the extent of groundwater drawdown surrounding the Open Pit, to delineate the groundwater capture zone, and estimate the rate of groundwater inflow. Groundwater elevation surrounding the Open Pit is predicted to decrease by up to 350 meters as the pit is excavated and dewatered. Local groundwater drawdown associated with operational dewatering of the Open Pit is shown on Figure 4.7. The 1 meter drawdown contour extends approximately 1 km south from the pit rim towards the TMF and approximately 3 km north from the pit rim towards Baker Creek and the North Thompson River.

Simulation results indicate that groundwater inflow to the Open Pit will occur at a rate of approximately 16 L/s at ultimate extents. MODPATH particle tracking (Pollok, 1994) was used to delineate the groundwater capture zone for the Open Pit. The resulting capture zone is shown on the inset of Figure 4.7. Inward groundwater flow to the Open Pit is expected within the capture zone. The majority of groundwater inflow to the pit comes from up-gradient catchment areas southeast and northwest of the Open Pit with a small portion originating from the foundation of the Non-PAG Waste Rock Stockpile. Additional discussion of the MODPATH procedure used to delineate the capture zone is provided in Section 6.3.

Table 4.1 presents a comparison of simulated baseflow from the Operations and Baseline models. The greatest percent baseflow reductions during Operations occur in P-Creek and T-Creek with flow reductions of 86% and 60%, respectively. Flow reductions in P-Creek are caused by reduction of contributing watershed area due to operational dewatering of the Open Pit and water collection at the Non-PAG Waste Rock and Low-Grade Ore Stockpiles in the P-Creek sub-catchment. Similarly, baseflow is reduced in lower T-Creek as a result of water retention within the TMF. These flow reductions are carried downstream into the Harper Creek watershed for the HARPERUS and WSC gauges, as shown in Table 4.1. Baseflow is reduced in Baker Creek, and to a lesser extent in Jones Creek, as a result of excavation and dewatering of the Open Pit.

Table 4.1 Operations Comparison of Baseflow at Hydrometric Stations

Hydrometric Station (Gauge I.D.)	Baseline Simulated Baseflow (m^3/day)	Operations Simulated Baseflow (m^3/day)	Flow Reduction Percentage (%)
P-Creek Above Harper Creek (OP Gauge)	1,228	177	86%
Harper Creek Above T-Creek (HARPERUS Gauge)	20,267	16,745	17%
T-Creek Above Harper Creek (TSFDS Gauge)	9,103	3,629	60%
Harper Creek at WSC Station (08LB076 Gauge)	62,266	47,289	24%
Baker Creek Above N. Thompson (BAKER Gauge)	2,976	2,009	32%
Jones Creek Above N. Thompson (JONESUS Gauge)	5,333	4,983	7%



NOTES:
1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:140,000 FOR 8.5x11 (LETTER) PAPER.

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OPERATIONS MODEL REGIONAL GROUNDWATER TABLE ELEVATION

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FIGURE 4.4

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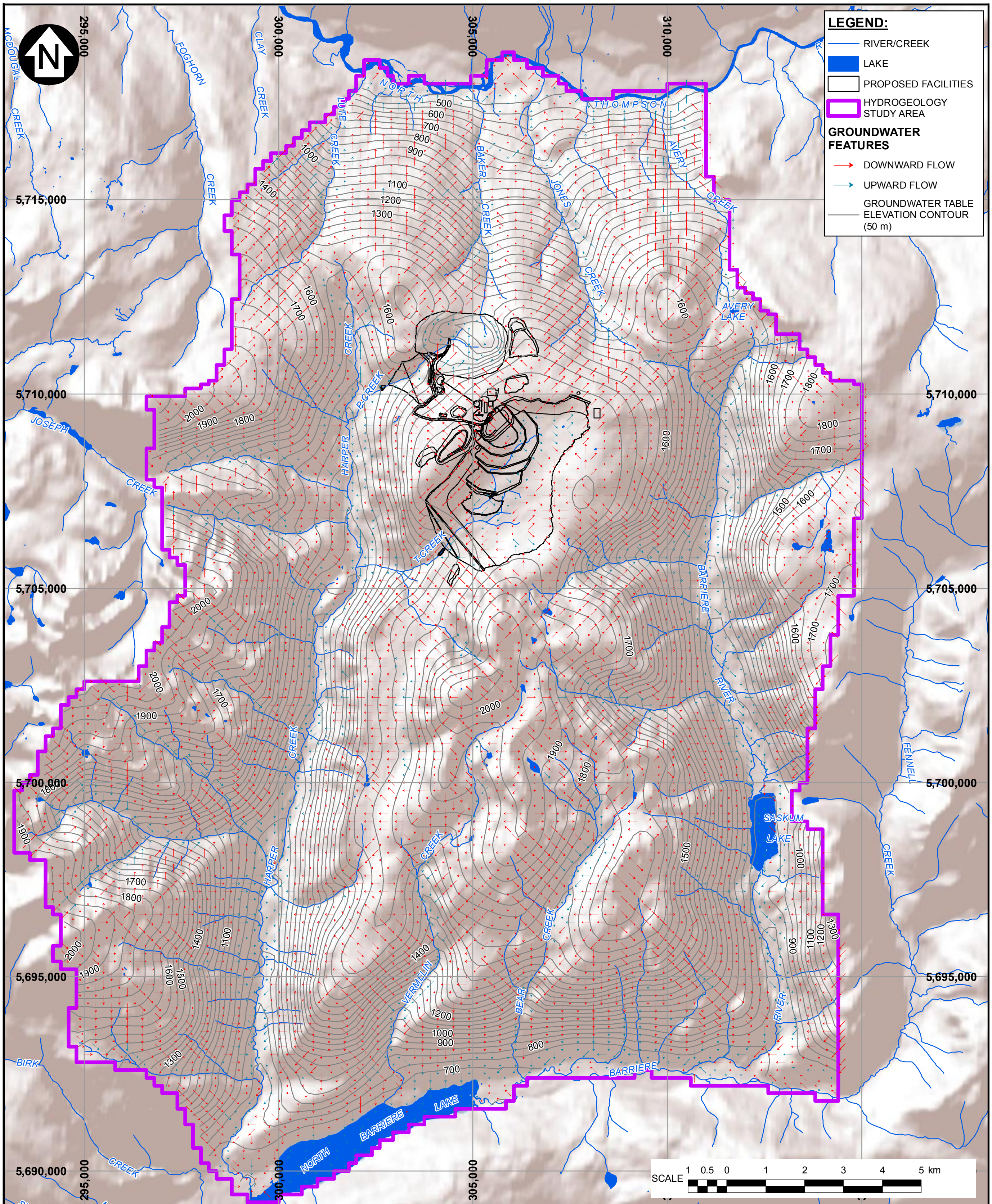

NOTES:

1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:140,000 FOR 8.5x11 (LETTER) PAPER.
4. THE GROUNDWATER FLOW DIRECTION VECTORS SHOWN WERE GENERATED IN GIS SOFTWARE USING CELL DATA EXPORTED FROM THE MODEL.

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OPERATIONS MODEL GROUNDWATER FLOW DIRECTION (PLAN VIEW)

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FIGURE 4.5

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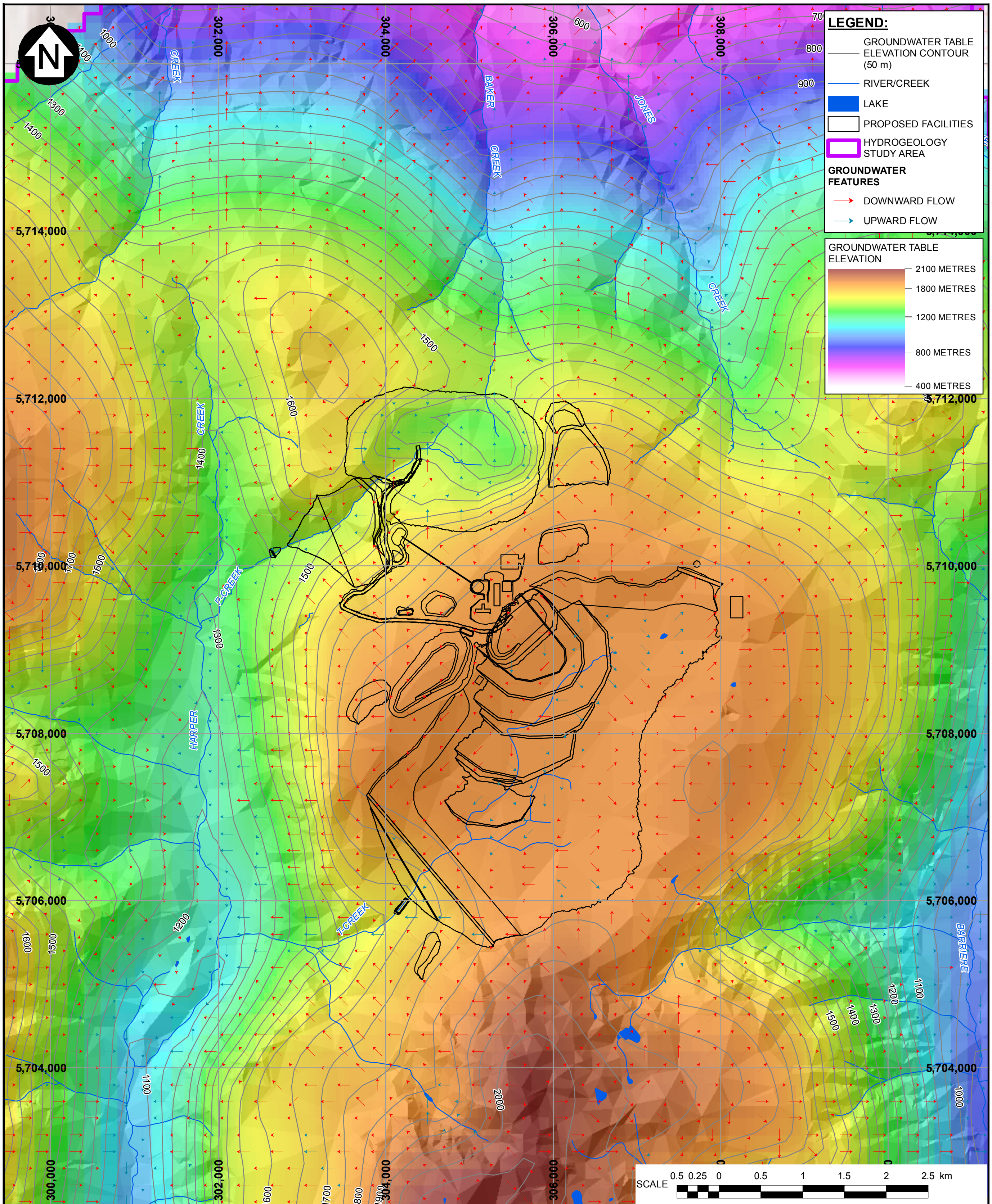

NOTES:
1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:65,000 FOR 8.5x11 (LETTER) PAPER.

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OPERATIONS MODEL PROJECT SITE
GROUNDWATER TABLE ELEVATION

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FIGURE 4.6 | REV 0

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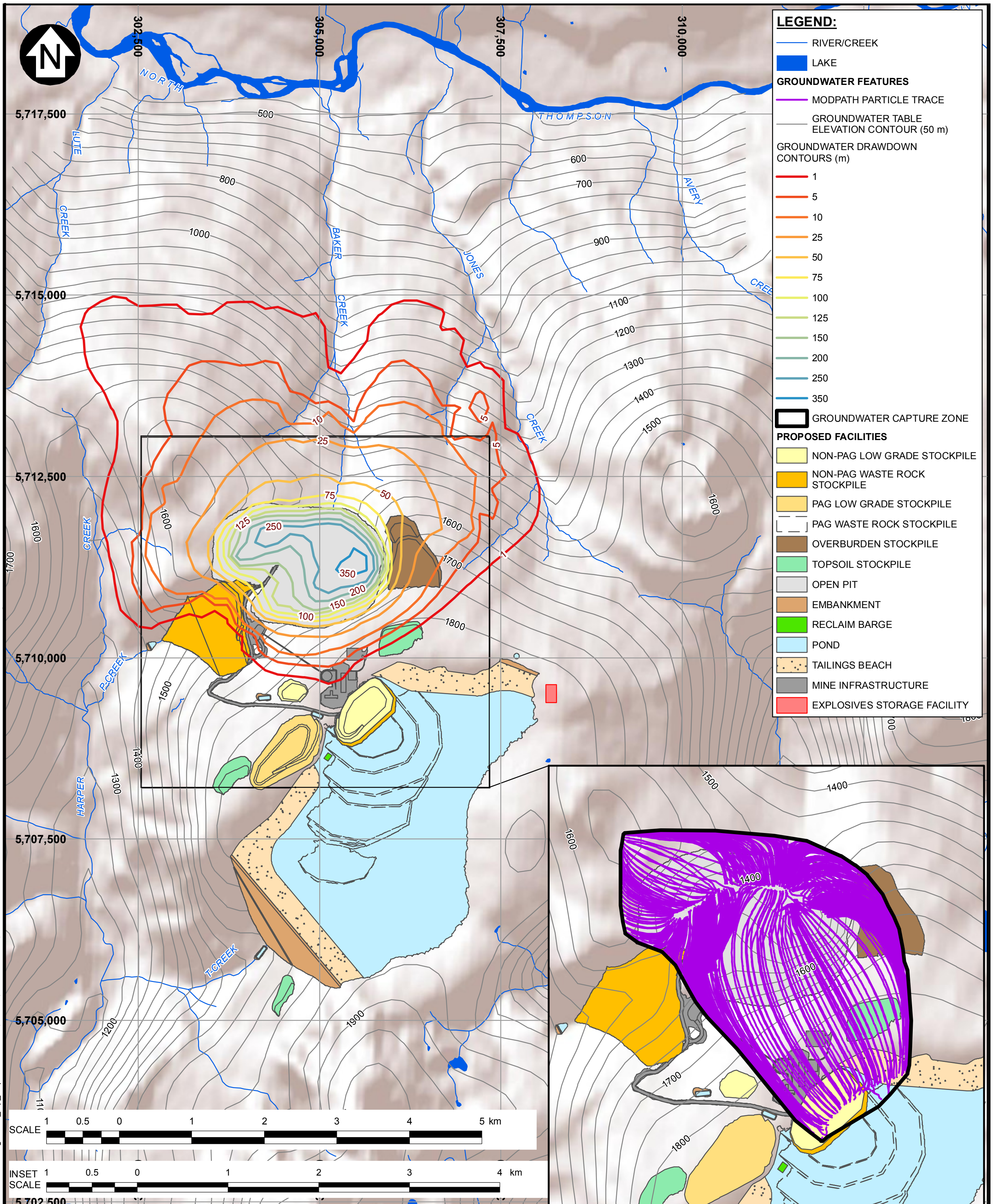

NOTES:

1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:75,000 FOR 8.5x11 (LETTER) PAPER.
4. THE GROUNDWATER DRAWDOWN CONTOURS AND MODPATH PARTICLE TRACES SHOWN WERE GENERATED IN MODFLOW AND IMPORTED INTO GIS SOFTWARE.

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HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

OPERATIONS MODEL
OPEN PIT GROUNDWATER
CAPTURE ZONE AND DRAWDOWN

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P/A NO. VA101-458/14 | REF NO. 2

FIGURE 4.7 | REV 0

5 – POST-CLOSURE SIMULATION

A steady-state Post-Closure Model was developed to characterize potential seepage pathways from key mine infrastructure during the post-closure period. The Post-Closure Model was developed by modifying the Operations Model to represent water storage in the Pit Lake using a constant head boundary condition.

The main objectives of the steady-state post-closure modelling were to:

- Estimate total seepage rates from the Pit Lake during post-closure.
- Delineate the potential seepage pathways from key mine facilities, including the Pit Lake, TMF, Low-Grade Ore (LGO) Stockpiles and Non-PAG Waste Rock Stockpile.
- Provide a conceptual understanding of the source of groundwater to existing domestic water wells located downslope from the pit lake in the Baker Creek watershed.

The results of the Post-Closure Model along with the methodology and assumptions used to develop the model are presented in the sections that follow.

5.1 MODEL CONSTRUCTION

5.1.1 Model Geometry and Grid

The numerical model geometry, layering and grid remained unchanged from the Operations Model.

5.1.2 Hydraulic Conductivity

The hydraulic conductivity values assigned to the Post-Closure Model remained unchanged from the Operations Model. A hydraulic conductivity value of 1×10^{-7} m/s was assigned to cells representing the seepage cut-off (same as in the Operations Model) for the TMF embankment and the rockfilled core of the embankment was assigned a hydraulic conductivity of 1×10^{-5} m/s. No hydraulic conductivity zone was assigned to tailings and waste rock within the TMF. Hydraulic conductivity of these materials is controlled by the river boundary conditions used to simulate the TMF supernatant pond, as discussed in Section 4.2.3.

5.1.3 Post-Closure Model Boundary Conditions

With the exception of the pit lake, boundary conditions remained the same as those in the Operations Model. A discussion detailing how each proposed facility is represented in the model is provided in the following subsections.

5.1.3.1 Tailings Management Facility (TMF)

The TMF Pond was represented using the same river boundary (RIV) cells defined in the Operations Model for model Layer 1 within the pond footprint. A stage elevation of 1,834 masl was assigned to the RIV cells of the TMF, which is the maximum design elevation of the supernatant pond.

5.1.3.2 Pit Lake

Constant head cells were added to the model to represent the pit lake and were assigned in Layers 1 through 4 within the maximum extent of the open pit shell. A stage of 1530 masl was assigned to the pit lake cells. The constant head cells allow groundwater inflow to the pit lake in locations where the surrounding aquifer head exceeds the pit lake stage and seepage outflow from the pit lake where the stage exceeds aquifer head.

5.1.3.3 Existing Domestic Water Wells

Four domestic groundwater wells were added to the model downslope from the pit lake in order to simulate groundwater extraction at these locations. One of the wells (I.D.00084) is located in the Jones Creek watershed and the other three (97736, 97740 and 39609) within the Baker Creek watershed. According to the BC Ground Water Wells and Aquifer Database (WELLS), wells 97736 and 97740 are screened in bedrock to a depth of 153 meters below ground surface (mbgs) and wells 00084 and 39609 are completed in overburden materials along the North Thompson River Valley to depths of 17 mbgs and 28 mbgs, respectively. The reported yield, well depth, screen interval geology and the MODFLOW layer to which the wells are assigned are summarized in Table 1. The wells were added to the model using the analytical pumping well package and were assigned to a layer based on the total depth of the well. Wells were assigned a steady-state pumping rate sufficient to accommodate estimated domestic household usage for a family of four (70 gallons per day per person), as shown in Table 5.1.

Table 5.1 Domestic Water Well Installation Details

Well Tag No.	UTM Coordinates (Zone 11N)		Reported Yield (gpm)	Well Depth (mbgs)	Screen Interval Geology	MODFLOW Simulated Pumping Rate (gpd)	MODFLOW Layer
	Easting (m)	Northing (m)					
97736	305,857	5,716,294	0.75	153	Bedrock	280	3
97740	305,289	5,717,289	0.25	153	Bedrock	280	3
39609	305,791	5,717,745	25	17	Sand and Gravel	280	1
00084	307,236	5,717,736	10	28	Sand and Gravel	280	1

5.1.3.4 Groundwater Recharge

The spatial distribution and rate of groundwater recharge for areas undisturbed by proposed mine facilities remained unchanged from the baseline model. Changes to the specified groundwater recharge boundary condition in the Post-Closure model were made for the following mine components:

- Non-PAG Waste Rock Stockpile
- tailings beach and embankments
- tailings pond, and
- Pit Lake.

A recharge rate of 130 mm/year was applied to the tailings beach in the Post-Closure Model. This value was reduced from that used in the Operations model (175 mm/year) to reflect the expected reduction in recharge associated with the end of tailings spiggoting on the tailings beaches. A recharge rate of 150 mm/yr was assigned the Non-PAG Waste Rock Stockpile and baseline recharge rates (by elevation band) were restored within the footprints of the Low-Grade Ore

Stockpiles. No recharge was assigned within the TMF pond or Pit Lake footprints as recharge to the pond areas is controlled by the RIV cells of the supernatant pond and constant head cells for the pit lake, respectively.

5.2 POST-CLOSURE RESULTS

Simulated water table contours from the Post-Closure Model are provided on Figure 5.1 and a plan view of groundwater flow directions for model Layer 1 is presented on Figure 5.2. The predicted water table and flow directions from Figures 5.1 and 5.2 were combined on Figure 5.3 to provide a larger-scale view of groundwater flow and elevation within the Project area.

Groundwater elevation surrounding the Pit Lake is expected to recover to the lake water surface elevation (1530 masl). Hydraulic head contours and groundwater flow directions indicate that groundwater flow from the Pit Lake is expected to be towards the Baker Creek watershed. Based on a water balance assessment of the Pit Lake using the Post-Closure model, groundwater inflow to the pit lake is estimated to be 4 L/s and seepage from the pit lake is expected to be approximately 8 L/s. Groundwater inflow to the pit is combined with precipitation on the lake and pit wall runoff and the excess after seepage losses is pumped to the TMF. Additional details on seepage pathways from the Pit Lake are discussed below in the Section 6.2.2.Table 5.2 presents a comparison of simulated baseflow from the Post-Closure and Baseline models. The greatest percent baseflow reductions during Post-Closure occur in P-Creek and T-Creek with flow reductions of 86% and 60%, respectively. Flow reductions in P-Creek are caused by reduction of contributing watershed area due diversion of the Pit Lake pond and water collection at the Non-PAG Waste Rock Stockpile in the P-Creek sub-catchment. Similarly, baseflow is reduced in lower T-Creek as a result of water retention within the TMF. A portion of the baseflow contribution previously attributed to the upper T-Creek watershed that is cut-off by the TMF embankment, and reports as inflow to the TMF and is discharged to lower T-Creek via the TMF spillway in Post-Closure. This surface discharge contribution is not considered in the baseflow analyses provided herein for Post-Closure. A summary of surface flow changes is presented in the watershed modelling report (KP, 2014c). These flow reductions are carried downstream into the Harper Creek watershed for the HARPERUS and WSC gauges, as shown in Table 5.2. Baseflow is reduced from Baseline in Baker Creek as a result of water retention in the Pit Lake.

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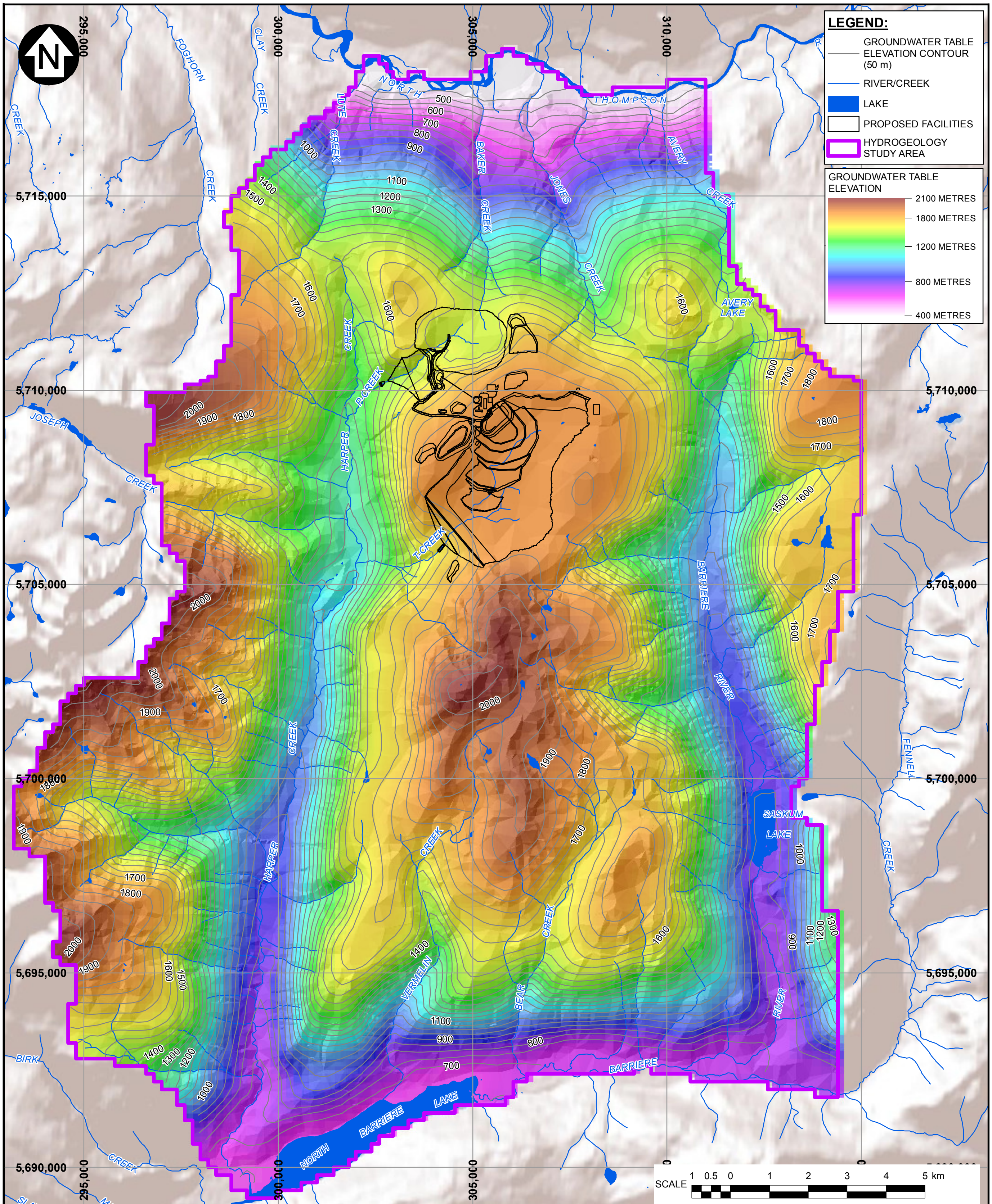
Table 5.2 Post-Closure Comparison of Baseflow at Hydrometric Stations

Hydrometric Station (Gauge I.D.)	Baseline Simulated Baseflow (m^3/day)	Post-Closure Simulated Baseflow (m^3/day)	Flow Reduction Percentage (%)
P-Creek Above Harper Creek (OP Gauge)	1,228	177	86%
Harper Creek Above T-Creek (HARPERUS Gauge)	20,267	16,777	17%
T-Creek Above Harper Creek (TSFDS Gauge)	9,103	3,630	60%
Harper Creek at WSC Station (08LB076 Gauge)	62,266	47,307	24%
Baker Creek Above N. Thompson (BAKER Gauge)	2,976	2,170	27%
Jones Creek Above N. Thompson (JONESUS Gauge)	5,333	5,050	5%

NOTES:

1. The 60% baseflow reduction for T-Creek does not include the contribution from TMF spillway discharge in Post-Closure.



NOTES:
1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:140,000 FOR 8.5x11 (LETTER) PAPER.

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HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

POST-CLOSURE MODEL REGIONAL GROUNDWATER TABLE ELEVATION

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FIGURE 5.1 REV 0

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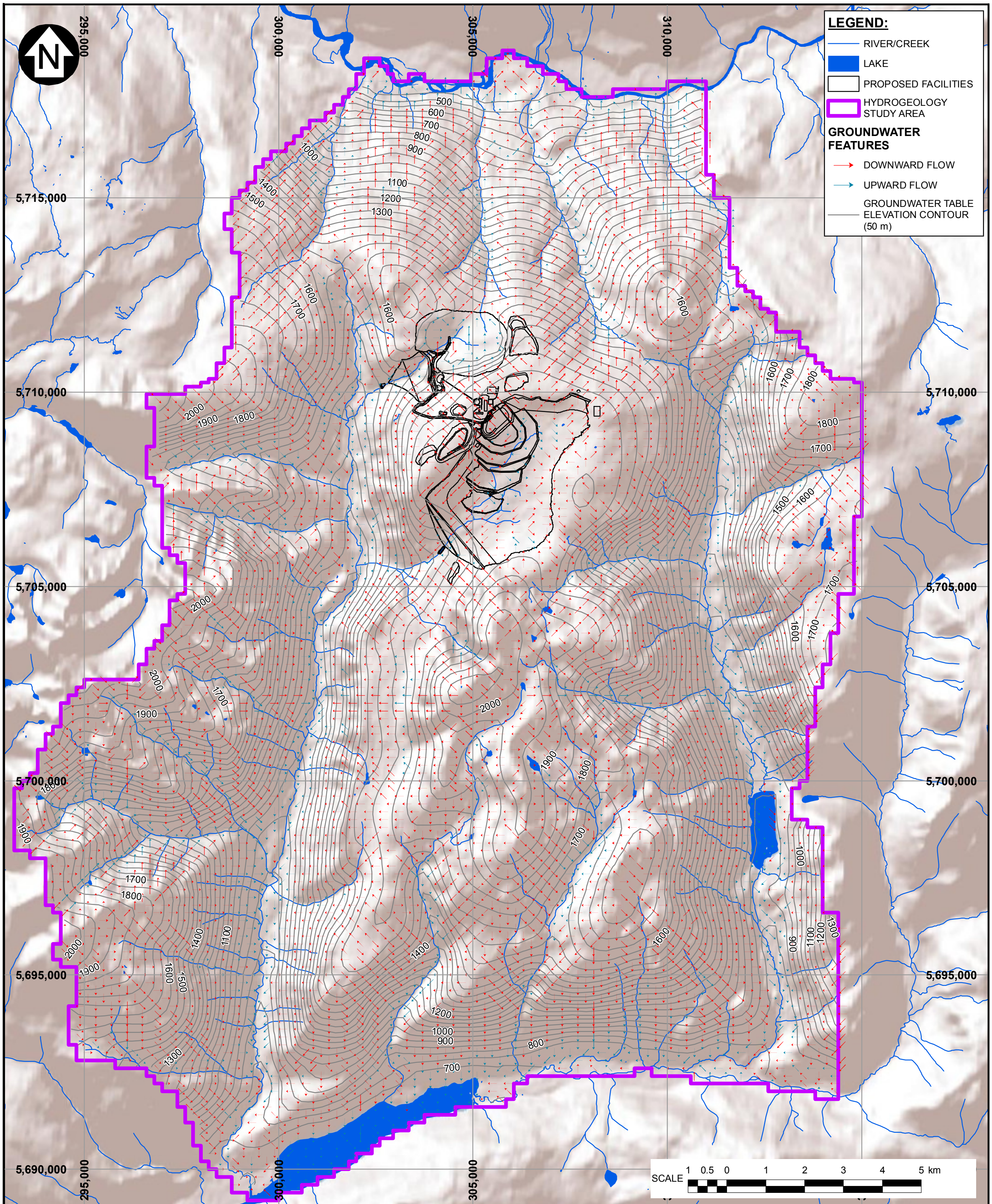

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NOTES:

1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:140,000 FOR 8.5x11 (LETTER) PAPER.
4. THE GROUNDWATER FLOW DIRECTION VECTORS SHOWN WERE GENERATED IN GIS SOFTWARE USING CELL DATA EXPORTED FROM THE MODEL.

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HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

POST-CLOSURE MODEL GROUNDWATER FLOW DIRECTION (PLAN VIEW)

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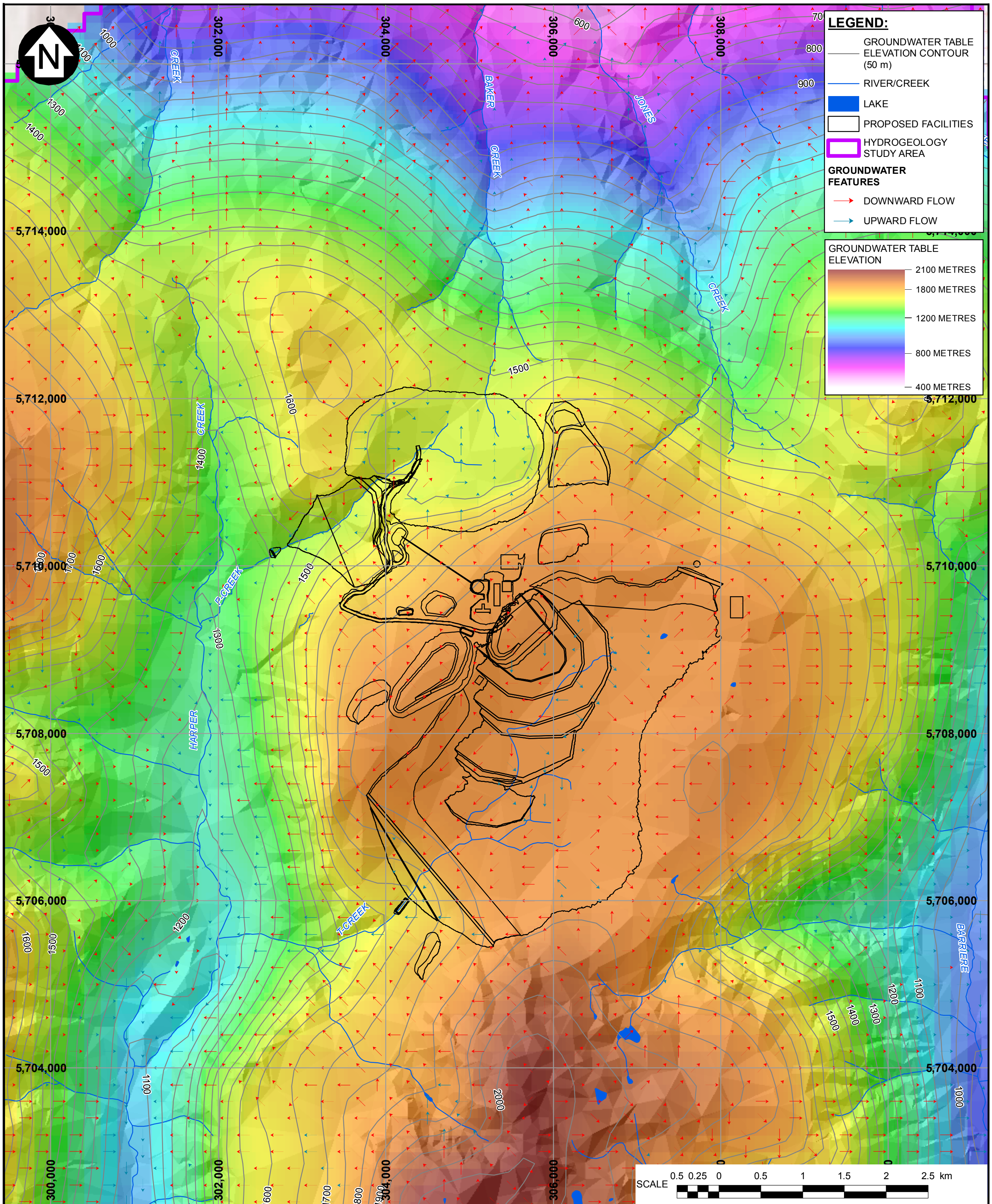
P/A NO. VA101-458/14

REF NO. 2

FIGURE 5.2

REV 0



NOTES:

1. BASE MAP: TRIM AND NTS MAPPING, ESRI ARCGIS ONLINE SHADED RELIEF.
2. COORDINATE GRID IS IN METRES. COORDINATE SYSTEM: NAD 1983 UTM ZONE 11N.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:65,000 FOR 8.5x11 (LETTER) PAPER.

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HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

POST-CLOSURE MODEL PROJECT SITE GROUNDWATER TABLE ELEVATION

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P/A NO. VA101-458/14

REF NO. 2

FIGURE 5.3

REV 0

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6 – MODPATH PARTICLE TRACKING SIMULATIONS

6.1 OVERVIEW

MODPATH particle tracking (Pollok 1994) was completed in the Operations and Post-Closure models to delineate flow directions and estimate seepage travel times to discharge locations from key mine infrastructure. The objectives of the MODPATH simulations were to:

- Delineate potential seepage pathways from the Tailings Management Facility
- Delineate potential seepage pathways from the Pit Lake (Post-Closure)
- Delineate potential seepage pathways from the Non-PAG Waste Rock Stockpile
- Delineate the Open Pit groundwater capture zone (Operations), and
- Determine the source of groundwater to four existing domestic water wells in Baker watershed.

The methodology and results of the above MODPATH analyses are discussed in the sections that follow. The simulated MODPATH particle traces resulting from each simulation are provided in Appendix A.

6.2 DELINEATION OF POTENTIAL SEEPAGE PATHWAYS

MODPATH forward particle tracking was implemented to delineate flow directions and estimate seepage travel times to discharge locations from key mine infrastructure. The following facilities were included in the MODPATH analysis:

- Tailings Management Facility (TMF)
- Pit Lake
- Non-PAG Waste Rock Stockpile
- Non-PAG Low-Grade Ore Stockpile, and
- PAG Low-Grade Ore Stockpile.

6.2.1 MODPATH Seepage Pathway Analysis Methodology

For the Pit Lake MODPATH simulation, particles were inserted along the rim of the pit shell in Layers 1 and 2 and within the footprint of the pit lake constant head cells in Layer 4. For MODPATH analyses at the other two facilities, particles were inserted at the top of Layer 1 within the facility footprint. These particles were forward tracked through the groundwater flow system to downstream discharge locations over an indefinite period of time.

The MODPATH simulation can be used to calculate approximate groundwater travel times along the seepage pathways by taking into consideration an assumed effective porosity. Effective porosities assigned to the model for the MODPATH velocity calculations are shown on Table 3.1 and include 0.1% (0.001) for weathered bedrock, 0.01% (0.0001) for competent bedrock and 15% (0.15) for overburden material (Freeze, 1979). Travel times are representative of advective transport and do not include effects from dispersion or diffusion.

MODPATH results are sensitive to specification of the “sink strength” input parameter, which defines the termination criterion for particle traces flowing through boundary cells (Pollock 1994). All MODPATH scenarios presented herein adopt a “stop at 50 percent strength” weak sink option to discontinue particle traces in boundary cells. Conceptually, this means that the model terminates a particle trace in a cell if more than 50% of the water in the cell is removed.

6.2.2 MODPATH Seepage Pathway Analysis Results

The results of the MODPATH simulations are summarized in Table 6.1 and include a description of potential seepage discharge locations and approximate advective groundwater travel times. The simulated MODPATH particle traces resulting from each simulation are provided in Appendix A. Results of the MODPATH analysis for each facility are described in the sections that follow.

6.2.2.1 Tailings Management Facility (TMF)

A forward particle tracking simulation was completed for the TMF in order to delineate potential seepage pathways from the facility during the Post-Closure phase. Particles were inserted into model Layer 1 within the TMF supernatant pond and tailings beach footprints. The simulated MODPATH particle traces resulting from the TMF simulation are shown on Figure A.1. Results indicate the presence of four potential seepage pathways from the TMF as follow:

1. Seepage through the Main TMF Embankment to T-Creek and Harper Creek
2. Seepage through the North TMF Embankment to Jones Creek
3. Seepage from the northwest of the TMF to the Pit Lake, and
4. Seepage from the northwest of the TMF towards the Non-PAG Waste Rock Stockpile.

The majority of seepage from the TMF is expected to move southwest through the TMF Main Embankment or underlying foundation materials (pathway 1). Approximately 87% of the particle traces exiting the facility via this pathway are captured by the TMF Embankment drains or water management pond. The remaining 13% of particles bypass collection infrastructure to surface in T-Creek (approx. 3%) or Harper Creek (approx. 10%). The bypass (unrecoverable) seepage pathways originate from the northwest side of the embankment rather than along the centerline of the embankment along the T-Creek valley. This flow regime is caused by the influence of non-contact groundwater flow from a topographic high to the southeast of the embankment, as shown on Figure A.1. This area contributes northwestward groundwater flow that cuts off potential seepage pathways from the southeastern half of the embankment and drives seepage pathways to the northwest of the embankment centerline.

Seepage from the TMF occurs at a total rate of 13 L/s and assuming that all particle traces convey seepage at an equal rate, unrecoverable seepage to Harper and T-Creeks is expected to occur at a rate of approximately 1.5 L/s. This agrees with the results of 2-dimensional seepage analyses completed for the TMF in SEEP/W which predicted an unrecoverable seepage rate of approximately 1 L/s, as discussed in Appendix B (KP 2014b). Simulated groundwater travel times for particle traces reporting to T-Creek range from 2 to 16 years with an average of approximately 10 years. Groundwater travel times to Harper Creek are expected to be approximately 12 years with a range of 2 to 30 years.

Table 6.1 Results of MODPATH Particle Trace Simulations

Facility I.D. MODPATH Discharge Location	**Percent of Total Seepage Discharge (%)**	**Travel Time to Discharge Location (Years)[1,2]**		
		Average	**Minimum**	**Maximum**
TMF Main Embankment and Foundation (Post-Closure)	-	-	-	-
Harper Creek	10%	12	2	30
T-Creek	3%	10	2	16
Main Embankment Drains	87%	8	1	25
TMF North Embankment and Foundation (Post-Closure)	-	-	-	-
Jones Creek	85%	12	2	20
North Embankment Drains	15%	-	-	-
Northwest Boundary of TMF Pond	-	-	-	-
Pit lake	95%	8	3	12
Non-PAG Waste Rock Stockpile [3]	5%	23	23	23
Non-PAG Waste Rock Stockpile (Operations)	-	-	-	-
P-Creek	2%	1	<1	2
Water Management Pond	69%	1	<1	2
Open Pit	29%	1	<1	1
Non-PAG Waste Rock Stockpile (Post-Closure)	-	-	-	-
P-Creek	1%	1	<1	2
Water Management Pond	99%	1	<1	2
Pit Lake	0%	-	-	-
PAG Low-Grade Ore Stockpile				
P-Creek	7%	18	6	22
Water Management Pond	47%	12	4	18
Harper Creek below P-Creek	22%	14	8	20
Harper Creek above P-Creek	4%	12	8	20
Tailings Management Facility	22%	1	<1	2
Non-PAG Low-Grade Ore Stockpile				
Water Management Pond	100%	1	<1	2
Pit Lake (Post-Closure)	-	-	-	-
Baker Creek	100%	13	2	21
P-Creek	0%	-	-	-

NOTES:

1. Approximate seepage travel times from the stockpiles to the discharge locations in the receiving environment were calculated using an assumed effective porosity of 0.1% (0.001) for weathered bedrock, 0.01% (0.0001) for unweathered bedrock and 15% (0.15) for alluvial materials.
2. Travel times are based on advective travel only and disregard the effects of dispersion and diffusion.
3. Travel times and seepage percentage based on a single MODPATH Particle that reports to the Non-PAG Waste Rock Stockpile.

A small amount of seepage is expected to flow northeast through the TMF North Embankment or underlying foundation materials (pathway 2). Model results show the source of this pathway to originate from recharge to the northeast tailings beach rather than from the TMF pond. A mass balance of the tailings beach area reporting to this seepage pathway was completed to provide an estimate of the total seepage. Based on this analysis approximately 1 L/s seepage is expected to report as seepage to the North Embankment or foundation materials. Approximately 15% of the particles that exit the TMF via this pathway are captured by the North Embankment drains or water management pond and the remaining 85% bypass to the headwaters of Jones Creek. Unrecoverable seepage to Jones Creek occurs at a rate of approximately 0.5 L/s, assuming all particle traces convey seepage at an equal rate. Simulated travel times to discharge locations in Jones Creek range from 2 to 20 years with an average of about 12 years. Particles with shorter travel times discharge higher in the Jones Creek watershed than deeper seepage pathways with longer travel times.

The presence of two topographic saddles along the northwest boundary of the TMF is expected to provide potential seepage pathways from the TMF to the Pit Lake (pathway 3) and Non-PAG Waste Rock Stockpile (pathway 4), as shown on Figure A.1. MODPATH results indicate that potential seepage pathways that discharge in the Pit Lake originate from the northeast TMF beach. A mass balance of this region indicates an approximate seepage rate of 0.5 L/s will report to the Pit Lake. Groundwater travel times along this pathway are expected to be about 8 years with a range of 3 to 12 years.

Trace amounts of seepage are expected to exit the TMF supernatant pond along its northwest boundary to report to the footprint of the Non-PAG Waste Rock Stockpile. This seepage pathway is expected to discharge within the stockpile base and will be captured as surface runoff in the water management pond. Groundwater travel times to the Non-PAG Waste Rock stockpile are expected to be approximately 23 years.

No particles were predicted to flow from the TMF through the east side of the basin towards the Barriére River, indicating groundwater recharge up-gradient provides containment on this side.

6.2.2.2 Pit Lake

Forward particle tracking analysis was completed for the Pit Lake to delineate potential seepage pathways at closure. Results indicate that all seepage from the Pit Lake is expected to discharge towards the Baker Creek watershed with no discharge as unrecoverable seepage towards the P-Creek watershed. All particle traces were predicted to discharge to Baker Creek upstream of its confluence with the North Thompson River. None of the particle traces originating in the pit lake report to the existing domestic groundwater wells. Approximately 8 L/s is expected to report to Baker Creek in Post-Closure. Simulated groundwater travel times to discharge locations in Baker Creek were an average of 13 years with a range from 2 to 21 years. The simulated MODPATH particle traces resulting from the pit lake simulation are shown on Figure A.2.

6.2.2.3 Non-PAG Waste Rock Stockpile

MODPATH particle analyses were completed for the Non-PAG Waste Rock Stockpile and water management pond in both the Operations and Post-Closure models to assess potential seepage pathways while the pit is actively dewatered and after establishment of a pit lake.

During Operations approximately 29% of particle traces were captured in the Open Pit and 2% bypass the water management pond and discharge as unrecoverable seepage to P-Creek. Approximately 5 L/s total seepage is expected within the stockpile foundation once it has reached its maximum extents. Assuming all MODPATH pathways convey the same groundwater flow, approximately 1.5 L/s seepage is expected to report to the Open Pit during Operations. A small amount of stockpile seepage (<0.1 L/s) is expected to be unrecoverable and bypass the collection pond discharging to P-Creek. Groundwater travel times to P-Creek were simulated with an average of 1 year. Travel times for seepage discharge within the stockpile footprint are expected to range from less than 1 year to 2 years with an average of 1 year. Travel times along the seepage pathway to the Open Pit were estimated to have an average travel time of 1 year.

The establishment of a Pit Lake slightly alters the groundwater flow regime beneath the Non-PAG Waste Rock Stockpile and affects potential seepage discharge locations. Results indicate that during Post-Closure approximately 1.3% of the particle traces bypass the water management pond to report downstream in P-Creek. Of the total stockpile seepage (5 L/s), approximately 4.9 L/s is expected to collect in the water management pond with the remainder (0.1 L/s) bypassing to P-Creek. Groundwater travel times in Post-Closure are expected to be similar to those in Operations, as shown on Table 6.1.

The simulated MODPATH particle traces resulting from the Non-PAG Waste Rock Stockpile simulations are shown on Figure A.3. The MODPATH simulations discussed above indicate the potential for a small amount of unrecoverable seepage to P-Creek from the Non-PAG Waste Rock Stockpile during the Operations and Post-Closure phases of the Project. Subsequent modelling suggests that these seepage pathways can be easily intercepted with implementation of shallow collection ditches along the downstream extent of the stockpile footprint.

6.2.2.4 PAG and Non-PAG Low-Grade Ore Stockpile

Forward particle tracking was completed for the PAG and Non-PAG LGO Stockpiles to delineate potential seepage pathways during the Operations I and II periods. The Non-PAG LGO Stockpile (located in the P-Creek watershed) is active from the Start of Operations I through Year 5. The PAG LGO Stockpile (located along the watershed divide between P- and T-Creeks is active from the start of Operations 1 through the end of Operations II (Year 29). The results of the particle tracking analysis for the PAG and Non-PAG LGO Stockpiles are presented on Figures A.5 and A.6 of Appendix A, respectively.

A mass balance analysis indicates that total seepage from the PAG LGO Stockpile is expected to occur at a rate of about 2.9 L/s. Approximately 47% (1.3 L/s) of the total is expected to discharge to the footprint of the Non-PAG Waste Rock Stockpile upstream of the water management pond. Groundwater travel times for seepage discharge within the stockpile footprint are expected to range from 4 years to 18 years with an average of 12 years. 7% (0.2 L/s) seepage is expected to discharge to P-Creek below the water management pond. Approximately 22% (0.6 L/s) seepage is expected to discharge in Harper Creek below the confluence with P-Creek. An additional 4% (0.1 L/s) of seepage is expected to discharge to Harper Creek above the confluence with P-Creek. Travel times along these pathway range from 8 years to 20 years with an average of 14 years. Approximately, 22% (0.6 L/s) of the total stockpile seepage is expected to flow south to discharge in the TMF.

Seepage from the Non-PAG LGO Stockpile is expected to occur at a rate of approximately 0.5 L/s. MODPATH results indicate that all particle traces are expected to discharge within the footprint of the Non-PAG Waste Rock Stockpile upstream from the water management pond. These seepage pathways will be collected in the water management pond and pumped to the TMF. Travel times for seepage discharge within the stockpile footprint are expected to range from less than 1 year to 2 years with an average of 1 year.

6.3 SEEPAGE INTO STREAMFLOW

The design of the mine waste and water management facilities included mitigation measures to prevent and capture groundwater seepage from the mine facilities to the maximum practical extent (KP, 2014d). The vast majority of seepage will be collected by the mitigation measures. Still, some unrecovered seepage is expected. There are several pathways of unrecovered seepage that are significant to the prediction of water quality for the project. These pathways include the following:

- Seepage towards T-Creek will result from infiltration of ponded water in the TMF directly through the embankment fill and the natural ground, and from expulsion of pore water as the tailings mass consolidates.
- Seepage towards P-Creek and Harper Creek will result from infiltration on the Non-PAG Waste Rock Stockpile and from seepage from the water management pond bypassing cut-off and collection measures.
- Seepage towards P-Creek and Harper Creek will result from infiltration on the PAG Low-Grade Ore (LGO) Stockpile bypassing collection measures infiltrating through the underlying low-permeability foundation liner.

The location of the discharge of unrecovered seepage from the mine facilities is a key aspect of the prediction of water quality for the project. MODPATH particle tracking was completed to delineate flow directions of unrecovered seepage from key mine facilities. Seepage flow rates were estimated in MODPATH to provide a comparison to unrecovered seepage estimates from SEEP/W analyses (KP, 2014b) and the Life-of-Mine (LOM) watershed model (KP, 2014c). The seepage rates were estimated using average groundwater recharge applied at ground surface beneath mine facilities, and do not account for reduced infiltration due to foundation lining. The unrecovered seepage rates and seepage directions from MODPATH particle tracking are included in Table 6.2.

Table 6.2 Unrecovered Seepage Rates from MODPATH Particle Tracking

FACILITY	SEEPAGE DIRECTION	UNRECOVERED SEEPAGE RATE (L/S)	SOURCE OF ESTIMATE
TMF	T-CREEK	1.5 L/s	MODPATH
NON-PAG WASTE ROCK STOCKPILE	P-CREEK	0.1 L/s	MODPATH
PAG LOW-GRADE ORE STOCKPILE	P-CREEK	0.2 L/s	MODPATH
	UPPER HARPER CREEK	0.1 L/s	MODPATH
	LOWER HARPER CREEK	0.6 L/s	MODPATH

Estimates of unrecovered seepage flow rates that were used in preparation of expected case water quality predictions for the project (KP, 2014e) were previously provided for the LOM watershed model (KP, 2014c). The seepage estimates from the previous study are reproduced in Table 6.3.

Table 6.3 Unrecovered Seepage Rates from Watershed Modelling

FACILITY	SEEPAGE DIRECTION	UNRECOVERED SEEPAGE RATE (L/S)	SOURCE OF ESTIMATE
TMF	T-CREEK	2 L/s (2)	SEEP/W ANALYSIS
TMF WMP	T-CREEK	0.5 L/s	WATERSHED MODEL
NON-PAG WASTE ROCK STOCKPILE WMP	P-CREEK	1 L/s (3)	WATERSHED MODEL AND SEEP/W ANALYSIS
PAG LOW-GRADE ORE STOCKPILE	P-CREEK AND HARPER CREEK	1 L/s	WATERSHED MODEL

NOTES:

1. ADOPTED FROM TABLE 5.3 OF THE WATERSHED MODELLING REPORT (KP, 2014c).
2. UNRECOVERED SEEPAGE RATE WAS SELECTED WITH CONSIDERATION OF BASE CASE SEEPAGE ESTIMATES AND SENSITIVITY ANALYSES. SEEPAGE RATE REPRESENTS CONSERVATIVE ESTIMATE FOR PURPOSE OF WATER QUALITY MODELLING (KP, 2014b).
3. UNRECOVERED SEEPAGE RATE WAS SELECTED TO REPRESENT A CONSERVATIVE SCENARIO WHERE THE WATER MANAGEMENT POND IS ALLOWED TO ACCUMULATE WATER AND IS MAINTAINED AT MAXIMUM CAPACITY. PREDICTED SEEPAGE RATES RANGED FROM 0.25 L/S TO 0.9 L/S DEPENDING ON POND FILL LEVEL. THE WATER MANAGEMENT SYSTEM IS DESIGNED TO REMOVE EXCESS WATER TO THE MINIMUM OPERATING LEVEL FOR PUMP SUBMERGENCE (KP, 2014d).

The unrecovered seepage rates and seepage directions predicted by MODPATH particle tracking agree reasonable well with predictions from the SEEP/W analyses and the LOM watershed model. The MODPATH estimates indicated that the flow rates included in the LOM watershed model are conservative.

The predicted behavior of groundwater, including both non-contact groundwater and unrecovered seepage from the mine facilities, is inherent to the methodology used in the analysis and will vary depending on the model and the facility being modelled. The MODFLOW models are steady-state models constructed at the maximum extents of the mine facilities and calibrated to a specific set of low flow measurements. The models provide an improved understanding of potential groundwater flow pathways.

The particle tracking completed from the mine facilities indicates that there is a potential for unrecovered seepage from the Non-PAG Waste Rock Stockpile and PAG LGO Stockpile to discharge to P-Creek and Harper Creek upstream of where the LOM watershed model predicts groundwater flow to discharge to streamflow. This indicates that there is a potential for water chemistry impacts in the upstream reach of Harper Creek that were not captured by the LOM watershed model and expected case water quality predictions. The location of unrecovered seepage discharge will vary over the life of the project, and may be impacted by both seasonal and annual trends. Water quality predictions include a sensitivity analysis to determine an expected range of water chemistry in P-Creek and upper Harper Creek based on sensitivity to discharge location. The seepage sensitivity analysis is described in the Water Quality Predictions Report (KP, 2014e).

6.4 DELINEATION OF OPEN PIT GROUNDWATER CAPTURE ZONE

MODPATH particle tracking (Pollok 1994) was used to delineate the capture zone of the proposed Open Pit at the predicted maximum extent of de-watering (Year 24) as shown on Figure 4.6. MODPATH simulation parameters including effective porosity and “weak sink” setting were set the same as discussed in Section 6.2.1. Particles were added to the model within model layers 1 through 4 along a capture zone boundary initially estimated based on water table contours. The capture zone was expanded or contracted iteratively until all MODPATH particle release locations reported to the Open Pit.

Results indicate that the capture zone will extend approximately 2 km southward from the southern pit rim, including a small portion of the TMF footprint, and about 0.5 km to the north towards Baker Creek and the North Thompson River. Groundwater inflow to the Open Pit from this capture zone is expected to reach a maximum of approximately 16 L/s. Simulated MODPATH particle traces resulting from the Open Pit MODPATH simulation are shown on Figure A.4.

6.5 MODPATH PARTICLE TRACKING ANALYSIS EXISTING DOMESTIC WATER WELLS

MODPATH reverse particle tracking was conducted to determine the potential source of groundwater to the four domestic wells discussed above in order to provide a conceptual evaluation of the potential for pit lake seepage to discharge to existing domestic groundwater wells downslope following closure. MODPATH simulation parameters including effective porosity and “weak sink” setting were set the same as discussed in Section 6.2.1. A circle of ten particles was inserted surrounding each well in the model layer corresponding to the screen interval. Particles were placed in Layer 3 for wells 97736 and 97740 and in Layer 1 for wells 39609 and 00084. These particles were tracked backwards through the groundwater flow system for an indefinite period of time to approximate the source of groundwater to the wells.

The simulated MODPATH particle traces resulting from the domestic water well simulation are shown on Figure A.7 in Appendix A. Results indicate that none of the particle traces that discharge to the domestic wells originate from the pit lake. The two deeper wells (97736 and 97740) receive groundwater from topographic highs adjacent to the pit lake to the east and west, respectively. Reverse particle tracking for wells 39609 and 00084 indicates that these wells source water predominantly from valley regions within 1 km of the North Thompson River (approx. 5 km downslope of the pit lake).

The MODPATH simulation was used to calculate approximate groundwater travel times to the domestic wells along the particle traces. Results indicate that groundwater discharging in wells 97736 and 97740 has an approximate travel time of 40 years from its source (topographic highs adjacent to the pit lake) to the wells. Groundwater discharging to wells 39609 and 00084 travels for approximately 90 days from source (valley regions along within 1 km of North Thompson River) to the wells. The travel times are representative of advective transport and do not include effects from dispersion or diffusion.

7 – CONCLUSION

A steady-state baseline numerical groundwater model was developed for the Harper Creek Project to provide a representation of baseline groundwater conditions and to serve as a basis from which to evaluate potential effects of the Project on hydrogeological conditions. The steady-state baseline model was calibrated to measured groundwater elevations and synthetic baseflow estimates. The calibrated baseline model was then modified to create two steady-state predictive models representing the Operations and Post-Closure phases of Project development. Major proposed mine facilities were represented in the predictive models, including the TMF, Open Pit, Non-PAG Waste Rock Stockpile, Low-Grade Ore Stockpiles and Pit Lake (Post-Closure).

Results from the Operations model indicate that the Project will have a localized effect on groundwater elevations immediately surrounding the TMF and Open Pit. Groundwater elevations near the TMF are expected to increase to the water surface elevation of the supernatant pond (1834 masl). Groundwater elevations surrounding the Open Pit will decrease by a maximum of 350 meters due to dewatering of the pit. The associated drawdown is expected to extend approximately 1 km south from the pit rim towards the TMF and approximately 3 km north from the pit rim towards Baker Creek and the North Thompson River. Groundwater inflow to the Open Pit was estimated to occur at a rate of approximately 16 L/s at the end of Operations I (Year 24). The majority of groundwater inflow to the pit will come from up-gradient catchment areas southeast and northwest of the Open Pit.

Groundwater elevations directly surrounding the Pit Lake during Post-Closure are expected to recover to the elevation of the Pit Lake water surface (1530 masl). Groundwater inflow to and seepage from the Pit Lake during Post-Closure when the Pit Lake is at its maximum elevation are expected to be approximately 4 L/s and 8 L/s, respectively. Model results indicate that groundwater seepage from the Pit Lake is expected to contribute to Baker Creek watershed.

A MODPATH particle analysis was conducted to assess pathways of potential seepage originating from the TMF, Pit Lake and Non-PAG Waste Rock Stockpile. Results are presented showing the estimated groundwater seepage pathways from each facility, the discharge location of the seepage pathways and estimated seepage travel times.

The results of baseline and predictive numerical groundwater models were used to inform watershed modelling and water quality modelling conducted to support the EIS submission. The numerical models presented in this report provide a foundation that should be updated as new data are collected. Additional head and streamflow data will help refine baseline model calibration and will improve model defensibility as a tool for making predictions of mine effects on hydrogeological conditions.

8 – REFERENCES

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9 – CERTIFICATION

This report was prepared, reviewed and approved by the undersigned.

Prepared:

Kevin Davenport, EIT
Staff Engineer

Reviewed:

Rod Smith, P.Eng.
Specialist Engineer

PROFESSIONAL PROVINCE OF BRITISH COLUMBIA ENGINEER
D. D. FONTAINE
36208
Oct 24, 2014

Reviewed:

Daniel Fontaine, P.Eng.
Senior Engineer

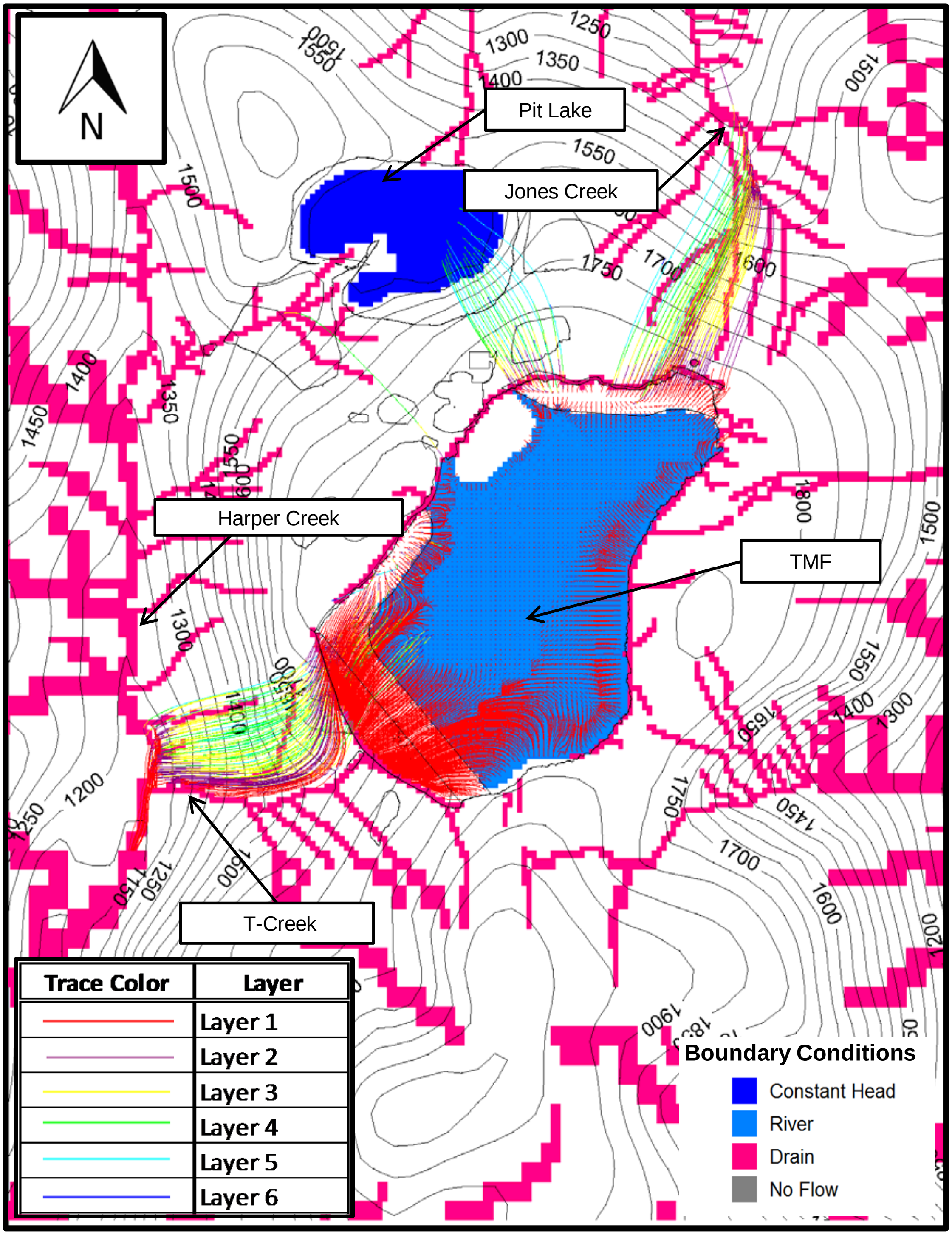
Approved:

Ken Brouwer, P.Eng.
President

APPENDIX A

RESULTS OF MODPATH PARTICLE TRACKING ANALYSES

(Pages A-1 to A-7)



NOTES:

1. PARTICLE TRACES ARE COLOURED ACCORDING TO THE MODEL LAYER THEY ARE TRAVELLING IN.
2. PARTICLES WERE INSERTED INTO LAYER 1 AND TRACKED FORWARD FROM SOURCE TO SINK.
3. THE CONTOURS SHOWN ABOVE ARE OF WATER TABLE ELEVATION (masl).

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

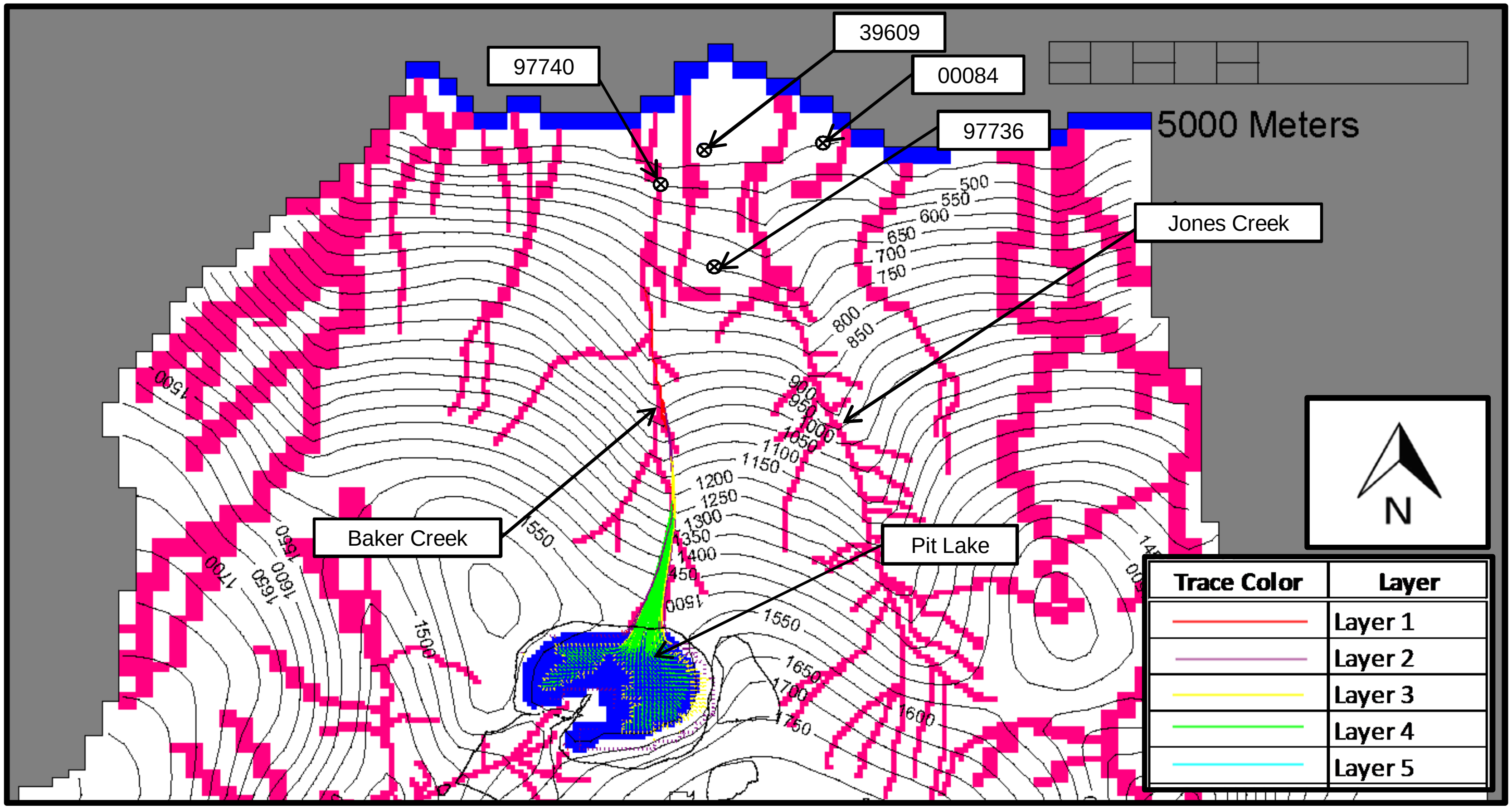
TMF MODPATH ANALYSIS
FORWARD PARTICLE TRACKING RESULTS

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-458/14	2

FIGURE A.1 REV 0

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	10OCT'14	ISSUED WITH REPORT	KTD	DDF	KJB



Boundary Conditions

- Constant Head
- River
- Drain
- No Flow

NOTES:

1. PARTICLE TRACES ARE COLOURED ACCORDING TO THE MODEL LAYER THEY ARE TRAVELLING IN.
2. PARTICLES WERE INSERTED INTO LAYERS 1 THROUGH 4 AND TRACKED FORWARD FROM SOURCE TO SINK.
3. THE CONTOURS SHOWN ABOVE ARE OF WATER TABLE ELEVATION (masl).
4. WELLS 97736 AND 97740 ARE INCLUDED IN MODEL LAYER 3 AND WELLS 39609 AND 00084 IN MODEL LAYER 1.

0	10OCT'14	ISSUED WITH REPORT	KTD	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

PIT LAKE MODPATH ANALYSIS
FORWARD PARTICLE TRACKING RESULTS

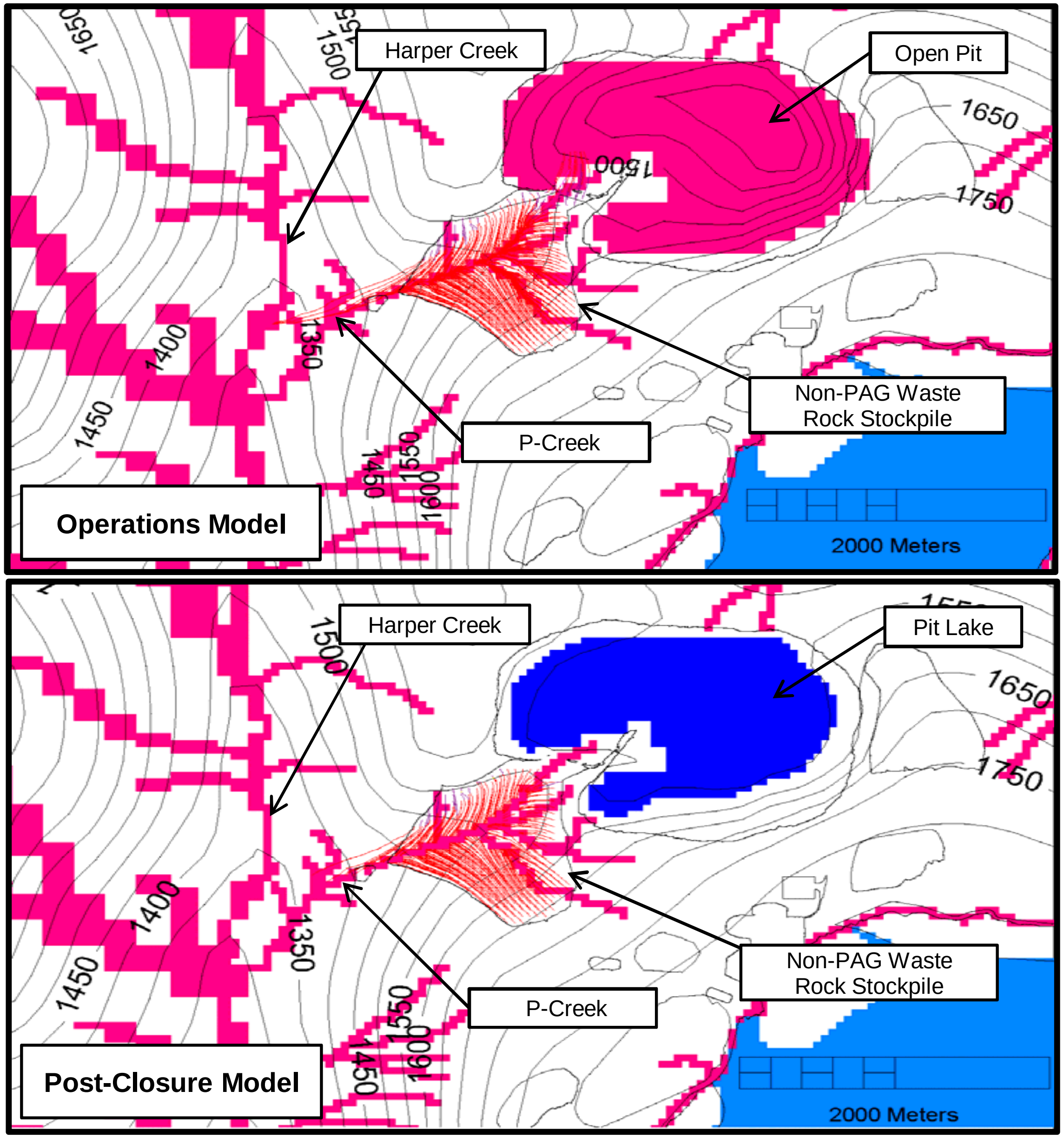
Knight Piésold CONSULTING

P/A NO. VA101-458/14

REF. NO. 2

FIGURE A.2

REV 0



Boundary Conditions

- Constant Head
- River
- Drain
- No Flow

N

Trace Color	Layer
	Layer 1
	Layer 2
	Layer 3
	Layer 4
	Layer 5

NOTES:

1. PARTICLE TRACES ARE COLOURED ACCORDING TO THE MODEL LAYER THEY ARE TRAVELLING IN.
2. PARTICLES WERE INSERTED INTO LAYER 1 AND TRACKED FORWARD FROM SOURCE TO SINK.
3. THE CONTOURS SHOWN ABOVE ARE OF WATER TABLE ELEVATION (masl).

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	10OCT'14	ISSUED WITH REPORT	KTD	DDF	KJB

HARPER CREEK MINING CORP.

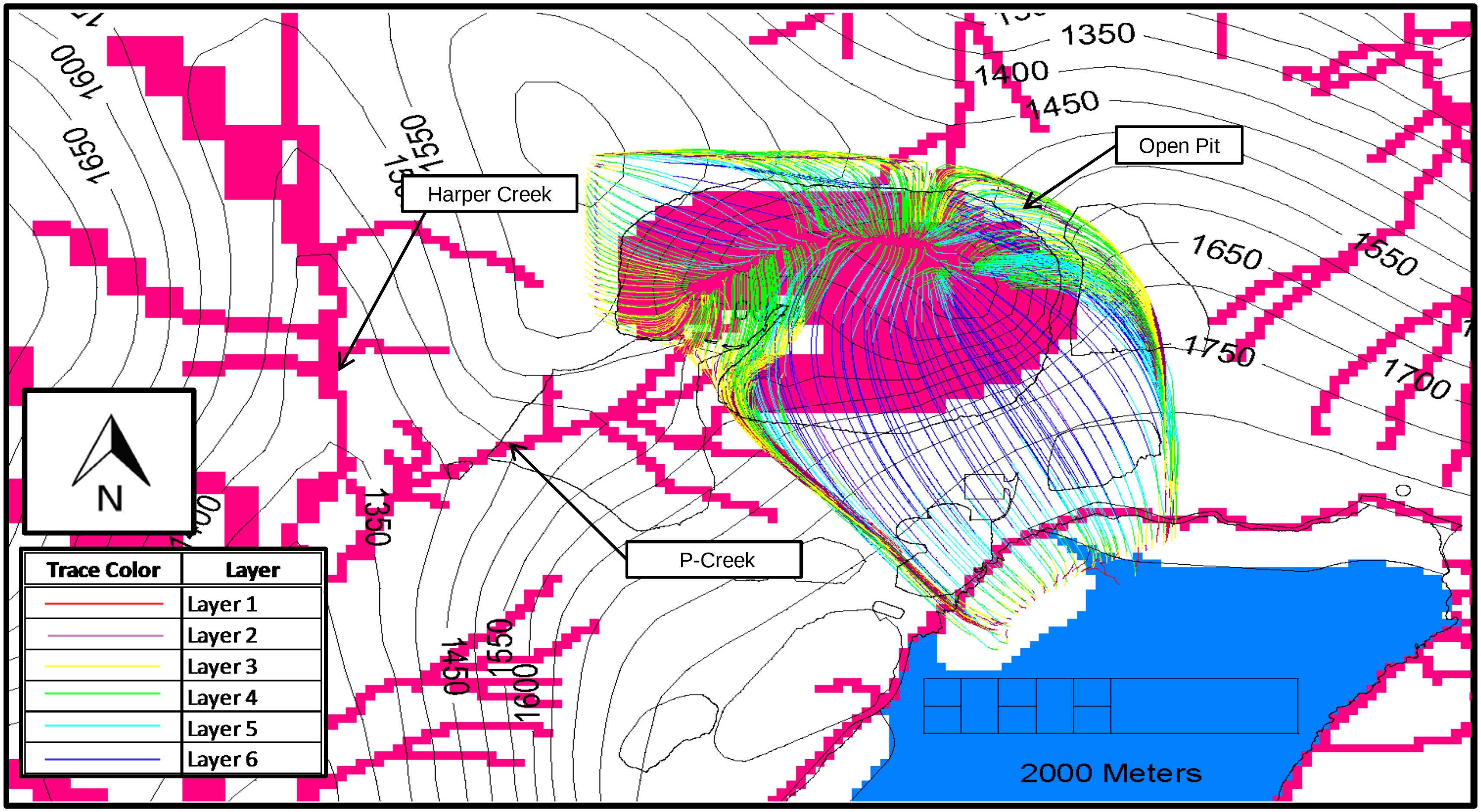
HARPER CREEK PROJECT

NON-PAG WASTE ROCK STOCKPILE MODPATH ANALYSES
FORWARD PARTICLE TRACKING RESULTS

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-458/14	2

FIGURE A.3 REV 0



Boundary Conditions

- Constant Head
- River
- Drain
- No Flow

NOTES:

1. PARTICLE TRACES ARE COLOURED ACCORDING TO THE MODEL LAYER THEY ARE TRAVELLING IN.
2. PARTICLES WERE INSERTED INTO LAYERS 1 THROUGH 4 AND TRACKED FORWARD FROM SOURCE TO SINK.
3. THE CONTOURS SHOWN ABOVE ARE OF WATER TABLE ELEVATION (masl).

0	10OCT'14	ISSUED WITH REPORT	KTD	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

DELINEATION OF OPEN PIT CAPTURE ZONE
FORWARD PARTICLE TRACKING RESULTS

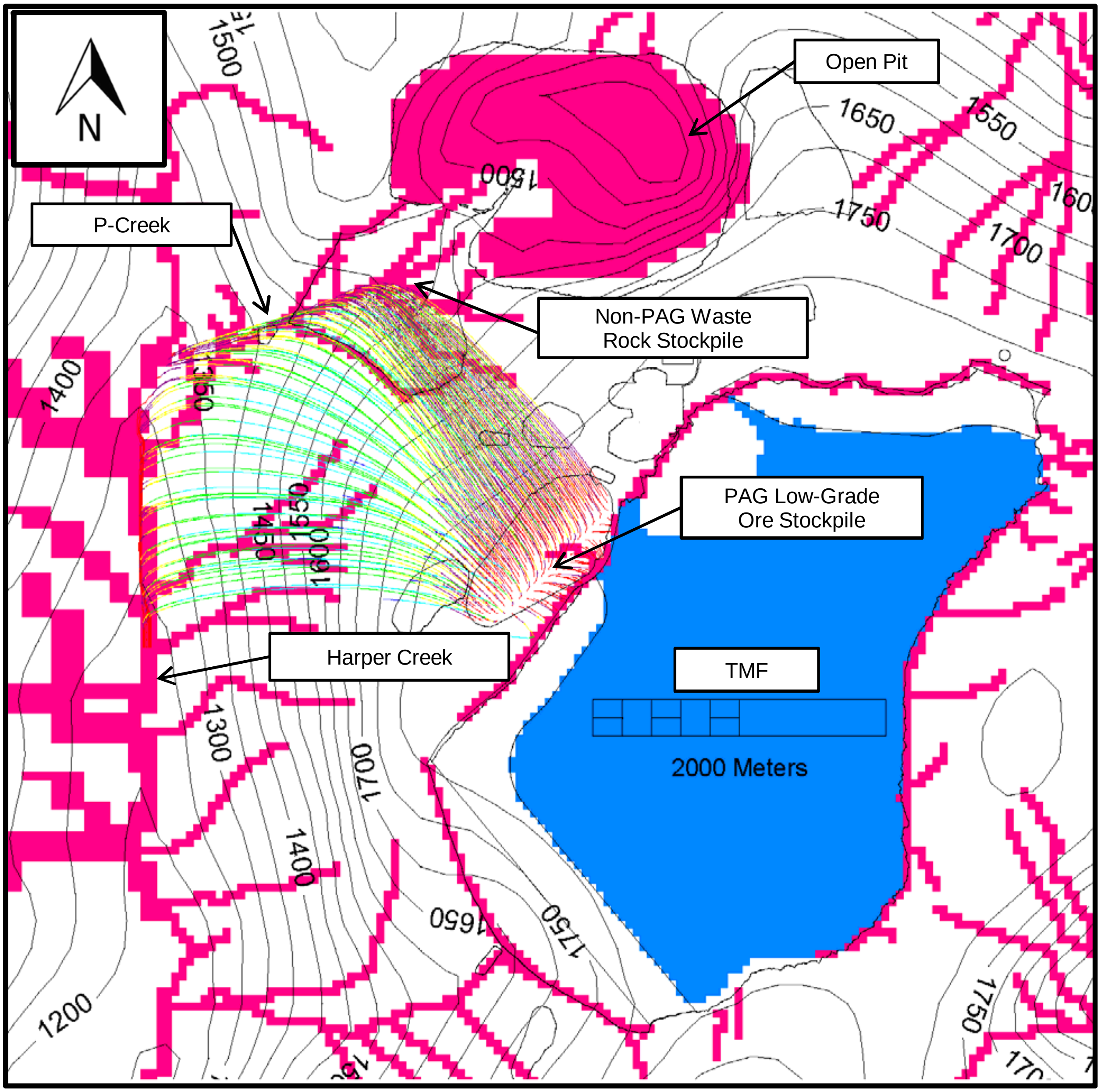
Knight Piésold CONSULTING

P/A NO. VA101-458/14

REF. NO. 2

FIGURE A.4

REV 0



Trace Color	Layer
	Layer 1
	Layer 2
	Layer 3
	Layer 4
	Layer 5
	Layer 6

Boundary Conditions

- Constant Head
- River
- Drain
- No Flow

NOTES:

1. PARTICLE TRACES ARE COLOURED ACCORDING TO THE MODEL LAYER THEY ARE TRAVELLING IN.
2. PARTICLES WERE INSERTED INTO LAYER 1 AND TRACKED FORWARD FROM SOURCE TO SINK.
3. THE CONTOURS SHOWN ABOVE ARE OF WATER TABLE ELEVATION (masl).

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	10OCT'14	ISSUED WITH REPORT	KTD	RS	KJB

HARPER CREEK MINING CORP.

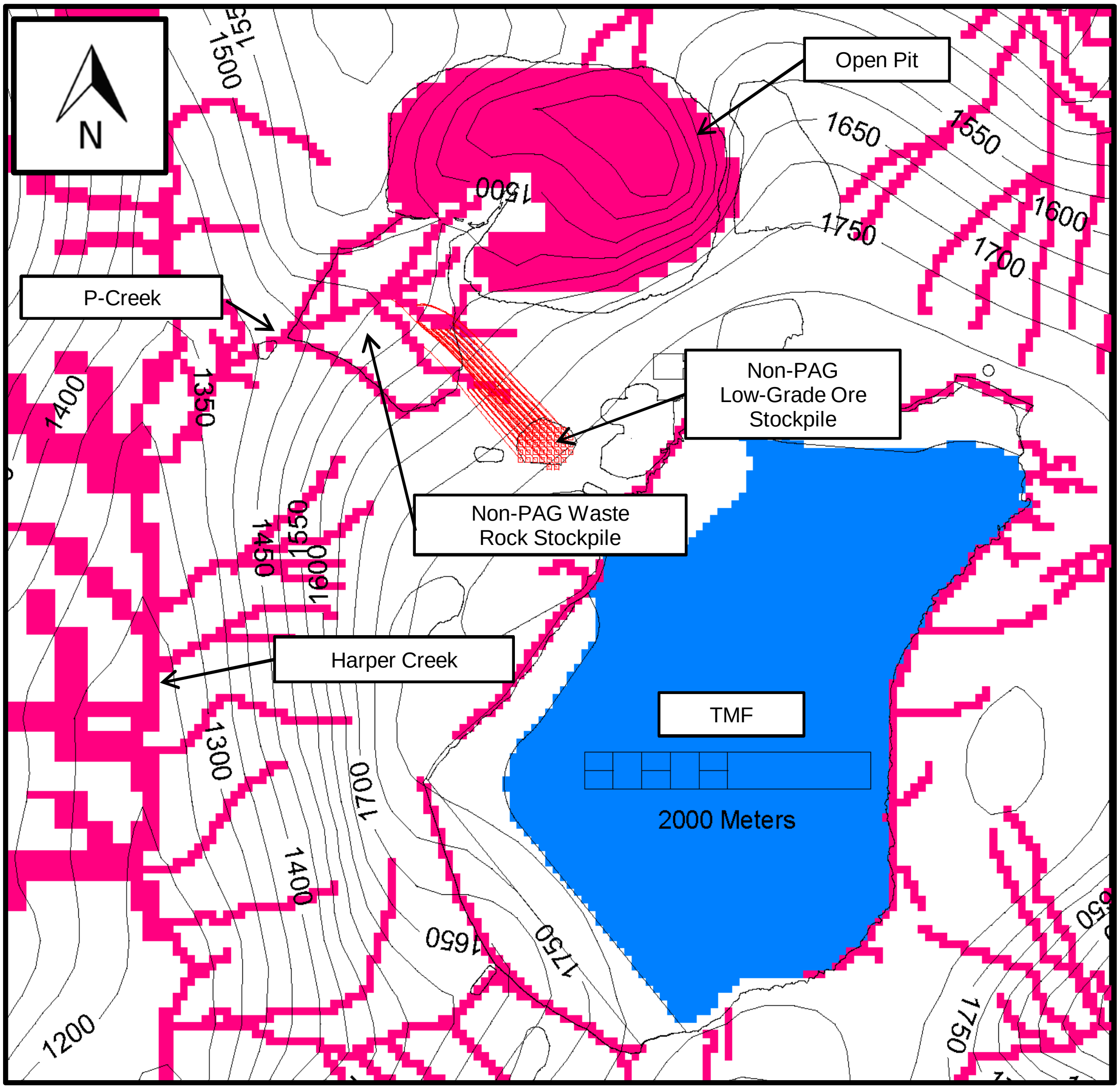
HARPER CREEK PROJECT

PAG LOW-GRADE ORE STOCKPILE FORWARD PARTICLE TRACKING RESULTS

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-458/14	2

FIGURE A.5 REV 0



Trace Color	Layer
	Layer 1
	Layer 2
	Layer 3
	Layer 4
	Layer 5
	Layer 6

Boundary Conditions

- Constant Head
- River
- Drain
- No Flow

NOTES:

1. PARTICLE TRACES ARE COLOURED ACCORDING TO THE MODEL LAYER THEY ARE TRAVELLING IN.
2. PARTICLES WERE INSERTED INTO LAYER 1 AND TRACKED FORWARD FROM SOURCE TO SINK.
3. THE CONTOURS SHOWN ABOVE ARE OF WATER TABLE ELEVATION (masl).

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	10OCT'14	ISSUED WITH REPORT	KTD	RS	KJB

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

NON-PAG LOW-GRADE ORE STOCKPILE FORWARD PARTICLE TRACKING RESULTS

Knight Piésold CONSULTING

P/A NO. VA101-458/14

REF. NO. 2

FIGURE A.6

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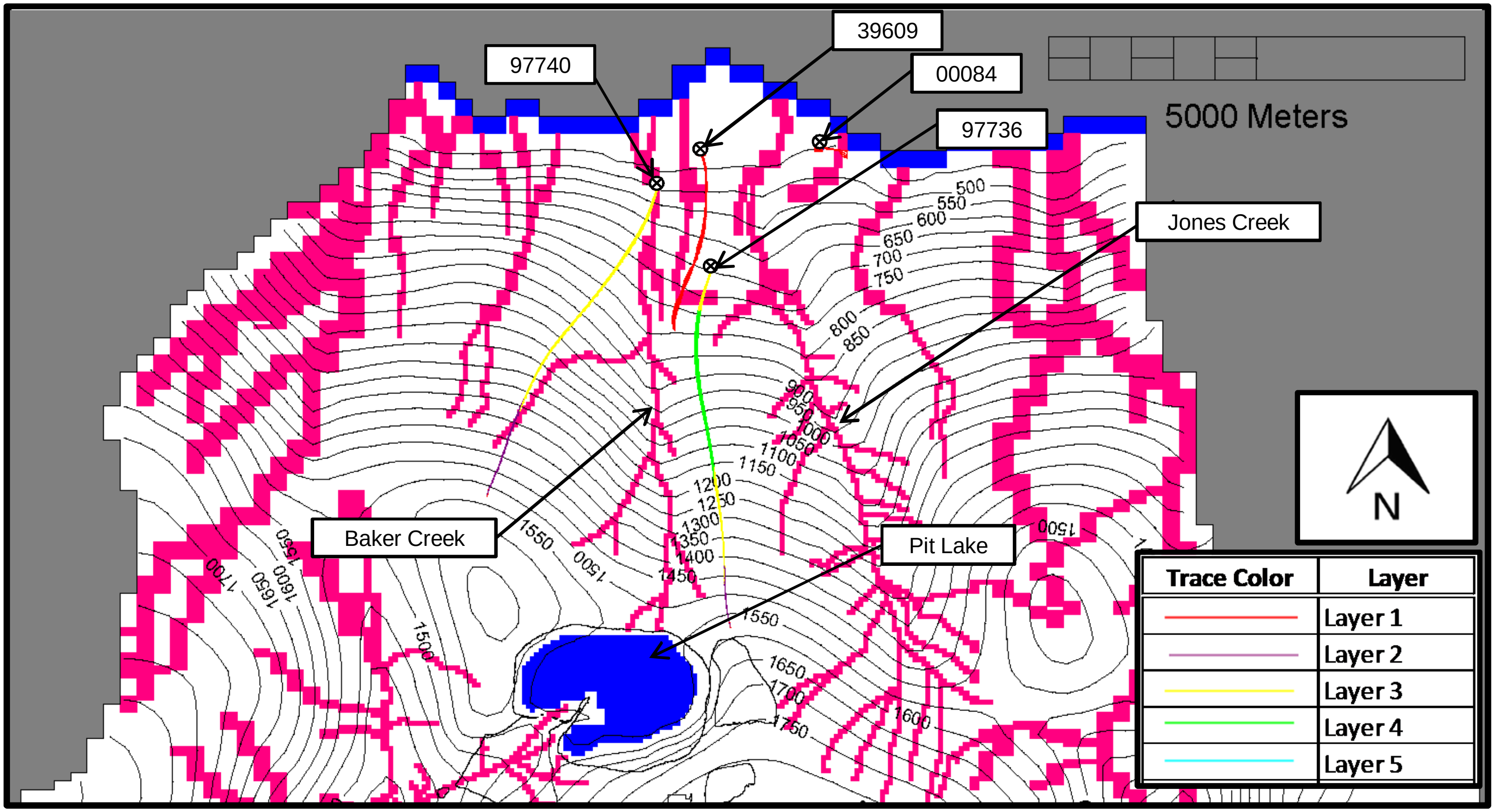

Boundary Conditions

- Constant Head
- River
- Drain
- No Flow

NOTES:

1. PARTICLE TRACES ARE COLOURED ACCORDING TO THE MODEL LAYER THEY ARE TRAVELLING IN.
2. PARTICLES WERE INSERTED INTO LAYERS 1 AND 3 AND AT THE WELL SCREEN LOCATIONS TRACKED BACKWATDS FROM SINK TO SOURCE.
3. THE CONTOURS SHOWN ABOVE ARE OF WATER TABLE ELEVATION (masl).
4. WELLS 97736 AND 97740 ARE INCLUDED IN MODEL LAYER 3 AND WELLS 39609 AND 00084 IN MODEL LAYER 1.

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	10OCT'14	ISSUED WITH REPORT	KTD	DDF	KJB

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

DOMESTIC WATER WELL MODPATH ANALYSIS REVERSE PARTICLE TRACKING RESULTS

Knight Piésold CONSULTING

P/A NO. VA101-458/14

REF. NO. 2

FIGURE A.7

REV 0

APPENDIX B

SEEPAGE AND STABILITY MODELLING

(Pages B-1 to B-42)

Knight Piésold
CONSULTING

www.knightpiesold.com

August 19, 2014

File No.:VA101-458/14-A.01
Cont. No.:VA14-00865

Mr. Alastair Tiver
Vice President Operations
Harper Creek Mining Corp
730 - 800 West Pender Street
Vancouver, BC V6C 2V6

Dear Alastair,

Re: Harper Creek Project - Seepage and Stability Modelling

1 – INTRODUCTION

Harper Creek Mining Corporation (HCMC) proposes to construct and operate the Harper Creek Project (the Project), an open pit copper mine near Vavenby, British Columbia (BC). HCMC is a wholly owned subsidiary of Yellowhead Mining Inc. (YMI), which is a public BC junior mineral development company trading on the Toronto Stock Exchange. The Project has an estimated 28-year mine life based on a process plant throughput of 70,000 tonnes per day (25 million tonnes per year). Ore will be processed on site through a conventional crushing, grinding and flotation process to produce a copper concentrate, with gold and silver by-products, which will be trucked from the Project site along approximately 24km of existing access roads to a rail load-out facility located at Vavenby. The concentrate will be transported via the existing Canadian National Railway network to the existing Vancouver Wharves storage, handling and loading facilities located at the Port of Vancouver for shipment to overseas smelters.

The Project consists of an open pit mine, on-site processing facility, tailings management facility (TMF) (for tailings solids, subaqueous storage of Potentially Acid Generating (PAG) waste rock, and recycling of water for processing), waste rock stockpiles, low grade and overburden stockpiles, a temporary construction camp, ancillary facilities, mine haul roads, sewage and waste management facilities, a 24km access road between the Project site and a rail load-out facility located on private land owned by HCMC in Vavenby, and a 12km power line connecting the Project site to the BC Hydro transmission line corridor in Vavenby.

2 – SCOPE OF REPORT AND KEY REFERENCE DOCUMENTS

In 2012, YMI commissioned Merit Consultants International Inc., Knight Piésold Ltd. (KP), Nilsson Mine Services Ltd., All North Consultants, and other specialist consultants to undertake a Feasibility Study (FS) for the Project. The Technical Report for the FS was filed on SEDAR on March 29, 2012 (Merit, 2012). The FS included technical modelling of seepage potential and stability analyses for the tailings management facility (TMF).

In 2014, KP was retained by HCMC to complete engineering studies and to update the design of the mine waste and water management facilities to contribute to an updated FS for the project. KP revised the technical modeling for the project, including updates to the 2 Dimensional (2D) stability and seepage analyses for the following:

- Tailings Management Facility (TMF)
- Non-PAG Waste Rock Stockpile

This letter presents the results of the revised 2D seepage and stability modeling for the project, and supersedes the findings discussed in the previous study (Knight Piésold, 2012a). This letter discusses the technical modelling approach and findings, and should be read in conjunction with other comprehensive reports that have been developed for the project. The following KP reports are essential to developing a complete understanding of the project mine waste and water management design and predicted project effects:

- **Mine Waste and Water Management Design** – KP report *Mine Waste and Water Management Design Report*, Ref. No. VA101-458/11-1. (Knight Piésold, 2014a)
- **Watershed Modelling** – KP report *Watershed Modelling*, Ref. No. VA101-458/14-1. (Knight Piésold, 2014b)
- **Numerical Groundwater Modelling** – KP report *Numerical Groundwater Modelling*, Ref. No. VA101-458/14-2. (Knight Piésold, 2014c)
- **Water Quality Predictions** – KP report *Water Quality Predictions*, Ref. No. VA101-458/14-3. (Knight Piésold, 2014d)

3 – TAILINGS MANAGEMENT FACILITY SEEPAGE ANALYSES

3.1 MODELLING APPROACH

Steady state seepage analyses were carried out for the main and north embankments to provide preliminary estimates of the seepage through the embankments and foundation materials for the final embankment configuration.

In order to determine the potential for seepage flow along the northwestern and southeastern flanks of the TMF, seepage analyses were completed at two sections of low topography (denoted east Saddle and west Saddle).

The analysed sections for the TMF are identified on Figure 1 and are described as follows:

- Main Embankment: Sections 1, 2 & 3
- North Embankment: Section 6
- East Saddle: Section 4
- West Saddle: Section 5

The seepage analyses were conducted using the 2D finite element computer program SEEP/W (Geostudio, 2007). Sensitivity analyses were also carried out to assess the range of the predicted seepage rates to variation in the saturated hydraulic conductivity of the foundation and embankment materials and variation in the model boundary conditions.

The seepage rate through foundation materials and embankment fill zones will be influenced by the following factors:

- Permeability of the natural glacial till materials that blanket the basin
- Permeability of the Orthogneiss bedrock foundation
- Thickness and permeability of the tailings stored within the TMF
- Permeability of the embankment core zones
- Seepage gradients in the embankment and foundation zones, and
- Seepage area (increases during operations).

The seepage flow rate is expected to vary over the life of the TMF as it is gradually filled with tailings, PAG waste rock materials and supernatant water. The tailings deposit will increase in thickness during operations and the tailings mass will also decrease in permeability due to on-going self-weight consolidation.

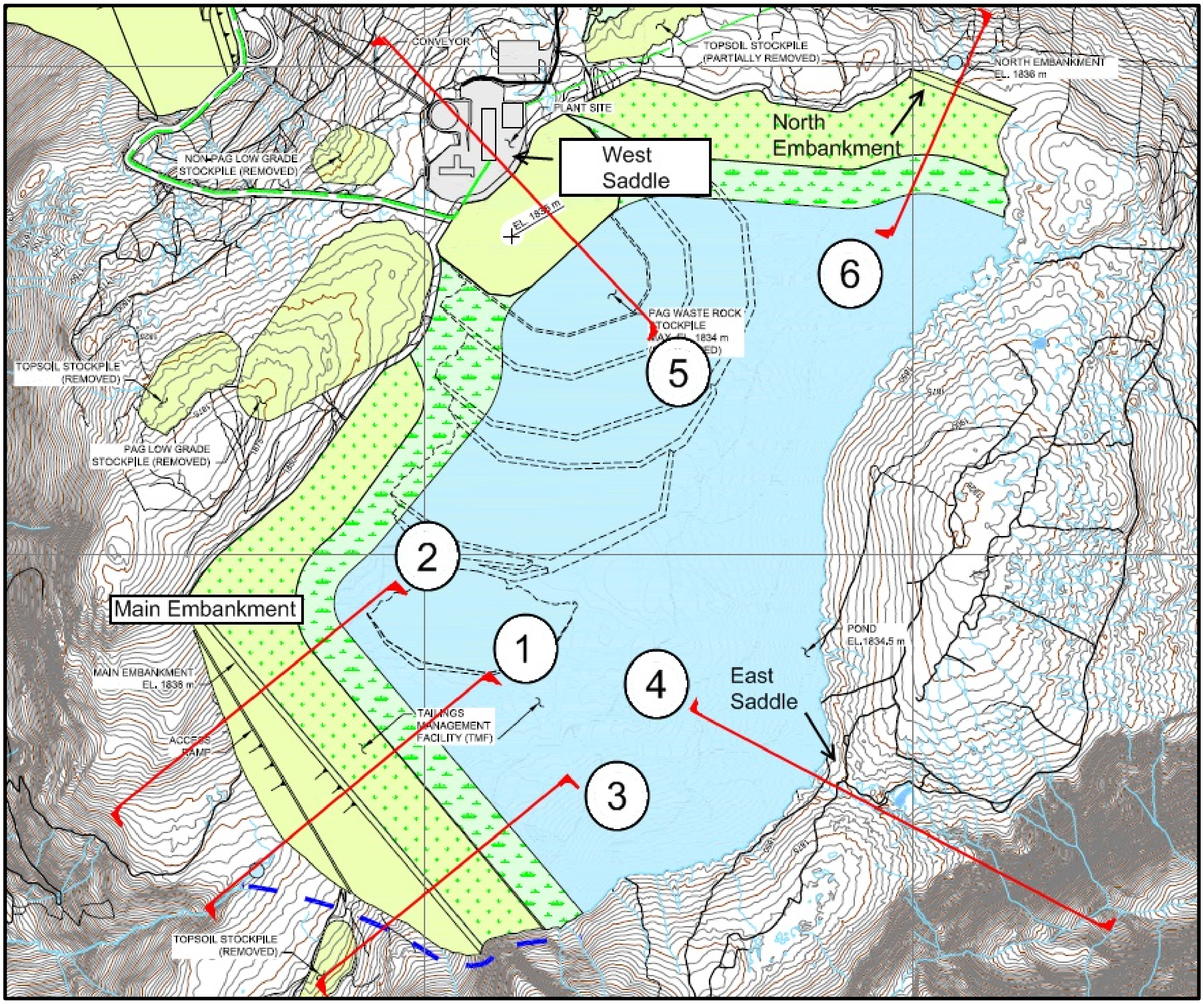

Figure 1 General Arrangement of TMF at Closure with 2D Analysis Sections Identified

3.2 SUMMARY OF MATERIAL PARAMETERS

The following sections provide a description of materials that have been included in the seepage analysis. The saturated hydraulic conductivity of each of the materials was based on published values for anticipated material types and compared with existing in-situ permeability testing or laboratory test results wherever possible to derive a best estimate value. Where the material permeability is expected to be variable, or is expected to have a significant impact on the estimated seepage rates, the sensitivity of the total seepage rates has been assessed by varying the saturated hydraulic conductivity within a reasonable range. Hydraulic conductivity functions for partially saturated soils were estimated based on material type.

The material parameters used in the seepage analyses are summarized in Table 1.

Table 1 Summary of Seepage Analysis Material Parameters

Unit	Saturated or Unsaturated	Horizontal Saturated Hydraulic Conductivity (m/sec)			Anisotropy Ratio (KV:KH)
		Lower Bound	Base Case	Upper Bound	
Embankment Materials					
Zone S (Core)	Saturated or Unsaturated	1E-08	5E-08	1E-07	1
Zone F (Filter)	Saturated or Unsaturated		5E-05		1
Zone T (Transition)	Saturated or Unsaturated		1E-04		1
Zone C (Waste Rock / Shell)	Saturated or Unsaturated		1E-04		1
Waste Materials					
Tailings Beach	Saturated or Unsaturated	1E-07	5E-07	1E-06	0.1
Consolidated Tailings	Saturated or Unsaturated	1E-08	5E-08	1E-07	0.1
Unconsolidated Tailings	Saturated or Unsaturated		5E-07		0.1
PAG Waste Rock	Saturated or Unsaturated		1E-04		1
Foundation Materials					
Overburden (SEE NOTE 1)	Saturated or Unsaturated		5E-07		1
Glacial Till (SEE NOTE 1)	Saturated or Unsaturated	5E-08	1E-07	5E-07	1
Orthogneiss Bedrock (to 30m depth)	Saturated or Unsaturated	5E-08	1E-07	1E-06	1
Orthogneiss Bedrock (30 to 50m depth)	Saturated or Unsaturated	2E-08	5E-08	2E-07	1
Orthogneiss Bedrock (50 to 200m depth)	Saturated or Unsaturated		1E-08		1
Orthogneiss Bedrock (200 to >500m depth)	Saturated		1E-10		1

NOTES:

1. 'Overburden' refers to the moderately permeable foundation material that is expected to comprise a combination of glacial till and colluvium in the vicinity of the non PAG waste rock stockpile and seepage collection dam, whilst 'Glacial Till' refers to the foundation material in the vicinity of the TMF.

3.2.1 Embankment Materials

The materials used in the construction of the embankments will be excavated and/or processed from the open pit and local borrow areas. The embankments will comprise the following zones:

- The core zone (Zone S) will be constructed from low-permeability glacial till from nearby external borrows and from pit stripping. The material will consist of well-graded silty sand with some gravel with a fines content of 20% to 60% passing the #200 sieve. The material will be compacted to 95% standard proctor maximum dry density (SPMDD).
- The filter zone (Zone F) will be processed material and will comprise clean, fine to coarse sand. Zone F will be placed and spread in maximum 600 mm lifts loose and compacted by four to six passes with smooth drum vibratory rollers.
- The transition zone (Zone T) will be processed material and will clean, sand and gravel. Zone T will be placed and spread in maximum 600 mm lifts loose and compacted by four to six passes with smooth-drum vibratory rollers.
- The shell zone (Zone C) will comprise random fill consisting of overburden and specific waste rock material types from the open pit. The material will be compacted by truck traffic in maximum lifts between 1 to 2 m depending on the equipment utilised.

3.2.2 Tailings and Waste Rock Materials

Laboratory testing has been completed on the tailings samples produced during lock cycle metallurgical test work. The tested tailings materials can be described as a non-plastic, fine-grained sandy-silt with traces of clay. The particle size distribution of the tailings sample comprised approximately 46-52% fine sand, 44-50% silt, and 4% clay. The Unified Soil Classification System (USCS) has been used for describing and categorizing soil within groups to allow for the development of distinct soil properties. The tailings can be classified as sand with fines (SM) and a fine-grained soil with very fine sands (ML) depending on the particle size distribution. The tailings material was grouped into three separate units for the purposes of the seepage analysis;

- The ‘tailings beach’ unit represents the higher permeability coarser grained fraction of the tailings that is expected to settle into the tailings basin over the length of the beach as the tailings slurries migrate towards the TMF pond
- The ‘consolidated tailings’ unit represents the tailings materials that have consolidated under considerable self-weight over the life of the project. A clear boundary between consolidated and unconsolidated tailings will not exist, however for modelling purposes this has been approximated to half the depth of the tailings impoundment.
- The ‘unconsolidated tailings’ unit represents the portion of tailings that are undergoing ongoing self-weight consolidation.

The PAG waste rock from the open pit will be placed in the TMF impoundment for subaqueous disposal. For the purposes of the seepage analysis, the PAG waste rock material is assigned the same saturated hydraulic conductivity as the shell zone (Zone C) waste rock.

3.2.3 Foundation Materials

<u>Overburden Materials</u>

The overburden thickness in the vicinity of the embankments is a glacial till material that is found to range in thickness from scarce to approximately 10 m. An average thickness was chosen to represent the overburden layer in the numerical models. The glacial till material was characterized through visual classification and laboratory particle size analysis testing. The details of the site investigation and laboratory program were presented in the 2011 Site Investigation Report (Knight Piésold Ltd., 2012a). The overburden typically consisted of silty-sand with some gravel, and is classified by the USCS as a coarse grained soil with gravel and fines (SM-SC and GM-GC).

The USCS classification group allows for comparison of anticipated geotechnical properties of the soil with published typical ranges of these properties. These properties include permeability, shear strength, compaction characteristics, workability and volume change potential of a soil, and how it will be affected by water, frost and other physical conditions. The range of material parameters was verified with respect to the expected hydraulic conductivity ranges published in Freeze and Cherry (1979).

Orthogneiss Bedrock

The bedrock unit in the vicinity of the TMF footprint comprises orthogneiss. Bedrock characterization undertaken during the 2011 site investigation program (Knight Piésold, 2012b) identified that the orthogneiss has a mean RMR of 68, a mean RQD of 74%, and a mean intact Uniaxial Compressive Strength of approximately 130 MPa. No distinct weathering profile was observed. During the site program, hydrogeological testing was completed in order to estimate the in situ hydraulic conductivity of the orthogneiss. Lugeon testing (single packer) was completed in all geotechnical and geomechanical drillholes, and falling head response testing was conducted following standpipe piezometer or monitoring well installation. The hydraulic conductivity of the orthogneiss was shown to generally decrease with depth. A plot of hydraulic conductivity values measured during the testing compared with test interval depth is shown on Figure 2.

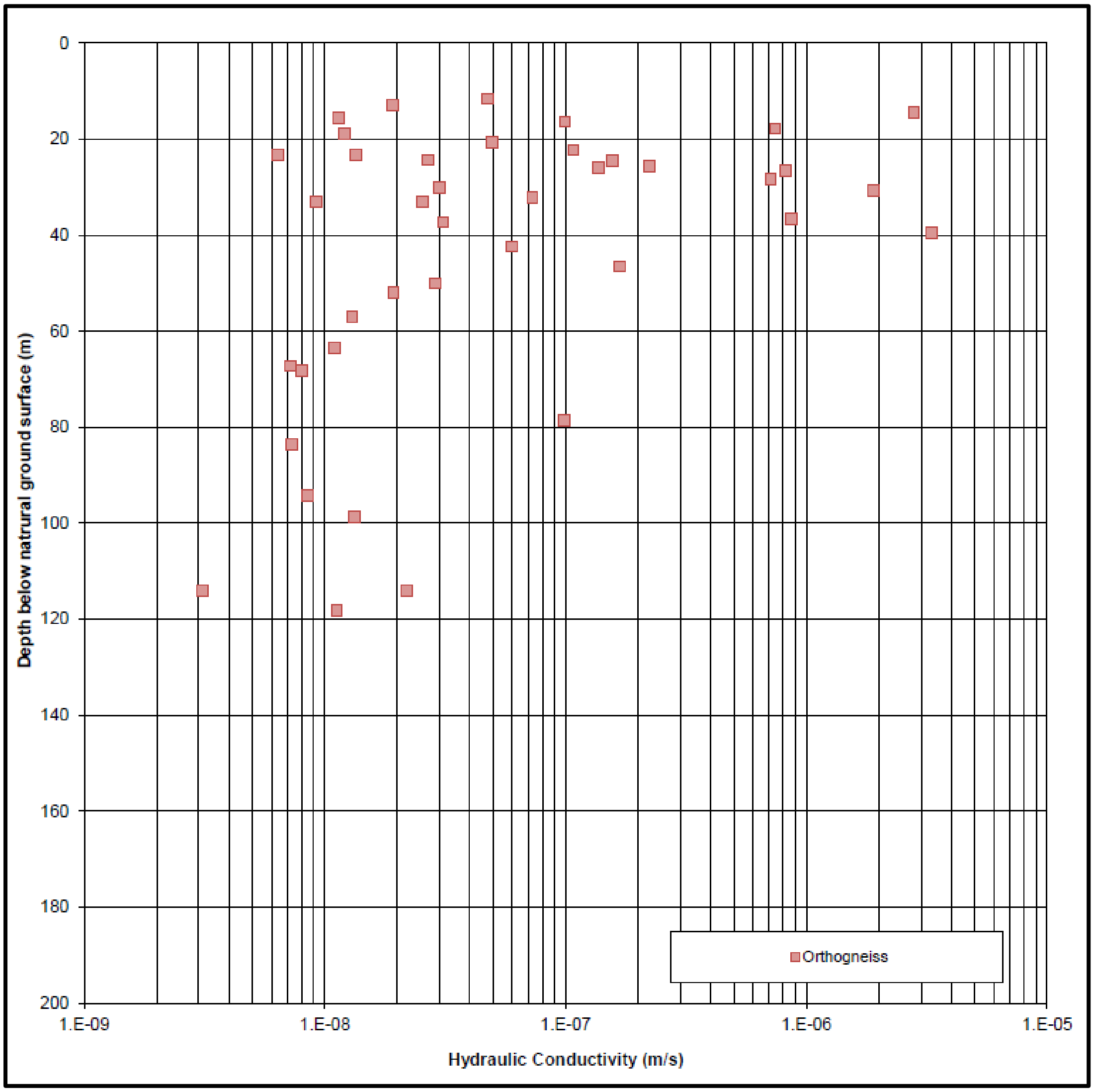

Figure 2 Hydraulic Conductivity Testing Summary – Orthogneiss Bedrock

3.3 BOUNDARY CONDITIONS AND FLUX SECTIONS

Boundary conditions used in the seepage analyses were selected to represent the hydrogeological conditions expected during operation of the TMF. The boundary conditions used in the analyses are summarized as follows:

- A total head boundary was used to represent the phreatic surface at the upstream side of the embankment for the final embankment elevations. A final embankment pond elevation of 1,834 m was modelled with a 300 m tailings beach as the base case condition.
- A total head boundary was used to represent the phreatic surface at the downstream extent of the models. The downstream phreatic surface was set at approximately 2 m below natural ground surface.
- A seepage face boundary condition was applied to the downstream face of the dam and the downstream natural ground surface to estimate the seepage flow expected to exit the ground within the model extents. The seepage flowing out of the embankment dam face was recovered and returned to the tailings pond via the seepage collection pond whilst the seepage exiting the ground downslope of the embankment dam lost to the watershed.
- A seepage face boundary condition was applied to the base of the transition zone to model the presence of a longitudinal PVC drain. The seepage flow exiting the model via this drain was recovered and returned to the tailings pond via the seepage collection pond.
- As a sensitivity case, a recharge value of 1×10^{-8} m/sec (315 mm/year) was applied to the beach of the main embankment dam sections to assess the effect of tailings water (transport water) and precipitation infiltration on the total seepage flow rates.
- As a sensitivity case, a recharge value of 1×10^{-9} m/sec (31.5 mm/year) was applied to the downslope ground surface of the saddle sections to assess the effect of precipitation infiltration on the total seepage flow rates.

Flux sections were located in key areas of the seepage models to estimate total, recovered and potentially unrecoverable seepage flows.

3.4 SEEPAGE FLOW CALCULATION METHODOLOGY

The seepage models provided an estimate of the unit seepage rate (per lineal metre of embankment) through each representative section.

The main embankment was divided into three sections (Sections 1, 2 and 3 as identified on Figure 1) and the unit seepage rates were estimated for each section. The total seepage flow was estimated by establishing a linear function between unit flow rate and dam height across the length of the dam from the three representative sections.

The seepage rates for the north embankment (Section 6), east saddle (Section 4) and the west saddle (Section 5) were estimated using a single representative cross section at each location. The total seepage flow was calculated by establishing a linear function between unit flow rate, section height, and a representative length for each section. The seepage estimate is reported by means of the following metrics:

- Total Seepage (l/s) – Indicates the total tailings seepage estimated to permeate through the TMF embankments and foundation for each section.
- Unrecoverable Seepage (l/s) – Indicates the total tailings seepage estimated to be unrecoverable and could reach the watershed downstream of the TMF with the planned seepage controls in place.
- Unrecoverable Seepage as a Percentage of Total Seepage (%) – Indicates the proportion of unrecoverable seepage relative to the total seepage originating from the TMF. This is considered a useful metric to evaluate the effectiveness of the water management features.

Representative cross sections through the main embankment, north embankment, east saddle and west saddle are shown on Figures A-1 through A-5 in Appendix A.

3.5 BASE CASE SEEPAGE RESULTS

3.5.1 Base Case Seepage Estimates

The base case seepage was estimated using the base case parameters identified in Table 1.

The base case total seepage through the main embankment and foundation was predicted to be approximately 14 l/s at the end of operations at a final embankment crest elevation of 1836 m. Approximately 1 l/s (7%) was estimated to be unrecoverable and lost to the watershed. The remaining amount was recovered in the seepage collection system and returned to the TMF.

The base case total seepage through the north embankment and foundation was predicted to be approximately 0.10 l/s at the end of operations at a final embankment crest elevation of 1836 m. The analysis indicated that the majority of this seepage will infiltrate into the foundation and will be unrecoverable, however in practice it is expected a portion of this total seepage will be recovered in the downslope seepage collection system and will be returned to the TMF.

The base case total seepage through the foundation in the vicinity of the east saddle and west saddle was estimated to be 0.11 l/s and 0.07 l/s respectively and at the end of operations and at a final pond elevation of 1834 m.

In practice, precipitation recharge on the downslope side of TMF the embankment is expected to reduce the hydraulic gradient across these saddles and the net total seepage is expected to be negligible.

3.6 MATERIAL PARAMETER SENSITIVITY ANALYSIS SEEPAGE RESULTS

A material parameter sensitivity analysis was completed for each of the sections. The sensitivity analyses were undertaken by investigating the change in total seepage estimate when the saturated hydraulic conductivity of a single material was varied in isolation. Hydraulic conductivity parameters were varied for the following materials:

- Zone S (core zone material)
- Tailings Beach Material (coarse grained tailings)
- Consolidated Tailings
- Glacial Till
- Orthogneiss Bedrock to 30 m depth, and
- Orthogneiss Bedrock from 30 m to 50 m depth.

The following sections describe the results of the material parameter sensitivity analysis completed for each of the analysis sections. Plots of the sensitivity analysis results are provided in Appendix B.

3.6.1 Main Embankment (Sections 1, 2 & 3)

The results of the sensitivity analysis for the main embankment are presented in Table 2 (below) and Figure B-1 and Figure B-2 (Appendix B). The results indicate that within the range of saturated hydraulic conductivity values selected, the main embankment dam seepage estimate is particularly sensitive to the saturated hydraulic conductivity of the 'Tailings Beach' material and the uppermost layer of the orthogneiss bedrock (<30 m depth below natural ground level (ngl)). The unrecoverable seepage is shown to be most notably sensitive to the saturated hydraulic conductivity of the uppermost layer of the orthogneiss bedrock (<30 m depth below ngl).

Table 2 Upper, Lower Bound and Base Case Seepage Estimates – Main Embankment

Sensitivity Analysis NOTE 1	Lower Bound	Base Case	Upper Bound
Material	**Total Seepage (l/s)**		
Zone C	13	14	15
Tailings Beach	9		17
Consolidated Tailings	13		15
Glacial Till	14		15
Orthogneiss Bedrock (to 30 m depth)	14		19
Orthogneiss Bedrock (30 to 50 m depth)	14		15
Material	**Unrecoverable Seepage (l/s)**		
Zone C	1	1	1
Tailings Beach	1		1
Consolidated Tailings	1		1
Glacial Till	1		1
Orthogneiss Bedrock (to 30 m depth)	1		4
Orthogneiss Bedrock (30 to 50 m depth)	1		2
Material	**Unrecoverable Seepage as a percentage of Total (%)**		
Zone C	9	7	7
Tailings Beach	11		6
Consolidated Tailings	8		7
Glacial Till	7		9
Orthogneiss Bedrock (to 30 m depth)	7		18
Orthogneiss Bedrock (30 to 50 m depth)	6		12

NOTES:

1. The Base Case seepage estimate was completed as a single case using the Base Case material parameters as identified in Table 1. The Lower Bound and Upper Bound seepage estimates were completed using the Lower and Upper bound seepage parameters as identified in Table 1, with the sensitivity of each material varied in isolation for each respective case.

3.6.2 North Embankment (Section 6)

The results of the sensitivity analysis for the north embankment are presented in Figure B-3 and Figure B-4 (attached). The results indicate that within the range of saturated hydraulic conductivity values selected, the north embankment seepage estimate is particularly sensitive to the saturated hydraulic conductivity of the uppermost layer of the orthogneiss bedrock (<30 m depth below ngl) and the second layer of orthogneiss bedrock (30 to 50 m depth below ngl). For each case, the estimate of unrecoverable seepage is expected to be over 95% of the total seepage estimate.

3.6.3 East Saddle (Section 4)

The results of the sensitivity analysis for the east saddle are presented in Figure B-5. The results indicate that that within the range of saturated hydraulic conductivity values selected, the east saddle seepage estimate is sensitive to the saturated hydraulic conductivity of the uppermost layer of the Orthogneiss bedrock (<30 m depth below ngl) and the second layer of orthogneiss bedrock (30 to 50 m depth below ngl) with an upper bound total seepage estimate of 0.20 l/s.

3.6.4 West Saddle (Section 5)

The results of the sensitivity analysis for the west saddle are presented in Figure B-6. The results indicate that within the range of saturated hydraulic conductivity values selected, the North embankment dam seepage estimate is sensitive to the saturated hydraulic conductivity of the uppermost layer of the Orthogneiss bedrock (<30 m depth below ngl) with an upper bound total seepage estimate of 0.39 l/s.

3.7 BOUNDARY CONDITIONS SEEPAGE SENSITIVITY ANALYSIS

3.7.1 Effect of Recharge Water on Tailings Beach

A recharge boundary condition of 1 x 10^{-8} m/sec (315 mm/year) was applied to the tailings beach at the main embankment to assess the effect of tailings transport water and precipitation on the total seepage rates. The total seepage estimate for the main embankment was found to increase to 19 l/s (132% of the base case estimate) with 1 l/s unrecovered seepage (unchanged).

3.7.2 Effect of Tailings Beach

In normal operating conditions, the tailings beach is expected to extend approximately 300 m from the main embankment crest. A scenario was modelled to determine an upper bound seepage estimate assuming the supernatant pond was allowed to reach the embankment dam (i.e. no tailings beach). The result was an increase in total seepage by an order of magnitude, with a total seepage of approximately 160 L/s. Unrecoverable seepage did not increase in this scenario, indicating in this upper bound case, seepage could still be captured at the downstream water management pond and recycled back to the TMF for long-term storage.

4 – TAILINGS MANAGEMENT FACILITY STABILITY ANALYSES

4.1 MODELLING APPROACH

Stability analyses of the TMF embankment were carried out to investigate the slope stability under both static and seismic loading conditions. The following cases were evaluated:

- Static conditions during operations and post-closure.
- Earthquake loading from the Operating Basis Earthquake (OBE), the Maximum Design Earthquake (MDE), and Earthquake loading from the 1:10,000 year earthquake event.
- Post-earthquake conditions using residual (post-liquefaction) tailings strengths.

Representative cross sections through the main and north embankments were based on the geotechnical foundation conditions and the maximum section for each embankment. The analyses were carried out for the following embankment configurations:

- Final embankment (crest elevation 1836 m) with full tailings storage and pond elevation at 1834 m.
- Stage 1 embankment (crest elevation 1720 m) with no tailings deposition and no retained water (main embankment only – upstream failure mode).
- Stage 1 embankment (crest elevation 1720 m) with no tailings deposition and pond water level at 1718 m (main embankment only – downstream failure mode).

The stability analyses were carried out using the limit equilibrium computer program SLOPE/W (Geostudio, 2007). In this program a systematic search is performed to obtain the minimum factor of safety from a number of potential slip surfaces. Factors of safety have been computed using the Morgenstern-Price Method.

In accordance with international recommendations (ICOLD, 1995) and standard industry practice, the minimum acceptable factor of safety for the tailings embankment under static conditions is 1.5 for normal operating conditions and for long-term (post-closure) of the TMF. A factor of safety of less than 1.0 is acceptable for earthquake loading conditions provided that calculated embankment deformations resulting from seismic loading are not significant and that the post-earthquake stability of the embankment maintains a factor of safety greater than 1.2, to ensure there is no potential for a flow-slide failure following liquefaction. Limited deformation of the embankment is acceptable under seismic loading from the MDE, provided that the overall stability and integrity of the TMF is maintained and that there is no release of stored tailings or water. Some remediation may be required following the MDE.

4.2 MATERIAL PARAMETERS AND ASSUMPTIONS

The following parameters and assumptions were incorporated into the stability analyses:

- Bulk unit weights for the embankment and foundation materials were based on laboratory testing or typical values for similar materials.
- An undrained shear strength was adopted to represent the tailings material strength for the static, seismic and post-earthquake cases, as described by the following relation:
  - $S_u/p' = 0.25$ (static and seismic loading)
  - Su/p' = 0.10 (post liquefaction residual strength), where;
    - S_u = undrained shear strength, and
    - p' = effective vertical stress.
- Effective strength parameters for the embankment fill and foundation materials were estimated based on typical values for similar materials.
- The shear strength for Zone C was defined using a conservative strength function that defines the variation with shear strength with normal stress. This strength function is based on published information on the shear strength properties of rockfill (Leps, 1970).
- A piezometric line was used to represent the predicted phreatic surface in the stability analysis as determined from the seepage analysis.

The material strength parameters adopted for the stability analyses are summarized in Table 3.

The embankment geometries analyzed for the main embankment are shown on Figures C-1 and C-2 (Appendix C) for the Stage 1A embankment and final embankment, respectively. The geometry of the final north embankment used in the stability analyses is shown in Figure C-3.

Table 3 Material Strength Parameters

Unit	Unit Weight (kN/m^3)	Friction Angle (deg)	Cohesion (kPa)
Embankment Materials			
Zone S (Core)	22	34	0
Zone F (Filter)	21	36	0
Zone T (Transition)	21	36	0
Zone C (Waste Rock / Shell)	23	See Note 1	
Tailings Materials			
Tailings Beach	18	See Note 2	
Consolidated Tailings	18	See Note 2	
Unconsolidated Tailings	18	See Note 2	
Waste Rock			
Non PAG Waste Rock	23	See Note 1	
PAG Waste Rock	23	See Note 1	
Foundation Materials			
Overburden (See Note 1)	22	36	0
Glacial Till (See Note 1)	22	36	0
Orthogneiss Bedrock	Impenetrable		

NOTES:

1. A relationship for friction angle and effective stress was developed for the rockfill materials, based on published information on the shear strength properties of rockfill (Leps, 1970).
2. A relationship for shear stress and effective normal stress (S_u/p') was used to model the tailings strength. The (S_u/p') values used for the analyses were 0.25 for static and seismic loading and 0.1 for liquefied tailings.

4.3 RESULTS OF STABILITY ANALYSIS

4.3.1 Static Analyses

The calculated Factors of Safety (FOS) for each of the dam sections considered in this study exceed the minimum Factor of Safety requirement of 1.5 for static normal operating (steady-state) conditions. In addition, calculated FOS for short term stability of the upstream starter embankment dam (prior to tailings deposition) exceeds the minimum Factor of Safety of 1.3 for static operating (steady-state) conditions. It should be further noted that the critical surface identified for each static analysis does not result in any loss of freeboard as the critical failure surface is shown not to pass through the dam crest. A summary of the Factors of Safety (FOS) for the cases analysed are presented in Table 4.

Table 4 Static Analyses Results Summary

Description	Minimum FOS	Comments
TMF Main Embankment at EL 1836 m (Final Height)		
Normal Operating Conditions	1.56	-
TMF Main Embankment at EL 1720 m (Starter Embankment – Stage 1A)		
Normal Operating Conditions	1.71	-
Normal Operating Conditions – Failure of upstream slope	1.42	-
Normal Operating Conditions – Pond at EL 1718 m	1.63	No tailings deposition, water in impoundment to EL 1718 m
TMF North Embankment at EL 1836 m (Final Height)		
Normal Operating Conditions	2.04	-

NOTES:
1. Only slip surfaces with a minimum of 2 m depth have been considered in the analysis.

4.3.2 Seismic Stability and Deformation Analyses

A seismic stability assessment of the TMF has included estimation of earthquake induced deformation of the embankment from the OBE, MDE, and the 1:10,000 event. The design ground motion parameters for the design earthquake events have been provided by the seismic hazard analysis completed for the project (Knight Piésold, 2012c).

The OBE has been defined as the 1 in 475 year earthquake with a mean Peak Ground Acceleration (PGA) of 0.08g. A design earthquake magnitude of 7 was adopted for the OBE.

The MDE has been assessed to correspond with the Earthquake Design Ground Motion (EDGM) as per table 6-1B of the 2013 revision to the 2007 CDA Dam Safety Guidelines. The guidelines revision states that the EDGM for a Dam Class ‘Very High’ should be selected based on the mean PGA corresponding to halfway between the PGA for the 1 in 2,475 year earthquake and the PGA for the 1 in 10,000 year earthquake. This corresponds to a PGA of 0.21g. A design earthquake magnitude of 7.3 was adopted for the MDE.

The PGA acceleration for the 1:10,000 year event has also been considered to demonstrate the robustness of the embankment design in closure to seismic loading. The 1 in 10,000 year earthquake corresponds with a PGA of 0.26g. A design earthquake magnitude of 7.3 was adopted for the 1:10,000 year event.

Embankment stability during earthquake loading from the OBE, MDE and 1:10,000 year event has been assessed by performing pseudo-static analysis, whereby a horizontal force (seismic coefficient) is applied to the embankment to simulate earthquake loading. The yield acceleration required to reduce the factor of safety to 1.0 was determined by iterative stability analyses. Deformation of the embankment is predicted to occur if the yield acceleration is lower than the average maximum ground acceleration along the potential slip surface from the earthquake.

Potential deformations under earthquake loading from the design earthquake events have been estimated using the simplified methods of Newmark (1965) and Makdisi-Seed (1977). These two methods estimate displacement of the potential sliding mass based on the average maximum ground acceleration along the slip surface and the yield acceleration.

The more recently published method of Bray (2007) was also used to predict seismically induced slide displacement of the embankment. In addition to the yield acceleration, this method considers the predominant period of response (Ts) of the embankment under seismic loading and the corresponding spectral ground acceleration (Sa). The predominant period is related to the stiffness characteristics of the embankment fill and to the height of the embankment. Spectral acceleration values were provided by the uniform hazard spectrum defined for each design earthquake event. The uniform hazard spectra for the design earthquake events were defined from the results of the site specific probabilistic seismic hazard analysis (Knight Piésold, 2012c).

The estimated yield acceleration is 0.2g for the Main Embankment at final height, between 0.18g and 0.23g for the Main Embankment at the starter height (elevation 1720 m) and 0.35g for the North Embankment at final height. Predicted embankment deformations under seismic loading are negligible, if any, as the calculated yield acceleration either exceeds, or is only slightly lower than the estimated average PGA values for the OBE and MDE events. For the 1:10,000 event, the estimated deformations are very small (<0.03 m) and do not impact the embankment freeboard or result in any loss of embankment integrity.

Some deformation of the embankment is expected to result from settlement of the fill materials during earthquake shaking. Potential settlement of the embankment crest has been estimated using the empirical relationship provided by Swaisgood (2003). This relationship was developed from an extensive review of case histories of embankment dam behaviour due to earthquake loading. Required inputs to the relationship are the earthquake magnitude, the maximum acceleration on rock at the site, the depth to rock (overburden thickness) and the embankment height. The predicted maximum crest settlements for the Main Embankment at final height are approximately 0.05 m for the OBE, 0.14 m for the MDE and 0.19 m for the 1:10,000 year event. The predicted maximum crest settlements for the North Embankment at final height are minor (<0.02) for all design earthquake events.

The calculated yield accelerations and corresponding estimated embankment deformations and crest settlements for each of the methods described above are presented in Table 5.

The predicted maximum embankment displacements and potential crest settlements under seismic loading from the OBE and MDE are acceptable and would not significantly impact embankment freeboard or result in any loss of embankment stability or integrity. The performance and integrity of the embankment core, drainage and filter zones would not be impacted by the predicted deformations.

The findings of the seismic stability analyses indicate that the TMF would remain stable and function normally after the OBE, MDE and 1:10,000 year event.

Table 5 TMF Seismic Displacement Results Summary

Description	Design PGA[1] (g) Mean[2]	Design Earthquake Magnitude	Calculated Yield Acceleration (K_Y)[3]	Displacement Along Slip Surface (m)			Crest Settlement (m)
				Newmark[4]	Makdisi-Seed (Average)[4]	Bray ($D_{84\%}$)[5]	Swaisgood[6]
TMF Main Embankment at EL 1836 m (Final Height)							
OBE	0.08	7	0.20	0.00	0.00	0.00	0.05
MDE	0.21	7.3	0.20	0.00	0.02	0.00	0.14
1:10,000 event	0.26	7.3	0.20	0.01	0.03	0.02	0.19
TMF Main Embankment at EL 1720 m (Starter Embankment – Stage 1A)							
OBE – Full tailings height volume	0.08	7	0.18	0.00	0.00	0.00	0.02
MDE – Empty Impoundment	0.21	7.3	0.18	0.01	0.04	0.01	0.05
MDE – Pond at EL 1718 m	0.21	7.3	0.23	0.00	0.00	0.01	0.05
TMF North Embankment at EL 1836 m (Final Height)							
OBE	0.08	7	0.35	0.00	0.00	0.00	0.01
MDE	0.21	7.3	0.35	0.00	0.01	0.01	0.02
1:10,000 Event	0.26	7.3	0.35	0.00	0.00	0.01	0.02

NOTES

1. The design maximum acceleration is for site class C conditions (defined as soft rock or very dense soils).
2. Mean acceleration values are conservatively estimated by multiplying the median acceleration value by 1.15. Mean acceleration values are recommended for dam design by the Canadian Dam Association "Dam Safety Guidelines" (2007).
3. The yield acceleration (ky) corresponds to the horizontal seismic coefficient (acceleration) required to reduce the factor of safety to 1.0
4. The Newmark (1965) and Makdisi-Seed (1977) methods estimate potential displacement along the critical slip surface.
5. The Bray (2007) method estimates potential displacement taking into consideration the fundamental period of the structure (Ts) and the ground motion's spectral acceleration at a degraded period equal to 1.5Ts.
6. The Swaisgood (2003) method estimates the predicted vertical settlement of the dam crest
7. Slip surfaces are a minimum of 2 m depth

4.3.3 Post-Liquefaction Stability Analysis

A stability assessment of the TMF has been undertaken to assess the static stability of the embankments following an earthquake event. The calculated Factors of Safety (FOS) for each of the dam sections considered in this study exceed the minimum Factor of Safety requirement of 1.2 for post liquefaction stability.

The post-earthquake condition conservatively assumes complete liquefaction of the tailing deposit and assumes a post-liquefaction residual strength for the entire tailings deposit. For each of the dam sections the calculated minimum factors of safety are the same as the static factor of safety as the critical potential slip surface does not pass through the liquefied tailing deposit. This indicates that the TMF embankment is not dependent on tailing strength to maintain stability and is not susceptible to a flow slide or large deformations resulting from earthquake-induced liquefaction of the tailing deposit.

A summary of the Factors of Safety (FOS) for the cases analysed are presented in Table 6.

Table 6 Post-Liquefaction Analyses Results Summary

Description	Minimum FOS	Comments
TMF Main Embankment at EL 1836 m (Final Height)		
Post Liquefaction Stability - Reduced Tailings Strength	1.56	Failure does not propagate into tailings (see Note 2)
TMF North Embankment at EL 1836 m (Final Height)		
Post Liquefaction Stability - Reduced Tailings Strength	2.04	Failure does not propagate into tailings (see Note 2)

NOTES:
1. Only slip surfaces with a minimum of 2 m depth have been considered in the analysis.
2. The post liquefaction Factor of Safety is the same as the pre earthquake static case as critical potential slip surfaces do not pass through the tailings deposit.

5 – NON PAG WASTE STOCKPILE STABILITY

The non PAG waste stockpile was assessed against the Dump Stability Rating (DSR) scheme from the Investigation and Design Manual Interim Guidelines (BC MWRPRC, 1991). A stability analysis was also undertaken to determine the factors of safety for the stockpile.

5.1 WASTE STOCKPILE STABILITY RATING SCHEME

The Investigation and Design Manual Interim Guidelines (BC MWRPRC, 1991) provides recommendations for stability assessment of mine waste piles. These guidelines include a Dump Stability Rating (DSR) scheme. The DSR system provides a semi-quantitative method for assessing the relative potential of dump stability and recommends the appropriate level of investigation and design. This is based on individual point ratings for each of the main factors affecting dump stability. Each factor is given a point rating based on qualitative and/or quantitative descriptions accounting for the possible range of conditions. An overall DSR is calculated as the sum of the individual ratings for each of the various factors. Copies of Table 5.1 “Dump Stability Rating Scheme” and Table 5.2 “Dump Stability Classes and Recommended Level of Effort” from the waste dump research committee guidelines are included in Appendix D.

The dump rating guidelines were used to classify the Non PAG Waste Stockpile. A summary of the results are presented in Table 7. The Non-PAG Waste Stockpile is classified as Class III, Moderate Hazard. The Moderate Hazard classification recommends that additional site investigations, including laboratory testing and a detailed stability analysis be completed for the next level of detailed design.

Table 7 Non-PAG Waste Rock Stockpile Stability Classification

Key Factors Affecting Stability[(1)]	Condition	Point Rating
Dump Height	100 - 200 m	100
Dump Volume	Large	100
Dump Slope	Moderate	50
Foundation Slope	Moderate	50
Degree of Confinement	Confined	0
Foundation Type	Intermediate	100
Dump Material Quality	Moderate	100
Method of Construction	Mixed	100
Piezometric & Climatic Conditions	Intermediate	100
Dumping Rate	Moderate	100
Seismicity	Moderate	50
DUMP STABILITY RATING		**850**
	Class	**Failure Hazard**
Dump Stability Class[(2)]	III	Moderate

In general, the dump stability classification indicates a basic stability analysis is required. In accordance with provincial guidelines (BC MWRPRC, 1991) and standard industry practice, the minimum acceptable factor of safety for waste dumps under static conditions is 1.3 for short-term operating conditions, 1.5 after reclamation and abandonment and 1.0 for a pseudo-static analysis. The BC Mine Waste Rock Pile Research Committee (MWRPRC) interim guidelines for design factors of safety are presented in Appendix D (Table 6.4).

5.2 NON-PAG WASTE STOCKPILE STABILITY ANALYSES

Slope stability analyses for the non PAG Waste Stockpile were carried out for the final design height of the stockpile (closure condition). The stability analyses were carried out using the 2D finite element software SLOPE/W (Geostudio, 2007) along the section identified in plan on Figure 3. The analysis was undertaken to assess the stability of the maximum height of the stockpile slope. The effect of the interaction of the waste stockpile on the open pit slope stability was not assessed for this study.

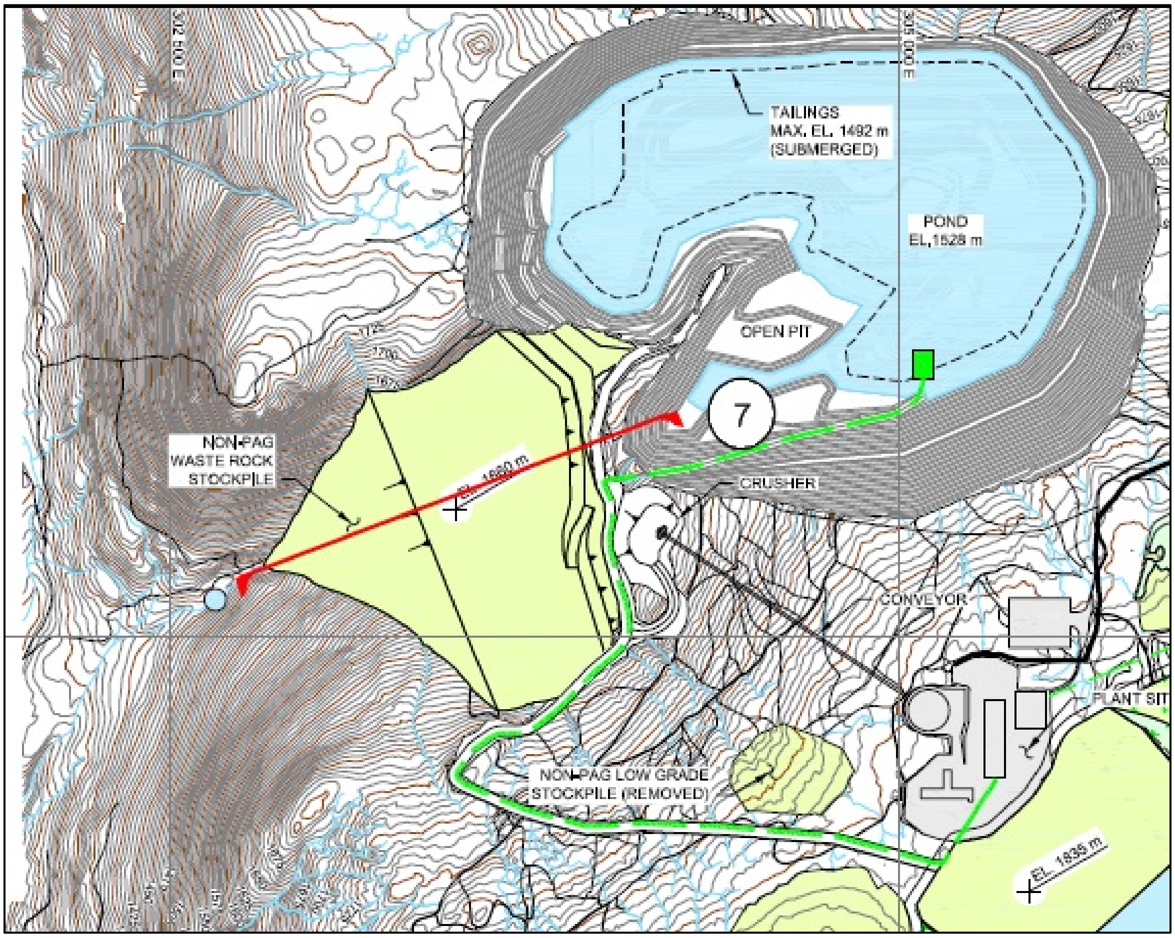

Figure 3 **Non PAG Waste Stock Pile General Arrangement at closure with 2D analysis section (Section 7) identified**

The static Factor of Safety against failure is 1.52 and the pseudo-static Factor of Safety against failure from an applied PGA corresponding to the 1:475 event (defined as the event which has a 10% probability of exceedance in 50 years) was determined to be 1.38. Both the static and pseudo-static Factors of Safety exceed the minimum design Factors of Safety as presented in Table 6.4 of the BC MWRPRC (1991) and included in Appendix D. The critical potential failure surface and factor of safety for the static condition is shown on Figure C-4.

In order to demonstrate the robustness of the design, seismic displacements were estimated according to the methods of Newmark (1965), Makdisi and Seed (1977), Bray (2007) and Swaisgood (2003) (described in detailed in Section 4.3.2). The ground motion parameters for the 1:10,000 year events as identified in the TMF stability analysis were used to estimate the seismic displacements for the waste stockpile. The estimated yield acceleration is 0.19g. Predicted displacements under seismic loading for the 1:10,000 event are shown to be negligible and estimated crest settlement is 0.29 m.

6 – REFERENCES

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7 – CLOSURE

This letter report presents a summary of the stability and seepage analyses undertaken for the Harper Creek mining project to date.

We trust the information contained herein meets your needs at this time. Should you required additional information please contact the undersigned.

Yours truly,
KNIGHT PIESOLD LTD.

PROFESSIONAL
PROVINCE OF
A. C. ROBB
174229
BRITISH COLUMBIA
ENGINEER
19.08.2014.

Signed: Angus Robb, P.Eng.
Project Engineer

Reviewed: Daniel Fontaine, P.Eng
Senior Engineer

Approved: Bruno Borntraeger for
Ken Brouwer, P.Eng.
President

Attachments:

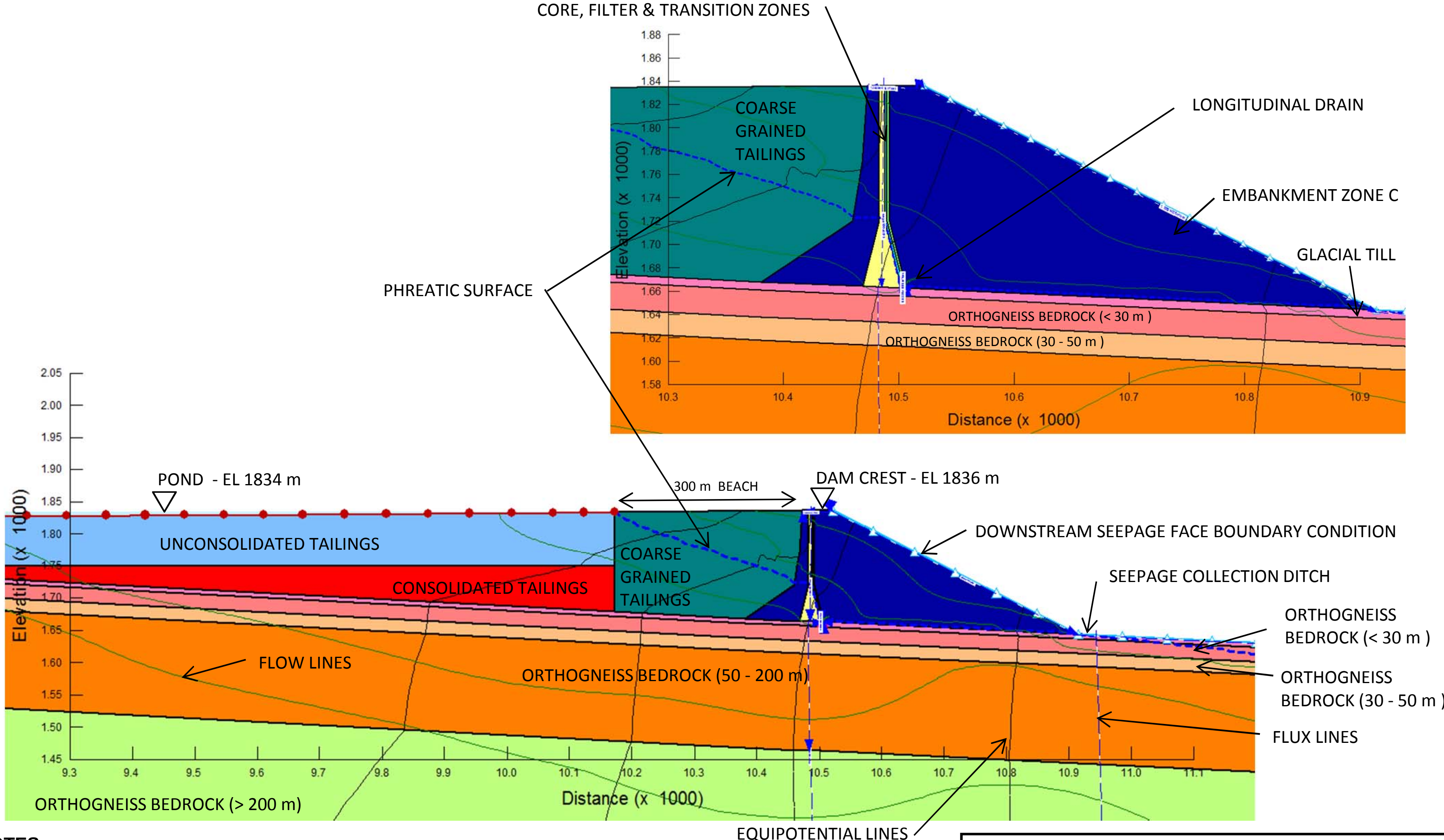
Appendix A	Seepage Analysis Figures
Appendix B	Seepage Sensitivity Analysis Plots
Appendix C	Stability Analysis Plots
Appendix D	Selected Tables from the Investigation and Design Manual Interim Guidelines (BC MWRPRC, 1991)

/ar

APPENDIX A

SEEPAGE ANALYSIS FIGURES

(Figures A-1 to A-5)



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS.

0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
SEEPAGE ANALYSIS
MAIN EMBANKMENT - SECTION 1

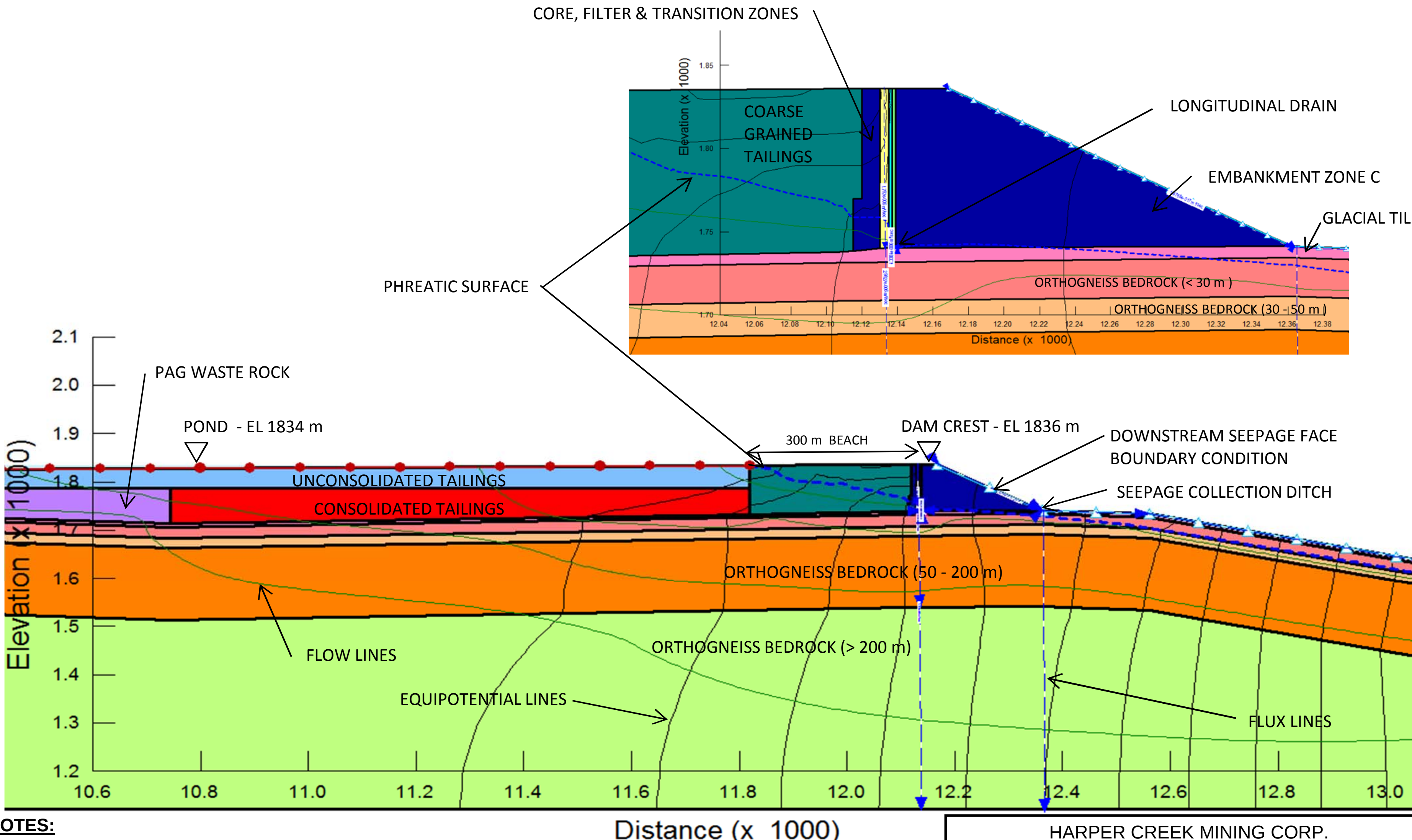
Knight Piésold CONSULTING

P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE A-1

REV 0



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS.

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
SEEPAGE ANALYSIS
MAIN EMBANKMENT - SECTION 2 & 3

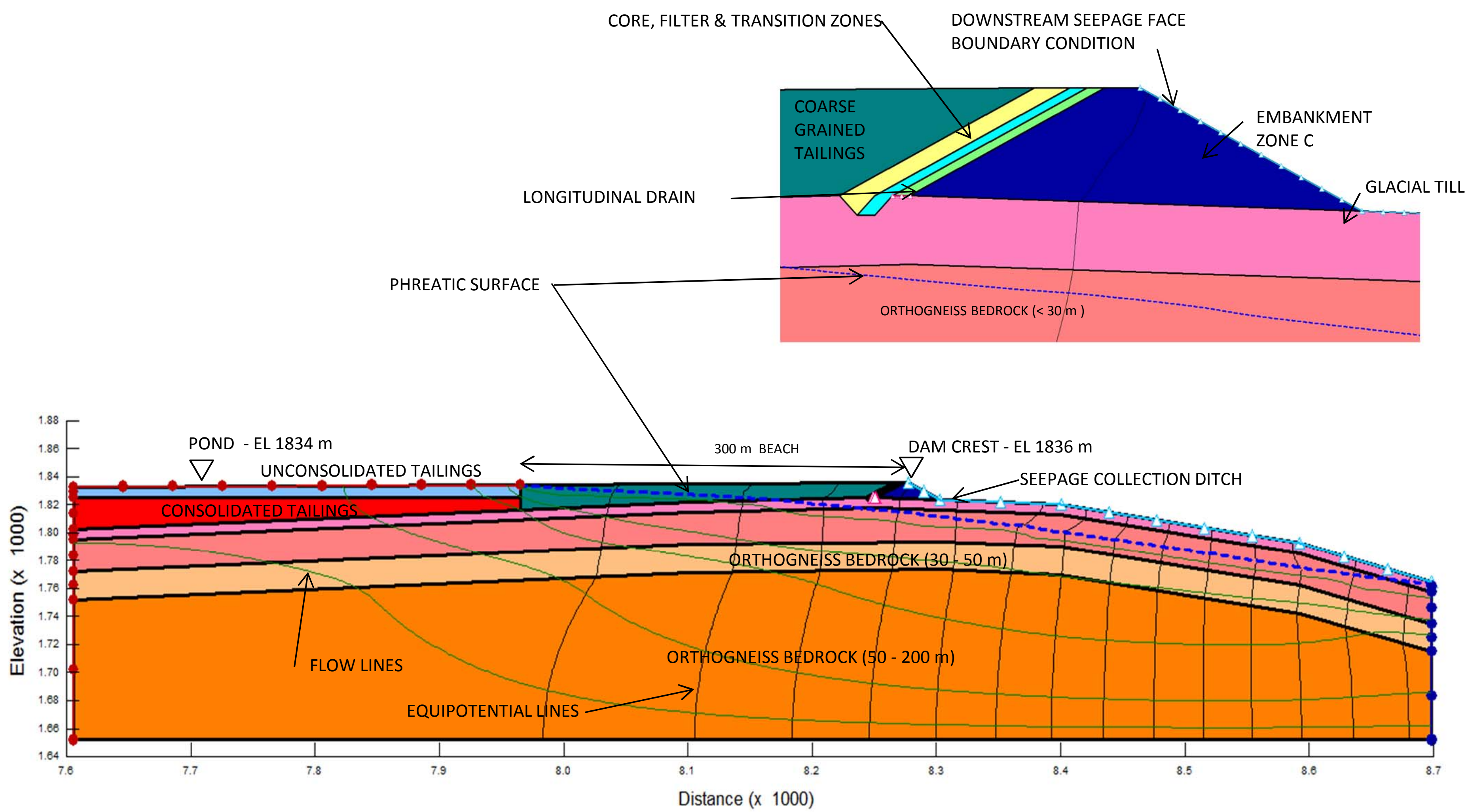
Knight Piésold CONSULTING

P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE A-2

REV 0



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS.

0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
SEEPAGE ANALYSIS
NORTH EMBANKMENT - SECTION 6

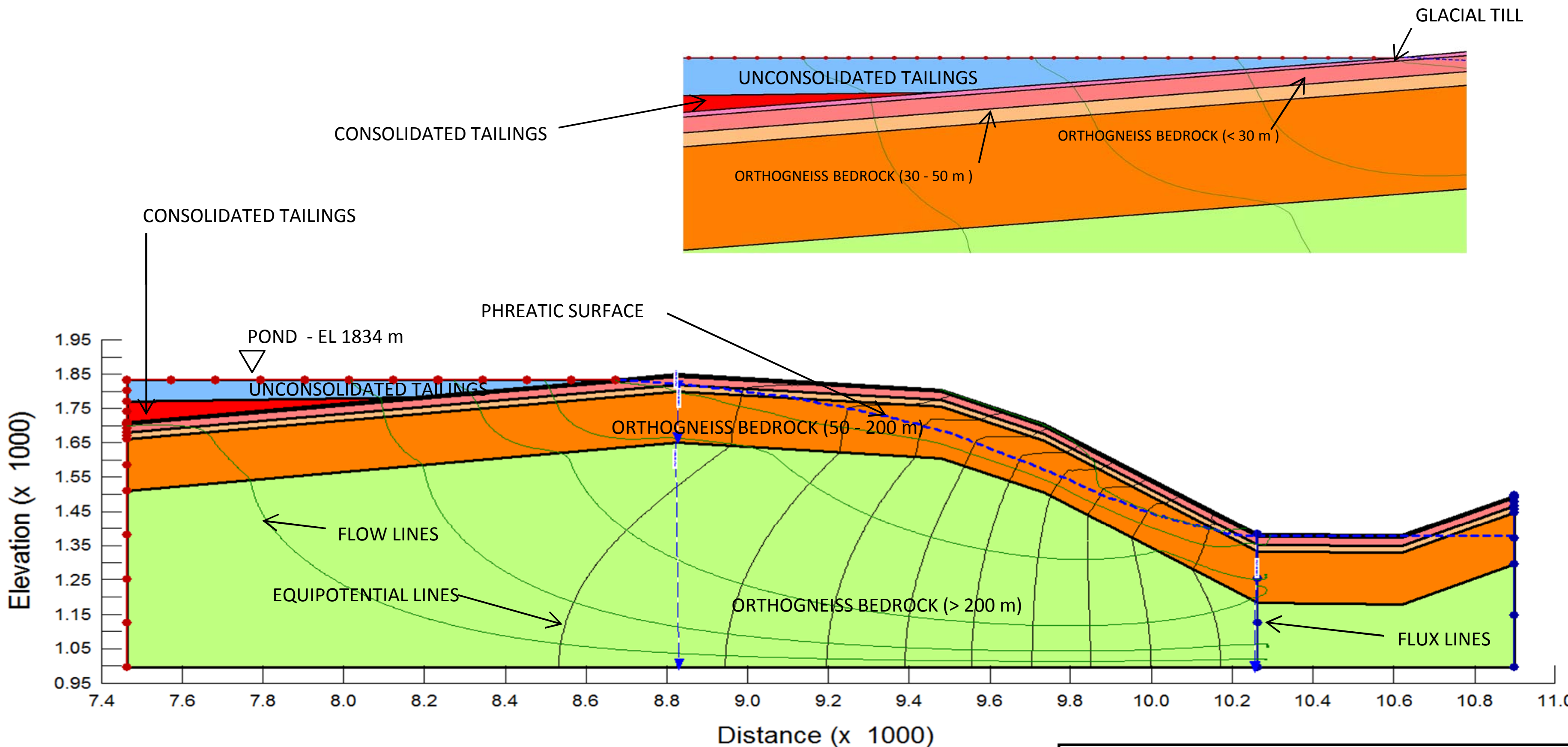
Knight Piésold CONSULTING

P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE A-3

REV 0



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS.

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
SEEPAGE ANALYSIS
EAST SADDLE - SECTION 4

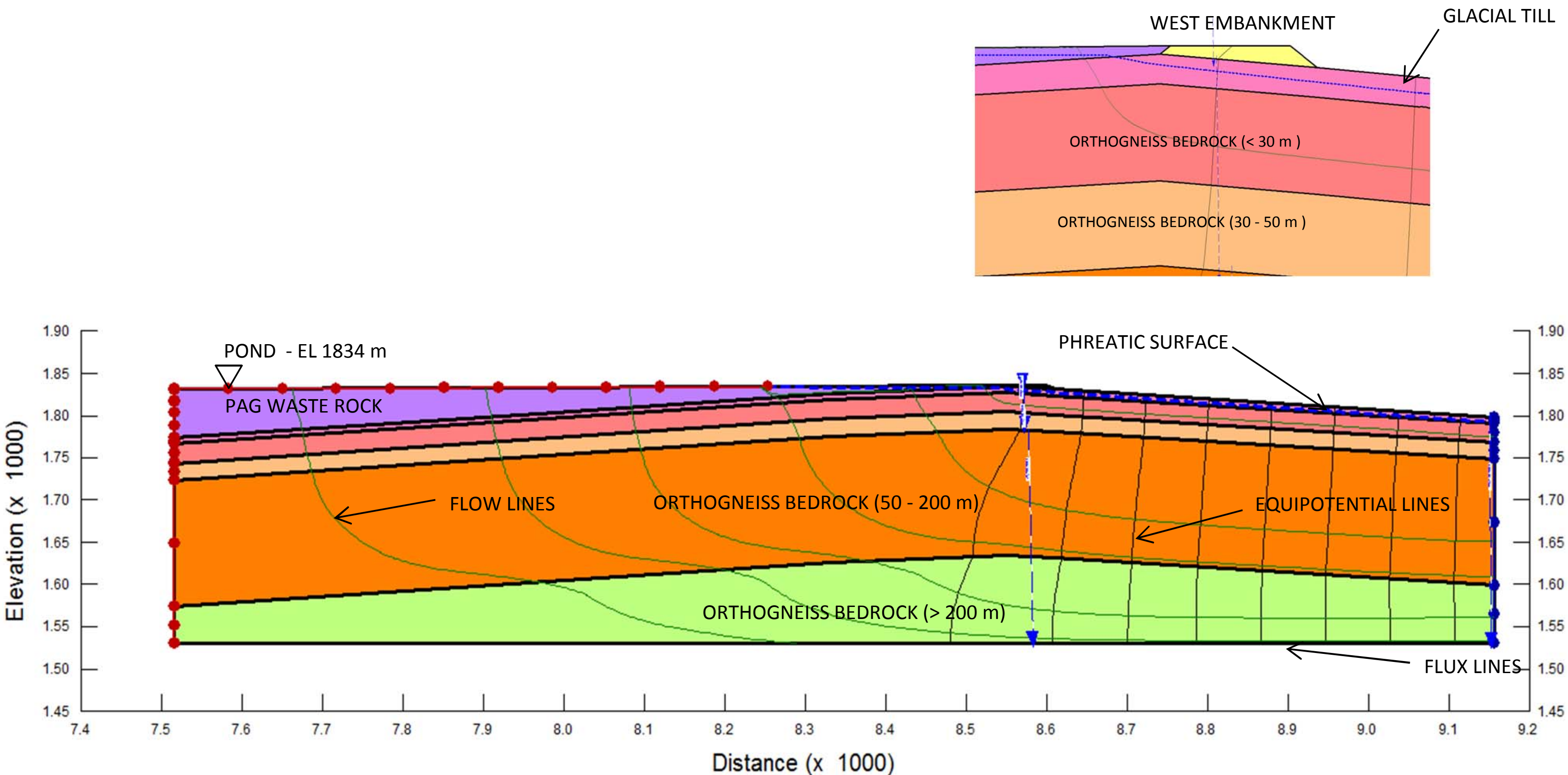
Knight Piésold CONSULTING

P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE A-4

REV 0



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS.

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
SEEPAGE ANALYSIS
WEST SADDLE - SECTION 5

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P/A. NO. VA101-458/14

REF NO. VA14-00865

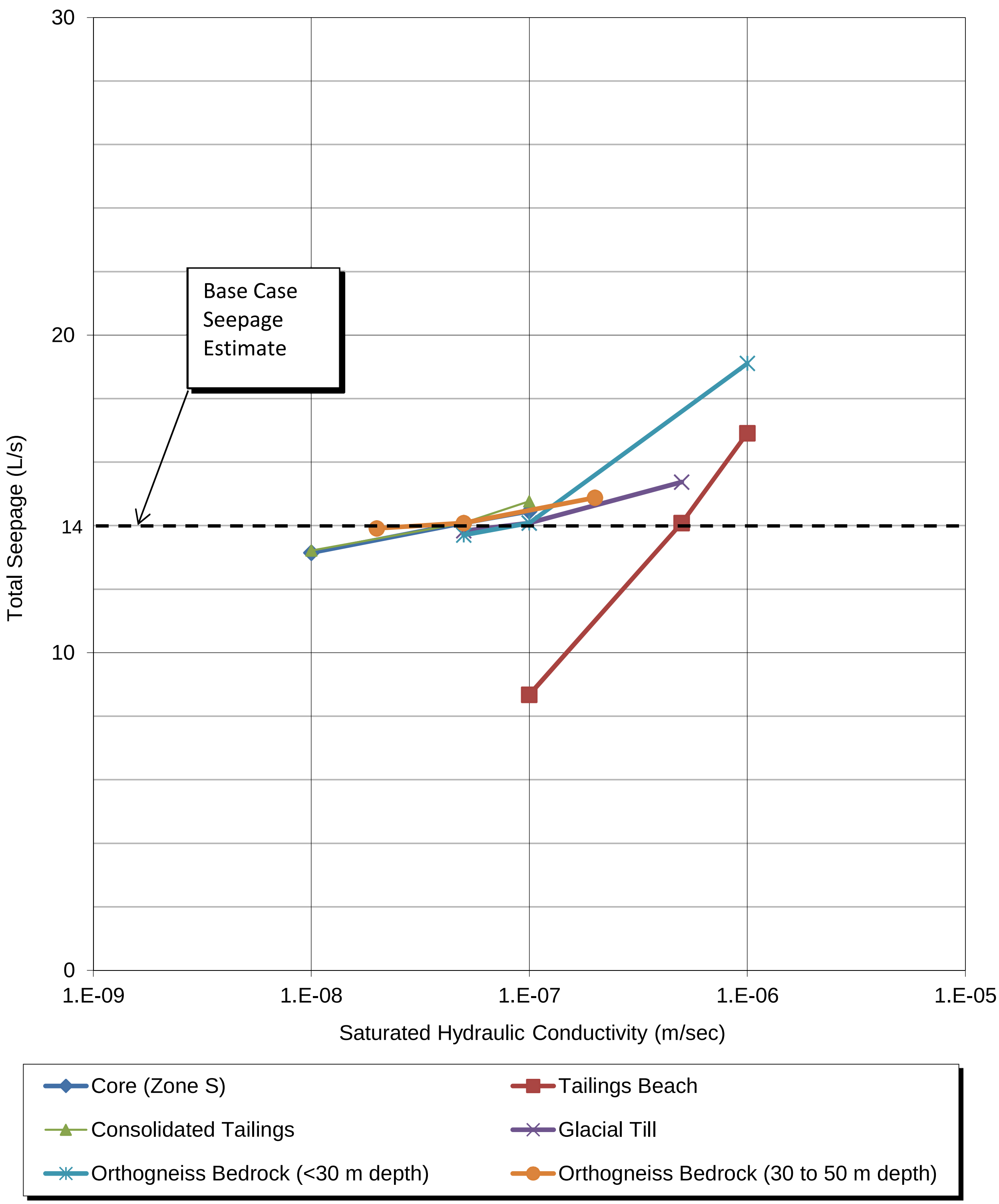
FIGURE A-5

REV 0

APPENDIX B

SEEPAGE SENSITIVITY ANALYSIS PLOTS

(Figures B-1 to B-6)



NOTES:
1. EACH SERIES REPRESENTS THE SENSITIVITY ANALYSIS PERFORMED FOR THE MATERIAL IDENTIFIED IN THE SERIES TITLE, WITH ALL OTHER MATERIAL PARAMETERS AS PER THE BASE CASE PARAMETERS

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

SECTIONS 1,2, & 3 - MAIN EMBANKMENT
TMF SEEPAGE SENSITIVITY ANALYSIS
TOTAL SEEPAGE

Knight Piésold CONSULTING

P/A NO. VA101-458/14

REF. NO. VA14-00865

FIGURE B-1

REV 0

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
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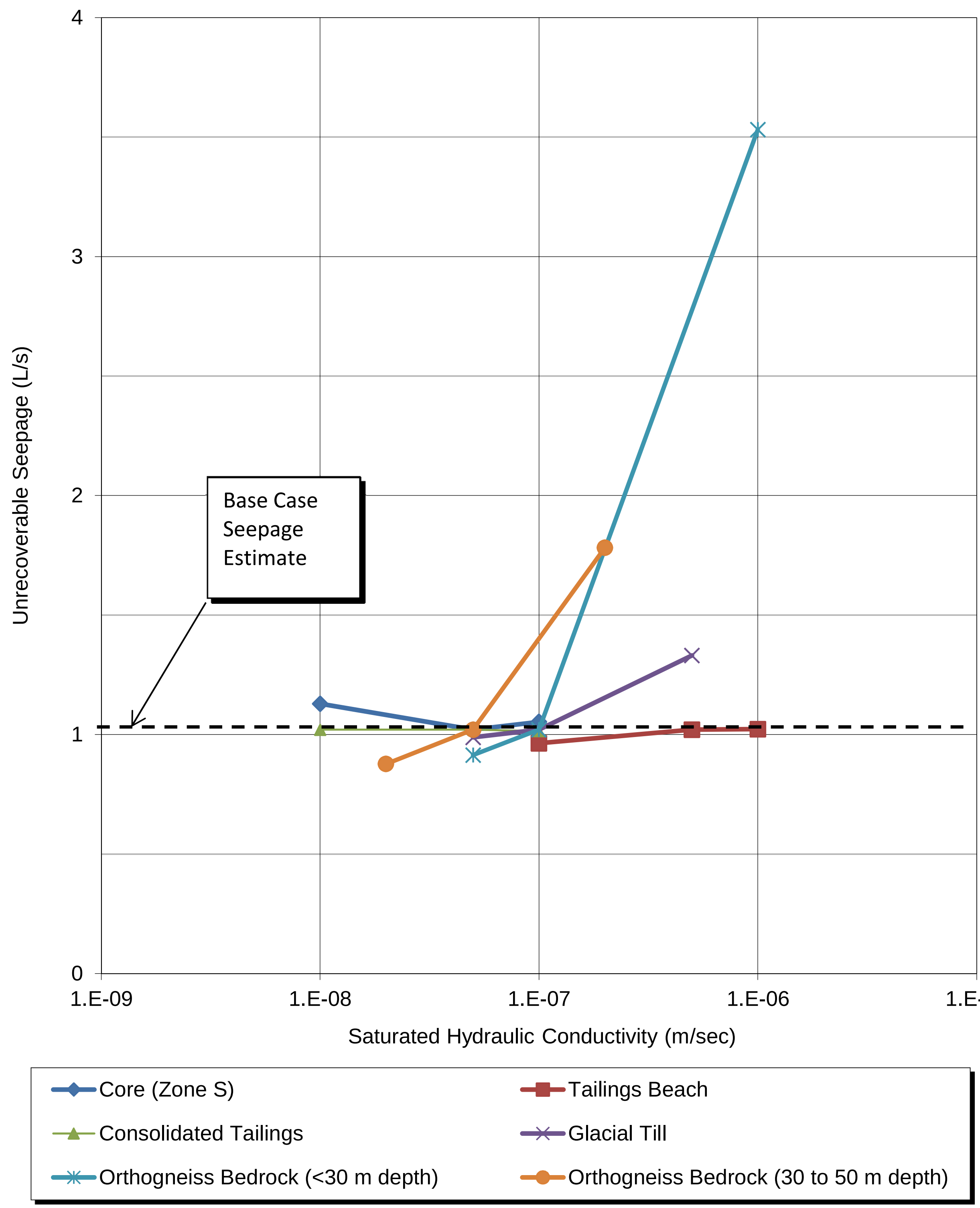

NOTES:
1. EACH SERIES REPRESENTS THE SENSITIVITY ANALYSIS PERFORMED FOR THE MATERIAL IDENTIFIED IN THE SERIES TITLE, WITH ALL OTHER MATERIAL PARAMETERS AS PER THE BASE CASE PARAMETERS

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

SECTIONS 1, 2 & 3 MAIN EMBANKMENT
TMF SEEPAGE SENSITIVITY ANALYSIS
UNRECOVERABLE SEEPAGE

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-458/14	VA14-00865

FIGURE B-2 REV 0

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
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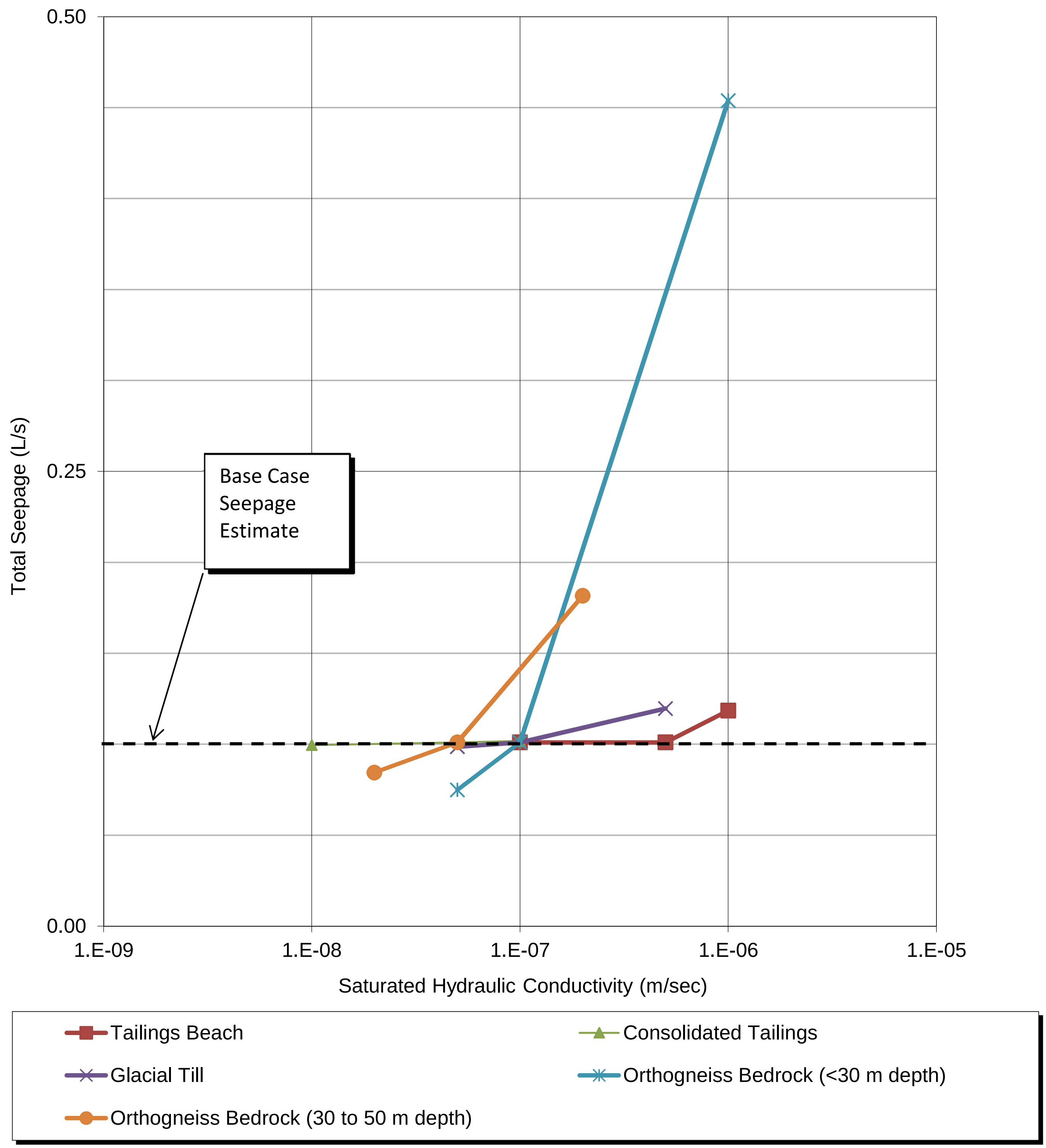

NOTES:
1. EACH SERIES REPRESENTS THE SENSITIVITY ANALYSIS PERFORMED FOR THE MATERIAL IDENTIFIED IN THE SERIES TITLE, WITH ALL OTHER MATERIAL PARAMETERS AS PER THE BASE CASE PARAMETERS
2. THE SENSITIVITY OF THE CORE ZONE MATERIAL WAS NOT INVESTIGATED FOR THE NORTH EMBANKMENT SECTION

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

SECTION 6 - NORTH EMBANKMENT
TMF SEEPAGE SENSITIVITY ANALYSIS
TOTAL SEEPAGE

Knight Piésold CONSULTING

P/A NO. VA101-458/14 | REF. NO. VA14-00865

FIGURE B-3 REV 0

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
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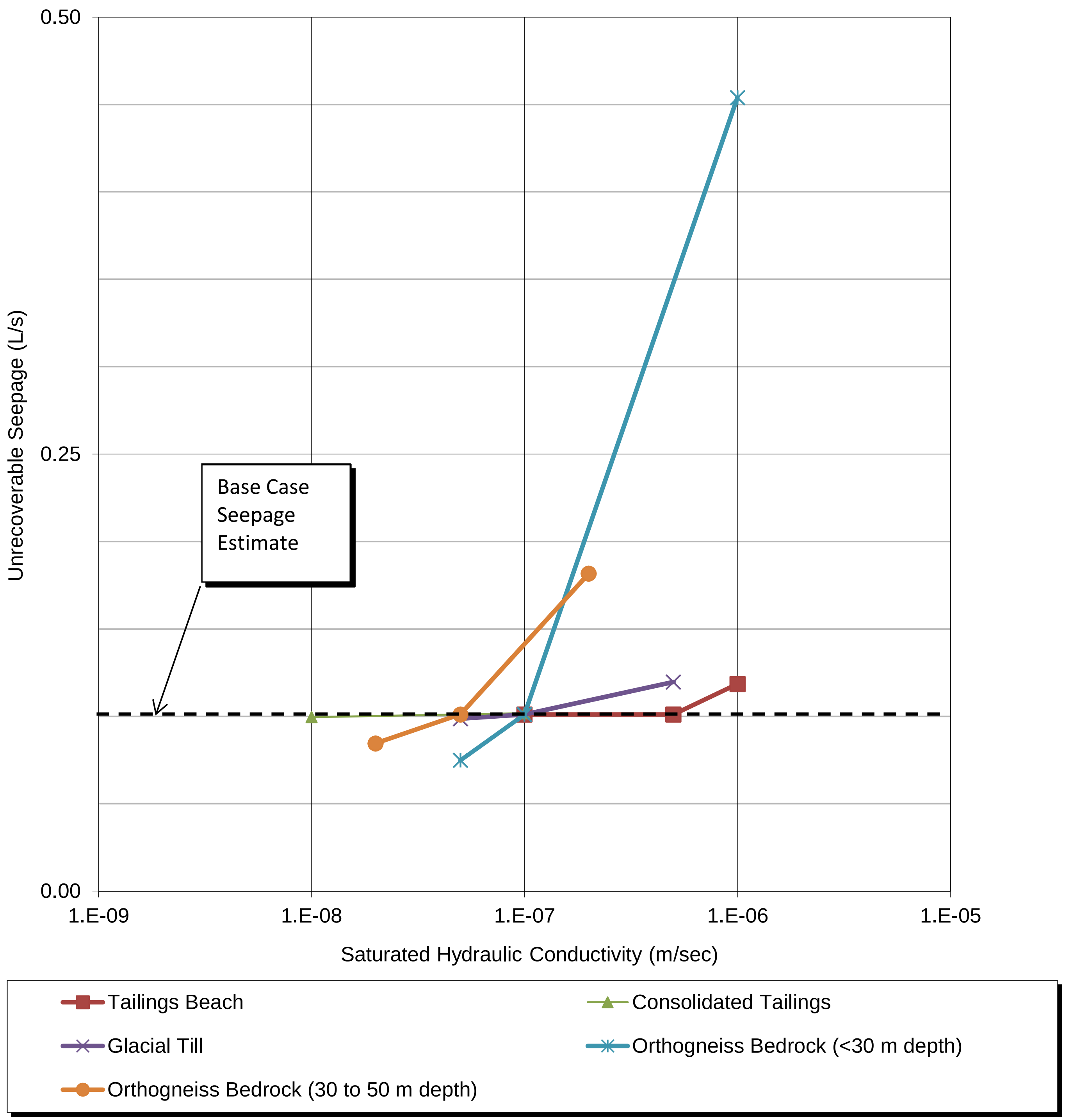

NOTES:

1. EACH SERIES REPRESENTS THE SENSITIVITY ANALYSIS PERFORMED FOR THE MATERIAL IDENTIFIED IN THE SERIES TITLE, WITH ALL OTHER MATERIAL PARAMETERS AS PER THE BASE CASE PARAMETERS
2. THE SENSITIVITY OF THE CORE ZONE MATERIAL WAS NOT INVESTIGATED FOR THE NORTH EMBANKMENT SECTION
3. LOST SEEPAGE ESTIMATES EQUAL THE TOTAL SEEPAGE ESTIMATES AS THE PHREATIC SURFACE IS FOUND TO DROP BELOW THE EMBANKMENT CORE AND NO SEEPAGE IS RECOVERABLE

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

SECTION 6 - NORTH EMBANKMENT
TMF SEEPAGE SENSITIVITY ANALYSIS
UNRECOVERABLE SEEPAGE

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA101-458/14	VA14-00865

FIGURE B-4 REV 0

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
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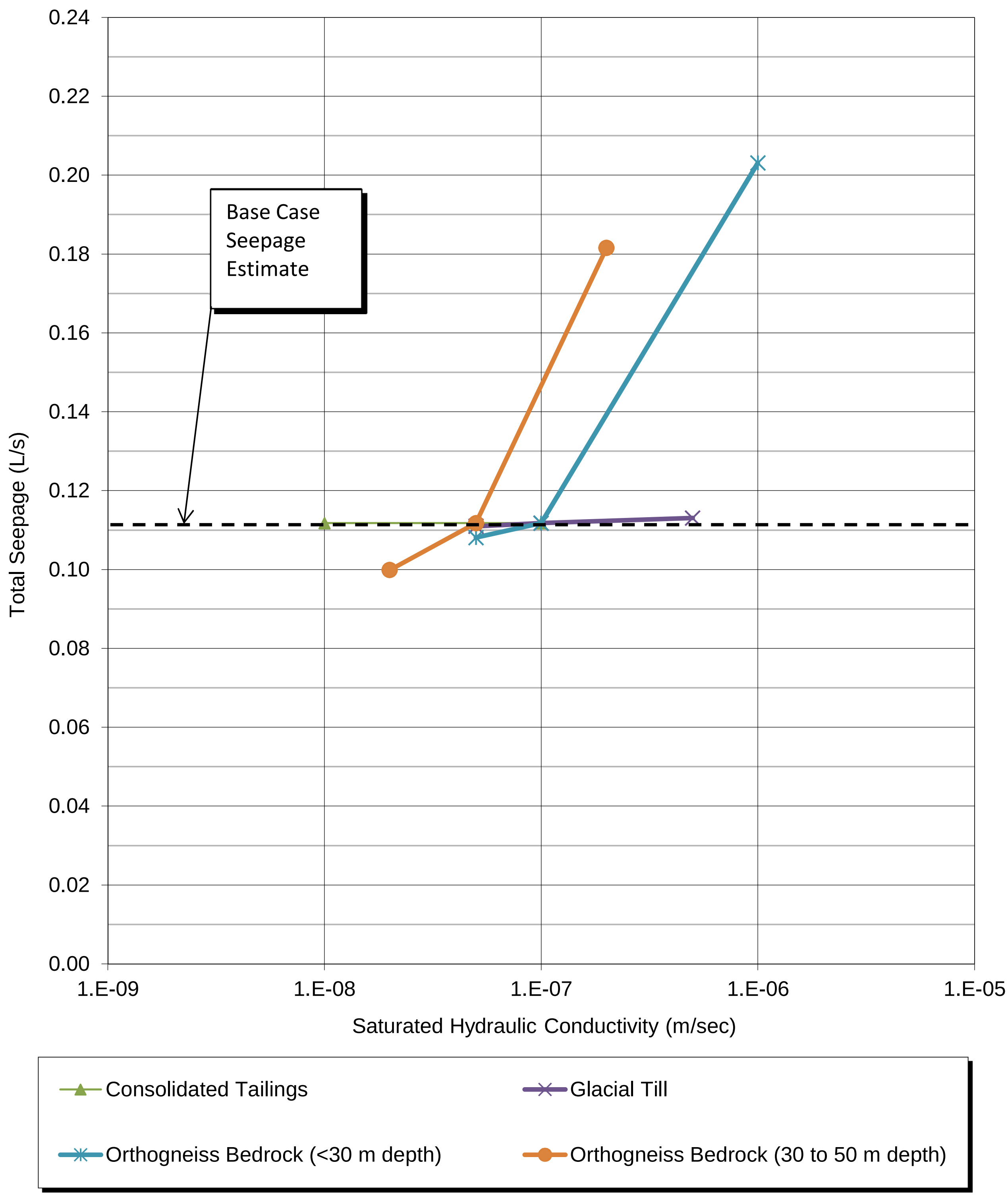

NOTES:
1. EACH SERIES REPRESENTS THE SENSITIVITY ANALYSIS PERFORMED FOR THE MATERIAL IDENTIFIED IN THE SERIES TITLE, WITH ALL OTHER MATERIAL PARAMETERS AS PER THE BASE CASE PARAMETERS
2. ALL SEEPAGE IS ASSUMED TO BE UNRECOVERABLE

HARPER CREEK MINING CORP.

HARPER CREEK PROJECT

SECTION 4 - EAST SADDLE
TMF SEEPAGE SENSITIVITY ANALYSIS
TOTAL SEEPAGE

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P/A NO. VA101-458/14

REF. NO. VA14-00865

FIGURE B-5

REV 0

0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB
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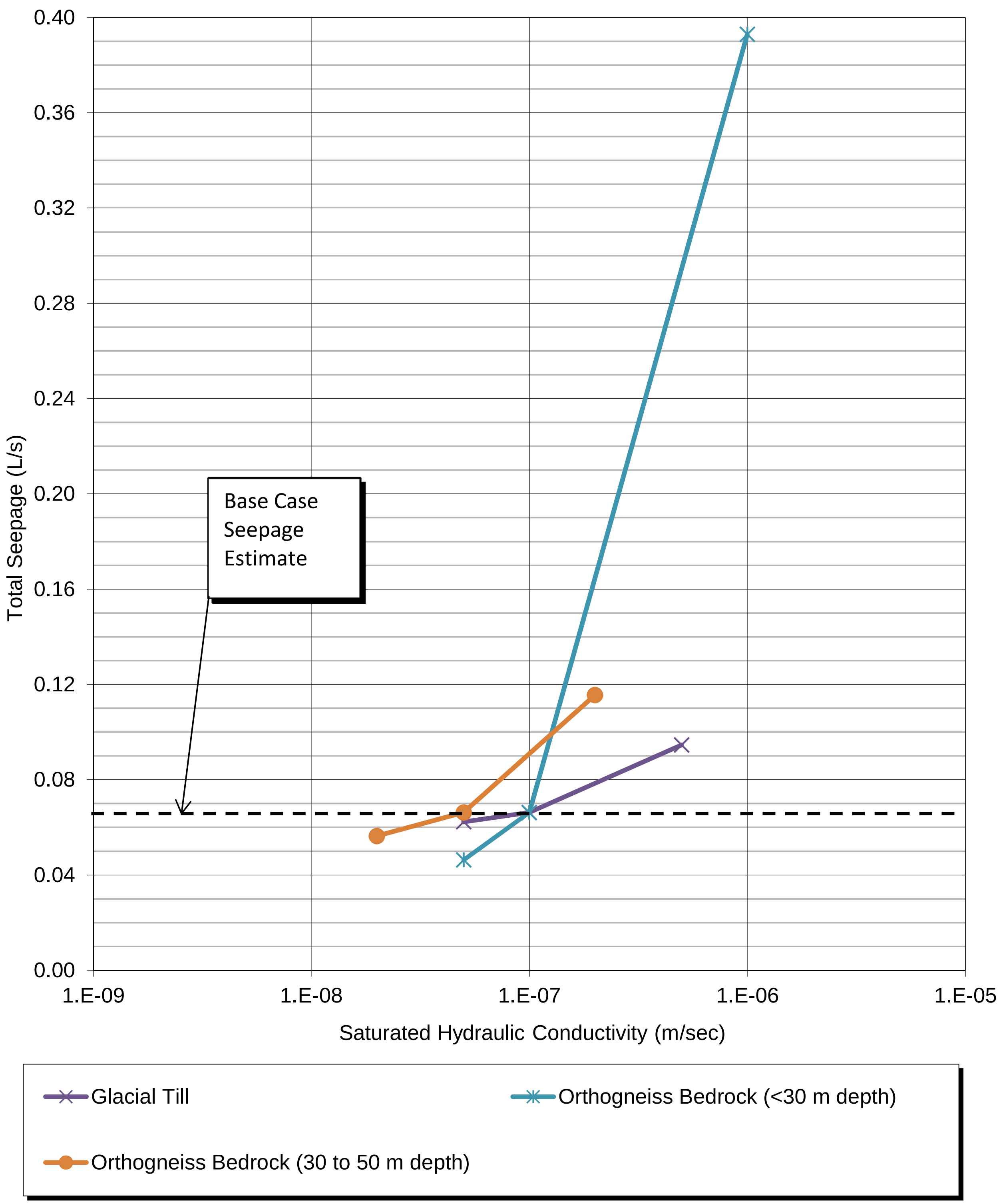

NOTES:
1. EACH SERIES REPRESENTS THE SENSITIVITY ANALYSIS PERFORMED FOR THE MATERIAL IDENTIFIED IN THE SERIES TITLE, WITH ALL OTHER MATERIAL PARAMETERS AS PER THE BASE CASE PARAMETERS
2. ALL SEEPAGE IS ASSUMED TO BE UNRECOVERABLE

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HARPER CREEK PROJECT

SECTION 5 - WEST SADDLE
TMF SEEPAGE SENSITIVITY ANALYSIS
TOTAL SEEPAGE

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VA101-458/14	VA14-00865

FIGURE B-6 REV 0

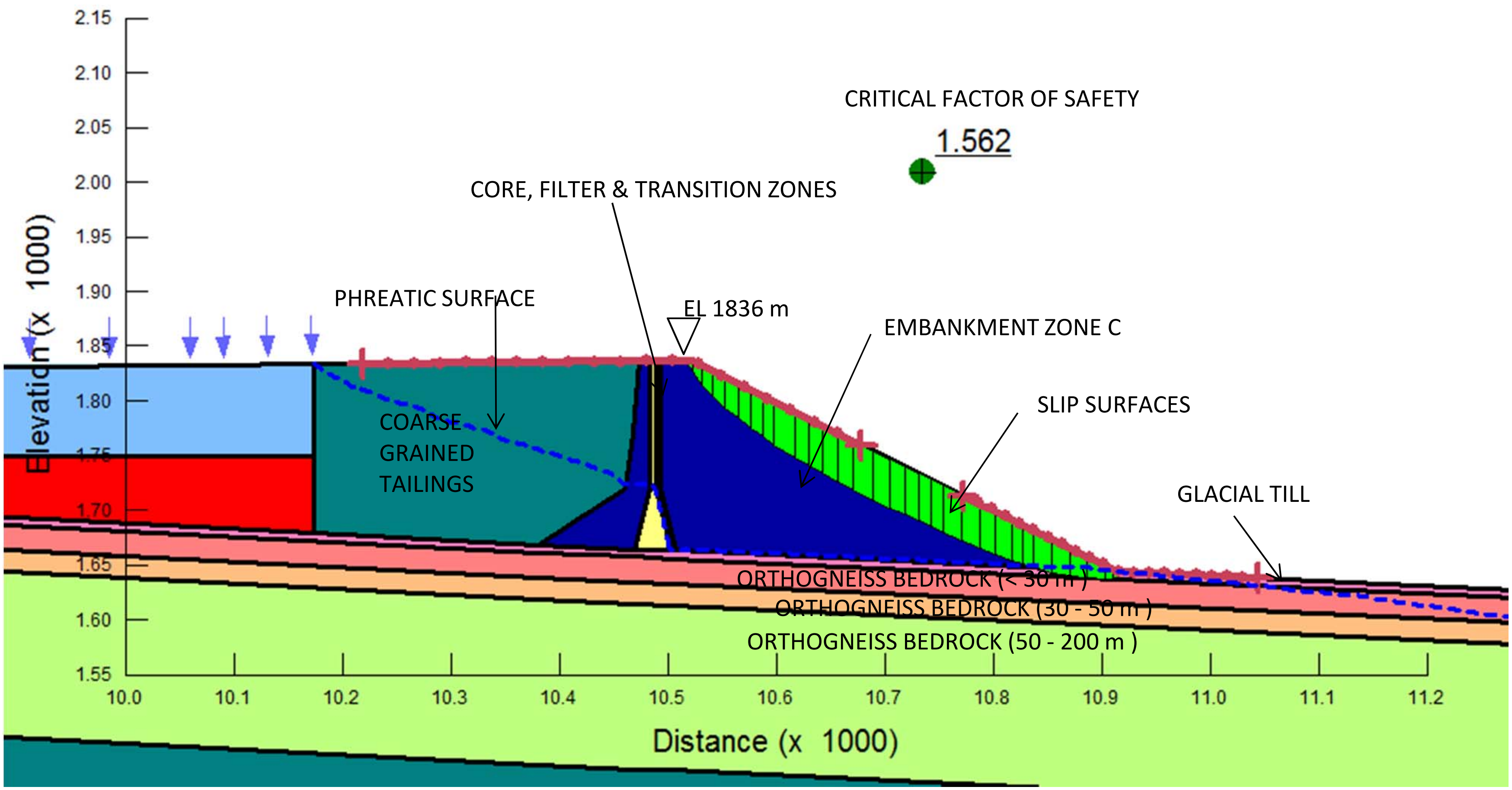
0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D

APPENDIX C

STABILITY ANALYSIS FIGURES

(Figures C-1 to C-4)

VA14-00865
August 19, 2014



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS.

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB

HARPER CREEK MINING CORP

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
MAIN EMBANKMENT STABILITY ANALYSIS
NORMAL OPERATING CONDITIONS AT

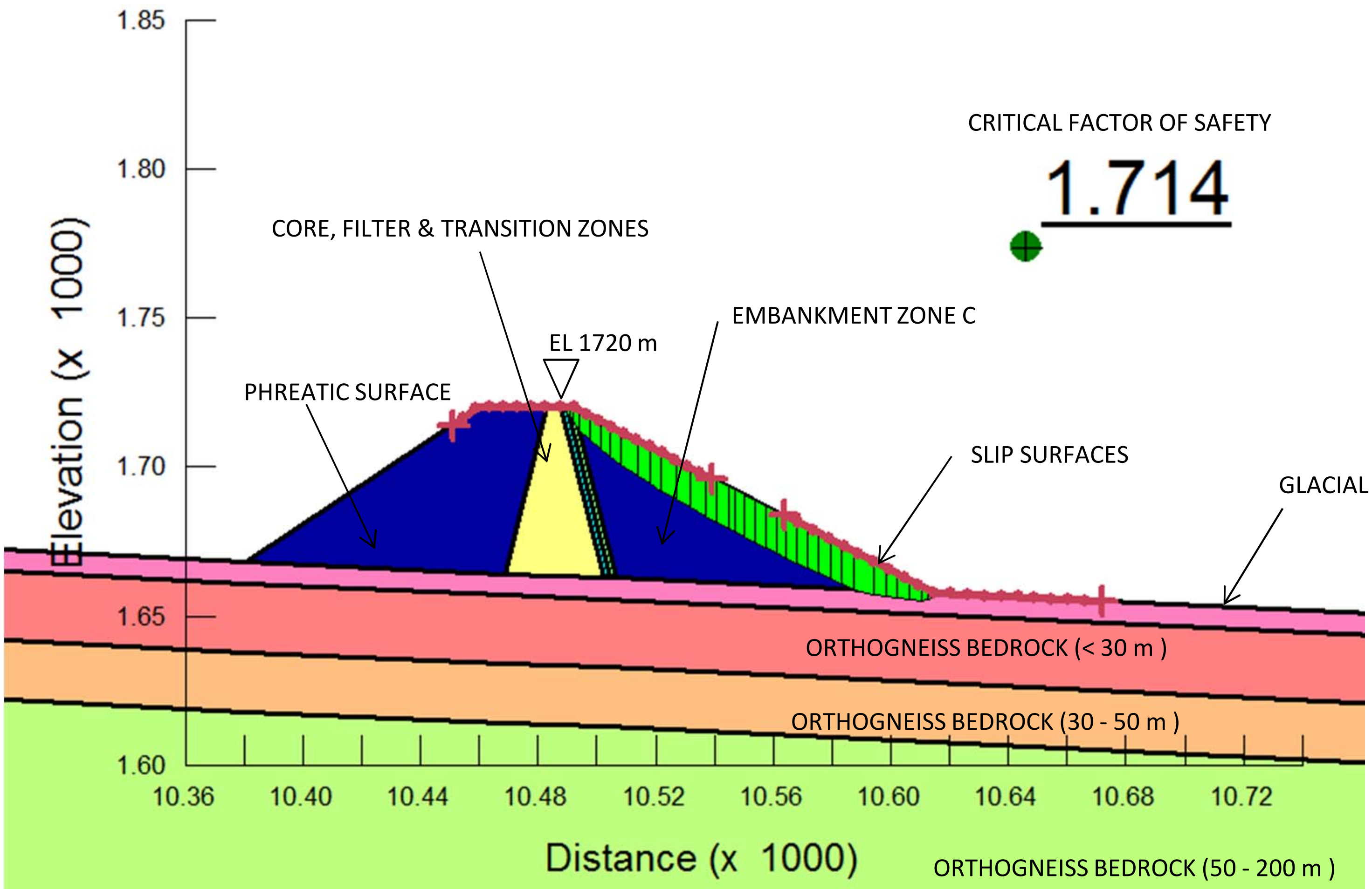
Knight Piésold CONSULTING

P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE C-1

REV 0



NOTES:

1. LOCATION OF PHREATIC SURFACE ASSUMED TO BE AT NATURAL GROUND LEVEL

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	19AUG'14	ISSUED WITH LETTER VA14-00865	ACR	DDF	KJB

HARPER CREEK MINING CORP

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
MAIN EMBANKMENT STABILITY ANALYSIS
NORMAL OPERATING CONDITIONS AT STAGE

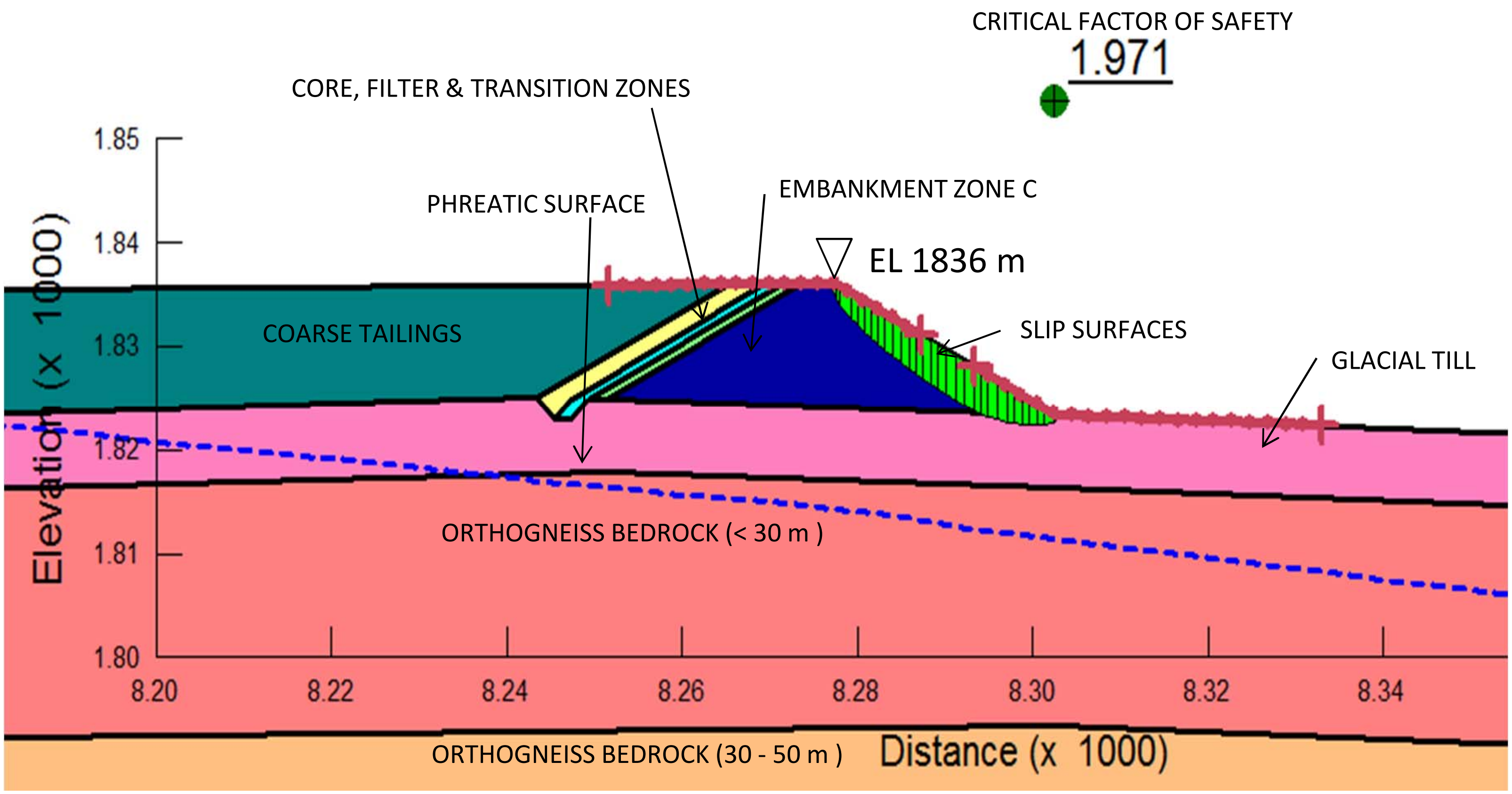
Knight Piésold CONSULTING

P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE C-2

REV 0



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS

REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D
0	19AUG'14	ISSUED WITH VA14-00865	ACR	DDF	KJB

HARPER CREEK MINING CORP

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
NORTH EMBANKMENTSTABILITY ANALYSIS
NORMAL OPERATING CONDITIONS AT

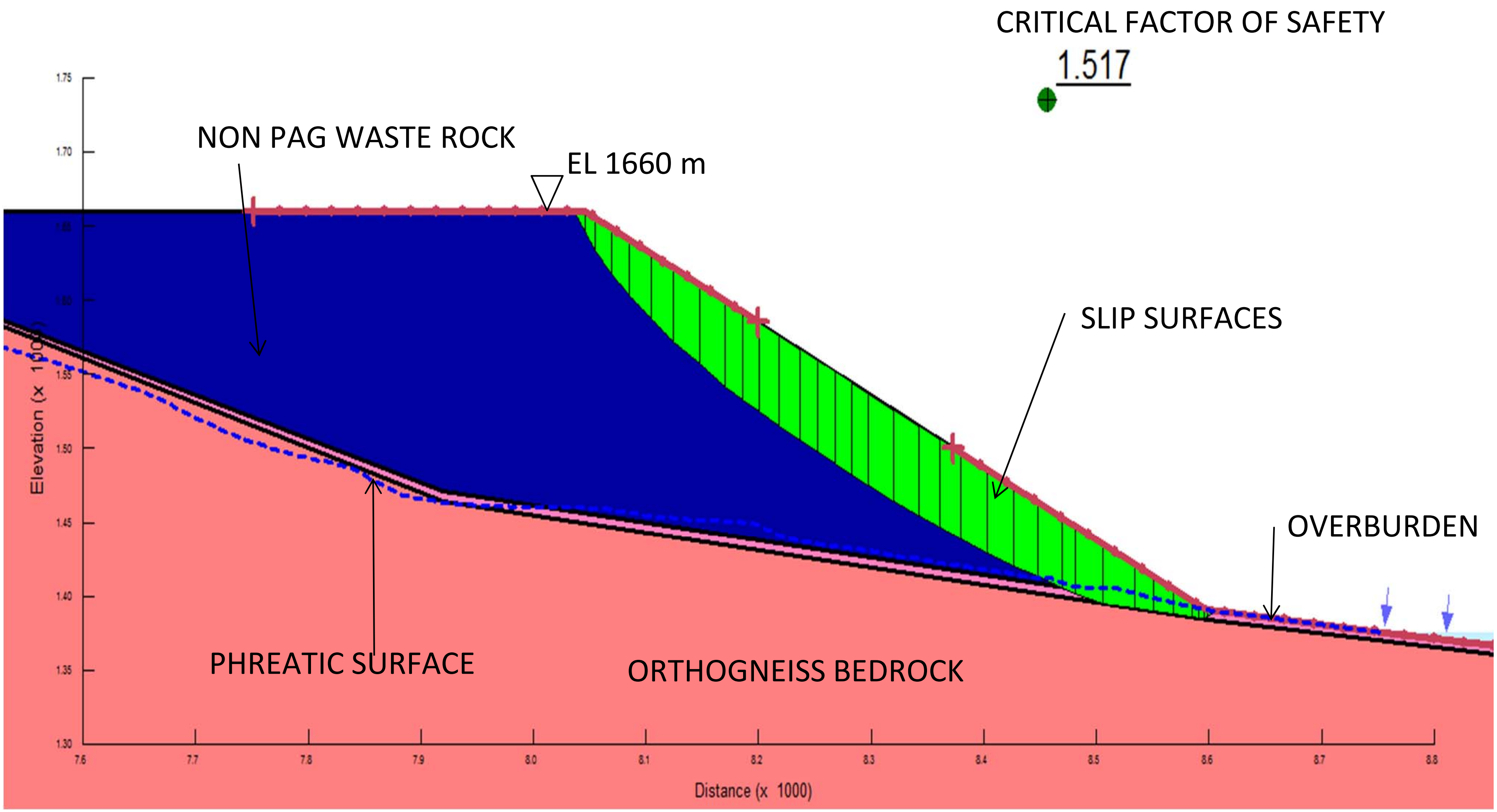
Knight Piésold CONSULTING

P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE C-3

REV 0



NOTES:

1. LOCATION OF PHREATIC SURFACE PREDICTED FROM STEADY STATE SEEPAGE ANALYSIS

0	19AUG'14	ISSUED WITH VA14-00865	ACR	DDF	KJB
REV	DATE	DESCRIPTION	PREP'D	CHK'D	APP'D

HARPER CREEK MINING CORP

HARPER CREEK PROJECT

TAILINGS MANAGEMENT FACILITY
NON PAG WASTE STOCKPILE STABILITY
OPERATING CONDITIONS AT CLOSURE

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P/A. NO. VA101-458/14

REF NO. VA14-00865

FIGURE C-4

REV 0

APPENDIX D

SELECTED TABLES FROM THE INVESTIGATION AND DESIGN MANUAL INTERIM GUIDELINES (BC MWRPRC, 1991)

(Pages D-1 to D-4)

TABLE 5.1

DUMP STABILITY RATING SCHEME

KEY FACTORS AFFECTING STABILITY		RANGE OF CONDITIONS OR DESCRIPTION	POINT RATING
DUMP CONFIGURATION		< 50m	0
		50m - 100m	50
DUMP HEIGHT		100m - 200m	100
		> 200m	200
DUMP VOLUME	Small	< 1 million BCM's	0
	Medium	1 - 50 million BCM's	50
	Large	> 50 million BCM's	100
DUMP SLOPE	Flat	< 26°	0
	Moderate	26° - 35°	50
	Steep	> 35°	100
FOUNDATION SLOPE	Flat	< 10°	0
	Moderate	10° - 25°	50
	Steep	25° - 32°	100
	Extreme	> 32°	200
DEGREE OF CONFINEMENT	Confined	-Concave slope in plan or section -Valley or Cross-Valley fill, toe butressed against opposite valley wall -Incised gullies which can be used to limit foundation slope during development	0
	Moderately Confined	-Natural benches or terraces on slope -Even slopes, limited natural topographic diversity -Heaped, Sidehill or broad Valley or Cross-Valley fills	50
	Unconfined	-Convex slope in plan or section -Sidehill or Ridge Crest fill with no toe confinement -No gullies or benches to assist development	100
FOUNDATION TYPE	Competent	-Foundation materials as strong or stronger than dump materials -Not subject to adverse pore pressures -No adverse geologic structure	0
	Intermediate	-Intermediate between competent and weak -Soils gain strength with consolidation -Adverse pore pressures dissipate if loading rate controlled	100
	Weak	-Limited bearing capacity, soft soils -Subject to adverse pore pressure generation upon loading -Adverse groundwater conditions, springs or seeps -Strength sensitive to shear strain, potentially liquefiable	200
DUMP MATERIAL QUALITY	High	-Strong, durable -Less than about 10% fines	0
	Moderate	-Moderately strong, variable durability -10 to 25% fines	100
	Poor	-Predominantly weak rocks of low durability -Greater than about 25% fines, overburden	200

Continued..

TABLE 5.1 (Continued)

DUMP STABILITY RATING SCHEME

KEY FACTORS AFFECTING STABILITY	RANGE OF CONDITIONS OR DESCRIPTION		POINT RATING
METHOD OF CONSTRUCTION	Favourable	-Thin lifts (<25m thick), wide platforms -Dumping along contours -Ascending construction -Wrap-arounds or terraces	0
	Mixed	-Moderately thick lifts (25m - 50m) -Mixed construction methods	100
	Unfavourable	-Thick lifts (> 50m), narrow platform (sliver fill) -Dumping down the fall line of the slope -Descending construction	200
PIEZOMETRIC AND CLIMATIC CONDITIONS	Favourable	-Low piezometric pressures, no seepage in foundation -Development of phreatic surface within dump unlikely -Limited precipitation -Minimal infiltration into dump -No snow or ice layers in dump or foundation	0
	Intermediate	-Moderate piezometric pressures, some seeps in foundation -Limited development of phreatic surface in dump possible -Moderate precipitation -High infiltration into dump -Discontinuous snow or ice lenses or layers in dump	100
	Unfavourable	-High piezometric pressures, springs in foundation -High precipitation -Significant potential for development of phreatic surface or perched water tables in dump -Continuous layers or lenses of snow or ice in dump or foundation	200
DUMPING RATE	Slow	-< 25 BCM's per lineal metre of crest per day -Crest advancement rate < 0.1m per day	0
	Moderate	-25 - 200 BCM's per lineal metre of crest per day -Crest advancement rate 0.1m - 1.0m per day	100
	High	-> 200 BCM's per lineal metre of crest per day -Crest advancement > 1.0m per day	200
SEISMICITY	Low	Seismic Risk Zones 0 and 1	0
	Moderate	Seismic Risk Zones 2 and 3	50
	High	Seismic Risk Zones 4 or higher	100

MAXIMUM POSSIBLE DUMP STABILITY RATING: 1800

TABLE 5.2

DUMP STABILITY CLASSES AND RECOMMENDED LEVEL OF EFFORT

DUMP STABILITY CLASS	FAILURE HAZARD	RECOMMENDED LEVEL OF EFFORT FOR INVESTIGATION, DESIGN AND CONSTRUCTION	RANGE OF DUMP RATING (DSR)
I	Negligible	-Basic site reconnaissance, baseline documentation -Minimal lab testing -Routine check of stability, possibly using charts -Minimal restrictions on construction -Visual monitoring only	< 300
II	Low	-Thorough site investigation -Test pits, sampling may be required -Limited lab index testing -Stability may or may not influence design -Basic stability analysis required -Limited restrictions on construction -Routine visual and instrument monitoring	300-600
III	Moderate	-Detailed, phased site investigation -Test pits required, drilling or other subsurface investigations may be required -Undisturbed samples may be required -Detailed lab testing, including index properties, shear strength and durability likely required -Stability influences and may control design -Detailed stability analysis, possibly including parametric studies, required -Stage II detailed design report may be required for approval/permitting -Moderate restrictions on construction (eg. limiting loading rate, lift thickness, material quality, etc.) -Detailed instrument monitoring to confirm design, document behaviour and establish loading limits	600-1200
IV	High	-Detailed, phased site investigation -Test pits, and possibly trenches, required -Drilling, and possible other subsurface investigations probably required -Undisturbed sampling probably required -Detailed lab testing, including index properties, shear strength and durability testing probably required -Stability considerations paramount. -Detailed stability analyses, probably including parametric studies and full evaluation of alternatives probably required -Stage II detailed design report probably required for approval/permitting -Severe restrictions on construction (eg. limiting loading rates, lift thickness, material quality, etc.) -Detailed instrument monitoring to confirm design, document behaviour and establish loading limits	> 1200

TABLE 6.4

INTERIM GUIDELINES FOR MINIMUM DESIGN FACTOR OF SAFETY [1]

STABILITY CONDITION	SUGGESTED MINIMUM DESIGN VALUES FOR FACTOR OF SAFETY	
	CASE A	CASE B
STABILITY OF DUMP SURFACE		
–Short Term (during construction)	1.0	1.0
–Long Term (reclamation – abandonment)	1.2	1.1
OVERALL STABILITY (DEEP SEATED STABILITY)		
–Short Term (static)	1.3 – 1.5	1.1 – 1.3
–Long Term (static)	1.5	1.3
–Pseudo–Static (earthquake) [2]	1.1 – 1.3	1.0

CASE A:
- –Low level of confidence in critical analysis parameters
- –Possibly unconservative interpretation of conditions, assumptions
- –Severe consequences of failure
- –Simplified stability analysis method (charts, simplified method of slices)
- –Stability analysis method poorly simulates physical conditions
- –Poor understanding of potential failure mechanism(s)

CASE B:
- –High level of confidence in critical analysis parameters
- –Conservative interpretation of conditions, assumptions
- –Minimal consequences of failure
- –Rigorous stability analysis method
- –Stability analysis method simulates physical conditions well
- –High level of confidence in critical failure mechanism(s)

NOTES:
1. A range of suggested minimum design values are given to reflect different levels of confidence in understanding site conditions, material parameters, consequences of instability, and other factors.
2. Where pseudo–static analyses, based on peak ground accelerations which have a 10% probability of exceedance in 50 years, yield F.O.S. < 1.0, dynamic analysis of stress–strain response, and comparison of results with stress–strain characteristics of dump materials is recommended.