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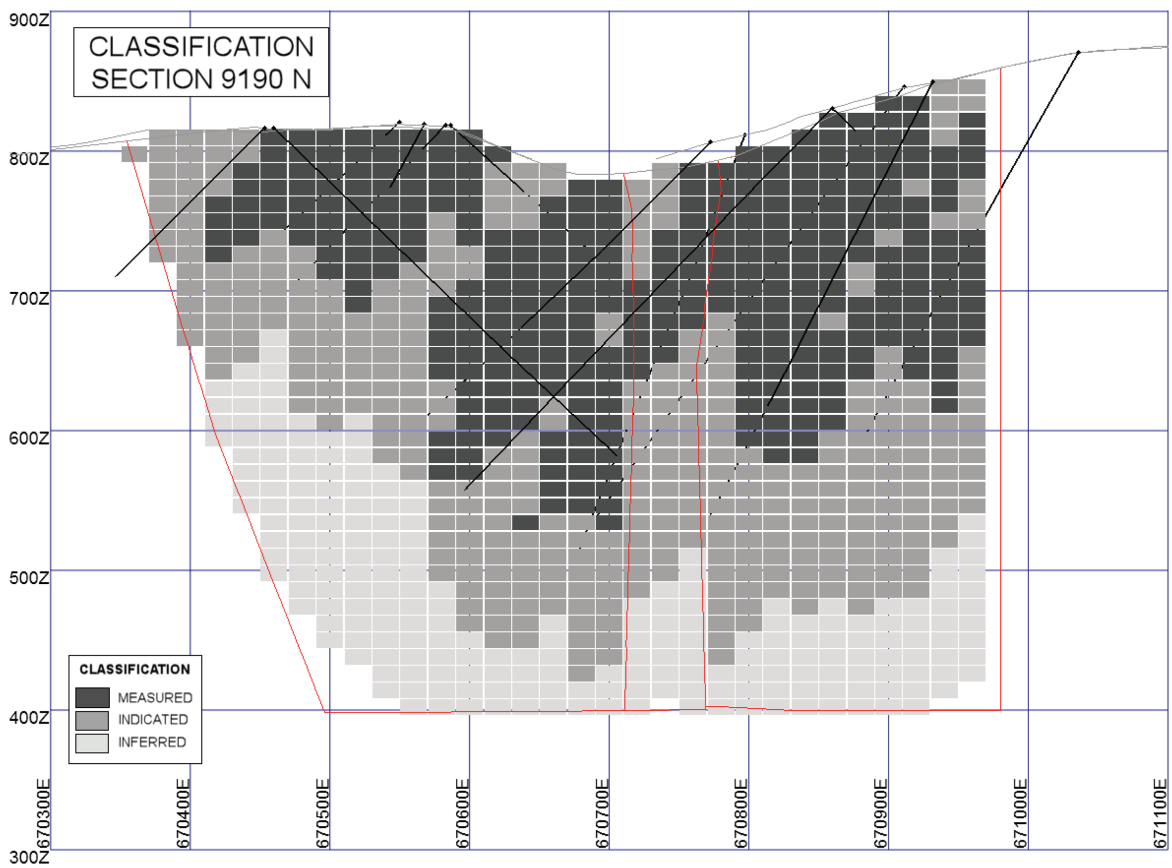


FIGURE 4.11-17

Morrison Copper/Gold Project
Block Classification – Section 9190 N

The global block grads mean at zero cut-off compare very well with the global means of the capped composites and raw assay data (Table 4.11-8).

Table 4.11-8
Global Mean Grade Comparison

Item	Kriged Mean	Mean Grades (g/t Au)	
		Comps	Raw Data
% Cu	0.33	0.36	0.36
g/t Au (Capped)	0.16	0.18	0.18
g/t Au (Uncapped)		0.18	0.19
% Mo	0.004	0.005	0.005

Swath plots were generated to assess the model for global bias by comparing kriged values with nearest neighbour estimates on 40 m vertical panels throughout the deposit. Results show a good comparison between the three methods, particularly in the main portions of the deposit indicated by the charts as illustrated in Figures 4.11-18 and 4.11-19.

4.11.12 Mineral Resource Summary

The Morrison mineral resource presented in Tables 4.11-9 through 4.11-11 reported at equivalent copper cutoff grades ranging from 0.1 to 0.5% with the base case of 0.3% Cu equivalent shown in bold. The copper equivalent value was calculated using relative recovery and metal prices of $2.45/lb copper, $570/oz gold, and $28/lb molybdenum. For blocks containing molybdenum values greater than or equal to 0.005% Mo, the following equation was used:

$$\mathbf{Cu\ EQ = Cu+Au*0.23+Mo*7.794}$$

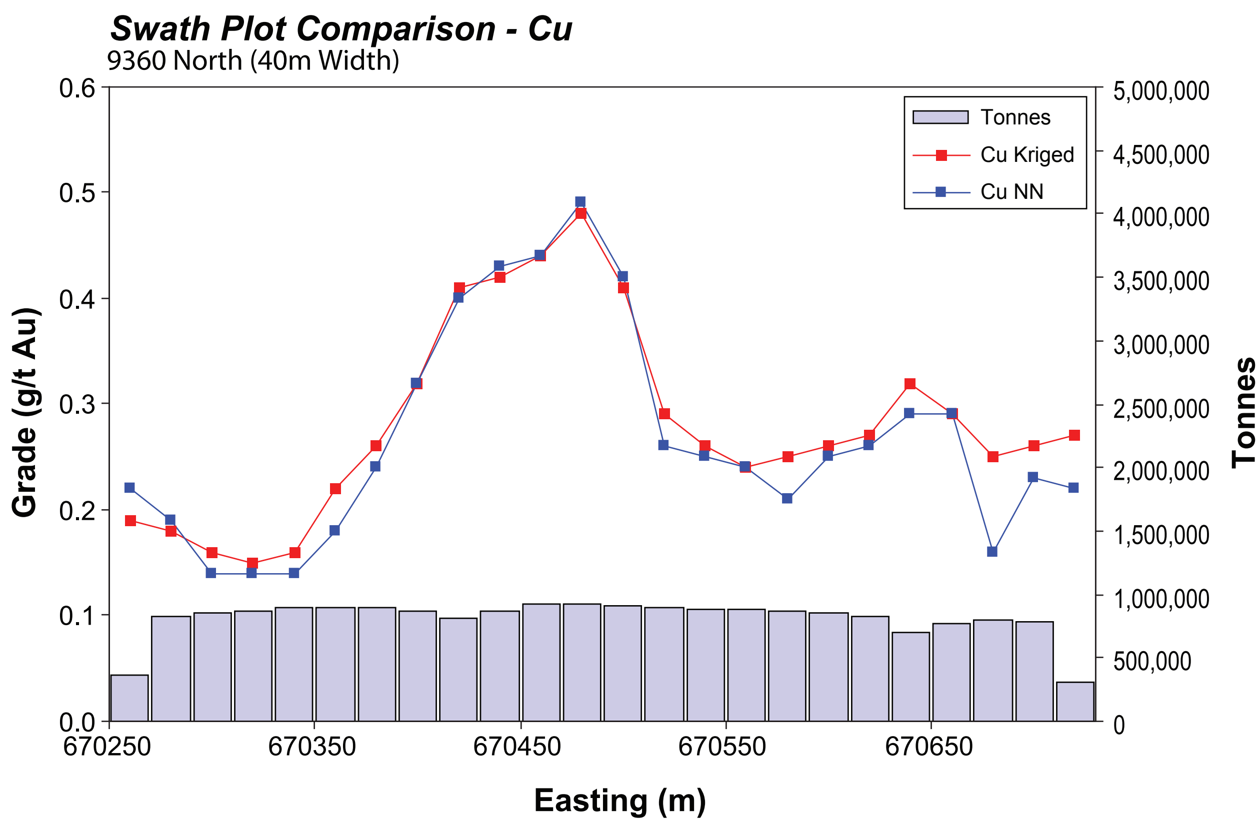
For blocks with <0.005% Mo, the molybdenum was considered unrecoverable and eliminated from the calculation.

Using a 0.3% equivalent copper cutoff grade, the Morrison deposit is estimated to contain a measured and indicated resource of 206.9 million tonnes averaging 0.39% Cu, 0.2 g/t Au, and 0.005% Mo. The deposit remains open at depth and a mineralized intercept by hole MO-01-24 on the hanging wall side of the west fault indicates a potential extension of the central zone to the northwest below the 700 m level.

4.12 Geotechnical Site Investigations and Laboratory Programs

4.12.1 Previous Studies

A geotechnical site investigation program was completed by Knight Piesold between November 2005 and April 2006 (Knight Piésold 2006). The area of investigation includes the TSF, an earlier proposed plant site near the open pit, and groundwater quality monitoring installations for the open pit. The investigation consisted of 17 geotechnical drill holes, 17 groundwater monitoring wells, and 35 test pits.



Morrison Copper/Gold Project
Swath Plot for Cu – Section 9360 North

FIGURE 4.11-18

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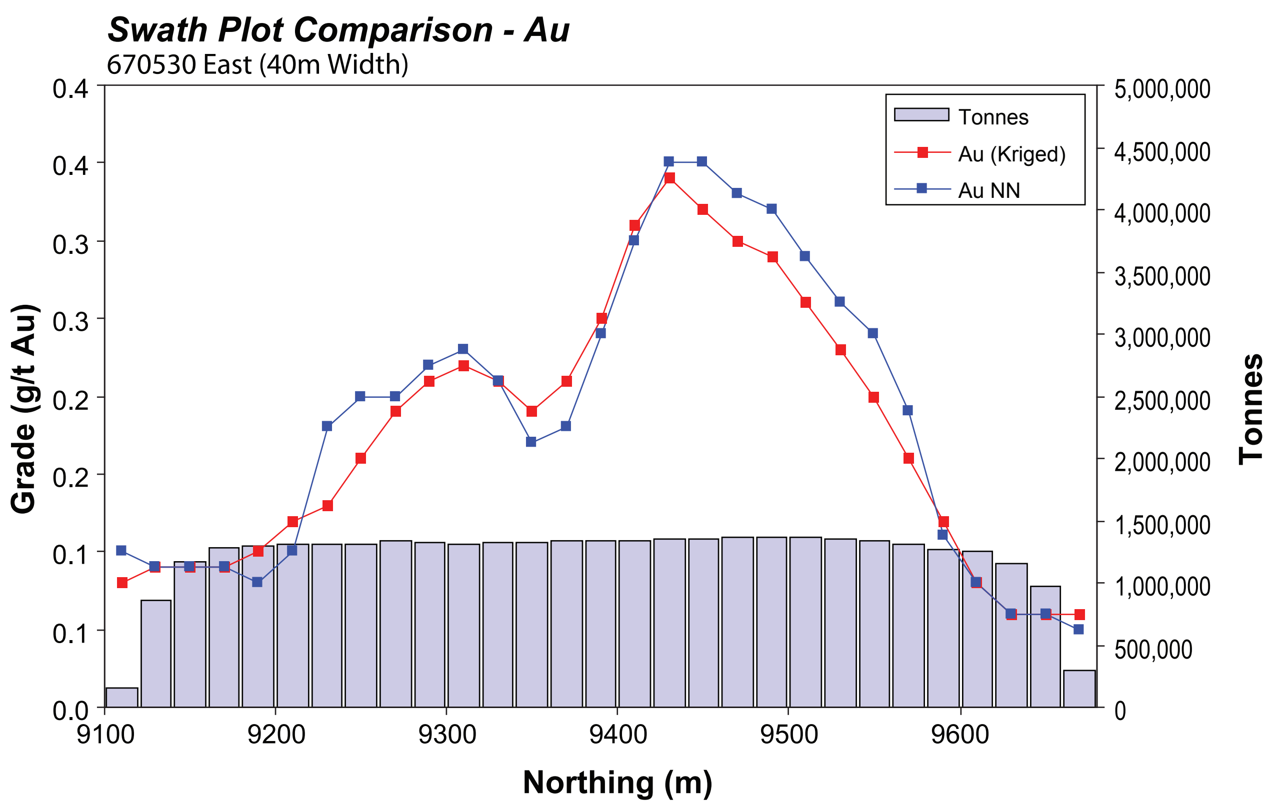

FIGURE 4.11-19

Morrison Copper/Gold Project
Swath Plot for Au – Long Section 670530 East

Table 4.11-9
Morrison Deposit – All Blocks Classified Measured or Indicated

Cut-off % Eq Cu	Measured					Indicated				
	Tonnes > Cutoff (000s)	Average Grade				Tonnes > Cutoff (000s)	Average Grade			
		% Eq Cu	% Cu	g/t Au	% Mo		% Eq Cu	% Cu	g/t Au	% Mo
0.10	122,098	0.42	0.36	0.18	0.004	167,328	0.38	0.32	0.15	0.005
0.15	120,670	0.43	0.36	0.18	0.004	162,427	0.39	0.32	0.16	0.005
0.20	116,486	0.43	0.37	0.18	0.005	150,772	0.41	0.34	0.17	0.005
0.25	109,119	0.45	0.38	0.19	0.005	132,892	0.43	0.36	0.18	0.005
0.30	98,142	0.47	0.40	0.19	0.005	110,108	0.46	0.39	0.19	0.005
0.35	81,644	0.50	0.42	0.21	0.005	89,290	0.49	0.41	0.20	0.005
0.40	64,209	0.53	0.45	0.21	0.005	69,831	0.53	0.44	0.21	0.006
0.45	48,076	0.56	0.48	0.23	0.006	53,005	0.56	0.47	0.22	0.006
0.50	33,909	0.60	0.51	0.24	0.006	36,484	0.59	0.50	0.23	0.006

Table 4.11-10
Morrison Deposit – Combined Measured and Indicated Resource

Cut-off % Eq Cu	Measured + Indicated				
	Tonnes > Cutoff (000s)	Average Grade			
		% Eq Cu	% Cu	g/t Au	% Mo
0.10	289,426	0.40	0.33	0.16	0.005
0.15	283,097	0.40	0.34	0.17	0.005
0.20	267,258	0.42	0.35	0.17	0.005
0.25	242,011	0.44	0.37	0.18	0.005
0.30	208,250	0.46	0.39	0.19	0.005
0.35	170,934	0.49	0.42	0.20	0.005
0.40	134,040	0.53	0.44	0.21	0.006
0.45	101,080	0.56	0.47	0.22	0.006
0.50	70,393	0.60	0.50	0.23	0.006

Table 4.11-11
Morrison Deposit – All Blocks Classified Inferred

Cut-off % Eq Cu	Inferred				
	Tonnes > Cutoff (000s)	Average Grade			
		% Eq Cu	% Cu	g/t Au	% Mo
0.10	93,021	0.38	0.32	0.16	0.004
0.15	88,898	0.39	0.33	0.16	0.005
0.20	83,055	0.41	0.34	0.17	0.005
0.25	73,505	0.43	0.36	0.18	0.005
0.30	62,839	0.46	0.38	0.19	0.005
0.35	48,695	0.49	0.42	0.21	0.005
0.40	37,735	0.53	0.45	0.22	0.005
0.45	28,567	0.56	0.47	0.23	0.006
0.50	20,745	0.59	0.50	0.24	0.006

Drilling methods consisted of ODEX drilling through the overburden and rotary drilling in bedrock using HQ Triple Tube. Standard penetration tests and Shelby tube samples were collected in soils, packer permeability tests were completed in competent bedrock, along with collecting geotechnical data. The results show a range of depth from 4 m to 20 m of moist, stiff till throughout the TSF area. The TSF area showed a mixture of sedimentary and volcanic bedrock beneath the overburden. Investigations at the plant site and surrounding area showed a consistent, moist, stiff till overburden with both volcanic and sedimentary bedrock. Groundwater monitoring wells were installed in geotechnical drill holes on the Morrison property, but groundwater was not sampled by Klohn.

4.12.2 2007 Klohn Crippen Berger Site Investigation Program

Klohn completed a two-phased site investigation program in 2007 to support the TSF feasibility design and a proposed plant site east of Booker Lake. Later, the plant site would be relocated to the knoll west of Booker Lake. The investigation consisted of two phases:

- Phase I – Geophysics: Electrical resistivity (ER) survey of proposed dam alignments and plant site;
- Phase II – Drill holes and test pits in overburden and bedrock, standpipe installation, hydraulic conductivity testing, and laboratory testing.

The site investigation plan is shown in Appendix 6. The following sections summarize the findings of the site investigation program. Further details can be found in Klohn's *2007 Geotechnical Site Investigation* dated November 19, 2008 (Appendix 9).

4.12.3 Phase II - Drilling and Test Pitting Program

The drilling program was designed using the results of the geophysical survey in which specific areas of interest were identified for drilling. The Phase II program was conducted from November 11 to December 17, 2007, and was completed under Klohn's technical supervision. In the TSF, the drilling program consisted of:

- 10 boreholes at 5 sites consisting of 1 deep and 1 shallow hole per site.
    - 5 deep boreholes drilled between 11 m and 39 m into bedrock, to total depths between 35 m and 58 m.
    - 5 shallow boreholes in overburden, with final depths between 11 m and 25 m.
- 13 piezometers constructed in 10 boreholes at 5 sites.

East of Booker Lake (at the previous plant site), the drilling program consisted of:

- 4 boreholes at 4 sites with a maximum depth of 25 m and up to 4 m into bedrock when encountered.
- 3 piezometers were installed in 3 boreholes.
- 1 borehole was cored 39 m into bedrock, to a total depth of 47 m, to ensure the plant site area does not contain ore mineralization.

4.12.3.1 Drilling Methods

The drill holes were completed using some or all of the following drilling methods, depending on the ground conditions:

- ODEX 90 hammer pushing 4.5” casing in overburden with a hole diameter of 123 mm;
- HQ triple tube mud rotary diamond drilling in rock with a hole diameter of 96 mm.

The drilling program and logs are summarized in Appendix 6. Standpipe installation methods are described in Section 4.12.1 and summarized in Table 4.12-1.

Table 4.12-1
2007 Drilling Program

Drilling Hole ID	Location	Date Started (2007)	Northing (m)#	Easting (m)#	Collar Elevation (m)*	Depth to Bedrock (m)	Hole Depth (m)
DH07-1A	North Dam	Nov. 11	6125281	671989	973	20.44	49.4
DH07-1B	North Dam	Nov. 15	6125279	671996	973	>17.4	17.4
DH07-2A	North Dam	Nov. 16	6125496	671403	990	23.9	35.1
DH07-2B	North Dam	Nov. 18	6125493	671396	990	>11.0	11.0
DH07-3A	Main Dam	Nov. 19	6123345	671446	974	21.9	41.6
DH07-3B	Main Dam	Nov. 22	6123335	671450	974	>15.4	15.4
DH07-4A	Main Dam	Nov. 23	6123637	671060	960	12.8	46.2
DH07-4B	Main Dam	Nov. 26	6123634	671070	960	>11.4	11.4
DH07-5A	Main Dam	Nov. 27	6123951	670477	935	19.2	21.5
DH07-5B	Main Dam	Nov. 29	6123965	670477	935	19.2	58.2
DH07-6	Plant Site	Dec. 3	6120025	671245	863	>23.2	23.2
DH07-7	Plant Site	Dec. 4	6120115	671105	851	>22.9	22.9
DH07-8	Plant Site	Dec. 5	6120422	671193	877	5.2	9.1
DH07-9	Plant Site	Dec. 5	6120197	671101	841	22.3	25.3
DH07-10	Plant Site	Dec. 6	6120299	671036	845	8.2	47.5

 # Coordinates were determined by handheld Global Positioning System (GPS).
*** Elevations were estimated from 2 m contours provided by PBM**
Packer testing was done in bedrock; falling head tests were performed in overburden. See below for details.

4.12.4 2008 Klohn Crippen Berger Site Investigation Program

Following the 2007 investigation the proposed plant site location changed from the east side of Booker Lake to the knoll on the west side of Booker Lake. This made room for a larger WRD and low grade ore stockpile around the north and east perimeter of the open pit. These changes prompted the need for further subsurface information, which led to the 2008 site investigation, completed in September 2008. Data from the geotechnical investigations are included in Appendix IV and comprised the following:

- geological and geomorphology mapping
- drilling and standpipe installation program

- test pit program
- geotechnical laboratory testing

4.12.4.1 2008 Drilling Program

The 2008 drilling program was carried out jointly by Klohn and Rescan to provide geotechnical data as well as environmental data and installations. During drilling of select monitoring well installations supervised by Rescan, Klohn recorded geotechnical data. The 2008 geotechnical drilling program is summarized in Table 4.12-2.

Table 4.12-2
Klohn 2008 Drilling Program

Drilling Hole ID	Location	Date Started	Northing (m)	Easting (m)	Collar Elevation (m)*	Depth to Bedrock (m)	Hole Depth (m)
DH08-1A	WRD	Sep 12, 2008	6120064	670403	819	14.17	20.12
DH08-1B	WRD	Sep 12, 2008				N/A	12.8
DH08-2	New plant site	Sep 13, 2008	6120472	670743	795	6.26	12.3
MW08-1	WRD	Sep 15, 2008	6119626	671032	849	55.88	86.2
MW08-3	Low grade ore Stockpile	Sep 21, 2008	6120820	669975	781	14.8	13.9

 # Coordinates were determined by handheld Global Positioning System (GPS).
*** Elevations were estimated from 2 m contours provided by PBM.**

The geotechnical drilling program ran from September 12 to September 22, 2008 under Klohn's supervision and was undertaken in the WRD and low grade ore stockpile foundations and at the new plant site west of Booker Lake. The program comprised the following:

- 2 boreholes (DH08-1 and DH08-2) drilled into overburden with standard penetration test (SPT) and large penetration test (LPT) split spoon samples taken every 1.5 m to 3 m, and then drilled 3 m into the underlying bedrock. The LPT is a larger diameter penetration sampler used for coarse-grained soils and LPT data are corrected to the SPT values for geotechnical assessment purposes.
- 2 pairs of monitoring wells, MW08-1(A,B) and MW08-3(A,B), installed into bedrock under Rescan's supervision, but with geotechnical information and SPT, LPT, and hydraulic conductivity data in overburden recorded by Klohn in one of each of the pairs.
- 6 thin-walled Shelby tube samples were collected in DH08-1A, MW08-1, and MW08-3, but recovery was limited because of the presence of gravel and cobbles in the soil.

Geological Mapping

Surface geological mapping of the property was undertaken in conjunction with mineral exploration efforts, the results of which are documented in earlier Noranda reports and BC

Mineral Assessment Reports. No additional geological mapping was undertaken during geotechnical or ML/ARD site investigations.

Subsurface geological mapping was undertaken by qualified professionals, and is represented by logging drill cores during the Morrison deposit exploration phase, and geological/geotechnical logging of drill core generated during geotechnical site investigations, in conjunction with groundwater monitoring well drilling. These latter, and more recent results, are documented in site investigation drill core logs.

4.12.4.2 Hydrogeological Investigations

Hydrogeological baseline conditions were established through field investigations between 2006 and 2008 and reported in Appendices 6, 7 and 24. A detailed report on 3D MODFLOW groundwater modelling for the Project is presented in Appendix 25.

Hydraulic Conductivity

Hydraulic conductivity testing was conducted during drilling investigations by Klohn in 2007 and Knight Piésold in 2006. Packer tests were completed in the glacial tills, the shallow fractured bedrock, and the deeper, more competent bedrock. The results of the testing are described in detail in Knight Piésold's geotechnical site investigation report (2006) and Klohn's 2007 geotechnical site investigation report (Appendix 9). Additionally, Klohn reviewed and incorporated falling head test results conducted by Rescan in monitoring wells MW07-1 through MW07-8, installed in October and November, 2007. The results of all hydraulic conductivity testing are summarized by depth in Table 4.12-3.

Table 4.12-3
Summary of Hydraulic Conductivity Testing

Drill Hole	Test Interval (mbgs) Top	Bottom	Hydraulic Conductivity K (m/s)	Geologic Unit
DH06-2	9.1	39.5	5.1E-07	Volcanic
DH06-3	6.7	36.9	3.3E-07	Volcanic
DH06-4	11.0	41.5	7.4E-07	Volcanic
DH06-6	9.6	36.7	1.4E-06	Volcanic
DH06-7	12.8	43.3	5.1E-06	Volcanic/Siltstone/Sandstone
DH06-11	8.8	36.9	7.2E-06	Siltstone
DH07-1A	23.35	35.66	6.0E-07	Sandstone/mudstone
DH07-2A	26.20	35.10	2.4E-07	Siltstone
DH07-3A	24.23	35.51	1.6E-06	Sandstone
DH07-4A	27.89	36.12	1.5E-07	Sandstone/siltstone
DH07-5B	26.37	42.98	9.2E-07	Meta-sedimentary
MW07-01A	16.76	28.04	1.2E-06	Volcanic/Siltstone
MW07-02A	4.57	21.94	6.8E-07	Limestone
	Geometric Mean		8.7E-07	

(continued)

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Table 4.12-3
Summary of Hydraulic Conductivity Testing (completed)

Drill Hole	Test Interval (mbgs) Top	Bottom	Hydraulic Conductivity K (m/s)	Geologic Unit
DH07-1A	35.51	49.38	7.0E-08	Sandstone/Mudstone
DH07-3A	35.51	41.61	1.8E-06	Siltstone
DH07-4A	36.12	46.18	2.3E-07	Siltstone/Sandstone
	Geometric Mean		3.1E-07	
DH06-1	59.4	89.9	2.4E-07	Volcanic
DH07-5B	45.26	58.22	6.6E-08	Siltstone/Sandstone Cong.
	Geometric Mean		1.3E-07	

For the fractured bedrock zone encountered at depths generally between 10 m and 40 m below ground surface (mbgs), the test results indicate hydraulic conductivities ranging from 1.5×10^{7} m/s to 5.1×10^{-6} m/s with a geometric mean of 8.7×10^{-7} m/s. The hydraulic conductivity decreased with depth: bedrock tested between approximately 40 m and 50 m has a range of 7.0×10^{-8} m/s to 1.8×10^{-6} m/s and a geometric mean of 3.1×10^{-7} m/s. Tests conducted at depths greater than 50 mbgs indicate conductivities ranging from less than 6.6×10^{-8} m/s to 2.4×10^{-7} m/s.

Hydraulic conductivity testing performed in the glacial tills indicated very low hydraulic conductivity. The tests were generally incomplete because of long recovery times. The hydraulic conductivity of the glacial tills is estimated to be on the order of 1×10^{-10} m/s.

<u>Standpipe Installations</u>

Sixteen standpipe piezometers were installed during the 2007 site investigation program (Table 4.12-4), and seven standpipe piezometers were installed in the 2008 Site Investigation Program as summarized in Table 4.12-5 to monitor groundwater piezometric levels Static water levels were measured, but response times in the overburden were typically much too slow to get an accurate water level. The water level in bedrock responded much more quickly. Table 4.12-5 also shows piezometric levels collected in April 2008, by Rescan. These levels will no longer be affected by drilling procedures.

Groundwater Recharge and Discharge

<u>Recharge to Overburden and Bedrock Groundwater</u>

The recharge to the overburden groundwater regime is mainly drawn from rainfall and spring snowmelt waters. In 2008, an automated precipitation monitoring system installed by Rescan recorded a total precipitation of 658 mm (Appendix 18). Environment Canada meteorological stations within 100 km of the Morrison property recorded an annual average precipitation of 500 mm over the period from 1971 to 2000. Secondary sources of recharge to overburden groundwater may include upgradient subsurface flows coming from higher elevation sites on and around the property.

Table 4.12-4
Morrison Copper/Gold Project : 2007 Standpipe Installations[1]

Drilling Hole ID	Nested Piezo	Location	Installation Date (2007)	Total Hole Depth (mbg[2])	Piezo Stickup (mags[3])	Screen Depth (mbg[2])	Filter Pack Interval (m)	Geologic Unit at Screen Depth	Static Water Level (mbg[2])[5]	Static Water Level (mbTOC[4])[6]
DH07-1A		North Dam	Nov. 15	49.4	0.90	43.1 - 49.2	42.8 – 49.4	Sandstone and siltstone	-4.5 (artesian)	(artesian)
DH07-1B		North Dam	Nov. 16	17.4	0.83	14.2 - 17.2	13.6 – 17.4	Gravelly clay/silt (TILL)	unknown	frozen
DH07-2A		North Dam	Nov. 18	35.1	0.97	31.7 - 34.7	31.1 – 35.1	Siltstone	27.7	28.71
DH07-2B		North Dam	Nov. 19	11	0.93	7.6 - 10.7	7 – 11	Gravelly clay (TILL)	unknown	7.25
DH07-3A		Main Dam	Nov. 22	41.6	0.92	38.4 - 41.5	37.8 – 41.6	Siltstone	10.7	9.51
DH07-3B		Main Dam	Nov. 23	15.4	0.86	12 - 15.1	11.6 – 15.4	Gravelly clay (TILL)	unknown	11.58
DH07-4A	S1	Main Dam	Nov. 25	46.2	0.82	43 - 46	42.5 – 46.2	Siltstone	9.7	11.1
DH07-4A	S2	Main Dam	Nov. 25	46.2	0.84	33.4 - 36.4	32.9 – 36.7	Sandy siltstone	10.5	11.78
DH07-4B	S1	Main Dam	Nov. 26	11.4	0.90	9.8 - 11.3	9.4 – 11.4	Gravelly clay (TILL)	unknown	11.67
DH07-4B	S2	Main Dam	Nov. 26	11.4	0.92	3 - 4.6	2.7 – 4.7	Gravelly clay (TILL)	unknown	4.96
DH07-5A	S1	Main Dam	Nov. 28	21.5	0.85	19.2 - 21.3	19.1 – 21.5	Volcaniclastic	9.3	12.32
DH07-5A	S2	Main Dam	Nov. 28	21.5	0.87	13.7 - 15.2	13.6 – 15.4	Gravel and clay (TILL)	10	11.49
DH07-5B		Main Dam	Dec. 2	58.2	0.88	55 - 58.1	54.3 – 58.2	Sandstone	unknown	12.38
DH07-6		Plant site	Dec. 3	23.2	0.86	21 - 22.6	20.7 – 23.2	Silty clay (TILL)	unknown	
DH07-7		Plant site	Dec. 4	22.9	0.91	21 - 22.6	20.7 – 22.9	Clay, some gravel (TILL)	unknown	
DH07-9		Plant site	Dec. 6	25.3	0.91	18.3 - 19.8	18 – 19.8	Silty clay (TILL)	unknown	

Notes:
1. **Pipe diameter: 26 mm.**
2. **mbg – metres below ground.**
3. **mags – metres above ground surface.**
4. **mbTOC – metres below top of casing/pipe.**
5. **Static water levels measured from Nov. 15 to Dec. 6, 2007 during the drilling program.**
6. **Static water levels measured on April 6 2008 by Rescan**

Table 4.12-5
Morrison Copper/Gold Project : 2008 Standpipe Installations[1]

Drilling Hole ID	Location	Installation Date	Total Hole Depth (mbg[2])	Screen Depth (mbg[2])	Filter Pack Interval (m)	Geologic Unit at Screen Depth	Static Water Level (mbg[2])[3]
DH08-1A	west of Booker Lake	Sep 13, 2008	20.1	16.15 – 19.2	15.24 – 20.12	Bedrock - Wacke	Unknown
DH08-1B	west of Booker Lake	Sep 14, 2008	12.8	9.14 – 12.19	8.53 – 12.8	Till	1.52
DH08-2	New Plant site	Sep 15, 2008	12.4	8.81 – 11.89	8.23 – 12.4	Bedrock – Shale	4.7
MW08-1A[4]	NE of open pit	Sep. 19, 2008	86.2	72.09~78.18	70.26~78.64	Bedrock	Unknown
MW08-1B[4]	NE of open pit	Sep 20, 2008	30.5	23.78~29.87	?~30.18	Till	Unknown
MW08-3A[4]	N of low grade ore stockpile	Sep. 21, 2008	36.2	30.48~35.05	28.96~35.51	Bedrock	Unknown
MW08-3B[4]	N of low grade ore stockpile	Sep. 22, 2008	14.8	8.99~13.72	8.23~13.89	Till	Unknown

1. Pipe diameter: 50.1 mm.
2. mbg – metres below ground.
3. Static water levels measured on Oct. 10 2008 by Rescan.
4. Data from Rescan.

Recharge to bedrock groundwater originates from two main sources: seepage from the overburden and groundwater flow by infiltration and seepage through the faults and fractures in the Morrison property. Direct recharge to bedrock can also occur where fractured bedrock is directly exposed at the surface (i.e., bedrock outcrop). The groundwater is possibly transmitted through fractures from outcrops at higher elevations. This might explain the flowing artesian conditions of some monitoring wells downgradient of bedrock outcrops in the Project footprint.

<u>Discharge to surface water</u>

The presence of flowing artesian wells on the property suggests that upward vertical hydraulic gradients contribute groundwater to surface water. Given that upward hydraulics occur generally (but not exclusively) at lower elevations in the Morrison property, this is likely to be where the groundwater contribution to the surface water regime is of higher importance. Groundwater discharge to surface water occurs along creeks, such as those adjacent to MW07-03A and MW07-01A, and smaller waterbodies such as Booker Lake.

Groundwater flow directions

Figures 4.12-1 and 4.12-2 present the potentiometric surface map created from the available water level data from monitoring wells and piezometers. In general, the groundwater surface contours follow the overlying topography contours. In the northern area of the Morrison property, groundwater flows in a northeasterly direction away from the TSF's proposed north dam. The TSF is surrounded by highlands to the east and northwest. The groundwater flows from these highlands into the proposed TSF area, where wetlands are common. From these wetlands, groundwater flows in a southwesterly direction towards Morrison Lake. In the southern portion of the Morrison property, in the proposed open pit location and surrounding area, groundwater flows in a southwesterly direction from the highlands to the east towards Morrison Lake.

All the Rescan monitoring wells were drilled and installed in nested well pairs consisting of a deep (A) and a shallow (B) monitoring well. This allows for characterizing the vertical flow gradients at the different monitoring well locations by comparing water levels measured near each other (approximately 5 m apart) and with screened units placed at different elevations. A general trend shows that well locations at higher elevations on the Morrison property have negative vertical gradients (i.e., downward vertical groundwater flow component), and monitoring well locations at lower elevations near valley bottoms show an upward groundwater flow component or positive vertical gradient. Exceptions to this trend occur where Morrison Lake or a geological structure, such as a major fault, is close to the monitoring well location.

4.12.4.3 Groundwater Quality

Five groundwater sampling events (November 2007, and January, April, July, and October 2008) were completed over a one-year period. A total of 71 groundwater samples were collected at all the Rescan monitoring wells and sent to ALS Laboratory Group of Vancouver, BC, for analysis. Samples were also taken from three Knight Piésold monitoring wells (DH06-7, DH-06-11, and DH06-12) in July, 2006, and included in the data set presented. Table 4.12-4 summarizes the locations and collection dates as well as the water type of the groundwater samples collected.

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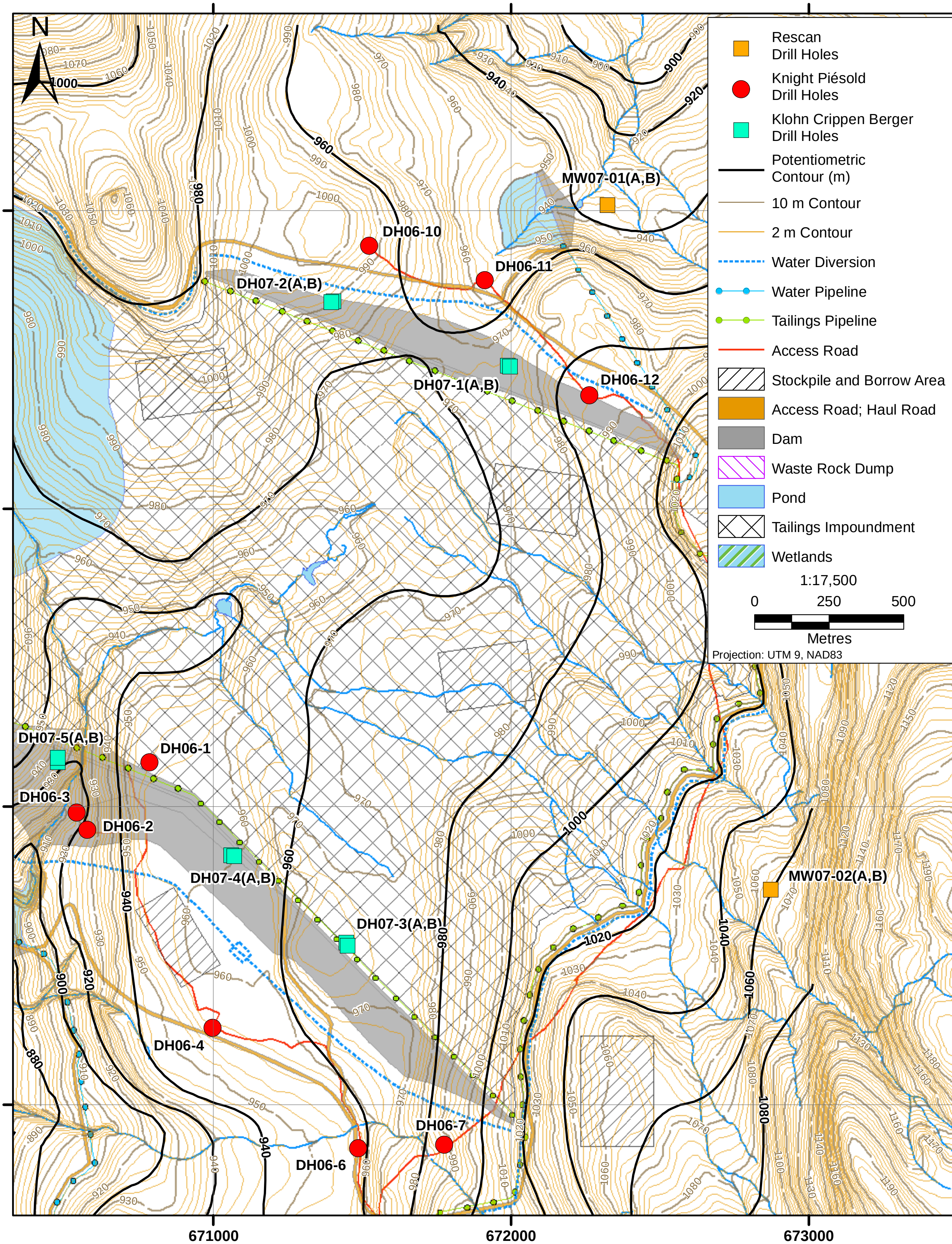


FIGURE 4.12-1

Morrison Copper/Gold Project: Potentiometric Surface Map (TSF Area)

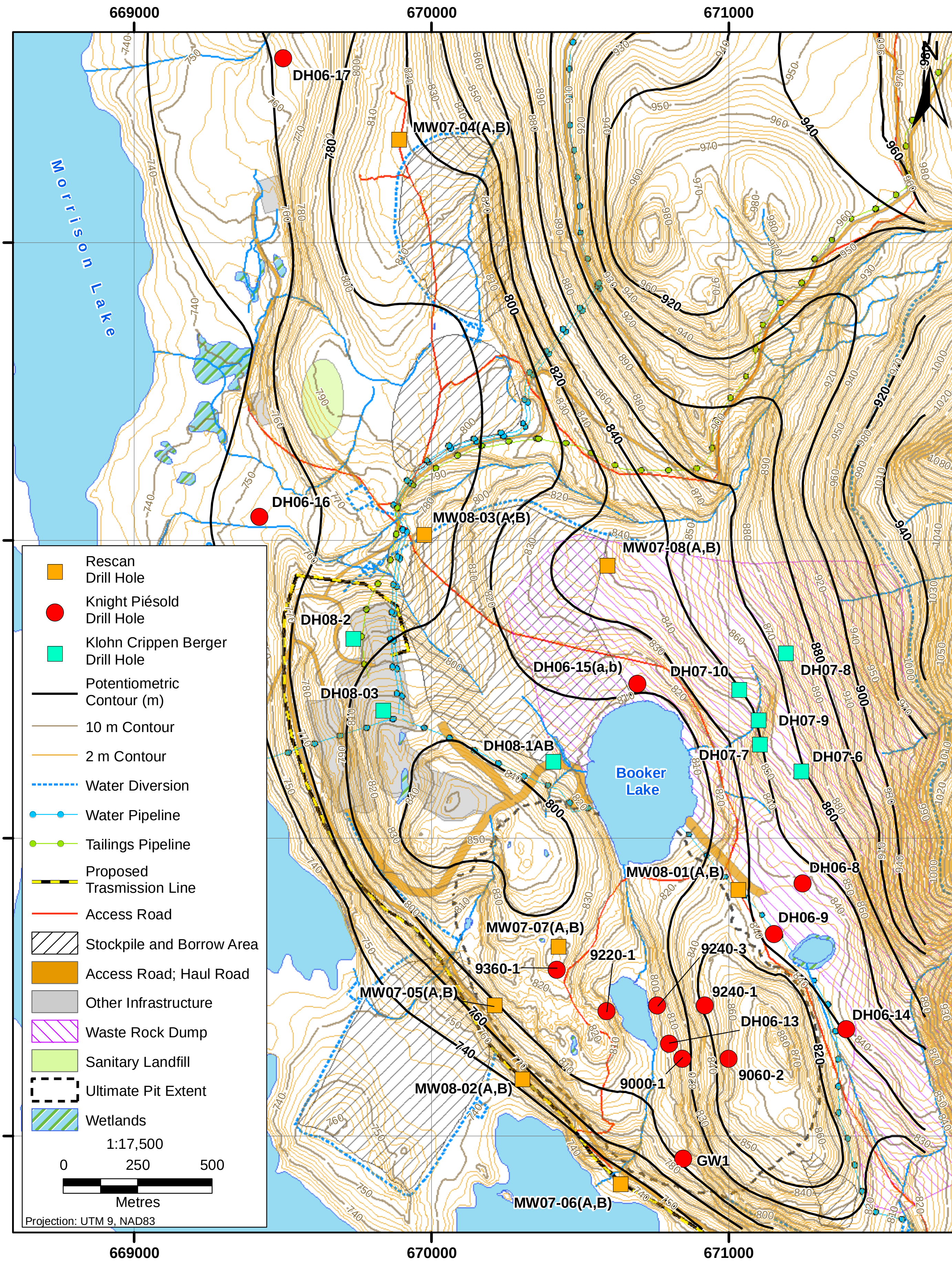


FIGURE 4.12-2

Morrison Copper/Gold Project: Potentiometric Surface Map (Open Pit Area)

The requested analyses were: general chemistry, total organic carbon (TOC), and total and dissolved metals. The concentrations obtained were compared to three guidelines: the British Columbia Approved and Working Water Quality Guidelines (BCWQG) for drinking water, the BCWQG for freshwater aquatic life, and the Canadian Council of Ministers of the Environment (CCME) Canadian Water Quality Guidelines for the protection of freshwater aquatic life.

Anions and metals concentrations in many of the groundwater sampling sites in the Morrison property exceed maximum allowable concentrations listed in the CCME water quality guidelines and BC water quality guidelines for the protection of freshwater aquatic life and the BCWQG for drinking water. Groundwater quality characterized by various exceedances of these guidelines should be taken into consideration when managing and displacing groundwater on the property during mine development, operations, and closure, especially near Morrison Lake or other groundwater-discharge areas that might be sensitive to a change in water chemistry, such as fish-bearing streams.

Geophysical Investigations

Six electrical resistivity lines totalling approximately 9.5 km were surveyed between May 4 and June 12, 2007 by Frontier Geosciences Inc. Lines were located as shown in Appendix 6. Data processing and inversion were done by Frontier. The inverted resistivity sections were interpreted by Klohn. The interpreted resistivity sections are presented in Appendix 9.

In general, the interpreted resistivity sections correlate well with drill hole data and available regional structural data suggesting that data quality is good. Three main units were identified: conductive overburden, resistive bedrock, and conductive bedrock. The conductive overburden unit is interpreted to be till. Highly conductive areas likely correspond to regions with higher moisture content, while resistive layers are interpreted to be layers of coarse material within the till. The extent and thickness of the overburden unit varies across the site. The northwestern region covered by resistivity lines: RL-KC07-4A and RL-KC07-4B shows only small patches of overburden with a maximum thickness of approximately 10 m, while the other areas show large extents of overburden, averaging approximately 15 m thick but in places showing >30 m (greater than the depth of the survey). Given that the resolution of the resistivity data is approximately 2 m vertical and 5 m horizontal, it is not suitable for identifying small or thin features and thus the till appears relatively homogeneous.

The bedrock was broadly classified into two types, resistive and conductive. Conductive bedrock was interpreted to be siltstone or other fine-grained sedimentary rock, while resistive bedrock was interpreted to be sandstone. The highly resistive bedrock on lines RL-KC07-4A and RL-KC07-4B could be sandstone, but given the resistivity values and local geology observations, is more likely to be basalt. Several local faults were interpreted on the inverted resistivity sections. Some, such as the one at the northwestern end of RL-KC07-1B are interpreted based on a change in resistivity, while others are interpreted from linear anomalies.

Test Pit Excavations

The test pit program consisted of eight test pits and was conducted from December 5 to 7, 2007. A 345 Cat excavator supplied by Babine Barge was used to dig the test pits. Test pit depth ranged from 0.8 m to 11.0 m. Grab samples were taken for moisture content, grain size, and Atterberg testing. The test pit program is summarized in Table 4.12-6. The test pit logs are presented in Appendix 11.

Table 4.12-6
2007 Test Pit Program

Test Pit	Date (2007)	Northing	Easting	Elevation (m)	Depth (m)	Depth to Bedrock (m)	Surface Material
TP07-1	Dec. 5	6120423	670641	821	5	Unknown	Glacial Till
TP07-2	Dec. 6	6121305	670015	795	6	Unknown	Sand and Gravel
TP07-3	Dec. 6	6121117	669994	795	6	Unknown	Sand and Gravel
TP07-4	Dec. 6	6120999	669939	789	6	Unknown	Sand and Gravel
TP07-5	Dec. 5	6120486	670347	828	6	Unknown	Glacial Till
TP07-6	Dec. 6	6120827	669928	776	6	Unknown	Glacial Till
TP07-7	Dec. 7	6123188	672197	1,040	2	1	Glacial Till
TP07-8	Dec. 7	6123524	672499	1,025	3.4	2.4	Glacial Till

Laboratory Testing

Geotechnical testing of selected representative soil samples was performed in Klohn's Vancouver laboratory. Grab samples were collected from test pit excavations, and SPT split spoon samples were obtained at regular intervals in overburden drill holes.

A suite of geotechnical laboratory tests was performed on selected soil samples to characterize gradation and plasticity properties. The following is a summary of the tests performed:

- 104 moisture content tests (ASTM D2216) to determine *in situ* moisture contents
- 21 washed sieve analyses (ASTM D422) to determine gradation
- 10 hydrometer analyses (ASTM D422) to determine gradation of the fine portion
- 10 Atterberg Limit tests (ASTM D4318) to assess the soil classification of the fine portion
- 2 Standard Proctor tests (ASTM D698) to determine a moisture-density relationship

4.12.4.4 Geotechnical Design Criteria

The consequence classification of the tailings facility was assessed to guide selecting criteria for the flood and seismic design. That assessment was based on a preliminary screening-level review with considering the potential incremental life safety, socioeconomic, financial, and environmental consequences of failure, and the associated hazard ratings as provided for in the Canadian Dam Safety Guidelines (CDA 2007) and the BC *Water Act* (1996f).

Given the potential environmental damage and substantial clean-up costs, the TSF is classified as a "Very High" consequence facility (1996f).

The tailings dam is designed to international standards, using International Commission of Large Dams (ICOLD) Guidelines (1996) and Canadian Dam Safety Guidelines (2007), BC Dam Safety Regulations, 2000. The main design criteria are summarized in Table 4.12-7.

Table 4.12-7
Summary of Design Criteria

Item	Criteria
Storage Capacity Tailings Storage Facility: Starter Dam Ultimate Dam Waste Rock Dumps: Waste Rock Dump Overburden Dump Organic Sediment Storage Temporary Stockpiles: Low Grade Ore Stockpile Organic Stockpiles	Year 1 tailings production (9.9 Mt tailings @ 1.4t/m^3) Total tailings production (224 Mt tailings @ 1.5t/m^3 less cycloned sand) Total Not-PAG, PAG waste (170 Mt @ 2t/m^3) Total Overburden and Unknown, less amounts used in dam construction (15.2 Mt @ 1.7t/m^3) Excavated lake bottom sediments 29 Mt @ 2t/m^3 Organic bearing soils from foundation areas (1,185,000t @ 1.5t/m^3)
Water & Flood Management during Operation Diversion of upland catchment, if required for water balance purposes Flood management – dam safety Flood discharge	1: 100 year peak flow storage of 30 day PMF or discharge of PMF peak flow with an emergency spillway flows exceeding 1:200 year peak flow can be discharged.
Seismic Return Period Tailings Dam – During operation and closure Waste Dumps – During closure	MCE PGA= 0.13 and M_w6.2 5,000 year return period PGA = 0.1
Geotechnical Factors of Safety Tailings Dam Operational & Closure PAG Dump, Low Grade Ore Stockpile, and Overburden Dump Operational & Closure Organic Stockpiles Operational Only	Static FOS = 1.5 Post-Construction FOS = 1.3 Pseudo-static FOS = 1.1 (seismic coefficient = 50% of pga = 0.065 g) Static FOS = 1.3 Post-Construction FOS = 1.3 Pseudo-static FOS = 1.1 (seismic coefficient = 50% of pga = 0.065 g) Static FOS = 1.3 Post-Construction FOS = 1.3
Environment - Operations Total seepage flows out of waste dump	Seepage to be collected and returned to the process plant
Closure Flood Handling Diversion Ditches Spillway Reclamation	All diversion ditches to be decommissioned PMF period routed peak flow Contingency spillway to ensure Main Dam would not be breached. Dam slopes to be reclaimed. Impoundment area to be combination of reclaimed beaches, if possible, and water pond areas. All PAG material to be permanently saturated.

Table 4.12-8 lists the feasibility level design criteria for dump stability that have been adopted from the BC Mine Waste Rock Pile Research Committee (BC MWRPRC 1991). The FOS criteria selected for static stability are 1.3 for operations and 1.5 for closure, which are conventionally used for engineered waste dumps. Stability analyses are conducted using peak frictional strength parameters and estimated operating pore water pressures in the dump materials and underlying foundation soils.

Stability evaluations were performed on all dumps with the exception of the organic stockpile #2 dump, which is the most stable of the two organic stockpile dumps and also of negligible consequence.

The seismic design criterion adopted for this study is a ground displacement of less than 1 m for the (probabilistically derived) 10,000-year return period peak ground motion of 0.13 g. Only the PAG waste dump was subjected to a seismic deformation analysis.

Table 4.12-8
Stability Criteria for Dump Design

Loading Condition	Design Standard	Design Criteria
1. Static Loading Conditions	Two-Dimensional Limit-Equilibrium Factor of Safety with operating pore pressures. Deep-seated failure surfaces through dump foundation.	F.S. 1.3 For operating conditions F.S. 1.5 For long-term closure conditions
2. Pore Water Pressure Conditions[1]	Using B bar pore water pressure piezometric line.	$\overline{B}$ = 0.4 for operating conditions and $\overline{B}$ = 0 for long-term closure
3.. Seismic Loading Conditions[1,2]	Pseudo-Dynamic Deformation Analyses	<1 m horizontal displacement for long-term closure conditions F.S. 1.1 for long-term closure conditions 10,000 – year return period for operation and closure.

Note:
1. **Pore water pressure conditions and seismic loading conditions were not applied to organic stockpiles.**
2. **Only the Closure PAG waste dump was subjected to a seismic horizontal displacement analysis.**

4.12.4.5 Hydrologic Design Criteria

Selecting the design flood for the TSF considered the CDA Safety Guidelines (2007), which recommends an inflow design flood (IDF) of 2/3 between 1 in 1,000-year occurrence and probable maximum flood (PMF). Similarly to the design earthquake criterion, the IDF flood criterion has been upgraded to the PMF. In addition, flood criterion considers combinations of rain on snow events.

The design flood can be managed with storage or an open channel spillway or, in part, with pumping. Several potential scenarios were considered, as follows.

Scenario A – 30 day PMF

The 30-day PMP of 0.54 m would, assuming a runoff coefficient of 1.0, produce approximately 5.3 Mm^3. Storing this flood would require approximately 10 m of freeboard on the starter dam, reducing to 2 m of freeboard on the final dam. Managing the flood could also be achieved by storing the 7-day PMP of 0.35 m, or 3.4 Mm^3, and pumping or discharge in an emergency spillway for the remaining flood waters.

Scenario B 2 week - 200 year return period – rain on snow

This event is equivalent to 0.32 m of precipitation, or 3.2 Mm^3. This could be stored in the impoundment, with an emergency spillway constructed to pass any additional flows.

In consideration of Scenarios A and B, Klohn recommends a flood management criterion of storage of the 7-day PMP, which would also store a 2-week, 200-year rain on snow event, with an emergency spillway to discharge any flows in excess of this criterion. A freeboard of 1 m above the flood volume would be provided to permit construction of the spillway.

On closure, a permanent spillway in rock would be designed to safely pass the peak flow from a PMF.

The design flood criteria selected for various other components of the water management facilities are summarized in Table 4.12-9. The expected operating life of the mine was taken into account in selecting the design floods for temporary facilities, such as the surface runoff diversion ditches, the starter dam emergency spillway, the seepage collection ditches and the seepage recovery pond. Based on current resources, the mine is expected to be active for a period of about 19 years. During this time, all facilities related to the tailings impoundment would be closely and frequently monitored, and personnel, equipment, and materials are expected to be readily available in the event that remedial measures are required under routine and/or emergency maintenance.

Table 4.12-9
Flood Design Criteria for Water Management Facilities

Facility	Design Flood Return Period (years)	Flood Storage & Freeboard Allowance	Comments
Tailings Impoundment	PMF	1 m freeboard and storage of 3.4 Mm^3	Store 7-day PMP of 0.35 m and discharge excess flows in an emergency spillway Assume upland ditches are not operational and a runoff coefficient of 1.0.
Surface water diversion ditches	100		Ditches may be required for temporary water control.
Tailings Dam Closure Spillway	PMP		Assume that upland surface water diversion ditches have been decommissioned.

(continued)

Table 4.12-9
Flood Design Criteria for Water Management Facilities (completed)

Facility	Design Flood Return Period (years)	Flood Storage & Freeboard Allowance	Comments
Seepage Collection Ditches	100		-
Seepage Recovery Pond Spillway	100		Assume that upland surface water diversion ditch is functioning.
Tailings Impoundment on Closure	PMF	2 above spillway invert	Routing of peak flow from the PMF over a closure spillway.
Waste rock dump collection ditches and sediment pond	100		
Plant site and waste dump diversion ditches	100		

4.12.4.6 Seismic Design Criteria

The design earthquake selected for the tailings dam considered the CDA's Safety Guidelines (2007) guideline for a "Very High" consequence dam, which recommends an annual exceedance probability of 1:5,000 year. However, the design criterion has been upgraded to a 1:10,000 return period because the dam structures can readily meet this criterion and it provides a greater degree of certainty, particularly for long-term closure. The design criterion also meets the maximum credible earthquake (MCE) based on a deterministic assessment.

The design earthquake selected for the WRD and LGO stockpile was based on a low consequence of failure as the dump would deform but would not fail catastrophically. A return period of 5,000 years was conservatively selected.

Seismic Hazard Assessment

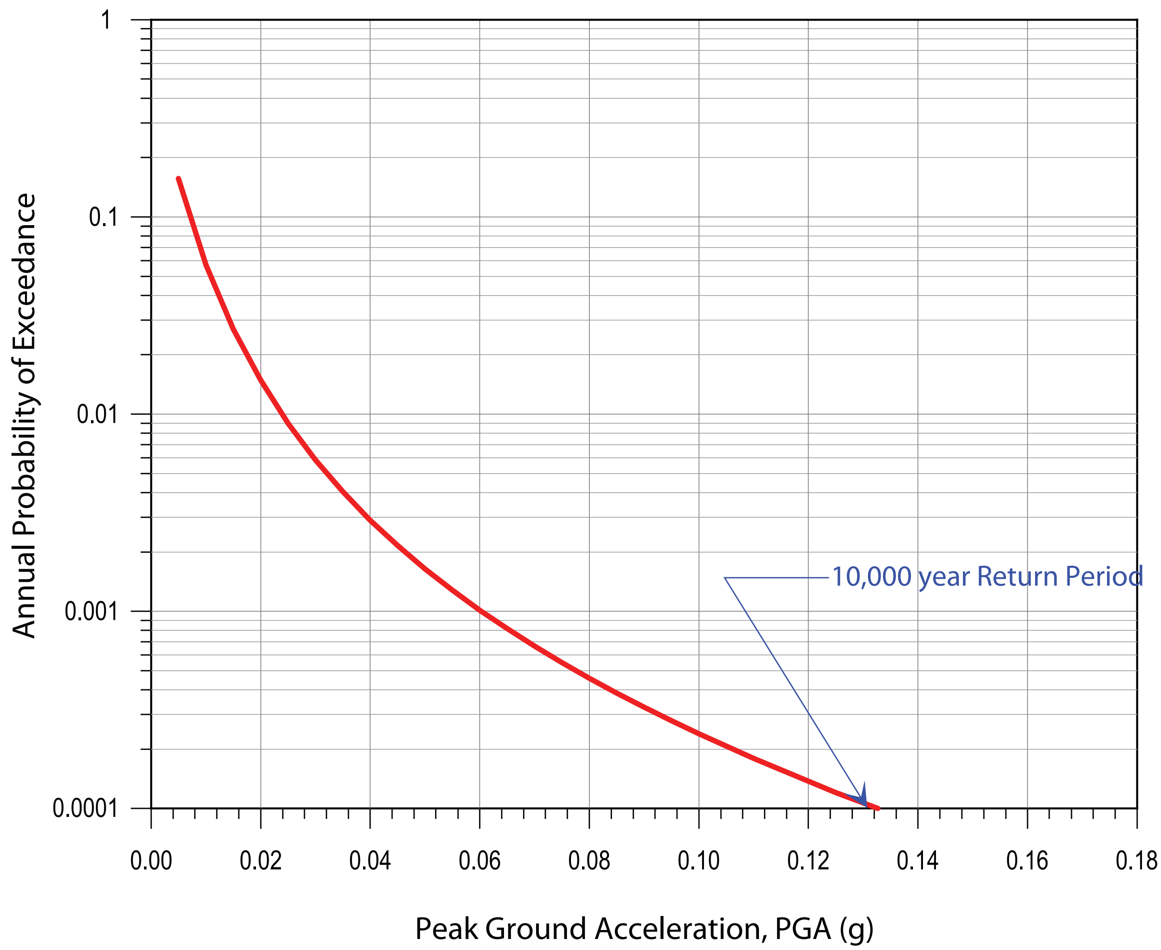
Probabilistic and deterministic seismic hazard analyses were conducted to derive the (MCE) peak ground acceleration (PGA) for the Project site. See Appendix 9 for the details. Figure 4.12-3 illustrates the seismic hazard curve from probabilistic analyses.

Recommendations for MCE Ground Motions

Based on the seismic hazard assessment, an MCE PGA of 0.13 g is recommended for the Project site and it should be associated with an earthquake magnitude of M_w6.2 in seismic deformation and liquefaction assessments. The recommended ground motion is applicable to Site Class C (i.e., very dense soil or soft rock with shear wave velocity of 360 m/s to 760 m/s in the top 30 m) conditions (NEHRP 2003).

Seepage

The environmental design is based on protecting Morrison Lake and Nakinilerak Lake where aquatic habitat is present. Surface water will target BC aquatic life water quality guidelines (BC MOE 2006), generally more strict than drinking water guidelines, 30-day-average values.



Morrison Copper/Gold Project:
Seismic Hazard Curve from Probabilistic
Seismic Hazard Assessment

FIGURE 4.12-3

The design basis for determining the "allowable" seepage rate from the impoundment is typically based on comparing the tailings supernatant water quality with receiving water quality criteria for drinking water, irrigation, and aquatic habitat. The assessment is based on determining the fate and transport of potential contaminants from the impoundment to the receiving environment. During operations, seepage through the dam will be collected with a seepage collection ditch and pond for return to the impoundment, as required. However, on closure it is preferable to have a passive system, which does not require ongoing controls.

Accordingly, the design seepage rate is "as low as reasonably possible" and, given the generally low permeability of the impoundment soils, is targeted as <5 L/s.

4.12.4.7 Metal Leaching and Acid Rock Drainage Studies on Tailings for Cycloned Dam Construction

Geochemistry – Cycloned Sand

The cycloned sand is predicted to be not potentially acid generating (not-PAG) and ongoing testing will be carried out during operations to confirm that the cycloned sand is not-PAG and that there is no risk of metal leaching and acid rock drainage (ML/ARD). Static ABA tests will be carried out weekly or bi-weekly to confirm the consistency of the cyclone sand. In addition, a larger scale field leach pad will be constructed with several tailings samples to confirm the longer term leaching properties of the cyclone sand.

If the cycloned sand does not meet specifications for geochemistry it will not be placed in the dam, and therefore, dam materials would be sourced from borrow areas within the impoundment.

Geochemistry – Fine Tailings and Total Tailings

The fine tailings and the total tailings are currently predicted to have some risk of ARD and the base case strategy is to maintain the tailings in a saturated state to mitigate ARD. The geochemistry of the tailings will be monitored bi-weekly or monthly with ABA tests. Several large-scale field leach pads will be constructed to monitor the longer term ML/ARD properties of the tailings. Longer term column tests will be carried out to confirm the expected negligible effect of metals leaching into a surface water cover for the closure condition.

4.12.4.8 Plant Site Foundations

The process plant will be built on a knoll north of the mine pit, on two approximately level platforms to suit the terrain. A balanced cut and fill site grading is proposed, resulting in cuts and fills of up to 15 m depths and heights. The drilling and test pitting program conducted for this study indicates that the site is underlain by very stiff to hard till over sandstone and porphyry bedrock.

Results of the geotechnical foundation design indicate that the undisturbed native till and bedrock are considered suitable as foundation soils to support the process plant structures on conventional spread footings. Major settlement sensitive machine foundations, such as the crushers, HPGR, and ball mills, should be founded on bedrock. Lightly loaded or settlement-tolerant structures may be founded on compacted structural fill. The excavated till is considered a suitable material to be reused as fill at the site; however, the excavated silty/clayey till material is moisture sensitive and care must be taken during construction to keep the material dry before placement and compaction.

4.12.4.9 Waste Rock Dump and Stockpile Factor of Safety Design Criteria

The following factor of safety and design criteria are determined for the waste rock dump and other stockpiles.

Geotechnical Factors of Safety Tailings Dam Operational & Closure	Static FOS = 1.5 Post-Construction FOS = 1.3 Pseudo-static FOS = 1.1 (seismic coefficient = 50% of pga = 0.065 g)
PAG Dump, Low Grade Ore Stockpile, and Overburden Dump Operational & Closure	Static FOS = 1.3 Post-Construction FOS = 1.3 Pseudo-static FOS = 1.1 (seismic coefficient = 50% of pga = 0.065 g)
Organic Stockpiles Operational Only	Static FOS = 1.3 Post-Construction FOS = 1.3

Stability

Two-dimensional limit equilibrium stability analyses of the WRD and stockpiles were carried out using the computer program SLOPE/W. Planar and circular slip surfaces were analyzed using the Morgenstern-Price method of slices. Pore pressures in the waste rock and foundation soils were input as piezometric elevations with pore pressure ratios ($\overline{B}$). The soil parameters and piezometric conditions assumed in the analysis are summarized in Table 4.12-10.

Table 4.12-10
Summary of Material Properties – Waste Dump Stability

Material	Total Unit Weight (kN/m^3)	Static Drained Strength	Piezometric Condition
PAG Waste Rock	20	$\phi' = 40°$ c' =0 kPa	No piezometric surface
Low Grade Ore	20	$\phi' = 40°$ c' =0 kPa	No piezometric surface
Colluvium/Till Foundation	18	$\phi' = 31°$ c' =0 kPa	Piezometric line at top of till $\overline{B}$ = 0.4 during operation only.
Overburden	18	$\phi' = 31°$ c' =0 kPa	No piezometric surface
Organic Overburden	18	$\phi' = 27°$ c' =0 kPa	Ru = 0.1

The results of the stability analysis are summarized in Table 4.12-11 and the results are included in Appendix VII.

Table 4.12-11
Summary of Stability Analyses –
Waste Rock Dump and Overburden Piles

Facility	Operations Static	Operations Pseudostatic[1]	Closure Static	Closure Pseudostatic[1]
PAG Waste Rock Dump (south side)	1.29	1.1	2.03	> 1.47
PAG Waste Rock Dump (north side)	1.38	1.1	2.07	
Low Grade Ore Stockpile	1.32	1.1	1.60	> 1.26
Overburden Pile	1.2		1.56	1.1

Note:
1. Pseudostatic factor of safety was calculated with a ground acceleration of 0.10 g, actual effective ground acceleration is 0.065 g.

4.12.4.10 Low Grade Ore Stockpile

The following factor of safety and design criteria are determined for the low grade ore stockpile.

Geotechnical Factors of Safety	Static FOS = 1.5
Tailings Dam	Post-Construction FOS = 1.3
Operational & Closure	Pseudo-static FOS = 1.1 (seismic coefficient = 50% of pga = 0.065 g)
PAG Dump, Low Grade Ore Stockpile, and Overburden Dump	Static FOS = 1.3
Operational & Closure	Post-Construction FOS = 1.3
Organic Stockpiles	Pseudo-static FOS = 1.1 (seismic coefficient = 50% of pga = 0.065 g)
Operational Only	Static FOS = 1.3
	Post-Construction FOS = 1.3

During operation, potentially acid-generating (PAG) waste rock and low grade ore will be placed in 10-to-20 m high maximum lifts, with intra-slope benches to achieve an average overall slope of 2H:1V. The minimum 2H:1V slope facilitates re-contouring of the PAG dump slope for reclamation at mine closure to an overall slope of 2.75H:1V and intra-slope benches at least every 50 m.

Stripped overburden and organic material will be stored in temporary stockpiles on-site and be placed in 3–to-5 m high lifts. The temporary stockpiles will have intra-slope benches to achieve an average overall slope of 3H:1V.

All waste dumps and temporary stockpiles should have a minimum setback of 100 m from the toe to the open pit.

The 1991 Investigation and Design Manual developed by the BCMWRPRC (1991) proposed a Dump Stability Rating classification system to assess the likelihood of a proposed dump experiencing significant instability or failure conditions during construction. Table 4.12-11 presents the rating of the four dump sites. All sites classify in Dump Stability Class II, which has a failure hazard of low.

Recommended levels of effort for investigation, design, and construction include:

- test pit site investigation
- limited laboratory index testing, as required
- basic stability analyses of dump construction
- limited restrictions on dump construction
- routine visual and instrument monitoring

The level of site investigations undertaken at the dump sites is consistent with the low failure hazard.

4.12.4.11 Tailings Dams

Location

Alternative studies completed to date indicate that Site B is the preferred location for the Morrison waste management facility. Describe alternatives assessment.

The site for the TSF was selected because:

- no fish habitat was found;
- creek diversion is simple and any water flow make-up appears easy to implement;
- the site is fare enough from Morrison Lake that there will be little effect, if any on the lake;
- the site appears to be relatively easy to reclaim;
- the scheduled requirement for dam building material can be accommodated by material from the open pit operations and tailings cycloning, eliminating any need for additional rock quarrying;
- the dam heights are low; thus, the aesthetics will be more pleasing to the eye.

The proposed TSF is approximately 3 km north-northeast of the plant site at an elevation of 110-to-200 m above the mill facility. The topography in the area of the proposed TSF is generally flat to gentle and is at the head of the watershed draining into Creek 54300 (00221 BABL system), which ultimately drains to Morrison Lake.

The tailings impoundment will require constructing three embankment dams to an elevation of approximately 1,017 m check to provide storage for tailings and required pond level freeboard for the life of mine. The main dam will be on the south side of the impoundment and span Creek 54300. This dam will reach an ultimate height of approximately 95 m. A dam along the north side of the impoundment (north dam) will be constructed at the drainage divide and reach an ultimate height of 45 m. A third, smaller saddle dam, the West Dam, will be constructed to an ultimate height of 35 m along the northwestern limit of the impoundment to provide the final containment capacity. Seepage cut-off trenches and groundwater recovery wells may be required at each dam.

Initial development of the impoundment for tailings storage will include construction of the 50 m high starter dam for the main dam to provide storage for one to two years of tailings production. From Year 2 onwards it will be necessary to complete construction of the south dam and construct the north dam. A reclaim barge will be positioned on the surface water pond to reclaim water to the mill and remove any excess water if necessary.

A 3.7-km long access road/haul road with a fairly constant 4% grade will be constructed to provide access from the plant site to the TSF, following the tailings supply pipeline route and a separate access road will be constructed along the water reclaim pipeline route.

The glacial till foundation has a low permeability and excess pore water pressures may develop during dam construction loading. Piezometers will be installed in the tailings dams to confirm predicted conditions.

Geologic Conditions

<u>Main Dam Foundation</u>

The foundation soils at the proposed main dam consists of medium dense glacial till overlying bedrock.

The main dam will straddle a broad, flat area for much of its length, with steeper terrain at the right (west) and left (east) abutments. The central flat area is underlain by clayey glacial till varying in thickness from 0 m to 21 m. Glacial till masks the undulating bedrock, in the rolling plateau topography (see Klohn Crippen Berger resistivity line RL-KC07-1A). Bedrock was observed to outcrop at the crest of small slopes separating flat areas. Water-saturated organic silt and peat infill the poorly drained areas between the small steps, and are between 1 m and 2 m thick. The right (west) abutment is a moderately sloping gully, transitioning into a bedrock controlled slope rising to the west. Near the gully, coarser fluvial and glaciofluvial sediments overlie glacial till. Post-glacial fluvial sediments appear to be thin and restricted to the modern channel. The bedrock controlled slope has a decreasing thickness of glacial till with increasing elevation, and accumulations of colluvium (reworked glacial till) were seen in the resistivity profile at the toe of steeper slopes.

<u>North Dam Foundation</u>

The foundation soils at the proposed north dam consists of medium dense glacial till overlying bedrock.

The north dam will straddle a saddle separating southward drainage towards Morrison Lake and the Skeena River, from northward drainage towards Nakinilerak Lake and the Fraser River. Topography near the saddle is subdued with generally gentle slopes. The right (east) abutment has a thin cover of glacial till between 2 and 5 m thick, and the left (west) abutment has a variable thickness of glacial till between 1 and 24 m thick. Glacial till was the only surficial sediment observed besides very thin and localized fluvial and organic sediments. Glacial till is very similar to that observed at the main dam foundation. An exception is clay with no stones observed just above the bedrock contact in Klohn Crippen Berger drill hole DH07-1A.

West Dam Foundation

The foundation soils at the proposed west dam consists of medium dense glacial till overlying bedrock.

No drilling was performed at the proposed west dam, but resistivity data show a very thin cover of surficial material over bedrock, typically less than 2 m thick.

Overburden Types

The overburden deposits at each of the dam sites were classified into three generalized units based on physical and depositional characteristics, such as method of deposition, gradation, and permeability. These soil units are classified and described as water-saturated surface organics; permeable glaciofluvial sand and gravel; and dense impermeable glacial till.

The water-saturated surface organics presented peat on the surface and organic silt at >0.3m deep. The peat is dark brown, moist to wet, fine to coarse fibrous, with some silt. The organic silt is low plasticity, soft, dark brown, wet, massive, low dry strength, rapid dilatancy, and organics are amorphous to fine fibrous. Pockets of organics are in flat, poorly drained areas.

Gravel composed the permeable glaciofluvial sand and gravel classification. It is fine-to-medium grained, fine-to-coarse sandy to some sand, loose, rounded, brownish grey, up to trace cobbles, no fines, moist, and glaciofluvial. Glaciofluvial sediments were not observed within the TSF footprint, but are suspected within the gully of the main dam.

The dense impermeable glacial till classification comprised sandy lean clay, with trace to some gravel, low to intermediate plasticity, soft to stiff, brown, no odour, moist, uncemented, high dry strength, slow dilatancy, and glacial till. Till is uniform and structureless, with rare lenses of gravel and sand, usually mixed with fines. It is widespread.

Tailings Seepage Design

Seepage analysis was carried out using GeoSlope International SEEP/W (2004 version). It is a two-dimensional, finite element numerical model that simulates the movement and pore-water pressure distribution within porous materials such as soil and rock. Two models were constructed and were used to quantify steady-state tailings water seepage rates to the foundation soils through a representative section of the TSF and the main dam.

Seepage collection systems downstream of each dam will include a dam and water return system to recycle seepage and cyclone sand drainage water back to the impoundment.

Tailings Dam Design

The tailings dams have been designed with a central glacial till core to act as a low permeability seepage barrier with cycloned sand shells on the upstream and downstream sides. The dams will be raised by the centreline construction method with downstream slopes of 3H:1V.

The design will involve:

- The starter dam for the main dam will be constructed as a homogeneous fill dam using glacial till borrow material from the TSF interior and from stripping the open pit.
- A drainage system comprising a sand-and-gravel drainage blanket placed under the downstream toe of the dam will be used to intercept and remove any seepage passing through the low permeability zone to ensure the downstream shells remain well drained.
- Constructing a grout curtain, if needed, in the rock foundation expected in the steep valley under the east side of the main dam.
- Graded filters on the downstream sides of the cores to form a filter relationship between the cores and the downstream shells, to protect the cores against internal erosion.
- A drainage collection system in the dams to intercept and remove any seepage passing through the core and ensure that the downstream shells remain fully drained.
- The starter dam for the Main Dam will be a 50 m high homogeneous compacted earthfill dam designed to store one year of tailings. The upstream slope will be 2H:1V and the downstream slope will be 3H:1V to 6H:1V. Seepage and piping control will be with a 1 m thick sand and gravel blanket filter drain placed under the downstream shell of the dam.

The main and north dams will be raised by the centerline cyclone sand method, with compacted cycloned sand placed in the downstream shell of the dam. A vertical low permeability core would extend from the starter dam to the ultimate dam crest.

Klohn anticipated that the foundation till will behave as an undrained material at the start dam construction time, and like a drained material during the mine life and after the mine closure. Instrumentation will be installed to confirm expected conditions and foundation behaviour. The geotechnical properties used for design are summarized in Table 4.12-12.

4.12.4.12 TSF Diversion Ditches and Spillways

Diversion Ditches

To minimize water storage requirements in the TSF it is necessary to maximize the amount of surface water diversion. Accordingly, diversion ditches will be constructed for the Year 4, Year 8, and Year 20 TSF impoundments to divert approximately 9 km^2, 5 km^{2,} and 4.4 km^2, respectively. The ditches are designed to pass the 100-year average snowmelt/runoff flows of

4.4 m^3/s (based on BC Flood Maps). The main diversion ditches, on the east side of the TSF, will typically be 2 m wide and 1.1 m deep. Riprap will be required on slopes steeper than 1%.

Table 4.12-12
Summary of Geotechnical Design Properties

Soil Unit	Bulk Unit Weight (kN/m^3)	Static Drained Material Strength	Static Undrained Strength- Cohesion	Piezometric Condition[1]
Cycloned Sand at Downstream	18	φ'=35˚ C'=0 kpa	-	Piezometric line
Cycloned Sand at Upstream	19	φ'=29˚ C'=0 kpa	-	Piezometric line
Tailing	19	φ'=29˚ C'=0 kpa	-	Piezometric line
Till Core	21	φ'=32˚ C'=0 kpa	C=200 kPa	Piezometric line
Start Dam-Till	21	φ'=32˚ C'=0 kpa	C=200 kPa	Piezometric line
Glacial Till Foundation	21	φ'=32˚ C'=0 kpa	C=200 kPa	Piezometric line with $\overline{B}$ =0.3
Blanket Drainage Sand	18	φ'=35˚ C'=0 kpa	-	Piezometric line
Bedrock		impenetrable	drainage layer	

Seepage Recovery Pond Spillways

Emergency spillways are required for the seepage recovery dams for dam safety protection. The spillway is sized for the 1,000-year return period peak flow of 1.5 m^3/s. The spillway will consist of a 2.0 m wide open channel with 2H:1V side slopes and a 0.5 m water depth plus 0.3 m freeboard. The spillway would be at a 0.5% slope.

TSF Closure Spillway

The TSF closure spillway will be constructed after completion of mining operations. The spillway will be in the east abutment of the main dam and will consist of a channel excavated in bedrock and exiting into an existing drainage channel. The spillway is designed for the maximum probable flood peak flow, assuming attenuation of the flood event within the impoundment. The estimated peak flood flow is 4.5 m^3/s, which will require a spillway 2 m wide and 0.7 m deep

4.12.4.13 Tailings Dam Construction

Tailings will be deposited from spigot off-takes along the crests of the dams to form tailings beaches.

The foundation area will be stripped of all loose and deleterious material. All topsoil will be stripped and stored for use in reclamation. A seepage cutoff trench will be excavated to 3 m deep along the upstream toe of the dam to ensure a positive connection between the dam core and the foundation. The trench will be 10 m wide and will extend a minimum of 2 m into glacial till.

Glacial Till Dam Fill

Glacial till will be borrowed from areas within the impoundment as shown in plan on Drawing D-1101 – Appendix 10. The borrow areas will be stripped and topsoil stockpiled for future use in reclamation. The glacial till is a well-graded silt-sand-gravel mixture, which will be placed in 300 mm thick loose lifts and compacted with a minimum of 6 passes with a 10 tonne vibratory roller. Scarifying between lifts may be required if very smooth surfaces are observed on the lifts surfaces. Coarser graded borrow material will be preferentially placed towards the downstream side of the dam. A minimum fine till section, defined as borrow having >30% passing the 75 micron size fraction, will be placed on the upstream side as shown in section on Drawing D-1103 – Appendix 10.

The borrow pit will be developed to leave a minimum of 2 m of glacial till over the underlying bedrock to control seepage into the bedrock.

Sand and Gravel Borrow

Sand and gravel will be obtained from Klohn's Borrow Area #2 (Appendix 6), approximately 1.5 km northwest of the plant site as shown in plan on Drawing D-1002 – Appendix 10. The sands and gravels typically contain 3% to 5% fines (< 75 micron sieve size) and it may be necessary to wash the material as part of the processing to produce a material with <3% fines. Additional testing will be carried out to confirm the gradation specification and the requirements for wash processing. The sand and gravel borrow will be placed in 500 mm thick lifts and compacted with 6 passes with a 10 tonne vibratory roller.

Cycloned Sand

Sand will be produced for dam construction with hydrocyclones along the crest of the dam. Tailings will be delivered to cyclones on the crest of the dam. The cyclone underflow, at a solids density of 72% by weight, will be discharged with a flexible pipe into construction cells parallel to the dam centerline. Each cell will be infilled with approximately 0.5 thickness of tailings, which will be spread with bulldozers. The bulldozers, and the downward hydraulic gradients, provide sufficient sand compaction. Excess water will be decanted from the cell and directed towards the seepage recovery pond for reclaim to the impoundment.

Borrow for Dam Construction

Another potential borrow area is approximately 650 m southeast of the open pit. Surface area of the borrow site is estimated to be 84,500 m^2 based on interpretation of the local topography.

The depth of sand and gravel was determined from site investigation data collected by Knight Piésold in 2006. KCBL's test pits TP06-41, TP06-42, TP06-43, and TP06-44 show that the sand and gravel deposit extends to a minimum depth of 3 m. Test pit location and stratigraphy is shown in the logs (Appendix 10).

The borrow area is estimated to contain 250,000 m^3 of sand and gravel based on an estimated average depth of 3 m.

Knight Piésold (2006) lab tests indicate that the borrow area material has grain-size distribution as shown in Table 4.12-13.

Table 4.12-13
Sand and Gravel Borrow Area #1 Sample Grain Size Distributions

Hole	Sample id*	Depth (m)	% Gravel	% Sand	% Fines
TP06-41	TP06-41	0.8	6.4	69.8	23.8
TP06-41	TP06-41	2.4	73.6	24.6	1.8
TP06-42	TP06-42	0.9	67.3	28.4	4.3
TP06-42	TP06-42	2.7	58.6	35.7	5.7
TP06-43	TP06-43	1.2	40.2	41.7	18.2
TP06-43	TP06-43	2.4	61.8	21.4	16.8
TP06-44	TP06-40	0.9	33.3	63.4	3.3
TP06-44	TP06-40	2.7	28.5	61.7	9.7

***Note: Test Pit TP06-44 does correspond to Sample ID TP06-40**

Sand and Gravel

KCBL investigated a potential sand-and-gravel borrow area approximately 1,500 m northwest of the open pit as shown in Drawing D-1003 (Appendix 10), which is referred to as Borrow Area #2. Surface area of the borrow site is estimated to be 180,000 m^2 based on interpretation of the local topography.

The depth of sand and gravel was determined from site investigation data collected by Klohn during the 2007 site investigation program (Appendix 6). Test pits TP07-02, TP07-03, and TP07-04 show that the sand and gravel deposit extends to a minimum depth of 6 m. Test pit stratigraphy is shown in the attached logs and test pit locations are shown on Drawing D-1003 (Appendix 6).

The borrow area is estimated to contain 1 M m^3 of sand and gravel based on an estimated average depth of 6 m.

Klohn's lab tests indicate that the borrow area material has a grain-size distribution as outlined in Table 4.12-14

Table 4.12-14
Sand and Gravel Borrow Area #2 Sample Grain Size Distributions

Test Pit	Depth (m)	%Gravel	%Sand	%Fines
TP07-2	3	46.8	46.0	7.2
TP07-2	5.8	22.2	72.7	5.1
TP07-3	3	60.9	36.4	2.7
TP07-4	3	54.0	43.3	2.7

The grain-size distribution curves are also shown in Figure 4.12-4.

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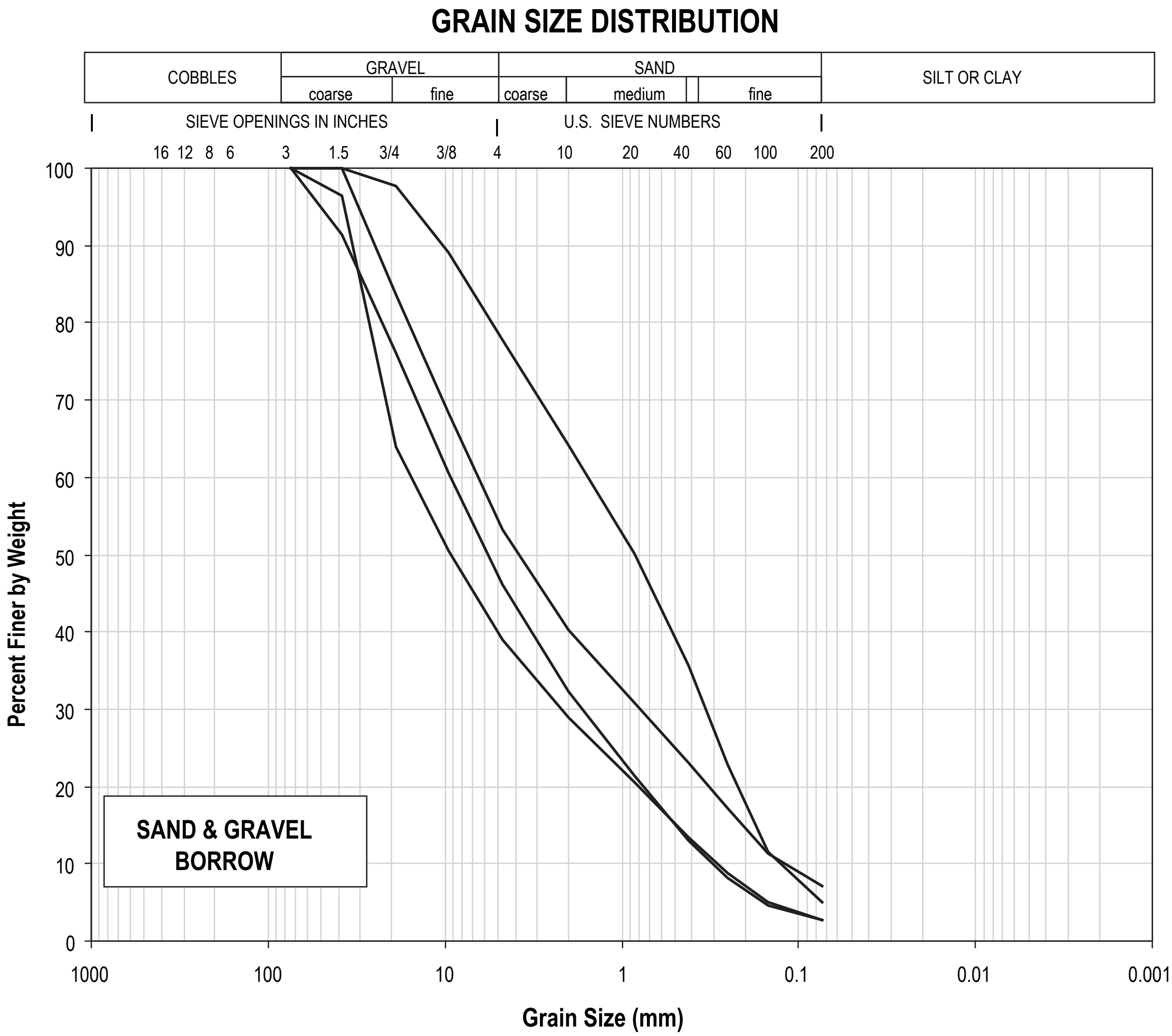

Morrison Copper/Gold Project
Sand and Gravel Borrow Area #2
Sample Grain Size Distribution Curves

FIGURE 4.12-4

These volume estimates are based on limited information and represents Klohn's best estimate with the information available at the time. A more extensive test pitting program was recommended to confirm the available volume.

The tailings dam will be constructed using locally available glacial till and sand and gravels. These deposits were derived from glacial activity and are generally from longer distances and a wide variety of soil and rock types, as opposed to mass wasting or colluvial erosional processes from the open pit mine area. Consequently, it is highly unlikely that the borrow materials will be PAG. Nonetheless, a program of ABA testing for all borrow materials will be carried out as part of the QA/QC program for dam fills. Material that is PAG will not be used for construction.

Stability Analysis

For Project design purposes, an analysis of regional seismicity and earthquake potential based on data generated by the Pacific Geosciences Centre was completed. Seismic data was incorporated into designs for the tailings impoundment dams and other structures

Stability analysis was carried out using GeoSlope International SLOPE/W (2004 version). In the analysis the following assumptions were made.

- The total pore pressure in foundation till was assumed to be $U=U_0+ (\overline{B}*P)$, U_0 is hydrostatic water pressure due to piezometric line, $\overline{B}$ is the ratio of remaining excess pore pressure to the total stress applied to the foundation, and P is total stress applied on the existing grade. Foundation till was regarded as drained material at operations and at closure. Immediately after a load was applied onto foundation, the load was supported by water in the void of the foundation soil. Because pore water had no way to drain out of the void in a short time, pore pressure had to increase to a higher level than hydrostatic water pressure, the pore pressure higher than the hydrostatic water pressure was defined as excess pore pressure. The excess pore pressure would decrease as water gradually drained out of the void, and the dissipated pore pressure would be transferred to the soil frame, and therefore the effective stress in soil and soil strength would increase. The ratio of remaining excess pore pressure to the total stress applied to the foundation was designated as $\overline{B}$.
- KCBL assumed that $\overline{B}=0.3$ at operations and $\overline{B}=0$ at closure for all ultimate dams. The $\overline{B}$ value was estimated by 1-D consolidation model using Cv=3.1 cm^2/S and 30 m thick of till layer with double drainage layer. The loading sequence was estimated using the dam crest vs. operation time. The calculated maximum $\overline{B}$ value was 0.3 in 21 years of operation and it will decrease to less than 0.07 after closure 10 years.
- For the starter dam foundation, till was assumed to be undrained material with cohesion C=200 kPa, considering high stress changes in a short time and the permeable nature of foundation till. Blow counts from SPTs of DH07-5B at the main dam location indicated that SPT N value was in the range of 25 to 80 with an average over 30, laboratory tests indicated that the foundation till was of medium plasticity. Using the correlation proposed by Terzaghi and Peck (1996), the unconfined compressive strength Uc was 400 kPa and the cohesion of the foundation till was estimated to be 200 kPa.

- The pizeometeric line was assumed to be 4 m below the dam crest and was horizontal from tailing pond to till core; the piezometric line was assumed to decline concavely toward the blanket drainge layer, the piezometric line was applied to all soil units.
- Seismic load induced by 0.065g horizontal acceleration was considered in pseudostatic analyses, and in yielding acceleration calculation. Upper bound seismic displacements were assessed using Hynes and Franklin (1984) relationships.

Start dam analyses using undrained shear strength material model for till foundation were summarized in Table 4.12-15. The critical slip faces are presented in Appendix 9**.** The analyses indicated that the major factor controlling stability of the main starter dam is the undrained shear strength of the glacial till foundation. A ratio of 2H:1V upstream slope for the start dam had an FOS >1.3, but 2.5H: 1V downstream slope could not satisfy minimum design requirement for FOS>1.3 if till cohesion was 200 kPa. Therefore, Klohn recommended a 3H:1V slope for downstream slope.

The analyses using drained material strength for till were summarized in Table 4.12-16. The main factors controlling stability of the ultimate dams were the assumed piezometric line and the excessive pore pressure ratio $\overline{B}$ value of the glacial till foundation.

Table 4.12-15
Summary of Stability Analyses Results for Start Dam Using Undrained Shear Strength of Foundation Till

Dam Location Section	Crest El.(m)	Piezometric Condition	Static Undrained Strength C (kPa)	FOS FOS ≥ 1.3
Main Dam Section D-D' Starter Dam 2.0H:1V upstream Without Water[1]	Starter Dam Crest El.=968.5	1 Piezoline applied to all soil unit	200	1.56
Main Dam Section D-D' Starter Dam 2.5H:1V downstream	Starter Dam Crest El.=968.5	1 Piezoline applied to all soil unit	200 250	1.15 1.44
Main Dam Section D-D' Starter Dam 3H:1V downstream	Starter Dam Crest El.=968.5	1 Piezoline applied to all soil unit	200	1.31

Note 1: With water or tailing presents at the upstream, the upstream will have higher FOS

More accurate piezometric line form seepage analyses can be used in the stability analyses at the next design stage. The $\overline{B}$ value was dictated by consolidation coefficient Cv, thickness of the till layer and available time for excessive pore pressure dissipation. Consolidation coefficient Cv was estimated using the hydraulic conductivity tests, which can be updated at the next design stage when the results of laboratory consolidation tests are available.

Table 4.12-16
Summary of Stability Analyses Results

Dam	Section	Slope	Stage	FOS – Operations ($\overline{B}$ =0.3) Static FOS ≥ 1.3	FOS – Operations ($\overline{B}$ =0.3) Pseudostatic seismic load = 0.065 FOS ≥ 1.1 & D<1.0 m	FOS – Closure ($\overline{B}$ =0) Static FOS ≥ 1.3	FOS – Closure ($\overline{B}$ =0) Pseudostatic seismic load = 0.065 FOS ≥ 1.1 & D<1.0 m
Main Dam	C-C'	3H:1V Downstream	Ultimate	1.46	FOS=1.124 D=0.64 m	1.80	FOS=1.45 D=0.14 m
	D-D'	2.5H:1V Downstream	Starter	1.37	N/A	N/A	N/A
	D-D'	3H:1V Downstream	Ultimate	1.34	FOS=1.1 D=0.9 m	1.80	FOS=1.42 D=0.13 m
North Dam	E-E'	3H:1V Downstream	Ultimate	1.51	FOS=1.1 D=0.44 m	1.85	FOS=1.48 D=0.15 m
Northwest Dam	G-G'	2.5H:1V Downstream	Ultimate	1.42	FOS=1.17 D=0.46 m	1.60	FOS=1.28 D=0.26 m

Note
1: Section D-D’ Starter Dam and Section G-G’ upstream 2H: 1V slope, other sections have a vertical upstream with ~1% tailing beach slope.
2: The upstream slopes have higher FOS than the corresponding downstream slopes as specified.

Seepage Analysis

Seepage analysis was carried out using GeoSlope International SEEP/W (2004 version). It is a two-dimensional, finite element numerical model that simulates the movement and pore-water pressure distribution within porous materials such as soil and rock. Two models were constructed and were used to quantify steady-state tailings water seepage rates to the foundation soils through a representative section of the TSF and the main dam.

Seepage collection dams will comprise a lined polishing pond to receive any excess water together with flows from the dam drainage and seepage systems for final cleaning, if necessary, and monitoring before release. There will also be a discharge system from the polishing pond into the bottom of Morrison Lake terminating with an outlet diffuser.

4.12.4.14 Collecting Contact Water

All contact water, from the WRDs, open pit, low grade ore stockpile, and plant site areas, will be collected and recycled to the plant site. Runoff collection sumps will be along the toe of the waste dumps as shown on Drawing D-1201. Sumps, which are uphill of the open pit area, have been sized to rout the average spring freshet runoff of 610 m^3/hr and will require a minimum sump capacity of 100 m^3. Although the pumping capacity could be reduced, to a minimum of 150 m^3/hr, it would result in more water reporting to the open pit during wet periods.

Sumps, in which an overflow could lead to a release to the environment, have been sized for the 1:10 year flow of 680 m^3/hr and will require a minimum sump size of 670 m^3.