BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

VICTORIA GOLD CORPORATION

EAGLE GOLD PROJECT
DUBLIN GULCH, YUKON

2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

FINAL

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January 31, 2012
Project No: 0792-006

Mike Padula, Project Manager
Victoria Gold Corporation
584 – Bentall #4
1055 Dunsmuir Street, PO Box 49215
Vancouver, BC, V7X 1K8

Dear Mr. Padula,

Re: Eagle Gold Project: 2011 Geotechnical Investigation For Mine Site Infrastructure Foundation Report - Final

Please find attached the final version of the aforementioned report. Should you have any questions or comments, please do not hesitate to contact the undersigned.

Yours sincerely,

BGC ENGINEERING INC.
per:

Pete Quinn, Ph.D., P.Eng.
Senior Geotechnical Engineer

Att.

PQ

EXECUTIVE SUMMARY

Introduction

Victoria Gold Corporation (Victoria), with assistance from Wardrop a Tetra Tech Company (Wardrop) is completing a feasibility study (FS) for development of the proposed Eagle Gold mine at Dublin Gulch, Yukon. BGC Engineering Inc. (BGC) was contracted by Victoria to complete geotechnical investigation work in support of FS design for mine site infrastructure. This report presents geotechnical engineering recommendations for selected mine site infrastructure resulting from the 2011 site investigation program performed between June and August 2011. The results of the site investigation have been published in a data report under separate cover.

The Eagle Gold property is located approximately 40 km north of Mayo, and 15 km northwest of Elsa (Drawing 01). The mine will comprise an open pit and heap leach pad; haul roads; waste rock storage areas; process plant; crushers and conveyors; truck shop; camp; water diversion structure; process water ponds; drainage ditches; sediment control structures and various other ancillary facilities. The current layout for the proposed mine facilities was received from Wardrop on November 23, 2011 (Drawing 02).

In the summer of 2011, BGC completed field investigations in support of geotechnical recommendations for mine site infrastructure. That work involved the excavation of ninety-six test pits, advancement of forty-six drillholes (29 Diamond holes and 17 Auger holes), and mapping of fifty-nine outcrops (natural exposures, existing road cuts and drill pads cuts) to characterize subsurface conditions relevant for foundation and earthworks design. Samples were taken from selected test pits and drillholes for index testing of soil and rock. Bulk samples of rock and placer tailings were also analyzed for a range of parameters related to the potential for re-use as select fill or aggregate. Downhole and surface geophysical investigations were completed, and plate load tests were conducted at selected locations of proposed building and machine foundations.

Several engineering reports were issued in draft by BGC in early 2011, with preliminary foundation and earthworks recommendations for a number of key facilities based on site investigation data from 2010, and in relation to the layout available at that time. Those reports are superseded by the recommendations contained herein.

This report provides geotechnical engineering recommendations for selected mine site infrastructure. This report does not provide recommendations for the open pit, the waste rock storage areas (WRSAs) or the heap leach facility (HLF), with the exception of cut slope recommendations for the Dublin Gulch diversion. Recommendations for the open pit and WRSAs will be provided by BGC under separate cover. Geotechnical design of the HLF, including the heap embankment, heap leach pad, Dublin Gulch diversion and events ponds, is the responsibility of Tetra Tech.

The report is organized as follows:

- Section 1.0 Introduction – general introductory material;
- Section 2.0 Proposed Facilities – general description of facilities under consideration in this report, along with design criteria used in the geotechnical analysis;
- Section 3.0 Site Conditions – a high level summary of generalized site conditions to provide basic context;
- Section 4.0 Material Properties – this section summarizes the assumed engineering properties of the in-situ foundation materials and processed engineering materials expected to be encountered or used in site development and earthworks construction;
- Section 5.0 Foundation Recommendations – this section provides recommendations of primary interest to the Structural designers, and includes recommendations for foundations and retaining walls;
- Section 6.0 Earthworks Recommendations – this section provides recommendations of primary interest to the civil designers, and includes recommendations for bulk earthworks, including cutting and filling to provide design grades for building pads, roads and other required surfaces;
- Section 7.0 Construction Materials – this section includes descriptions of different material types for use in earthworks construction, with discussion of quantities and schedule of required engineering materials, and quality and quantity of different material sources; and
- Section 8.0 Recommendations for Further Investigation – this section highlights areas of uncertainty where additional data will be required to support further development of geotechnical design, and presents recommendations for further work.
- Section 9.0 Closure.

Proposed Facilities

The proposed layout provided by Wardrop includes a number of buildings; including those containing the crushers, and those at the process plant site, truck shop and explosives storage areas (Drawing 02). Anticipated foundation dimensions and loads, and tolerable foundation deformations were provided by Wardrop on November 18, 2011.

The crushers will include three separate facilities – the primary, secondary and tertiary crushers – connected by conveyors. These facilities will include heavy vibratory machinery (i.e. the crushers) and associated machine foundations, in addition to the building foundations. The primary crusher will be accessed at the top by trucks from the pit, and will be founded some 25 to 30 m lower. Thus the primary crusher building will also function as a large retaining wall.

The line of crushers will be built on steeply sloping terrain, thus requiring cutting and filling to allow development of building pads. The cut above the lower platform below the primary crusher is shown on the general arrangement as approximately 90-95 m high at its highest point. This cut is shown as being lower above the secondary and tertiary crushers.

A line of conveyors will connect the tertiary crusher with the heap leach facility in the valley bottom below. The conveyors will be supported on sleepers where appropriate, and on elevated bents where necessary.

The process plant facilities will be developed on a cut/fill pad constructed on the hillside below Tin Dome, above and to the north of the Dublin Gulch valley bottom. The truck shop will be developed on a cut/fill pad constructed on the hillside below and to the west of the planned open pit, above and to the east of Haggart Creek.

A number of significant earth and rock cuts will be required for development of roads and building pads on the sloping ground at the project site, including the following:

- Main cut above primary crusher, currently shown in Wardrop grading information to be about 90-95 m in height at 1.75H:1V;
- Main cut at plant site building pad, currently shown by Wardrop to be 31 m in height at 2H:1V;
- Cuts along the Dublin Gulch diversion channel, currently shown by Wardrop to be up to about 26 m in height at 1.75H:1V to 2H:1V;
- Main cut at truck shop building pad, currently shown by Wardrop to be 20 m in height at 2H:1V;
- Main cut at upper edge of 100 day storage pad, currently shown by Wardrop to be 36 m in height at 1.75H:1V;
- Numerous other cuts of up to 15-20 m in height.

Foundation Recommendations

Recommendations have been provided for building foundations allowing for a minimum factor of safety of 3 against bearing capacity failure, and minimizing settlements within the objectives specified by Wardrop. Summary recommendations are:

Recommended Allowable Bearing Pressures for Ancillary Facilities

Bearing Stratum	Allowable Bearing Capacity (kPa)	
	Up to 2 m x 2 m Pad Footing	Up to 2 m x 20 m Strip Footing
Structural Fill[1]	250	150
Highly to Completely Weathered Rock	250	150
Type 3 Rock	500	300
Type 2 Rock	1000	600
Type 1 Rock	1500	1000

Notes:

1. Footings founded on structural fill require a minimum of 1.5 m of embedment (depth of bottom of footing below surrounding grade) to obtain the indicated allowable bearing capacity. Separate consideration of frost protection may be necessary.

Recommended Allowable Bearing Pressures for Specific Facilities

Facility	Expected Pad Elevation (m ASL)	Foundation Dimensions[1]	Expected Subgrade Conditions	Allowable Bearing Pressure[2] (kPa)
Primary Crusher	1026/1050[3]	Up to 12 m x 18 m mat	Type 1 rock	1000
Secondary Crusher[4]	1032	Up to 16 m x 16 m mat, 12 m x 5 m spread footing	Type 2 rock[4]	400
Tertiary Crusher[4]	1014	Up to 14 m x 9 m mat	Type 2 rock[4]	400
Conveyors from Tertiary Crusher to Heap Leach Facility	Varies along conveyor	Bents on 1.5 m x 6 m footing	Type 3 rock at ~ 5 m to 20 m depth below grade, typically 10 m expected	200 mm concrete-filled steel pipe piles socketed 2 m into Type 3 rock at ~ 10 m depth below grade can support 700 kN
		Sleepers at grade, on timber cribbing, where possible	Colluvium below stripped topsoil	N/A – adjustable foundations
Plant Site	860	3.5 m x 12 m	Highly to Completely weathered rock or structural fill	200
Truck Shop	855	3 m x 8 m	Type 3 rock	300

Notes:

1. Provided by Wardrop on 18 Nov 2011.
2. Based on factor of safety of 3 against bearing capacity failure and limiting settlements to those specified by Wardrop on 18 Nov 2011.
3. The lower portion of the primary crusher is at 1026 m. The elevation of the top of the primary crusher, where trucks will deposit ore is at 1050 m.
4. Crushers cannot be supported on regular structural fill. If the secondary and tertiary crushers must be built at planned grades well above the suitable bearing stratum, the gap between the bearing stratum and foundation grades can be made up by lean concrete or some other form of stiff fill material.

Earthworks Recommendations

There are a number of ground-related challenges to construction of earthworks and buildings at the proposed mine site. These include, generally:

- Presence of discontinuous permafrost, including some areas with excess ground ice;
- Relatively short “traditional” (i.e. spring/summer/fall) construction season, with specific challenges and limitations during other parts of the year (e.g. poor trafficability and material workability on hillsides before mid-summer; and long, harsh winter);
- Uncertain quality and quantity of required borrow materials;
- Presence of significant quantities of existing random fill (placer tailings); and
- Presence of steep slopes and geological hazards.

Each of these specific challenges requires consideration in the planning, design and construction of mine site infrastructure, as discussed in the report.

Engineered slopes constructed of structural fill or rock fill may be made at 2H:1V or flatter. Buildings should be set back a minimum of 10 m from the crest of fill slopes.

Where a structural fill is to be constructed on an existing natural slope, the fill should be keyed into the natural slope by excavating steps into the slope at the edge of successive lifts of structural fill.

Selected high fills, including those below the pit-crushers haul road and at the lower (north) end of the 100 day storage pad, may encroach into seasonal drainage areas or depressions with shallow groundwater. Particular care should be taken in these potentially wet areas to choose free draining, coarse granular fill materials, preferably angular durable rockfill, to prevent buildup of excess pore pressures in the fills.

Recommended slope geometry for cut slopes follows:

Recommended Permanent Cut Slope Angles – Area Specific

Area	Overburden		Slope below Overburden		Notes
	Thick-ness (m)	Steepest Cut Angle	Material	Steepest Cut Angle [1]	
Primary Crusher	2 - 4	2.5H:1V	Type 1, 2, 3 Rock	1.75H:1V	Design FS = 1.5; maximum slope height ~107 m; slope angle controlled by dip of foliation at about 30-32 degrees; benched slope design recommended; 8 m maximum bench height; 13 m minimum bench width; 0.25H:1V bench face angle.
100 Day Storage	3 - 4	2.5H:1V	Type 2, 3 rock	1.75H:1V	Design FS = 1.5[2]; slope angle controlled by dip of foliation at about 30-32 degrees; minimum distance of 80-100 m required between slope crest and toe of haul road / crusher platform fill slopes. Benched slope design is recommended as detailed above for primary crusher.
Truck Shop	5 - 8	2.5H:1V	Type 3 rock	1.75H:1V	Design FS = 1.5; maximum slope height = ~22 m; slope angle controlled by dip of foliation. Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Plant Site	3 - 7	2.5H:1V	Highly to completely weathered rock	2H:1V	Design FS = 1.5; maximum slope height ~35 m; Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Dublin Gulch Diversion	2 - 5	2.5H:1V	Till	2H:1V[3]	Design FS = 1.5; maximum slope height ~28 m; maximum cut angle assumes that the cut slope is dry.

Notes:

1. Maximum overall slope angle in the slope materials below the overburden depth. Overall slope angle defined by the line that connects the toe of the slope with the slope crest at the rock-overburden contact.
2. Recommended FS for the 100 day storage cut is 1.5 due to proximity to crushers and potential to undermine them in case of failure. FS = 1.3 could be considered when the cut is moved 80-100 m further from the crushers, however, the overall slope angle will still be controlled by the dip of the foliation and cannot be steepened significantly.
3. Assumed groundwater level is greater than 6 m below existing ground surface, which is inferred but not confirmed and requires further investigation.

At the primary crusher, 100-day storage, and truck shop areas, the cut slope design is controlled by the potential for failure of the rock along discontinuities defined by foliation in the metasedimentary rock. The foliation is expected to dip out of the slope at angles ranging from about 20° to 40°. The potential failure wedge that could form on slopes of this size is large enough to make mechanical support of the slopes impractical. Therefore a relatively shallow overall slope angle has been recommended. This overall slope angle is approximately parallel to the observed dip of the foliation, which essentially eliminates the potential for a planar failure at the slope-scale.

Bench scale failures are expected, including minor raveling and slumping, where the foliation is undercut; however, failures occurring on upper bench faces are not expected to adversely affect the infrastructure at the base of the slope due to the presence of the 13 m wide rockfall catchment benches. However, an allowance should be made in the design for spot bolting of loose blocks of rock on the bench faces in case specific weak structures are encountered. Mesh may also be required to contain poor quality rock that could ravel, should it be encountered, particularly on the bottom bench where service vehicles may be entering. Additionally, an 8 m wide rockfall catchment area should be included in the design at the upper and lower platform elevations. A 1 m high barrier (concrete or earth, or permanent fence) is recommended to be placed at the outer edge of the rockfall catchment area to deter encroachment into the catchment area by vehicles or personnel.

At the primary crusher, it is expected that blasting will be required to excavate the rock; therefore a benched slope design has been recommended. The recommended bench face angle is 0.25H:1V, which has been selected to facilitate controlled blasting. The maximum recommended bench height is 8 m. The minimum recommended bench width is 13 m to facilitate installation of a safety berm and to allow access for bench clean up. The bench width may need to be adjusted at detailed design to maintain the recommended overall slope angle of 1.75H:1V.

The recommendations provided for the primary crusher cut are based on assumed water levels and ground conditions, which are based on relatively sparse site characterization data. The consequences of a slope-scale failure at the primary crusher cut are perceived to be very high. Additional site investigations are recommended to reduce the current level of uncertainty in the understanding of ground conditions. The recommendations provided in this report assume that the design is controlled by the foliation of the metasedimentary rock. Future site investigation should verify that additional unfavorable conditions are not present and should be designed to characterize the orientation and condition of the contact between the meta-sedimentary and igneous rock, which is expected to daylight near the base of the cut.

At the 100-day storage area, the crest of the cut slope may daylight near the toe of the fill slope from the haul road and crusher platform. A minimum distance of 80-100 m between the cut slope crest and toe of fill is recommended to reduce the possibility of a slope failure at the 100-day storage area which could affect the crusher or haul road.

The recommended cut angle at the Dublin Gulch diversion assumes that the slope materials are unsaturated. If the slope materials are saturated, the recommend cut angle would decrease to 2.5H:1V. Current information regarding the depth to groundwater along the diversion is sparse. Future site investigation programs should be designed to characterize the groundwater depth along the diversion, and update the cut slope design, if appropriate.

The following Table provides general cut slope angle recommendations based on material type, for general application across the site for cut slopes less than 10 m high. It is assumed these cuts will be unsaturated and without adverse geologic structure. Cut slopes that do not meet these conditions should be reviewed on a case-by-case basis by the geotechnical engineer.

A rockfall catchment area should be provided at the base of all cut slopes. For soil slopes, the catchment area should be sloped back toward the cut slope at an angle of 4H:1V. The recommended minimum width of the rockfall catchment is 2.5 m below soil cuts, and 8 m below rock cuts.

Recommended Permanent Cut Slope Angles – General

Slope Material	Maximum Cut Slope Angle[1]	Maximum Cut slope Height	Notes
Colluvium	2.5H:1V	10 m	
Till	2H:1V	10 m	
Highly to completely weathered rock (excavatable)	2H:1V	10 m	
Type 3 rock (generally excavatable)	1.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 2 rock (generally rippable)	1H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 1 rock (may require blasting)	0.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered

Note:

1. Maximum cut slope angles assume the slope is < 10 m high, unsaturated, and without adverse geologic structure.

Construction and Waste Materials

Material take offs (MTOs) with earthworks quantity estimates were provided by Merit Consultants International on January 6, 2012. The MTOs include numerous line items for quantities of earth or rock materials and various types of granular borrow required for construction of the mine site facilities, including the following approximate quantities of specific materials:

- Approximately 2.9 million m^3 of engineered fill, which includes approximately 2.1 million m^3 of engineered fill for the heap containment dyke and diversion embankment, selected from a variety of sources, including processed placer tailings, non-durable rock obtained during bulk earthworks activities, and possibly durable waste rock from mining. This engineered fill includes the following general categories of materials:
  - General fill,
  - Structural fill,
  - Durable rock fill, and
  - Non-durable rock fill;
- 298,000 m^3 of crushed durable rock to produce a well-graded material for the heap overliner;
- Various minor quantities of miscellaneous engineering materials, including silt/fines for liner construction, transition/filter materials, drainage materials, rip rap, concrete aggregate, and road pavement structure materials.

The report includes suggested specifications for various materials to be used in earthworks construction.

This project will involve the movement of large quantities of earth and rock fill in a relatively short construction period (currently understood to be about three years) and within a limited footprint in rugged terrain. It will be challenging to manage material movement to meet construction schedule requirements. An effort has been made to understand the temporal nature of planned material movement, with consideration of MTOs provided by Wardrop, Tetra Tech, and Knight Piésold, as compiled by Merit Consultants and received by BGC on January 06, 2012.

The report presents a breakdown of material quantities over time, based on an analysis of quarterly supply and demand, as listed in the following table. Cut quantities are shown as positive numbers, being quantities available for use (or intended for disposal). Fill quantities are shown in brackets to represent negative numbers, being deficit quantities required for construction.

Quarterly Demand for Cut and Fill Quantities, as inferred from MTOs from Merit

Used for Material Balance	Category	Material Quantity (m^3)												
		Q4 2012	Q1 2013	Q2 2013	Q3 2013	Q4 2013	Q1 2014	Q2 2014	Q3 2014	Q4 2014	Q1 2015	Q2 2015	Q3 2015	Total
No	Strip and stockpile topsoil	50,738	26,026	147,437	0	0	0	37,015	7,701	0	0	44,485	0	**313,402**
	Excavate and dispose waste rock in waste dump	77,319	0	261,201	0	0	0	54,344	35,234	0	0	67,842	0	**495,940**
	Excavate colluvium	35,050	0	168,300	0	0	0	18,000	5,700	0	0	0	0	**227,050**
	Excavate rock	10,758	0	375,555	0	0	0	4,278	0	0	0	0	271,369	**661,960**
	Excavate permafrost	3,500	0	34,900	0	0	0	0	1,200	0	0	0	0	**39,600**
	Local cut and fill	76,791	0	239,749	0	0	0	45,210	13,251	0	0	0	839,463	**1,214,464**
Yes	Excavate and stockpile suitable materials	208,271	0	185,885	0	0	0	0	24,632	0	0	0	133,699	**552,487**
	General excavation	333,280	0	1,182,390	0	0	0	75,400	120,000	0	0	0	0	**1,711,070**
	Excavate placer tailings	0	0	876,000	0	0	0	0	0	0	0	0	0	**876,000**
	Subgrade preparation	0	0	(18,300)	(104,600)	0	0	0	(3,500)	0	0	0	0	**(126,400)**
	Other materials	(3,520)	0	(58,823)	(3,100)	0	0	(12,000)	(298,000)	0	0	0	0	**(375,443)**
	Fill from stockpile	(18,110)	0	(355,643)	0	0	0	(149,191)	(17,461)	0	0	0	(7,430)	**(547,835)**
	Fill	(70)	0	(126,518)	(1,119,000)	0	0	(743,000)	0	0	0	0	0	**(1,988,588)**
Material balance - each quarter		519,851	0	1,684,991	(1,226,700)	0	0	(828,791)	(174,329)	0	0	0	126,269	**101,291**
Material balance - cumulative		519,851	519,851	2,204,842	978,142	978,142	978,142	149,351	(24,978)	(24,978)	(24,978)	(24,978)	101,291	**101,291**

Note:

1. Quantities (in brackets) indicate deficit quantities, or fill to be derived from elsewhere. The material categories have been modified slightly from those received in information provided by Merit.

Summary of Borrow Material Availability

Borrow Source	Material Types	Estimated Volumes (in situ volumes, except where noted)	Comments
Pit Pre-Strip	Durable rock fill Non-durable rock fill Concrete aggregate Heap overliner Rip rap	Very large. Available volumes depend on the sequence of mining activities, although materials can be developed prior to mining activities by developing a quarry prior to pre-strip.	Source consists of weathered granodiorite and weathered silicified metasedimentary rock, typically quartzite. Suitable concrete aggregate has not yet been identified, and requires further study. Testing of material for use as heap overliner was commissioned by Tetra Tech, and the results are not available to BGC at the time of writing. Availability of rip rap in desired block size of 500-600 mm will require further input from mine plan, and careful selection. Most near surface weathered rock suggests excavated block size of approximately 100-300 mm.
Ann Gulch Central Knob	Non-durable rock fill	Up to approximately 900,000 m^3, subject to further input from Tetra Tech.	Grading plans showing the volumes of anticipated rock excavation are not available to BGC at the time of writing.
Steiner Zone	Same as for Pit Pre-strip	Up to approximately 200,000 m^3, assuming quarry depth of 5 m	Very little information is known about this area. Further subsurface investigation is required to confirm quality and quantity of available materials.
Dublin Gulch Placer Tailings	General Fill Structural Fill Concrete aggregate Heap overliner Rip rap	Approximately 2.0 million m^3, of which about 1.1 million m^3 is above the groundwater level	Materials are highly variable, and will require processing through screening, crushing and/or washing to develop the required material specifications. Oversized materials (> 75 mm) screened from the tailings may be suitable for use, after crushing, as heap overliner or concrete aggregate pending further analysis. Some rip rap can be developed from the screened oversize material; however, the quantity of 500-600 mm particles is expected to be small and would require careful selection.

Borrow Source	Material Types	Estimated Volumes (in situ volumes, except where noted)	Comments
Haggart Creek Placer Tailings	General Fill Structural Fill	Approximately 750,000 m^3 available above the elevation of Haggart Creek	No subsurface information is available to support the quantity estimate. Available volume of suitable material is estimated from visual classification of surficial materials present in several distinct piles.
Silt Borrow	Silt liner	Approximately 220,000 m^3	Available silt materials are frozen and potentially ice-rich, and will require thawing and drying prior to use.

The current analysis shows a peak excess of approximately 2.2 million cubic metres of excavated material which will require storage during the first year of the project. This excess supply will be drawn down over the following year, leaving a small excess of available fill at the end of construction.

The material categories listed in the previous table have been modified slightly from categories provided in the MTOs from Merit Consultants on January 6, 2012.

Bulk earthworks activities will generate several types of material that are unsuitable for immediate use, or may not be suitable for any use, thus necessitating temporary storage or permanent disposal. Decisions on ultimate disposition may require further consideration of the need for soil for reclamation. Preliminary information suggests the development of the following materials requiring storage or disposal:

- Topsoil – these materials will be required for reclamation. It will be necessary to develop stockpiles to store these materials during construction and mine operation. The current estimate of 313,000 m^3 does not yet include open pit pre-stripping;
- Ice-rich permafrost – these materials will be unsuitable for immediate re-use in any application. They may be suitable for re-use in reclamation after thawing and draining of excess water. These materials will require careful storage after excavation and prior to use, as they will be weak and unstable when thawed. It may be necessary to develop specific storage areas with containment structures and water management infrastructure. Current estimates indicate approximately 40,000 m^3 of ice-rich permafrost will be removed during development of the heap leach facility, and with additional volumes from other areas on site (quantity currently unknown), all requiring management during construction and mining operations;
- Colluvium – some of the shallow colluvial soils removed during bulk excavation work will contain excessive amounts of deleterious materials, such as organic inclusions or excess proportions of fines. Current estimates suggest approximately 227,000 m^3 of colluvium requiring permanent disposal or storage for re-use in reclamation.
- Waste rock - these materials are indicated by Merit and Wardrop as unsuitable for re-use as construction fills and are intended to be permanently disposed in designated disposal areas. In general they correspond to soils or rock with deleterious materials and may include excess fines or excess ice. Current estimates indicate approximately 500,000 m^3 of unsuitable material that needs to be excavated, removed and disposed, either in the waste rock storage areas, or other disposal areas to be determined.

Work was done to explore potential borrow sources, including effort to determine the characteristics of the placer tailings; investigation of potential silt borrow near the proposed laydown area, evaluation of various rock sources for use as engineered fill; and, evaluation of placer tailings and rock near the proposed open pit for potential use as concrete aggregate. Summary information for various borrow sources is presented in the Table "Summary of Borrow Material Availability."

Recommendations for Further Work

This report has provided feasibility study level geotechnical recommendations for mine site infrastructure. There are several areas of uncertainty that should be further explored as part of detailed design. The following list provides recommendations for further investigation.

- Diamond drillholes:
  - Vertical holes at all three crushers to better establish depth to suitable bearing stratum across the facilities' footprints;
  - Inclined holes in the area of proposed rock cuts at the crushers and 100 day storage pad;
  - Vertical holes at the plant site to better determine depth to suitable bearing stratum within the extent of the building pad;
  - Allowance for additional holes within the footprint of the heap leach facility, in the event Tetra Tech consider additional data warranted;
  - Allowance for additional holes at major cuts such as that along the phase 1 heap access road;
  - Allowance for holes for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Auger holes (with CRREL barrel available):
  - Conveyor bent foundation locations between tertiary crusher and heap leach facility;
  - Along the alignment of the proposed Dublin Gulch diversion channel;
  - In Eagle Pup to confirm the extent of the ice-rich lobate feature in the valley bottom;
  - At the revised truck shop buildings and cut locations;
  - Allowance for holes for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Plate load tests at plant site and all three crushers;
- Design and construction of a test fill embankment to determine whether high quality structural fill would be suitable to support the secondary and tertiary crushers;
- Sampling and strength testing of materials selected for heap embankment fill, if considered necessary by Tetra Tech;
- Additional sampling and testing of granodiorite from the pit area and Steiner zone for possible use as concrete aggregate. Obtain materials engineering advice to guide this process, and including trial mix designs possibly with additives to make use of local aggregates, and trial design mix for lean concrete for use in raising grades at crushers;
- Sample mixes for low strength concrete as stabilized fill

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LIMITATIONS

BGC Engineering Inc. (BGC) prepared this document for the account of Victoria Gold Corporation (Victoria). The material in it reflects the judgment of BGC staff in light of the information available to BGC at the time of document preparation. Any use which a third party makes of this document or any reliance on decisions to be based on it is the responsibility of such third parties. BGC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this document.

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1.0 INTRODUCTION

1.1. General

Victoria Gold Corporation (Victoria), with assistance from Wardrop a Tetra Tech Company (Wardrop) is completing a feasibility study (FS) for development of the proposed Eagle Gold mine at Dublin Gulch, Yukon. BGC Engineering Inc. (BGC) was contracted by Victoria to complete geotechnical investigations for mine site infrastructure. This report presents geotechnical engineering recommendations for selected mine site infrastructure resulting from geotechnical investigations performed between June and August 2011.

1.2. Project description

The Eagle Gold property is located approximately 40 km north of Mayo, and 15 km northwest of Elsa, as illustrated in Drawing 01. The mine will comprise an open pit and heap leach pad; haul roads; waste rock storage areas; process plant; crushers and conveyors; truck shop; camp; water diversion structure; process water ponds; drainage ditches; sediment control structures and various other ancillary facilities. The current layout for the proposed mine facilities, as received from Wardrop on November 23, 2011, is illustrated in Drawing 02.

1.3. Previous Investigations

Site conditions at the Eagle Gold site have been partially described in several reports as follows:

- Report on 1995 Geotechnical Investigations for Four Potential Heap Leach Facility Site Alternatives, First Dynasty Mines, Dublin Gulch Property. (Knight Piésold, 1996a).
- Report on Feasibility Design of the Mine Waste Rock Storage Area, First Dynasty Mines, Dublin Gulch Property. (Knight Piésold, 1996b).
- Field Investigation Data Report, Dublin Gulch Project, New Millennium Mining. (Sitka Corp, 1996).
- Hydrogeological Characterization and Assessment, Dublin Gulch Project, New Millennium Mining. (GeoEnviro Engineering, 1996).
- BGC Engineering Inc. 2010. Site Facilities Geotechnical Investigation Factual Data Report. Eagle Gold Project, Victoria Gold Corporation.
- Stantec. 2010. Project Proposal for Executive Committee Review. Pursuant to the Yukon Environmental and Socio-Economic Assessment Act. Eagle Gold Project, Victoria Gold Corporation.
- BGC Engineering Inc. 2011a. 2010 Geotechnical Investigation for Mine Site Infrastructure, Factual Data Report. Eagle Gold Project, Victoria Gold Corporation.
- BGC Engineering Inc. 2011b. 2011 Geotechnical Investigation for Mine Site Infrastructure, Factual Data Report. Eagle Gold Project, Victoria Gold Corporation.

In 1996, Knight Piésold completed a feasibility level geotechnical study to evaluate the surficial materials and bedrock conditions at four potential heap leach pad locations, two potential waste rock storage areas, and the open pit. Groundwater wells and two thermistors were installed in selected drillholes at that time. Test pitting and diamond drilling were completed from June to September 1995 at upper Bawn Bay Gulch, lower Dublin Gulch, the north side of Lynx Creek, and at the confluence of Haggart and Lynx Creeks.

In 1996, Sitka Corp completed test pits and diamond drillholes in Bawn Bay Gulch, Eagle Pup, Stewart Gulch, and Platinum Gulch for preliminary design of the heap leach and waste rock facilities. Auger holes were drilled in Gill Gulch to evaluate it as a potential borrow source of silt material for use as a liner for the heap leach facility. Monitoring wells were installed in Bawn Bay Gulch and Eagle Pup. Eight thermistor strings were installed.

In 2009, BGC was engaged to gather factual data describing subsurface conditions at the proposed heap leach and waste rock storage facility locations. The work involved the excavation of sixty-nine test pits and advancement of seven boreholes. Thermistor strings were installed in three boreholes to gather temperature data. Dynamic cone penetration profiles were obtained at two borehole locations to obtain information about material density. Dynamic cone soundings were attempted in two other holes and met with refusal. Groundwater monitoring wells were installed by Stantec in two of the seven BGC boreholes.

In 2010, Stantec presented a Project Proposal which included general site conditions such as regional geology, physiography, drainage, climate and seismicity. Air-photo based terrain mapping and an evaluation of geological hazards affecting the project area were both also described in this report.

In 2010, BGC was engaged to develop a geotechnical site investigation program in support of FS for proposed mine site infrastructure. A total of forty-nine test pits and twenty-five drill holes were completed to characterize the overburden material and bedrock conditions. Additionally, three cut slopes were logged for exposed soil and rock conditions, and core from one client-drilled condemnation hole was logged for geotechnical purposes. Laboratory testing was completed on selected samples for moisture content, and representative samples were also tested for Atterberg Limits and grain size analysis. Various other lab tests were also completed on bulk samples of placer tailings being considered for potential use as select fill or aggregate.

In the summer of 2011, BGC completed additional field investigations in support of geotechnical recommendations for mine site infrastructure. That work involved the excavation of ninety-six test pits, advancement of forty-six drill holes (29 Diamond holes and 17 Auger holes), and mapping of fifty-nine outcrops (natural exposures, existing road cuts and drill pads cuts) to characterize subsurface conditions relevant for foundation and earthworks design. Samples were taken from select test pits and boreholes for index testing of soil and rock. Bulk samples of rock and placer tailings were also tested to evaluate their potential for re-use as select fill or aggregate. Downhole and surface geophysical investigations were completed, and plate load tests were conducted at selected locations of proposed building and machine foundations.

BGC issued a series of draft reports in March and April 2011, with preliminary foundation and earthworks recommendations for a number of key facilities. The recommendations presented in those reports are superseded by those contained herein

1.4. Scope of Work

BGC was engaged to provide further geotechnical investigation work to address gaps in the data required in support of FS design for mine site infrastructure. The 2011 site investigation program was conducted between June and August 2011 and the results have been published under separate cover (BGC 2011b). BGC was also engaged to provide geotechnical engineering recommendations in support of the FS-level design of mine site infrastructure.

This foundation design report provides geotechnical engineering recommendations for selected mine site infrastructure as noted in Section 2.0. This report does not provide recommendations for the open pit, the waste rock storage areas (WRSAs), or the heap leach facility (HLF), with the exception of cut slope recommendations for the Dublin Gulch diversion. Recommendations for the open pit and WRSAs will be provided by BGC under separate cover. Geotechnical design of the HLF, including the heap embankment, heap leach pad, Dublin Gulch diversion structures and events ponds, is the responsibility of Tetra Tech.

1.5. Report Outline

The report is organized as follows:

- Section 1.0 Introduction – general introductory material;
- Section 2.0 Proposed Facilities – general description of facilities under consideration in this report, along with design criteria used in the geotechnical analysis;
- Section 3.0 Site Conditions – a high level summary of generalized site conditions to provide basic context. Readers are referred to BGC (2011b) for greater detail as required, and a more detailed summary of site conditions is provided in Appendix A of this report;
- Section 4.0 Material Properties – this section summarizes the assumed engineering properties of the in-situ foundation materials and processed engineering materials expected to be encountered or used in site development and earthworks construction;
- Section 5.0 Foundation Recommendations – this section provides recommendations of primary interest to the Structural designers, and includes recommendations for foundations and retaining walls;

- Section 6.0 Earthworks Recommendations – this section provides recommendations of primary interest to the civil designers, and includes recommendations for bulk earthworks, including cutting and filling to provide design grades for building pads, roads and other required surfaces;
- Section 7.0 Construction Materials – this section includes descriptions of different material types for use in earthworks construction, with discussion of quantities and schedule of required engineering materials, and quality and quantity of different material sources; and
- Section 8.0 Recommendations for Further Investigation – this section highlights areas of uncertainty where additional data will be required to support further development of geotechnical design, and presents recommendations for further work.
- Section 9.0 Closure.

2.0 PROPOSED FACILITIES AND DESIGN OBJECTIVES

2.1. General

The geotechnical recommendations contained in this report rely on information from several key sources, including:

- General Arrangement, Revision J, received from Wardrop, 23 November, 2011;
- Topographic contours and aerial imagery provided by Victoria, February, 2011;
- Grading information provided by Wardrop November and December, 2011; and
- Anticipated foundation dimensions, loads and settlement tolerances provided by Wardrop, 18 November, 2011.

The following subsections present brief overviews of anticipated building foundations, major earthworks, and geotechnical design parameters used for design.

2.2. Buildings

The proposed layout illustrated in Drawing 02 shows a number of buildings including those containing the crushers, the process plant site, truck shop and explosives storage areas. Anticipated foundation dimensions and loads, and tolerable foundation deformations have been provided by Wardrop on 18 November, 2011, and are summarized in Table 2-1.

The crushers will include three separate facilities – the primary, secondary and tertiary crushers – connected by conveyors, as illustrated in Drawing 03. These facilities will include heavy vibratory machinery (i.e. the crushers) and associated machine foundations, in addition to the building foundations. The primary crusher will be accessed at the top by trucks from the pit, and will be founded some 25 to 30 m lower. Thus the primary crusher building will also function as a large retaining wall.

The line of crushers will be built on steeply sloping terrain, thus requiring cutting and filling to allow development of building pads. The cut above the lower platform below the primary crusher is shown on the general arrangement as approximately 90-95 m high at its highest point. This cut is expected to be lower above the secondary and tertiary crushers.

A line of conveyors will connect the tertiary crusher with the heap leach facility in the valley bottom below. The conveyors will be supported on sleepers where appropriate, and on elevated bents where necessary. The conveyor layout is illustrated on Drawing 04.

The process plant facilities will be developed on a cut/fill pad constructed on the hillside below Tin Dome, above and to the north of the Dublin Gulch valley bottom. The proposed layout is illustrated on Drawing 05.

The truck shop will be developed on a cut/fill pad constructed on the hillside below and to the west of the planned open pit, above and to the east of Haggart Creek. The proposed layout is illustrated on Drawing 06.

Specific foundation dimensions and loads have not been provided for the camp facilities or explosives storage area. It is presently assumed that the camp will consist of settlement-tolerant structures (e.g. portable structures on timber cribbing that can be jacked and shimmed as required) that do not require specific foundation recommendations. It is also assumed that the explosives storage will consist of portable containers placed a grade on level pads, rather than permanent structures on concrete foundations. Therefore, specific foundation recommendations are not provided for either the camp site or the explosives storage facilities.

Table 2-1. Foundation Loads and Settlement Tolerances Provided by Wardrop[1]

Area And Facility	Equipment	Presumed Foundation Type And Footprints	Type Of Loadings/Presumed Bearing Pressure	Maximum Allowable Settlement
Primary Crushing	Crusher	Mat – 18m x 12m	Vibrating – 350 to 400 kPa	10 mm in 6 m.
Tertiary/Secondary Crushing/Silos	Tertiary/ Secondary Crushing/Silos	Building - spread footings – 2m x 2m to 12m x 5m Silos/Crushers – mat. – 14m x 9m to 16m x 16m	Building - 250 to 300 kPa Crushers – Vibrating - 350 to 400 kPa	Building- • 20 mm individual footings • 10 mm in 7 m bays differential • 12 mm across crane aisle between crane rails. Crushers/silos – 5 mm in 8 m differential.
Conveyors[2]	Gallery and Bents	Spread footings – 1.5m x 6m (typ.)	Static - 100 to 150 kPa[2]	25 mm – individual footings[2].
Reagent/Refinery[3]	Cranes	Building - Spread footings – 1.5m x 1.5m to 12m x 3.5m[3]	Static - 250 kPa Static - 200 kPa for 12 m x 3.5 m[3]	20 mm individual footings 10 mm in 7 m bays differential 12 mm across crane aisle between crane rails.
Process shop/Truck shop	Cranes	Spread footings - 2m x 2 m Spread footings - 8m x 3m	Static - 250 kPa	20 mm individual footings 10 mm in 7 m bays differential 12 mm across crane aisle between crane rails.
Ancillary Buildings		Spread footings – 1.5m x 1.5 m to 2m x 2 m	Static - 150 to 200 kPa	20 mm individual footings 15 mm in 6 m bay differential.

Notes:

1. As provided by Wardrop on 18 Nov 2011, except where noted otherwise.
2. Per email from Wardrop 12 January 2012, conveyor footing loads are expected to be limited to 100-150 kPa, with maximum tolerable settlement 25 mm.
3. Per email from Wardrop 2 December 2011.

2.3. Major Earthworks

The proposed mine will be located in rugged terrain, necessitating large cuts and fills in some areas, including the following:

- Main cut above primary crusher, currently shown in Wardrop grading information to be about 90-95 m in height at a slope of 1.75H:1V;
- Main cut at plant site building pad, currently shown by Wardrop to be 31 m in height at a slope of 2H:1V;
- Cuts along the Dublin Gulch diversion channel, currently shown by Wardrop to be up to about 26 m in height at a slope of 1.75H:1V to 2H:1V;
- Main cut at truck shop building pad, currently shown by Wardrop to be 20 m in height at 2H:1V;
- Main cut at upper edge of 100 day storage pad, currently shown by Wardrop to be 36 m in height at a slope of 1.75H:1V;
- Numerous other cuts of up to 15-20 m in height, at typical slopes of 1.5H:1V to 2H:1V.

Drawing 07 shows areas of planned cutting and filling associated with the bulk earthworks for infrastructure development. Several of the larger planned cut slopes are identified on that drawing, and illustrated in cross section in subsequent drawings. Drawing 08 shows the planned cut near the primary crusher. Drawing 09 shows the planned cuts near the plant site and at the Dublin Gulch diversion channel. Drawing 10 shows planned cuts at the truck shop and at the 100 day storage pad.

It is noted that the cut slope angles shown on these cross sections, and described above, are from the grading plan received from Wardrop on 23 November, 2011. Recommended slope angles are presented in Section 6.3 and may differ from those listed above and shown on these drawings.

2.4. Design Criteria

2.4.1. Allowable Bearing Pressures for Foundations

Allowable bearing pressures for the static performance of foundations must consider allowable settlements, and must also consider the potential for bearing capacity (shear) failure. Settlement tolerance criteria have been presented previously in Table 2-1. A minimum factor of safety of 3 against bearing capacity failure has been included in all bearing pressure recommendations presented later in the report.

Machine foundations must also be designed to limit vibrations to acceptable levels. Vibratory loads and vibration tolerances are equipment-specific, and therefore further analysis will be required during detailed design once equipment suppliers have been identified. Based on input from Wardrop, for preliminary planning purposes, it has been assumed that machine foundations can be designed and constructed economically if the bearing strata can provide at least 400 kPa allowable bearing pressure for static loads on the large mat foundations indicated in Table 2-1.

2.4.2. Slope Stability

Cut and fill slopes associated with the civil earthworks discussed in this report are designed to meet specific criteria for static and pseudo-static earthquake loading. The recommended safety factors are summarized in Table 2-2 below.

Table 2-2. Recommended Factors of Safety for Slope Design

<table>
<tr><th>Consequence of Failure</th><th>Cut/Fill Location</th><th>Minimum Static Factor of Safety (FS)</th><th>Minimum Pseudo-Static FS; Slope Displacement-Based (Seed, 1979)</th><th>Minimum Pseudo-Static FS; Slope Displacement-Based (Bray, 2007)</th></tr>
<tr><td rowspan="7">High</td><td>Plant Site</td><td rowspan="7">1.5</td><td rowspan="13">≥1.15 for k_h = 0.1g
M = 6.5
Maximum slope displacement of 100 cm</td><td rowspan="13">≥1.0 for k_{15} = (0.006+0.038M)*S(0.5)-0.026; S<1.5g and 2% in 50-year ground motion
Maximum slope displacement of 15 cm</td></tr>
<tr><td>Truck Shop</td></tr>
<tr><td>100-Day Storage SE Section (Close To Crushers)</td></tr>
<tr><td>Crushers</td></tr>
<tr><td>Crushers Haul Road</td></tr>
<tr><td>Substation</td></tr>
<tr><td>Diversion Channel</td></tr>
<tr><td rowspan="6">Moderate to Low</td><td>Laydown Area</td><td rowspan="6">1.3</td></tr>
<tr><td>Explosive Magazines</td></tr>
<tr><td>100-Day Storage (Distant From Crushers)</td></tr>
<tr><td>Main Pond</td></tr>
<tr><td>Truck Shop - Pit Road</td></tr>
<tr><td>General Site Roads</td></tr>
</table>

2.4.3. Seismic Design

Site specific seismic hazard information was obtained from Natural Resources Canada at www.EarthquakesCanada.ca. The National Building Code of Canada (NBCC 2005) design ground motions, corresponding to a 2 % probability of exceedence in 50 years (0.000404 per annum) are detailed in Table 2-3 below.

Table 2-3. National Building Code of Canada Recommended Design Motions

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.513	0.312	0.155	0.086	0.245

Ground motions for other return periods are provided in Table 2-4 below.

Table 2-4. Ground Motions for other Probabilities

Probability of exceedence per annum	0.010	0.0021	0.001
Probability of exceedence in 50 years	**40 %**	**10 %**	**5 %**
Sa(0.2)[1]	0.131	0.272	0.368
Sa(0.5)	0.076	0.160	0.219
Sa(1.0)	0.037	0.077	0.107
Sa(2.0)	0.020	0.043	0.059
PGA[2]	0.072	0.139	0.182

Notes:

1. S_a is spectral acceleration at the selected period (e.g. 0.2 seconds), in units of acceleration due to gravity, g.
2. Peak ground acceleration, in units of acceleration due to gravity, g.

The seismic hazard described above can be re-stated in terms of a representative earthquake event. An earthquake of M5.65 located at a distance of 17 km from the site would yield ground motions similar to those reported above. This de-aggregation of the seismic hazard was provided by the Geological Survey of Canada (GSC) on the basis of site coordinates. They were requested to do the de-aggregation for peak ground acceleration, and using the return period/annual probability specified in the National Building Code (therefore applicable to buildings). Slightly different values may apply for other structures to which the NBCC does not apply, and for which other components of the hazard (specific spectral acceleration values, rather than PGA) may be more important.

The information provided by the GSC (email to BGC dated July 2009) was accompanied by the following qualifying notes:

> *De-aggregations of the NBCC Robust seismic hazard generate a suite of files, one for each period, for each site.*
>
> *"Robust" hazard values are the ones used in the NBCC and are the higher of the H, R, C, and F model values at each site. Where any of the three other models give hazard values "sub-equal" to that from the highest model for any period, for that period the de-aggregations for those other models should also be considered for engineering purposes. This is because certain hazard and risk contributions of those other models may exceed those of the Robust model.*
>
> *A hazard example might be for liquefaction, where nearby, small-magnitude sources from the H model may give the Robust value of PGA (suitable for structural design of short-period buildings), but the liquefaction hazard may come from mid-distance large-magnitude earthquakes in the R model (because of the longer duration of ground motions from those sources).*
>
> *A risk example might be for structural damage, to the degree that it is influenced by duration effects not captured by the 5%-damped spectral values.*
>
> *"Sub-equal" can be generally taken as 70% or greater of the Robust value for any period, but there is no certainty that this is the correct value for all cases. The user needs to decide.*

3.0 SITE CONDITIONS

3.1. General

A detailed presentation of the findings of the 2011 site investigations has been provided in BGC (2011b). The following sections provide a brief overview of site conditions relevant to the development of mine site infrastructure, based on the available data. A more detailed synthesis of subsurface data relevant to geotechnical design is presented in Appendix A.

3.2. Generalized Site Conditions in the Mine Site Area

3.2.1. General Site Conditions

The site topography involves moderate to high relief, with ground elevation varying from approximately 800 to 1400 m ASL.

Ground conditions are highly variable across the site. Subsurface data are available from various sources in most areas of proposed development, as shown on Drawing 11. This drawing also subdivides the project area into a number of distinct functional areas for grouping data in relation to key facilities.

Overburden thickness varies substantially across the site as illustrated in Drawing 12. Overburden soils in the Dublin Gulch valley bottom are predominantly placer tailings (fill). Observed thickness of placer tailings is illustrated in Drawing 13.

Groundwater was observed at varying depths across the site, generally close to the elevation of streams in the valley bottoms, and often below the depth of test pit excavation (typically 5 m or greater) on the hillsides (Drawing 14).

Permafrost is present in the area, and is relatively warm (typically 0 to -1 degrees Celsius), discontinuous and occasionally contains excess ground ice. Although not dominantly controlled by slope aspect, permafrost is found more frequently in the north-facing lower slopes above the south side of Dublin Gulch. The distribution of frozen ground (including ice-rich frozen soils) observed in the testholes to date is illustrated in Drawing 15.

Bedrock at the site has been classified in three broad categories on the basis of expected engineering properties: Types 3, 2 and 1. The observed depths to Type 3, 2 and 1 rock are illustrated in Drawings 16, 17 and 18, respectively. These rock types are described in Section 4.0.

Bedrock strength may be controlled in some cases by structures such as joints, faults, bedding and foliation. A compilation of structural data relevant to mine site infrastructure is presented on Drawing 19, which also divides the site into five broad structural domains. This includes one domain for the granodiorite intrusion that hosts the ore body, and four domains in the surrounding metasediments, with domain boundaries determined largely on the basis of orientation of foliation and its relationship with regional bedrock structure.

Geological hazards were mapped by Stantec (2010). Inferred geological hazards within the areas of proposed mine site infrastructure development are illustrated in Drawing 20.

Appendix A provides a detailed compilation of subsurface data relevant to the geotechnical design of specific facilities considered in this report

3.2.2. Site Class

Seismic design parameters (i.e. uniform hazard spectra) applicable for buildings were presented in Table 2-3. A peak ground acceleration value of 0.245 g corresponds to the 1/2475 year design motion (2 % probability of exceedence in 50 years), and has been used for analysis in this report.

Seismic design parameters may require local modification for ground conditions. Site classes and soil profile names inferred based on downhole shear wave velocity profiles from each borehole tested are presented in Table 3-1. These site class designations may be used to modify the design ground motions listed in Table 2-3 for site specific conditions, where appropriate.

Table 3-1. Site Classes for Seismic Design of Buildings from Shear Wave Velocities

Facility/Area	Borehole ID	Depth[1] Analyzed (m)	Average Shear Wave Velocity, Vs_{30} (m/s)	Site Class and Soil Profile Name[2]
Crushers	BH-BGC11-36	30	825	"B" - Rock
Crushers	BH-BGC11-40B	30	800	"B" - Rock
Crushers	BH-BGC11-62	30	655	"C" - Very Dense Soil and Soft Rock
Events Ponds	BH-BGC11-32	21	365	"C" - Very Dense Soil and Soft Rock
Heap Embankment	BH-BGC11-33	30	690	"C" - Very Dense Soil and Soft Rock
Heap Embankment	BH-BGC11-34	30	540	"C" - Very Dense Soil and Soft Rock
Heap Embankment	BH-BGC11-59	28	650	"C" - Very Dense Soil and Soft Rock
Heap Pad	BH-BGC11-28	30	655	"C" - Very Dense Soil and Soft Rock
West end of Dublin Gulch	BH-BGC11-39	18	305-325	"D" - Stiff Soil
Diversion Channel	BH-BGC11-52	21	440	"C" - Very Dense Soil and Soft Rock
Plant Site	BH-BGC11-69	19.5	8302	"B" - Rock

1. Site classifications for depths analyzed less than 30 m do not meet the Vs_{30} criteria and thus should be considered as guidance only.
2. National Building Code of Canada 2005 Volume 1, pp.4-22 Division B, tables 4.1.8.4.A and 4.1.8.4.B National Building Code of Canada 2005 - User's Guide- Structural Commentaries (Part 4 of Division B) - Commentary J, pp. J-30-31.

3.2.3. Anticipated Site Conditions Relevant for Design of Cut Slopes

The proposed major cuts are shown in cross section in Drawings 08, 09 and 10, which also present the interpreted subsurface conditions including lithology and groundwater depth. The orientation, persistence and character of structural discontinuities in the rock are described in Appendix A.

3.2.4. Anticipated Site Conditions Relevant for Foundation Design

Expected subgrade conditions at planned foundation grades are presented for various facilities in Table 3-2. Given the topographic variability at the proposed mine site, the pads for building foundations are to be constructed as cuts or as balanced cut-fill. The subgrade conditions presented in Table 3-2 have been generalized; these conditions are expected to vary within the facility footprints. In particular, suitable bearing strata should be expected to have highly variable and likely sloping surfaces within the footprints of planned facilities.

Table 3-2. Anticipated Subsurface Conditions at Selected Building Foundations

Facility	Expected Pad Elevation (m ASL)	Expected Groundwater Conditions	Expected Subgrade Material at Foundation Grade
Primary Crusher	1026/1050[1]	Pad excavation is expected to encounter groundwater	Type 1 Rock above elevation of lower pad
Secondary Crusher	1032	Groundwater is expected below pad elevation	Type 2 rock at 4 m below existing ground surface[2]
Tertiary Crusher	1014	Groundwater is expected below pad elevation	Type 2 rock at 4 m below existing ground surface[2]
Conveyors from Tertiary Crusher to Heap Leach Facility	Varies along conveyor alignment	Groundwater is expected at rock/colluvium contact	Ice rich colluvium from ground surface to Type 3 rock at 10-20 m below ground surface
Plant Site	860	Pad excavation is expected to encounter groundwater	Varies, completely weathered rock at north end, fill at south end
Truck Shop	855	Pad excavation may encounter groundwater	Varies, Type 3 rock or better at east end to fill at west end

Notes:

1. According to grading information provided by Wardrop, the lower pad adjacent to the primary crusher is at 1026 m. The elevation of the top of the primary crusher, where trucks will deposit ore, is shown at 1050 m.
2. The current grading plan from Wardop shows the pads for the secondary and tertiary crushers constructed as a cut and fill balance with the crusher buildings spanning the cut and fill, and founded close to existing grade, some few metres above the type 2 rock subgrade

4.0 MATERIAL PROPERTIES

Material properties assumed for geotechnical design for in-situ soils, in-situ rock, and imported engineered materials are summarized in Table 4-1, Table 4-2 and Table 4-3, respectively. Additional descriptive information about material definitions, quantities and sources is provided in Section 7.0.

Material properties have been derived from information available from a variety of sources, including visual classification, index testing, field and laboratory shear strength testing, in-situ penetration testing, downhole and surface geophysical investigations, and plate load testing.

Rock has been classified as Type 1, 2 or 3. “Type 3” rock is usually the first “rock-like” material underlying the overburden soil materials, however sharp contacts between overburden and type 2 or type 1 rock have been observed occasionally. Type 3 rock is defined as being rock that is highly or less weathered (i.e. W4 or better), and has intact strength greater than R0 (i.e. minimum UCS strength 1 MPa). It is expected that Type 3 rock can generally be excavated with normal excavating equipment, with approximately 40 % requiring ripping.

“Type 2” rock is defined as rock with Geological Strength Index (GSI, Hoek and Marinos, 2000) or Rock Mass Rating (RMR, Bieniawski, 1976) of 30 or greater, and core recovery during drilling of 50 % or greater. Alternatively, where GSI and RMR data are unavailable, average Rock Quality Designation (RQD) of 10 or greater serves as an equivalent criterion. It is expected that Type 2 rock will generally require ripping, with approximately 35 % that can be excavated with normal excavation equipment.

“Type 1” rock is defined as having GSI, RMR or average RQD exceeding 40. It is expected that Type 1 rock will mostly require ripping, potentially hard ripping, with approximately 10-20 % requiring blasting.

The estimated shear strength of foliation for use in the cut slope design for the metasedimentary unit was determined using lab testing results from the open pit design work. Base friction values were determined through small scale direct shear testing. An increase in shear strength for large-scale roughness was applied based on the variability of the orientated discontinuity measurements, the direct shear results, and field and core observations of joint roughness. As a result, the design foliation strength was assumed to be 35°.

Table 4-1. Recommended Material Properties for Design – In Situ Natural Soils

Material	Bulk Density (kN/m^3)	Mohr-Coulomb Shear Strength			Stiffness[3]	
		Friction Angle (Deg)	Cohesion (kPa)	Concrete-Soil Friction (Degrees)	Deformation Modulus, E_s (MPa)	[1]Modulus Of Subgrade Reaction, Kv_1 (KPa/mm)
Colluvium, Debris Flow	18	34	0	N/A	N/A[2]	
Till	19	35	25	23	N/A	
Completely weathered rock	20	35	50	23	60	210
Placer Tailings	19	30 - 35	0	N/A	N/A	

Notes:

1. Modulus of subgrade reaction has been provided for a standard 1 foot plate diameter. Values need to be scaled to footing size, and will be lower for larger footings. BGC can provide further advice on request.
2. N/A: Not applicable.
3. Poisson's Ratio estimated to be 0.3 (Bowles, 1996).

Table 4-2. Recommended Material Properties for Design – Rock Mass

		Hoek-Brown input parameters[1]			Hoek-Brown strength properties				Rock Mass stiffness[4]	Dynamic Properties	
UNIT	Bulk Density[3] (kN/m^3)	GSI[2]	UCS[2] (MPa)	m_i	m_b	s	a	Rock Mass, sig_c (MPa)	E_{rm} (MPa)	G_{max} (MPa)	ν[5]
Type 1 Rock	25	51	54[6]	11	1.912	0.0043	0.505	3.4	2000 - 3000	3000 - 5000	0.2
Type 2 Rock	25	36	33[6]	6	0.610	0.0008	0.515	0.8	1000 - 2000	2000 - 4000	0.2
Type 3 Rock	25	28	25[7]	6	0.459	0.0003	0.526	0.4	100 - 500	N/A	0.2

Notes:

1. The Hoek-Brown failure criteria have been estimated using a disturbance factor ('D') of 0 for all units.
2. Median RMR'76 parameters are used for each geotechnical unit.
3. Unit Weights are based on average results of specific gravity testing.
4. Rock mass stiffness ranges are estimated considering Plate Load Test results, lab data and results of downhole geophysics
5. Poisson's ratio from average lab test results where failure mode was not along foliation
6. UCS for Type 1 and Type 2 rock are taken from median Is_{50}, multiplied by the corresponding k value (20 and 28, respectively).
7. UCS for Type 3 Rock is estimated from median strength grade (R2.5 = 25 MPa)
8. Shear modulus G_{max} obtained from V_s values from downhole geophysics

Table 4-3. Recommended Material Properties for Design – Construction Fill Materials

Material	Bulk Density (kN/m^3)	Mohr-Coulomb Shear Strength			Stiffness		Dynamic Properties	
		Friction Angle (Degrees)	Cohesion (kPa)	Concrete-Soil Friction Angle (Degrees)	Deformation Modulus, E_s (MPa)	Modulus[1] Of Subgrade Reaction, Kv_1 (kPa/mm)	Shear Modulus[2], G_{max} (MPa)	Poisson's Ratio, ν
General Fill	20	35	0	23	N/A			
[3]Structural Fill	21	40	0	27	50-100	150-300	100 - 200	
Rock fill – durable[4]	18	45	0	30	100-150	300-400	200 - 300	0.3
Rock fill - non-durable[4]	19	38	0	25	50-100	150-300	100 - 200	

Notes:

1. Modulus of subgrade reaction has been provided for a standard 1 foot plate diameter. Values need to be scaled to footing size.
2. Shear modulus is presented for very low strains of 10-6 to 10-5, and have been estimated from available shear wave velocities (Vs). Modulus should be reduced for larger strains, and BGC can provide further assistance in selection of appropriate moduli on request. Poisson's ratio inferred from Bowles, 1982.
3. It is assumed that the selected structural fill material will consist of high quality well graded sand and gravel with low fines content and durable particles, as described in section 7.3.
4. Non-linear strength envelopes may be derived for rock fill from Leps (1970) for applications under high loads, for example in the heap embankment.

5.0 FOUNDATION RECOMMENDATIONS

5.1. General

The most recent General Arrangement provided by Wardrop on 23 November 2011 shows the following key facilities considered in this report:

- Crushers and conveyors;
- Plant Site; and
- Truck Shop.

There are other ancillary facilities that have not been given specific consideration because their final locations have not been set, or due to expected light loads and/or high settlement tolerances. The camp facilities and explosives storage areas have not been considered explicitly in this report.

5.2. Foundations

5.2.1. General

Anticipated foundation conditions described herein should be verified in the field by a qualified geotechnical engineer during construction, and must be confirmed through additional site-specific subsurface investigation prior to final design. If conditions vary significantly from those presented, modifications to the foundation design parameters may be required, and BGC should be given the opportunity to review its recommendations in light of actual conditions.

It is expected that all buildings will be founded on conventional spread footings or other mass concrete foundation elements. Spread footings should be founded on Structural Fill or an approved subgrade of highly to completely weathered rock, or Type 1, 2 or 3 rock. All organics and colluvium must be removed to expose a subgrade of undisturbed rock. In areas where the required subgrade is lower than the proposed design grade, the difference may be made by placing structural fill, except where noted due to high anticipated loads.

It is recommended that foundations be designed to not straddle dissimilar subgrade materials, for example structural fill and type 3 rock. In cases where structural fill is required to make grades below part of a building, a minimum of 1 m of structural fill should be placed below foundation elements above the stiffer subgrade to minimize differential settlements.

It is presently understood that selected conveyor bent foundations will need to be founded on concrete-filled steel pipe piles socketed into bedrock.

Buildings should be set back a minimum of 10 m from the crest of fill slopes. A minimum embedment depth of 1.5 m below surrounding grade is required for adequate bearing capacity of foundations unless indicated otherwise; however, greater embedment may be required for frost protection.

5.2.2. Allowable Bearing Pressures

Anticipated foundation dimensions and loads, and deformation tolerances, were summarized in Table 2-1 in Section 2.1.

Allowable bearing pressures are a function of both the settlement tolerance of the supported facilities, and shearing resistance of the subgrade soil or rock (i.e. safe bearing capacity). The allowable bearing pressure is the minimum value satisfying both factored resistance against shear failure and tolerable settlement. These calculations depend on the foundation loads, dimensions and depths, in addition to groundwater levels and soil/rock strength and stiffness.

Allowable bearing pressures for machine foundations must also consider the vibration loads and tolerances. These were not available at the time that this report was prepared but should be considered at detailed design.

Allowable bearing pressures for small foundations (up to 2 m by 2 m and a strip footing 2 m wide and up to 20 m long) are provided in Table 5-1. These allowable bearing pressures are valid for up to 20 mm of total settlement. These bearing pressures should be considered only where facility specific recommendations are not provided and where footings are up to or smaller than the stated sizes. Facility specific recommendations are provided in Table 5-2.

Table 5-1. Recommended Allowable Bearing Pressures for Ancillary Facilities

Bearing Stratum	Allowable Bearing Capacity (kPa)	
	Up to 2 m x 2 m Pad Footing	**Up to 2 m x 20 m Strip Footing**
Structural Fill[1]	250	150
Weathered Rock	250	150
Type 3 Rock	500	300
Type 2 Rock	1000	600
Type 1 Rock	1500	1000

Notes:

1. Footings founded on structural fill require a minimum of 1.5 m of embedment (depth of bottom of footing below surrounding grade) to obtain the indicated allowable bearing capacity. Separate consideration of frost protection may be necessary.

Recommended allowable bearing pressures for key facilities are summarized in Table 5-2. In the current grading plan prepared by Wardrop, the secondary and tertiary crushers are planned to be founded on a pad constructed of a cut and fill balance with the crusher building spanning the cut and fill, and foundations at close to existing grade, with the suitable bearing stratum of Type 2 rock present some metres below grade. The crushers cannot be supported on structural fill. The available alternatives to bridge the distance between expected foundation grades and elevation of suitable type 2 rock bearing stratum include some form of stabilized fill material, such as lean concrete, or heavy deep foundations, such as caissons. It is recommended that further consideration be given to adjusting grades to put the foundations on type 2 rock to avoid either of these costly alternatives. Note, however, that it is expected that adjusting grades will require a higher permanent cut slope adjacent to the crushers, so this additional rock excavation should be balanced against the increased foundation costs. BGC can provide more detailed recommendations after further input from Victoria and Wardrop.

Table 5-2. Recommended Allowable Bearing Pressures for Specific Facilities

Facility	Expected Pad Elevation (m ASL)	Foundation Dimensions[1]	Expected Subgrade Conditions	Allowable Bearing Pressure[2] (kPa)
Primary Crusher	1026/1050[3]	Up to 12 m x 18 m mat	Type 1 rock	1000
Secondary Crusher[4]	1032[4]	Up to 16 m x 16 m mat, 12 m x 5 m spread footing	Type 2 rock[4]	400[4]
Tertiary Crusher[4]	1014[4]	Up to 14 m x 9 m mat	Type 2 rock[4]	400[4]
Conveyors from Tertiary Crusher to Heap Leach Facility	Varies along conveyor	Bents on 1.5 m x 6 m footing	Type 3 rock at ~ 5 m to 20 m depth below grade, typically 10 m expected	200 mm concrete-filled steel pipe piles socketed 2 m into Type 3 rock at ~ 10 m depth below grade can support 700 kN
		Sleepers at grade, on timber cribbing where possible	Colluvium below stripped topsoil	N/A – adjustable foundations
Plant Site	860	3.5 m x 12 m	Highly to completely weathered rock or structural fill	200
Truck Shop	855	3 m x 8 m	Type 3 rock	300

Notes:

1. Foundation dimensions provided by Wardrop on 18 Nov 2011.
2. Based on factor of safety of 3 against bearing capacity failure and limiting settlements to those specified by Wardrop on 18 Nov 2011.
3. The lower portion of the primary crusher is at 1026 m. The elevation of the top of the primary crusher, where trucks will deposit ore is at 1050 m.
4. Crushers cannot be supported on regular structural fill. If the secondary and tertiary crushers must be built at planned grades well above the suitable bearing stratum, the gap between the bearing stratum and foundation grades can be made up by lean concrete or some other form of stiff fill material.

5.2.3. Subgrade Preparation

Care should be taken to avoid disturbing subgrade materials that will remain in place. Areas of weathered rock subgrade that become softened or loosened during construction should be removed and replaced with compacted engineered fill (structural fill) or lean concrete. The base of all excavations should be dry and free of loose materials at the time of concrete placement. A layer of lean concrete can be placed on the subgrade for protection to allow work to continue in wet weather prior to pouring of footing concrete.

Subgrades should be reviewed by a qualified geotechnical engineer prior to placement of structural fill, protective blanket, or forms for foundations.

5.2.4. Foundations on Sloping Bearing Strata

Several proposed facilities will be constructed on sloping ground, notably the plant site, truck shop, explosives storage areas and crushers, and in these areas the acceptable bearing strata are also expected to be sloping. Foundation subgrades should be prepared so that foundation elements will be placed on horizontal surfaces, which may require excavation of notches or benches within the suitable bearing strata. BGC can provide further advice with respect to specific foundations on sloping ground during detailed design and construction.

5.2.5. Water Control

Final site grading should maintain positive drainage in the direction of natural drainage and should direct water away from the structures. Improper drainage and ponding of water near or under the structures can be detrimental to the foundation performance. The final grades should be sloped down, away from the structure, at a slope of 4% within 3 m of the structure and at a slope of 2% beyond.

It is recommended that permanent surface water control be provided at the base of all excavation slopes to direct water away from the proposed facilities and to allow the slopes to drain effectively. In addition, temporary surface water control during foundation excavations should be provided by the contractor so that foundation excavations and subgrade remain essentially dry when the foundation is being constructed.

Based on available groundwater level information, it is expected that the pad development for the proposed crushers, plant site and truck shop area will intercept the water table. Due to the fractured nature of the shallow bedrock, dewatering of excavations, if necessary, should be feasible with conventional sumps and pumps.

Development of the silt borrow area near the laydown area may encounter groundwater. Upward seepage gradients, if encountered, may result in softening and/or heaving of the subgrade soils in this area.

5.2.6. Minimum Foundation Depth for Frost Protection

Critical foundations, water lines, and other important infrastructure should be protected from frost. The maximum depth of frost penetration for the project site is estimated to be 3.0 m.

Exterior building foundations should be founded below the anticipated depth of frost penetration or will need to be properly insulated if founded above the maximum depth of frost penetration. Exterior footings at 1.0 m depth below finished grade should be insulated by 50 mm thick Dow Styrofoam SM or equivalent extruded polystyrene insulation buried 0.3 m below final grade and extending horizontally 1.8 m. The vertical portion along the foundation element should also be insulated with 50 mm thick insulation. The insulation should be sandwiched between two layers of bedding sand, 75 mm in thickness, and should be sloped down away from the structure at 1 percent grade. If exterior footings are raised to 1.0 m depth, allowable bearing pressures may need to be reduced. BGC can provide further comment if requested.

5.2.7. Concrete Slabs

A minimum of 150 mm thick layer of compacted free-draining sand and gravel, consisting of 19 mm minus durable material with less than 8 % fines (passing No. 200 sieve), should be placed beneath all slabs-on-grade as a leveling course.

5.2.8. Temporary Excavations

Construction may require temporary excavations into native soil and weathered bedrock. Safe, stable construction slopes should be made the responsibility of the contractor and will depend on the ground and site conditions encountered at the time of construction.

5.3. Retaining Walls

Retaining Walls must be designed to sustain various loads, and should be checked for satisfactory performance with respect to overturning, sliding, bearing capacity and global stability. These checks are generally the responsibility of the wall designer.

5.3.1. Design Basis

BGC provides the following guidance to aid in the design of retaining walls. It is assumed that all retaining walls are intended to be designed as rigid reinforced concrete walls. However, if requested, recommendations can be provided for flexible reinforced soil walls, tied-back walls or other retaining structures, and BGC can provide further advice and assistance in wall design if required.

The recommendations in this report are based on the assumption that the water table can be kept below the base of the wall, and therefore do not account for hydrostatic water pressures which could increase lateral loads and produce uplift on the foundation. In order to achieve this condition in practice, the following measures are suggested:

- Infiltration and seepage behind walls should be minimized. The upper surface of the backfill should be covered with a low permeability material, and the site should be graded away from the structure to prevent surface water from accumulating against the back of the wall.
- Retaining wall backfill should be free-draining granular soil (e.g. 19 mm minus sand and gravel with less than 8 % fines).
- Backfill should be drained using a perforated drain tile set at footing level and draining to a free outlet. In addition, where possible operationally, weep holes should be provided through the face of the wall.

Walls should be designed for internal, external and overall stability. Appropriate shear strength properties for geotechnical materials and concrete-subgrade or concrete-backfill contact are provided in Table 4-1 and Table 4-3.

For seismic design, inertial forces due to the mass of the wall (including the mass of the reinforced soil column in the case of reinforced earth walls) should be considered. For cantilever retaining walls, the additional inertial force due to the mass of the soil column above the heel section of the wall should also be considered.

Lateral earth pressures acting on retaining walls depend to a large degree on how much a wall is allowed to rotate under normal operating conditions. Walls are often defined as either “unrestrained” or “restrained,” where this distinction depends on the allowable wall rotation. Restrained walls, which are sometimes referred to as “non-yielding” walls, are subject to higher loads, assumed to be represented by the “at rest” (i.e. K_o) condition. Unrestrained walls, sometimes called “yielding” walls, are subject to the “active” condition (i.e. K_a) or “passive” condition (i.e. K_p), depending on direction of wall movement.

The distinction between “unrestrained” (or “yielding”) and “restrained” (or “non-yielding”) walls depends on the wall configuration and properties of the retained backfill. “Unrestrained” walls are free to move sufficiently to allow active earth pressures to develop behind the wall in the limiting condition. “Restrained” walls are those that are prevented from moving sufficiently for active pressures to develop behind the wall in the limiting condition, when bearing or sliding failure is occurring.

When a retaining wall is backfilled with compacted granular fill, the transition from the “at rest” to “active” condition occurs at an angular rotation of about 0.001 m/m (i.e. 1 mm of deflection per 1 m of wall height). The transition from the “at rest” to “passive” conditions, for a wall moving inward, toward the backfill, occurs at an angular rotation of about 0.02.

The “active” and “passive” conditions may be assumed to apply for analysis of sliding and overturning. Structural design of the wall should consider “at rest” conditions, plus any compaction pressures and additional line loads or surcharges. Minimum factors of safety of 1.5 may be considered for both sliding and overturning.

5.3.1.1. Static Earth Pressures

The calculation of static earth pressures depends on several factors, including: type and geometry of the wall; strength (friction angle) of the backfill, backfill/wall interface and wall/foundation interface; wall height; bulk weight of backfill; and, inclination of ground surface behind (and above) the top of the wall.

The method for calculating static earth pressures for “unrestrained” and “restrained” walls are shown in Drawings 21 and 22.

5.3.1.2. Dynamic Earth Pressures

The method for calculation of dynamic earth pressures depends on whether the wall is “unrestrained” or “restrained.” Both methods are illustrated in Drawings 23 and 24.

5.3.1.3. Compaction Pressures

To minimize compaction-induced earth pressures, use of small vibratory or hand-operated ram compaction equipment is recommended for the area within 2 m of the back of the wall.
If using small compaction equipment behind the wall is not practical, an additional load should be included to account for additional stresses due to compaction. The method for calculating and applying this additional load is detailed in Drawing 25.

5.3.1.4. Lateral Pressures from Surcharge Loads

The presence of vertical loads behind the top of the wall will increase lateral pressures acting on the wall. Three different types of vertical surcharge loads may be considered: point loads, acting at a single point behind the wall; line loads, acting along the full length of the wall at some specified distance behind the wall; and, area loads, which may be uniform pressures acting on the ground surface adjacent to the wall.

The effect of area loads may be determined by assuming a uniform horizontal pressure against the full height of the wall as detailed in Drawing 25, where “K” is the lateral earth pressure coefficient applicable for the wall type (i.e. “unrestrained” or “restrained”). The method for estimating the effect of point loads or line loads is illustrated in Drawing 26.

6.0 EARTHWORKS RECOMMENDATIONS

6.1. General

This section presents general recommendations for bulk civil earthworks necessary to obtain the required grades for building pads, roads and other required platforms.

There are a number of ground-related challenges to construction of earthworks and buildings at the proposed mine site. These include:

- Presence of discontinuous permafrost, including some areas with excess ground ice;
- Relatively short "traditional" (i.e. spring/summer/fall) construction season, with specific challenges and limitations during other parts of the year (e.g. poor trafficability and material workability on hillsides before mid-summer; and long, harsh winter);
- Uncertain quality and quantity of required borrow materials;
- Presence of significant quantities of existing placer tailings; and
- Presence of steep slopes and geological hazards.

Excavation of frozen ground, particularly ice rich permafrost, requires additional effort and care. Well-bonded, ice-rich frozen ground will be difficult to excavate, and as discussed previously, will require ripping. Further consideration needs to be given to the thaw behavior of this material, and allowances made for adequate drainage and associated erosion control, as well as additional time and effort for the work. Exposure of ice-rich permafrost and the associated thaw can result in wet, muddy, soft ground, and poor trafficability, along with local slumping and other nuisance effects.

Each of these specific challenges requires consideration in the planning, design and construction of mine site infrastructure, as discussed in the following sections.

6.2. Area-Specific Earthworks Commentary

6.2.1. General

The project area has been subdivided into a number of functional areas, as shown in Drawing 11. Summary observations for each functional area were presented in Table A-5 in Appendix A. That table provides an overview of the general conditions within each area, including the observed thickness of overburden, presence or absence of frozen ground and excess ice, and depth, where encountered, to Types 1, 2 or 3 bedrock.

The presence of placer tailings (fill) is an issue primarily in the Dublin Gulch valley bottom, and will affect the development of the heap leach pad, heap embankment, a portion of the Dublin Gulch diversion, and ponds or other facilities constructed in this area. The observed thickness of placer tailings at 16 test holes had a mean value of about 10 m, with a range between 0.3 m and 19.8 m.

There is typically a thin cover of organic soils overlying the other overburden units. The observed thickness of this unit varies across the site, ranging between 0 m and 3.7 m, with an average thickness of 0.3 m (285 observations). All organic materials are unsuitable for re-use as engineering fill materials, but should be suitable for reuse as cover materials for reclamation and should thus be segregated and separately stockpiled.

The following sub-sections present commentary related to earthworks construction in each functional area, based on the summary observations just presented. The need to remove surficial organic materials is not repeated in these sections. These general comments are intended to be interpreted in relation to gross earthworks within each identified functional area, and may not apply precisely for specific sites or facilities.

6.2.2. Area-Specific Commentary

The following subsections provide area-specific earthworks commentary. Where bedrock is encountered, it can generally be assumed that common excavation, ripping and blasting may be expected in Types 3, 2, and 1 rock, respectively. Excavated rock can generally be expected to be suitable for reuse as general fill, and potentially suitable for use as structural fill with due care in selection, placement and compaction control. Excavated rock used as structural fill will not be suitable for use in applications where a free-draining material is required, such as at shallow depths below buildings, or behind retaining walls.

Frozen ground will be most efficiently excavated by ripping where it contains excess ice or is otherwise well-bonded, and for planning purposes, all frozen ground may be assumed to require ripping. Excavated frozen ground will generally be unsuitable for reuse without substantial effort to thaw and drain, and may be suitable for reuse only for limited applications, depending on the moisture and fines contents.

It will be necessary to plan for temporary or permanent stockpiling of the wasted ice-rich frozen soil. These materials will be unstable when thawed and will not stand at steep angles or significant height, so a large footprint or containment berm may be required to store relatively small volumes. It may be possible to dispose of ice-rich spoil in areas developed for borrow, including the placer tailings piles in the Haggart Creek area as suggested by Victoria. Such disposal would require further study.

6.2.2.1. 100 Day Storage Pad

The overburden in this area is relatively thin, and is commonly frozen, with excess ice encountered in nearly half the test holes where frozen ground observations were made. Excavated overburden materials will not generally be suitable for re-use as a construction material. The shallow bedrock will be relatively easy to excavate to depths of 5-10 m below grade, and will be suitable for re-use as general fill. Excavations deeper than about 10 m, if required, may require ripping or blasting.

6.2.2.2. Conveyors

This area contains thick, frozen overburden, typically containing excess ground ice. Excavation of frozen ground will likely require ripping, and excavated materials will be unsuitable for re-use. Rock excavation is not anticipated in this area.

6.2.2.3. Crushers

This area contains moderately thick (typically 0 to 7.5 m) overburden, most of which is weathered rock, and is sporadically frozen. It should be assumed that about half of the overburden may be re-used as general fill. Shallow bedrock to approximately 5-10 m below grade will be Type 3. Deeper rock at 10-15 m or > 15 m depth can be expected to be Type 2 and Type 1, respectively. All excavated rock is expected to be suitable for re-use as general fill.

6.2.2.4. Dublin Gulch Diversion

In the portion west of Eagle Pup valley, there is widespread frozen ground with excess ice which may require ripping to excavate and will be unsuitable for re-use. The thickness of ice-rich permafrost, where present, was not delineated in the geotechnical investigations to date, but is expected to be up to about 5 m. Depth to rock is highly variable in this area, and borehole data are limited.

6.2.2.5. Dublin Gulch Pond

Very little subsurface information is available in this area. It should be assumed that loose, variable fill materials (placer tailings) will be present, including wet, silty materials that will likely be unsuitable for reuse.

6.2.2.6. Eagle Pup WRSA Pond

Overburden is relatively thick (typically 3 to 12 m), with locally shallower Type 3 or Type 2 bedrock. Ice-rich frozen ground was observed in one of four testholes probed in this area. An estimated half of excavated overburden materials may be suitable for re-use as general fill. Bedrock, where encountered, can be excavated but may require local ripping. Excavated bedrock will be suitable for re-use as general fill.

6.2.2.7. Eagle Pup WRSA

Overburden is moderately thick (0 to 10 m), but highly variable. Frozen ground is widespread (47 of 77 observations) and frequently contains excess ice (29 of 77 observations). Stripping of ice-rich materials, where required for WRSA foundation preparation, will require ripping, and excavated materials will not be suitable for re-use. Excavation of rock is not expected to be necessary for foundation preparation in the WRSA. There is a lobate feature approximately 100 m x 100 m in plan, with ice-rich colluvium to > 25 m depth, located in the valley bottom. This feature will be discussed in greater detail in the WRSA design report under separate cover.

6.2.2.8. Heap Leach Events Ponds

Overburden is thick (typically 10 m to 20 m) and comprised of placer tailings, which are expected to be generally suitable for reuse as general fill without processing, or for use as select fill (structural fill, and potentially concrete aggregate or heap overliner) with crushing and screening. Excavation of rock is not expected to be necessary in this area, unless pond grades intersect bedrock.

6.2.2.9. Explosives Storage

Overburden is relatively thin (typically 2-3 m). Some ice-rich frozen ground should be anticipated. It may be assumed that roughly half of excavated overburden will be suitable for re-use as general fill. Bedrock to about 5 m depth can be expected to be Type 3, and deeper rock will be Type 2 and will require ripping. If excavations deeper than about 10 m are required, blasting of Type 1 rock should be anticipated.

6.2.2.10. Heap Leach Embankment

Overburden in the valley bottom is thick (typically 4 to 14 m) and comprised of placer tailings, which are expected to be generally suitable for reuse as general fill without processing, or for use as select fill (structural fill, and potentially concrete aggregate or heap overliner) with crushing and screening. No rock excavation is expected to be necessary in this area, based on the current heap facility layout by Tetra Tech.

Overburden materials are more variable at the north and south ends of this area, where the abutments will be constructed. No general commentary can be provided for those areas in this report. Foundation preparation recommendations for the heap embankment and abutments are being undertaken by Tetra Tech.

6.2.2.11. Heap Leach Pad

The overburden within the proposed heap leach pad footprint is typically of moderate thickness (0 to 9 m), but highly variable. Frozen ground is present in some areas (14 of 71 testhole observations) and contains excess ice in isolated areas (6 of 71 observations). Non-frozen overburden will generally be granular colluvium that is expected to be easily excavated and generally suitable for reuse as grading fill for the heap subgrade. Bedrock depth is variable, and shallow bedrock to 5 m depth is generally Type 3. Type 2 rock can be expected at depths below 5 m, and Type 1 rock may be encountered at depths greater than about 10 m, but is locally shallower in the upper part of the heap.

6.2.2.12. Laydown Area

This area includes the area intended to be developed for silt borrow for pond liner material, as well as the proposed construction laydown area and permanent camp.

The proposed laydown area straddles thick (estimated to be 10 to 20 m, no data available) placer tailings in the Dublin Gulch valley bottom, and thick (up to 25 m thick), ice rich permafrost in the undisturbed area further south.

The ice rich permafrost will require ripping to excavate, and the silt borrow material will need to be thawed and dried before it can be re-compacted as liner material.

The placer tailings in this area have been recently re-worked to construct a pad for the 100-man exploration camp. The materials in this pad are silt, sand and gravel in varying proportions.

6.2.2.13. Main Site Water Management Pond

The proposed pond area straddles thick (estimated to be 10 m or greater, no data available) placer tailings in the Dublin Gulch valley bottom, and thick (up to 25 m thick), ice rich permafrost in the undisturbed area further south.

The placer tailings in this area are expected to be generally suitable for re-use as general fill. Ripping will be required to excavate frozen ice rich overburden in the undisturbed part of this area, which comprises roughly the southern three quarters. No rock excavation is expected to be necessary in this area.

6.2.2.14. Main Truck Road

The overburden in this area is of moderate thickness (approximately 1.5 to 7 m), with limited presence of frozen ground (1 of 7 observations). Most of the unfrozen excavated overburden is expected to be suitable for re-use as road grading fill. Excavations deeper than about 5 m may encounter Type 3 rock. Excavations deeper than 10 m and 15 m should be expected to encounter Type 2 and Type 1 rock, respectively.

6.2.2.15. Plant Site

This area has thick overburden, most of which is either till or completely weathered rock. Roughly two thirds of the excavated overburden materials in this area are expected to be suitable for re-use as general fill, assuming a deep cut for the plant site pad. It is expected that excavations in this area can be completed with conventional excavation equipment to at least 30 m depth. The Type 3 rock encountered below about 10 m depth may be suitable for re-use as structural fill with due care in quality control of material selection (possibly including screening), placement and compaction control.

6.2.2.16. Platinum Gulch WRSA Pond

There is very little information available for this area, however the distribution of permafrost may be limited, and bedrock may be locally shallow (i.e. 0 m to 6 m). Type 1 rock should be anticipated for excavations deeper than about 5 m.

6.2.2.17. Platinum Gulch WRSA

The overburden in this area is moderately thick (typically 0 m to 6 m), with significant variability in observed thickness. Frozen ground is locally present and occasionally contains excess ice. Stripping of ice-rich materials, where required for foundation preparation, will require ripping, and such excavated materials will not be suitable for re-use. Rock excavation is not expected to be necessary for foundation preparation in the WRSA.

6.2.2.18. Secondary Road

This functional area contains secondary roads from the main access road along Haggart Creek between the substation and truck shop to the bottom of the 100 day storage pad. Limited information suggests that overburden is thick and likely frozen and ice rich in this area. Ripping may be required for excavation of frozen overburden for road grade preparation. It should be expected that excavated spoil materials will not be suitable for immediate re-use as road grading fill, but may become suitable given adequate time to thaw and drain (perhaps after a minimum of one full summer, but will depend on seasonal weather).

6.2.2.19. Truck Shop

Overburden is moderately thick (typically 7 to 8 m) and consists of frozen silty colluvium with excess ice in the upper 2 to 4 m. The underlying bedrock is Type 3. The shallow frozen overburden will require ripping. The frozen colluvium and bedrock below about 4 to 5 m depth can be excavated with normal excavating equipment. Excavated overburden materials will not be suitable for immediate reuse, but excavated bedrock will be suitable for use as general fill, or for use as structural fill with due care in quality control of material selection, placement and compaction control.

6.3. Site Preparation

The shallow overburden materials, including organic soils and colluvium, should be removed below all building foundations or below pads for building development to expose undisturbed native subgrades of highly to completely weathered rock or type 1, 2 or 3 rock. Organic soils should be stockpiled for re-use in reclamation work. The excavated colluvium materials may be suitable for re-use as general grading fill (General Fill), provided they do not contain deleterious materials, such as organic inclusions or excess ice. Stripped materials should be segregated under the direction of a qualified geotechnical engineer. Selected poor-quality colluvial soils may need to be wasted, at the discretion of the Engineer.

The overburden soils contain a significant percentage of fines (materials passing the No. 200 sieve) and fine sand such that their consistency may be sensitive to moisture and freezing temperatures. These soils may also degrade to slurry-like consistency when subjected to construction traffic loads or otherwise disturbed in wet conditions. It is recommended that defined construction roads be used for repetitive construction traffic to minimize disturbance at prepared areas. Trafficability will be poor on recently thawed ground or in areas of poor drainage.

Permafrost is present in patches, and seasonally-thawed soils may remain frozen late into the summer. Some of these materials may contain excess ice and will therefore become wet when thawed. Care should be taken to segregate frozen materials removed during site grading activities.

Where construction activities are to be conducted during periods of freezing weather, fill should not be placed upon frozen material, snow or ice. Earth fill placement, including non-durable rock fill placement, should be temporarily suspended if freezing conditions exist. It is recommended that if the ambient air temperature is less than zero degrees Celsius for more than four (4) hours over the preceding twenty-four (24) hours, the temperature of the fill should be measured to determine if the fill is frozen. If frozen, the fill should be removed and replaced. To help protect the fill surface from freezing during periods of shutdown it is recommended that placed fills be covered with loose (sacrificial) fill, or blankets, to help insulate the fill from freezing temperatures. Placement of coarse durable rock fill, which does not require water for compaction, can proceed in freezing conditions.

6.4. Site Grading - Fills

6.4.1. General

Site grading, as described in this section, includes all major excavations and fills necessary to bring the site to the proposed design elevations, including fill to support buildings, foundations, floor slabs, and backfill of foundations.

6.4.2. Engineered Fill Slopes

Engineered fill slopes constructed of structural fill or rock fill may be made at 2H:1V or flatter. Buildings should be set back a minimum of 10 m from the crest of fill slopes.

Where a structural fill embankment is to be constructed on an existing natural slope, the fill should be keyed into the natural slope by excavating steps into the slope at the edge of successive lifts of structural fill.

Selected high fills, including those below the pit-crushers haul road and at the lower (north) end of the 100 day storage pad, may encroach into seasonal drainage areas or depressions with shallow groundwater. Particular care should be taken in these potentially wet areas to choose free draining, coarse granular fill materials, preferably angular durable rockfill, to prevent buildup of excess pore pressures in the fills.

6.5. Site Grading - Cuts

6.5.1. Excavation Effort

Bulk excavation activities will encounter various materials. All overburden soil materials, including organics, colluvium, till, debris flow material, alluvium and highly to completely weathered rock are expected to be excavatable by normal excavating equipment when encountered in an unfrozen state. These same materials will likely require ripping when frozen, and ice-rich frozen materials in particular will require hard ripping.

It is expected that Type 3 rock can generally be excavated with normal excavating equipment, with approximately 40 % requiring ripping. It is expected that Type 2 rock will generally require ripping, with approximately 35 % that can be excavated with normal excavation equipment. It is expected that Type 1 rock will mostly require ripping, potentially hard ripping, with approximately 10-20 % requiring blasting.

6.5.2. Permanent Cut Slopes

6.5.2.1. General

Area specific cut slope angle recommendations are provided for the highest and most critical of the proposed excavations (Table 6-1). General cut slope angle recommendations are provided for all other slopes that are less than 10 m high (Table 6-2). Except where noted, the recommendations are applicable to unsupported slopes, where no slope support, reinforcement, or extensive rockfall prevention is used. All constructed slopes should be reviewed in the field during construction to check that design assumptions remain valid. It may be necessary to revise slope design recommendations for specific structures following future site investigation or during construction as ground conditions are exposed.

6.5.2.2. Area Specific Cut Slope Recommendations

Table 6-1 provides area specific cut slope recommendations, which are also illustrated in Drawings 08 to 10. These recommendations assume the stratigraphy and water levels illustrated in Drawings 08 to 10, and the material strength properties listed in Table 4-1, Table 4-2 and Table 4-3. The location of each cross section is illustrated in Drawing 07. A two-dimensional limit equilibrium stability analysis was completed to evaluate the long term stability of each slope. Static and pseudo-static analyses were completed using Slope/W (Geo-Slope, 2007), a commercially available limit equilibrium slope stability analysis software.

Typically, the slopes are composed of variable thickness of overburden over bedrock that is weathered to varying degrees. Table 6-1 provides the estimated overburden thickness and recommended cut angles for both the overburden and the underlying rock. In design, the overburden cut angle should be used in the zone between the ground surface and a depth equal to the overburden thickness.

Table 6-1. Recommended Permanent Cut Slope Angles – Area Specific

Area	Overburden		Slope below Overburden		Notes (refer to Drawings 08 to 10)
	Thick-ness (m)	Steepest Cut Angle	Material	Steepest Cut Angle[1]	
Primary Crusher	2 - 4	2.5H:1V	Type 1, 2, 3 Rock	1.75H:1V	Design FS = 1.5; maximum slope height ~107 m; slope angle controlled by dip of foliation at about 30-32 degrees; benched slope design recommended; 8 m maximum bench height; 13 m minimum bench width; 0.25H:1V bench face angle.
100 Day Storage	3 - 4	2.5H:1V	Type 2, 3 rock	1.75H:1V	Design FS = 1.5[2]; slope angle controlled by dip of foliation at about 30-32 degrees; minimum distance of 80-100 m required between slope crest and toe of haul road / crusher platform fill slopes. Benched slope design is recommended as detailed above for primary crusher.
Truck Shop	5 - 8	2.5H:1V	Type 3 rock	1.75H:1V	Design FS = 1.5; maximum slope height = ~22 m; slope angle controlled by dip of foliation. Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Plant Site	3 - 7	2.5H:1V	Highly to completely weathered rock	2H:1V	Design FS = 1.5; maximum slope height ~35 m; Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Dublin Gulch Diversion	2 - 5	2.5H:1V	Till	2H:1V[3]	Design FS = 1.5; maximum slope height ~28 m; maximum cut angle assumes that the cut slope is dry.

Notes:

1. Maximum overall slope angle in the slope materials below the overburden depth. Overall slope angle defined by the line that connects the toe of the slope with the slope crest at the rock-overburden contact.
2. Recommended FS for the 100 day storage cut is 1.5 due to proximity to crushers and potential to undermine them in case of failure. FS = 1.3 could be considered when the cut is moved 80-100 m further from the crushers, however, the overall slope angle will still be controlled by the dip of the foliation and cannot be steepened significantly.
3. Assumed groundwater level is greater than 6 m below existing ground surface, which is inferred but not confirmed and requires further study.

At the primary crusher, 100-day storage, and truck shop areas, the cut slope design is controlled by the potential for failure of the rock along discontinuities defined by foliation in the metasedimentary rock. The foliation is expected to dip out of the slope at angles ranging from about 20° to 40°, typical values observed in Structural Domain C (Drawing 19). The potential failure wedge that could form on slopes of this size is large enough to make mechanical support of the slopes impractical. Therefore a relatively shallow overall slope angle has been recommended. This overall slope angle is approximately parallel to the observed dip of the foliation, which essentially eliminates the potential for a planar failure at the slope-scale.

Bench scale failures are expected, including minor raveling and slumping, where the foliation is undercut; however, failures occurring on upper bench faces are not expected to adversely affect the infrastructure at the base of the slope due to the presence of the 13 m wide rockfall catchment benches. However, an allowance should be made in the design for spot bolting of loose blocks of rock on the bench faces in case specific weak structures are encountered. Mesh may also be required to contain poor quality rock that could ravel, should it be encountered, particularly on the bottom bench where service vehicles may be entering. Additionally, an 8 m wide rockfall catchment area should be included in the design at the upper and lower platform elevations. A 1 m high barrier (concrete or earth, or permanent fence) is recommended to be placed at the outer edge of the rockfall catchment area to deter encroachment into the catchment area by vehicles or personnel.

At the primary crusher, it is expected that blasting will be required to excavate the rock; therefore a benched slope design has been recommended. The recommended bench face angle is 0.25H:1V, which has been selected to facilitate controlled blasting. The maximum recommended bench height is 8 m. The minimum recommended bench width is 13 m to facilitate installation of a safety berm and to allow access for bench clean up. The bench width may need to be adjusted at detailed design to maintain the recommended overall slope angle of 1.75H:1V.

The recommendations provided for the primary crusher cut are based on assumed water levels and ground conditions, which are based on relatively sparse site characterization data. The consequences of a slope-scale failure at the primary crusher cut are perceived to be very high. Additional site investigations are recommended to reduce the current level of uncertainty in the understanding of ground conditions. The recommendations provided in this report assume that the design is controlled by the foliation of the meta-sedimentary rock. Future site investigation should verify that additional unfavorable conditions are not present and should be designed to characterize the orientation and condition of the contact between the meta-sedimentary and igneous rock, which is expected to daylight near the base of the cut.

At the 100-day storage area, the crest of the cut slope may daylight near the toe of the fill slopes from the haul roads and crusher platform. A minimum distance of 80-100 m between the cut slope crest and toe of fill is recommended to reduce the possibility of a slope failure at the 100-day storage area which could affect the crusher or haul roads.

The recommended cut angle at the Dublin Gulch diversion assumes that the slope materials are unsaturated. If the slope materials are saturated, the recommend cut angle would decrease to 2.5H:1V. Current information regarding the depth to groundwater along the diversion is sparse. Future site investigation programs should be designed to characterize the groundwater depth along the diversion, and update the cut slope design, if appropriate.

A rockfall catchment area should be provided at the base of all cut slopes. The catchment area should be sloped back towards the cut slope at an angle of 4H:1V. The recommended minimum width of the rockfall catchment is 2.5 m below soil cuts, and 8 m below rock cuts.

6.5.2.3. General Cut Slope Recommendations

Table 6-2 provides general cut slope angle recommendations based on material type, for general application across the site for cut slopes less than 10 m high. It is assumed these cuts will be unsaturated and without adverse geologic structure. Cut slopes that do not meet these conditions should be reviewed on a case-by-case basis by the geotechnical engineer.

Table 6-2. Recommended Permanent Cut Slope Angles – General

Slope Material	Maximum Cut Slope Angle[1]	Maximum Cut slope Height	Notes[1]
Colluvium	2.5H:1V	10 m	
Till	2H:1V	10 m	
Highly to completely weathered rock (excavatable)	2H:1V	10 m	
Type 3 rock (generally excavatable)	1.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 2 rock (generally rippable)	1H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 1 rock (may require blasting)	0.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered

Notes:

1. Maximum cut slope angles assume the slope is < 10 m high, unsaturated, and without adverse geologic structure

7.0 CONSTRUCTION MATERIALS

7.1. General

This section discusses the demand for specific engineering fill materials, and provides some comment on quantities of excess materials requiring permanent disposal or temporary storage.

7.2. Borrow Requirements

7.2.1. Mine Site Area

Development of the proposed mine will involve excavation, stockpiling, processing, hauling, placing and compaction of a variety of earth and rock materials. Material take offs (MTOs) with earthworks quantity estimates were provided by Merit Consultants International on January 6, 2012. These MTOs include numerous line items for various types of excavation or granular borrow required for construction of the mine site facilities, including the following approximate quantities of specific materials:

- Approximately 2.9 million m^3 of engineered fill, which includes approximately 2.1 million m^3 of engineered fill for the heap containment dyke and diversion embankment, selected from a variety of sources, including processed placer tailings, non-durable rock obtained during bulk earthworks activities, and possibly durable waste rock from mining. This “engineered fill” includes the following general categories of materials:
    - General fill,
    - Structural fill,
    - Durable rock fill, and
    - Non-durable rock fill;
- 298,000 m^3 of crushed durable rock to produce a well-graded material for the heap overliner;
- Various minor quantities of miscellaneous engineering materials, including silt/fines for liner construction, transition/filter materials, drainage materials, rip rap, concrete aggregate, and road pavement structure materials.

7.3. Suggested Borrow Material Classifications

Construction fill materials at the project site have been identified by other disciplines without specific technical specifications. The following definitions are proposed for consideration by other disciplines responsible for earthworks construction.

<u>Silt/Clay Liners</u>

These are fine-grained fills used as a barrier for chemical and physical migration of fluids. The prefeasibility study report (SWRPA 2010) suggests a target hydraulic conductivity for compacted fine grained liner materials of no more than 1 x 10^{-5} cm/s, or 1 x 10^{-6} cm/s in the absence of a leachate detection and removal system.

Silt liner materials should contain a minimum of 35% passing the No. 200 sieve and be free of all deleterious materials including oversize clasts of 75 mm or greater, frozen soils, and organics. This material should be placed with uniform moisture content, typically within 2% (above) optimum moisture content (ASTM D698) and a USCS classification of CL, ML, CH or MH.

Rock Fill

Rock fill can be classified as one of two types: 1) that derived from strong rock, yielding durable rock fragments larger than gravel size and containing sand and gravel with less than 15% fines when excavated/blasted; and, 2) that derived from weak, fissile rock, generating non-durable rock fragments. The first type may be placed and compacted as a rock fill in 1 m lifts, whereas the second type should be placed and compacted in thinner lifts, with watering and compaction similar to that required for an earth fill.

Additional detail on construction of rock fills derived from strong rock or weaker rock may be found in Cooke (1990) and US Army Corps of Engineers (2005).

For the purpose of this report, rock fill is divided into two categories - durable rock fill; and, non-durable rock fill - each with different anticipated engineering properties, sources, and placement and compaction requirements. Most of the metasedimentary rock excavated at the site will yield non-durable rock fill. Relatively unweathered granodiorite from the pit area, and quartzite from the hornfels aureole around the granodiorite intrusion, are expected to yield durable rock fill.

Structural Fill

Structural Fill is an engineered soil material placed and compacted for use beneath lightly to moderately loaded structures to provide a uniform bearing surface with tolerable movements under load through the life of the structure.

Structural Fill should consist of well graded sand and gravel having a maximum size of 75 mm and less than 8% fines (materials passing the No. 200 sieve) and be free of all deleterious materials including frozen soils, clay lumps and organics. All structural fill should be placed and compacted to at least 95% Modified Proctor Maximum Dry Density (MPMDD). Placement and compaction should be performed in moisture-conditioned lifts less than 300 mm of loose thickness with equipment suitable to obtain the specified density.

Materials that do not satisfy the specifications for structural fill may be used as structural fill in specific applications, at the discretion of a qualified geotechnical engineer. For example, locally excavated weathered rock that contain more than 8 % fines may serve as structural fill provided compaction objectives can be met and drainage/frost susceptibility issues are less important, e.g., used only at depth in thick fills.

General Fill

General Fill is an inorganic granular material used for general site grading, thermal insulation cover and/or protection of pipes, or similar applications. Materials should be limited to maximum 200 mm particle size, and contain no more than 20% fines. General fill should be compacted to yield a stiff surface as determined acceptable to the geotechnical engineer by proof-rolling with fully loaded dump trucks. General Fill should not be used for support of settlement-sensitive structures.

Grading Fill

This is a soil material used as an intermediate layer between in-situ soil or rock subgrade and higher quality engineering materials above, such as road base, for example. Any granular material that can be placed and compacted to 95 % MPMDD to provide a uniform bearing surface may be suitable for this purpose. Selected materials should have a maximum particle size of 150 mm. Oversize materials may be screened out, or can be removed from the surface of placed materials by hand. Suitable materials would include and materials identified as suitable for structural fill or general fill, and may include local colluvium.

Rip rap

Riprap consists of cobble and boulder size rock fragments, typically angular or subangular as derived from blasting or crushing, and is used as a protective barrier from erosion and scour due to water currents and/or ice. Material should consist of hard, durable rock fragments free from splits, seams or defects that could impair its soundness. Thicknesses of riprap layers typically vary from 1.0 to 1.5 times the maximum rock size. Riprap is typically specified by the median particle size, D_{50}. Additional grain size criteria may be presented if the riprap needs to be either well graded or uniformly graded, depending on the specific application. Preliminary information from Tetra Tech suggests there will be a need for riprap with D_{50} of about 500 to 600 mm.

Drainage Material

This is an open or gap-graded granular material intended for allowing free drainage of fluids to pipes and/or seepage collection systems. Drainage material should consist of crushed or uncrushed screened rock or gravel free of fines and flat, elongated particles. Grain size requirements depend on the specific drainage application.

Filter/transition Material

Filters are a transition zone material used for preventing soil migration due to fluid flow between granular materials, and/or between rock fill and finer silt and clay layers. Filter material gradations are generally designed based on the specific material gradations that they will transition. Filter materials can be derived from rock excavations or gravel borrow areas, and may require crushing, screening and/or washing to attain the necessary gradations.

Concrete Aggregate

Concrete aggregate includes fine and coarse aggregate meeting CSA A23.1 specifications for designing and proportioning concrete mix. Aggregates can be derived from crushed durable rock or gravel.

Road Base

This is an engineered material, consisting of a well-graded, hard, durable, very clean (less than 5% fines), screened and crushed sand and gravel or rock, with a maximum particle size of 38 mm. Material should be free of flat and elongated pieces and have a minimum of 50 % fractured particle faces. Road base gravel should also have less than 25% loss by Micro-Deval. Road base materials should be placed and compacted to a minimum of 98% MPMDD.

Road Surfacing Material

Road surfacing material should consist of well-graded hard, durable, angular screened and crushed sand and gravel or rock with less than 15% fines, and maximum particle size of 25 mm. Granular material should have less than 25% loss by Micro-Deval and greater than 50% fractured faces.

Heap Overliner Material

The heap leach pad will include a protective layer of crushed gravel over the primary liner and solution collection piping system, known as the heap overliner. As specified by Tetra Tech on an email dated in November 23, 2011, the overliner drain fill shall consist of free-draining granular material with 38 mm maximum particle size and a maximum of 5 percent fines passing the No. 200 ASTM sieve size (0.075-mm). The material shall be free of organic matter and soft, friable particles in quantities objectionable to the geotechnical engineer. The drain fill shall have a hydraulic conductivity of $2x10^{-4}$ m/sec or higher when tested in accordance with the constant-head method described in ASTM D 2434, using a hydraulic gradient of 1.

7.4. Temporal Material Demand and Material Balance

7.4.1. General

This project will involve the movement of large quantities of earth and rock fill in a relatively short construction period (currently understood to be about three years) and within a limited footprint in rugged terrain. It will be challenging to manage material movement to meet the construction schedule. An effort has been made to understand the temporal nature of planned material movement, drawing from material take offs (MTOs) provided by Wardrop, Tetra Tech, BGC and Knight Piésold, as compiled by Merit Consultants and received on January 06, 2012.

Table 7-1 presents a breakdown of material quantities over time, based on an analysis of quarterly supply and demand. Cut quantities are shown as positive numbers, being quantities available for use (or intended for disposal). Fill quantities are shown, in brackets, as negative numbers, being quantities required for construction. Material supply/demand and total cut/fill balance are also illustrated graphically in Figure 1.

The current analysis shows a peak excess of approximately 2.2 million cubic metres of excavated material which will require storage during the first year of the project, as shown in Table 7-1. This excess supply will be drawn down over the following year, leaving a small excess of available fill at the end of construction.

The material categories listed in Table 7-1 correspond to categories provided in the MTOs from Merit Consultants and Wardrop received on January 06, 2012.

7.4.2. Excess Materials Requiring Storage or Disposal

Bulk earthworks activities will generate several types of material that are unsuitable for immediate use, or may not be suitable for any use, thus necessitating temporary storage or permanent disposal. Decisions on ultimate disposition may require further consideration of the need for soil for reclamation. Preliminary information suggests the development of the following materials requiring storage or disposal:

- Topsoil – these materials will be required for reclamation. It will be necessary to develop stockpiles to store these materials during construction and mine operation. The current estimate of 313,000 m^3 does not yet include open pit pre-stripping;
- Ice-rich permafrost – these materials will be unsuitable for immediate re-use in any application. They may be suitable for re-use in reclamation after thawing and draining of excess water. These materials will require careful storage after excavation and prior to use, as they will be weak and unstable when thawed. It may be necessary to develop specific storage areas with containment structures and water management infrastructure. Current estimates indicate approximately 40,000 m^3 of ice-rich permafrost will be removed during development of the heap leach facility, and with additional volumes from other areas on site (quantity currently unknown), all requiring management during construction and mining operations;
- Colluvium – some of the shallow colluvial soils removed during bulk excavation work will contain excessive amounts of deleterious materials, such as organic inclusions or excess proportions of fines. Current estimates suggest approximately 227,000 m^3 of colluvium requiring permanent disposal or storage for re-use in reclamation.
- Waste rock - these materials are indicated by Merit and Wardrop as unsuitable for re-use as construction fills and are intended to be permanently disposed in designated disposal areas. In general they correspond to soils or rock with deleterious materials and may include excess fines or excess ice. Current estimates indicate approximately 500,000 m^3 of unsuitable material that needs to be excavated, removed and disposed, either in the waste rock storage areas, or other disposal areas to be determined.

Table 7-1. Quarterly Demand for Cut and Fill Quantities, as inferred from MTOs from Merit, Wardrop and Tetra Tech

Used for Material Balance	Category	Material Quantity (m^3)												
		Q4 2012	Q1 2013	Q2 2013	Q3 2013	Q4 2013	Q1 2014	Q2 2014	Q3 2014	Q4 2014	Q1 2015	Q2 2015	Q3 2015	Total
No	Strip and stockpile topsoil	50,738	26,026	147,437	0	0	0	37,015	7,701	0	0	44,485	0	**313,402**
	Excavate and dispose waste rock in waste dump	77,319	0	261,201	0	0	0	54,344	35,234	0	0	67,842	0	**495,940**
	Excavate colluvium	35,050	0	168,300	0	0	0	18,000	5,700	0	0	0	0	**227,050**
	Excavate rock	10,758	0	375,555	0	0	0	4,278	0	0	0	0	271,369	**661,960**
	Excavate permafrost	3,500	0	34,900	0	0	0	0	1,200	0	0	0	0	**39,600**
	Local cut and fill	76,791	0	239,749	0	0	0	45,210	13,251	0	0	0	839,463	**1,214,464**
Yes	Excavate and stockpile suitable materials	208,271	0	185,885	0	0	0	0	24,632	0	0	0	133,699	**552,487**
	General excavation	333,280	0	1,182,390	0	0	0	75,400	120,000	0	0	0	0	**1,711,070**
	Excavate placer tailings	0	0	876,000	0	0	0	0	0	0	0	0	0	**876,000**
	Subgrade preparation	0	0	(18,300)	(104,600)	0	0	0	(3,500)	0	0	0	0	**(126,400)**
	Other materials	(3,520)	0	(58,823)	(3,100)	0	0	(12,000)	(298,000)	0	0	0	0	**(375,443)**
	Fill from stockpile	(18,110)	0	(355,643)	0	0	0	(149,191)	(17,461)	0	0	0	(7,430)	**(547,835)**
	Fill	(70)	0	(126,518)	(1,119,000)	0	0	(743,000)	0	0	0	0	0	**(1,988,588)**
Material balance - each quarter		519,851	0	1,684,991	(1,226,700)	0	0	(828,791)	(174,329)	0	0	0	126,269	**101,291**
Material balance - cumulative		519,851	519,851	2,204,842	978,142	978,142	978,142	149,351	(24,978)	(24,978)	(24,978)	(24,978)	101,291	**101,291**

Note:

1. Quantities (in brackets) indicate deficit quantities, or fill to be derived from elsewhere.

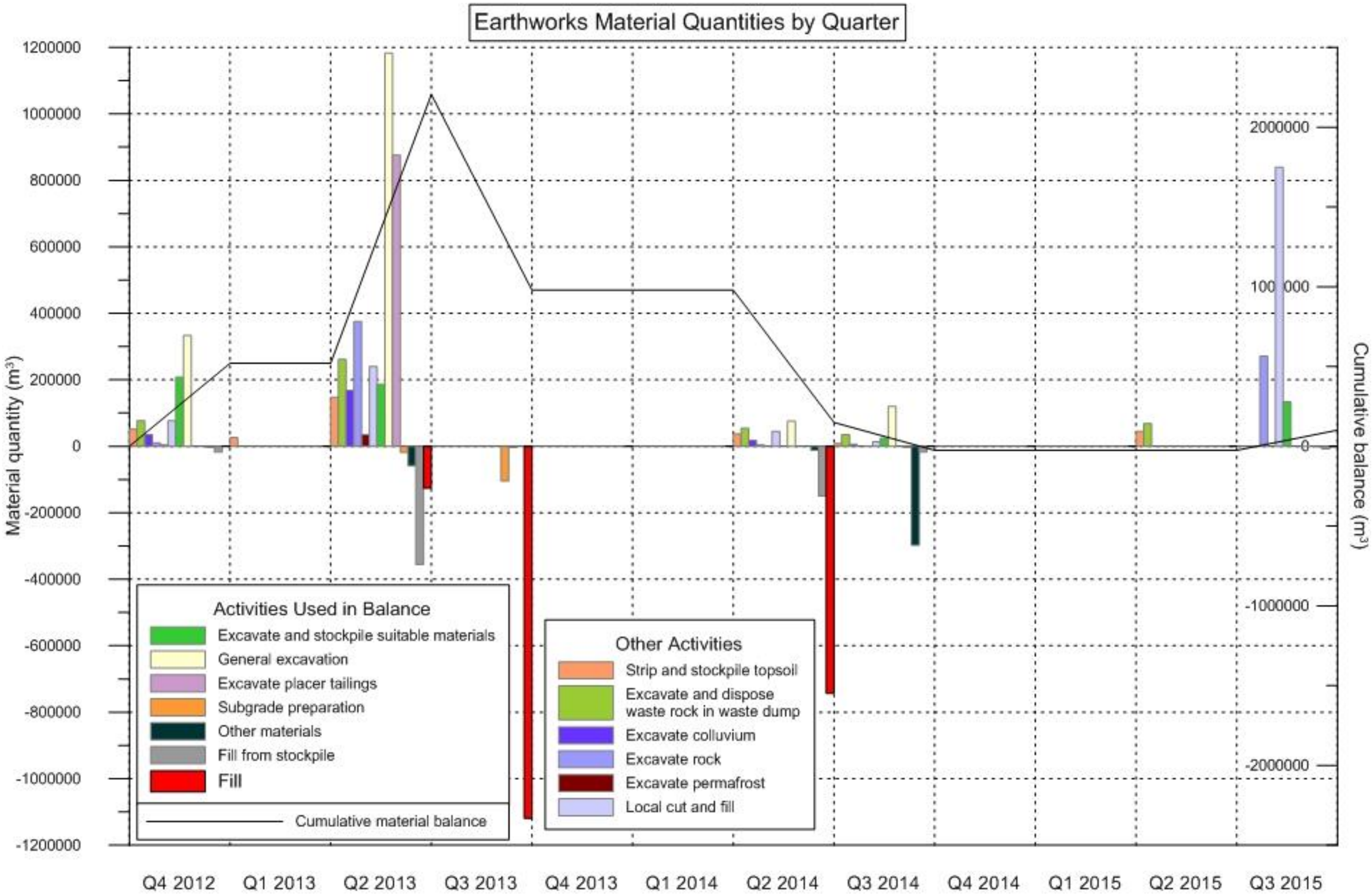

Figure 1. Quarterly Material Demand and Balance

7.5. Available Borrow Materials

7.5.1. General

Several sources of borrow material were identified in the BGC Borrow Evaluation Report (BGC 2011c). This included two potential silt borrow pits near the proposed laydown area and near the confluence of Platinum Gulch and Haggart Creek; the existing placer tailings in the Dublin Gulch valley bottom; and, proposed platform cuts into bedrock along sloping ground. Additional work was conducted in 2011, including investigation of the placer tailings; investigation of potential silt borrow near the proposed laydown area, evaluation of various rock sources for use as engineered fill; and, evaluation of placer tailings and rock near the proposed open pit for potential use as concrete aggregate.

The distribution of placer tailings in Dublin Gulch and Haggart Creek valley bottoms is illustrated in Drawing 27. The locations of potential borrow sources, including the placer tailings, rockfill sources and the proposed silt borrow, are illustrated in Drawing 28. Summary information for these various borrow sources is presented in Table 7-2.

7.5.2. Rock Sources

Rock will be required for use as rock fill (durable and non-durable), rip rap, and for crushing to produce concrete aggregate and heap overliner material.

Most of the rock encountered at the project site consists of weak, friable metasedimentary rock, suitable only for use as non-durable rock fill, or general fill. Such rock will be available in moderate quantities for local re-use from most major cuts for building pad development. A larger quantity is expected to be available from the “Ann Gulch central knob,” an area of extensive cutting within the first phase of heap pad subgrade development.

Durable rock may be available in very small quantities (i.e. several hundred to a few thousand cubic metres) from the oversize materials screened out of the placer tailings in Dublin Gulch valley bottom, and in larger quantities from weathered granodiorite in the open pit pre-strip or the Steiner zone. Suitability for use of these materials as concrete aggregate requires further testing and analysis, as preliminary data suggest that the rock will not meet normal standards for concrete aggregate. Expert advice will be required to determine whether the local rock materials can be used as concrete aggregate with admixtures to counteract the known limitations. It may be possible to use local rock as concrete aggregate without admixtures with more careful selection, e.g. using only granodiorite from the pit pre-strip; however, this alternative also requires tighter controls and further testing.

7.5.3. Placer Tailings

The placer tailings are found within the Dublin Gulch and Haggart Creek valley bottoms and consist of reworked materials from historical placer mining operations.

The distribution of materials within the placer tailings was examined by field reconnaissance, and the spatial distribution of typical material types is illustrated in Drawing 27. The visual observations of surficial materials are supported by test hole observations and associated lab testing in Dublin Gulch area. Interpretations in the Haggart Creek area are based solely on visual observations of surficial materials.

Examination of the surface topography of the tailings and the approximate bedrock surface, as inferred from test hole locations, suggests that approximately 2 million m^3 of fill materials are present in the Dublin Gulch valley bottom and potentially exploitable for use elsewhere as an engineering material. Note that if all of these materials are exploited to expose bedrock, it may be necessary to replace a significant quantity of material to restore grades in the pond development area to a level above the existing valley bottom drainage system. The net quantity of potentially available exploitable materials, currently present above the elevation of the creeks, is roughly 1.1 million m^3. An additional 750,000 m^3 of placer tailings is available in the Haggart Creek area above the level of Haggart Creek.

Producing engineered fills from the placer tailings will require targeted selection combined with crushing, screening and/or washing.

Removal of placer tailings down to bedrock, which may be necessary to provide an adequate foundation subgrade, will require significant efforts for dewatering.

7.5.4. Silt Borrow

Exploration for potential silt borrow was conducted in the general vicinity of the proposed laydown area, near the location of the existing exploration camp. The 2011 investigation work included four auger holes and six test pits.

Compacted samples of silt obtained from the vicinity of the proposed silt borrow area yielded a mean permeability of 4.5×10^{-8} cm/s at 95 % MPMDD, based on four tests. Note that these results are lower than anticipated and should be checked through further testing.

It should be noted that ice-rich permafrost is present to at least 25 m depth in the proposed silt borrow area, and the thickness of suitable silty material is, on average, approximately 4-5 m, ranging from about 2 m to more than 15 m thickness. The excavated silt material will need to be thawed and dried before use. Screening may be required to remove oversize particles.

It is estimated that up to about 220,000 m^3 of silty materials may be obtained from the indicated silt borrow area.

Table 7-2. Summary of Borrow Material Availability.

Borrow Source	Material Types	Estimated Volumes (in situ volumes, except where noted)	Comments
Pit Pre-Strip	Durable rock fill Non-durable rock fill Concrete aggregate Heap overliner Rip rap	Very large. Available volumes depend on the sequence of mining activities, although materials can be developed prior to mining activities by developing a quarry prior to pre-strip.	Source consists of weathered granodiorite and weathered silicified metasedimentary rock (i.e. typically quartzite). Suitable concrete aggregate has not yet been identified, and requires further study focussing on the granodiorite. Testing of material for use as heap overliner was commissioned by Tetra Tech, and the results are not available to BGC at the time of writing. Availability of rip rap in desired block size of 500-600 mm will require further input from mining, and careful selection. Most near surface weathered rock suggests excavated block size of approximately 100-300 mm.
Ann Gulch Central Knob	Non-durable rock fill	Up to approximately 900,000 m^3, subject to further input from Tetra Tech.	Grading plans showing the volumes of anticipated rock excavation are not available to BGC at the time of writing.
Steiner Zone	Same as for Pit Pre-strip	Up to approximately 200,000 m^3, assuming quarry depth of 5 m	Very little information is known about this area. Further subsurface investigation is required to confirm quality and quantity of available materials.
Dublin Gulch Placer Tailings	General Fill Structural Fill Concrete aggregate Heap overliner Rip rap	Approximately 2.0 million m^3, of which about 1.1 million m^3 is above the groundwater level	Materials are highly variable, and will require processing through screening, crushing and/or washing to develop the required material specifications. Oversized materials (> 75 mm) screened from the tailings may be suitable for use, after crushing, as heap overliner or concrete aggregate pending further analysis. Some rip rap (perhaps up to 2-3,000 m^3) can be developed from the screened oversize material, however the quantity of 500-600 mm particles is expected to be small and would require careful selection.
Haggart Creek Placer Tailings	General Fill Structural Fill	Approximately 750,000 m^3 available above the elevation of Haggart Creek	No subsurface information is available to support the quantity estimate. Available volume of suitable material is estimated from visual classification of surficial materials present in several distinct piles.
Silt Borrow	Silt liner	Approximately 220,000 m^3	Available silt materials are frozen and ice-rich, and will require thawing and drying prior to use.

8.0 RECOMMENDATIONS FOR FURTHER INVESTIGATION

This report has provided feasibility study level geotechnical recommendations for mine site infrastructure. There are several areas where additional investigation is recommended to provide sufficient data for subsequent detailed design. The following list provides recommendations for further investigation. This list should be read in conjunction with Drawing 29.

- Diamond drill holes:
  - Vertical holes at all three crushers to better establish depth to suitable bearing stratum across the facilities' footprints;
  - Inclined holes in the area of proposed rock cuts at the crushers and 100 day storage pad;
  - Vertical holes at the plant site to better determine depth to suitable bearing stratum within the extent of the building pad;
  - Allowance for additional holes within the footprint of the heap leach facility, in the event Tetra Tech considers additional data warranted;
  - Allowance for additional holes at major cuts such as that along the phase 1 heap access road;
  - Allowance for holes for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Auger holes (with CRREL barrel available):
  - Conveyor bent foundation locations between tertiary crusher and heap leach facility;
  - Along the alignment of the proposed Dublin Gulch diversion channel;
  - In Eagle Pup to confirm the extent of the ice-rich lobate feature in the valley bottom;
  - At the revised truck shop buildings and cut locations;
  - Allowance for holes in areas being considered for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Plate load tests at plant site and all three crushers;
- Design and construction of a test fill embankment to determine whether high quality structural fill would be suitable to support the secondary and tertiary crushers;
- Sampling and strength testing of materials selected for heap embankment fill, if considered necessary by Tetra Tech;

- Additional sampling and testing of granodiorite from pit area and Steiner zone for possible use as concrete aggregate. Obtain materials engineering advice to guide this process, including trial mix designs possibly with additives to make use of local aggregates and trial design mixes for lean concrete for use in raising grades at crushers;
- Sample mixes for low strength concrete as stabilized fill

9.0 CLOSURE

We trust the above satisfies your requirements at this time. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

BGC ENGINEERING INC.
per:

Daniela Welkner, M.Sc.
Senior Engineering Geologist

Pete Quinn, Ph.D., P.Eng.
Senior Geotechnical Engineer

Reviewed by:

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Senior Civil Engineer

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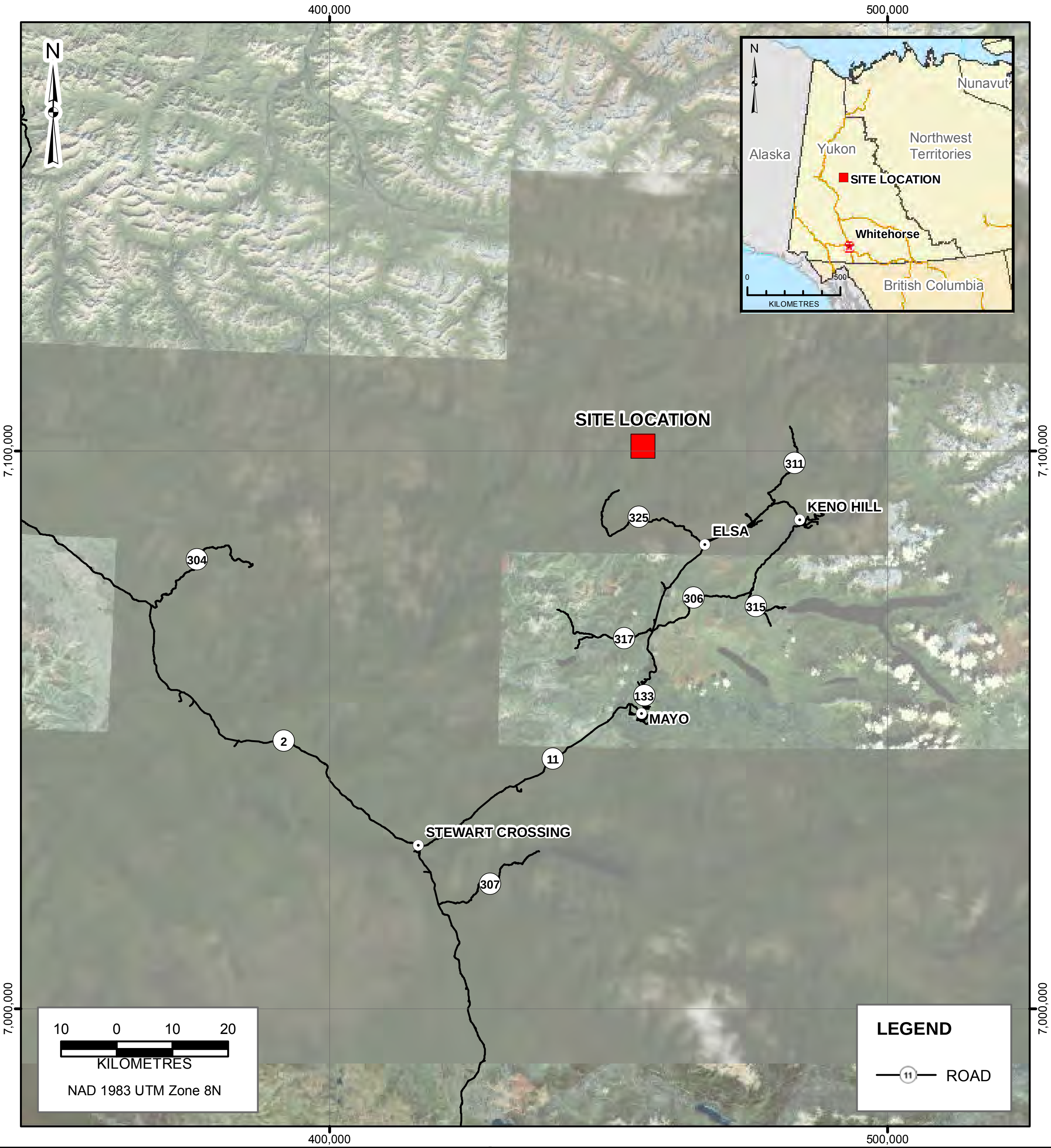
DRAWINGS



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TITLE:
GENERAL SITE LOCATION

CLIENT:
VICTORIA GOLD CORPORATION

PROJECT No.:	DWG No.:	REV.:
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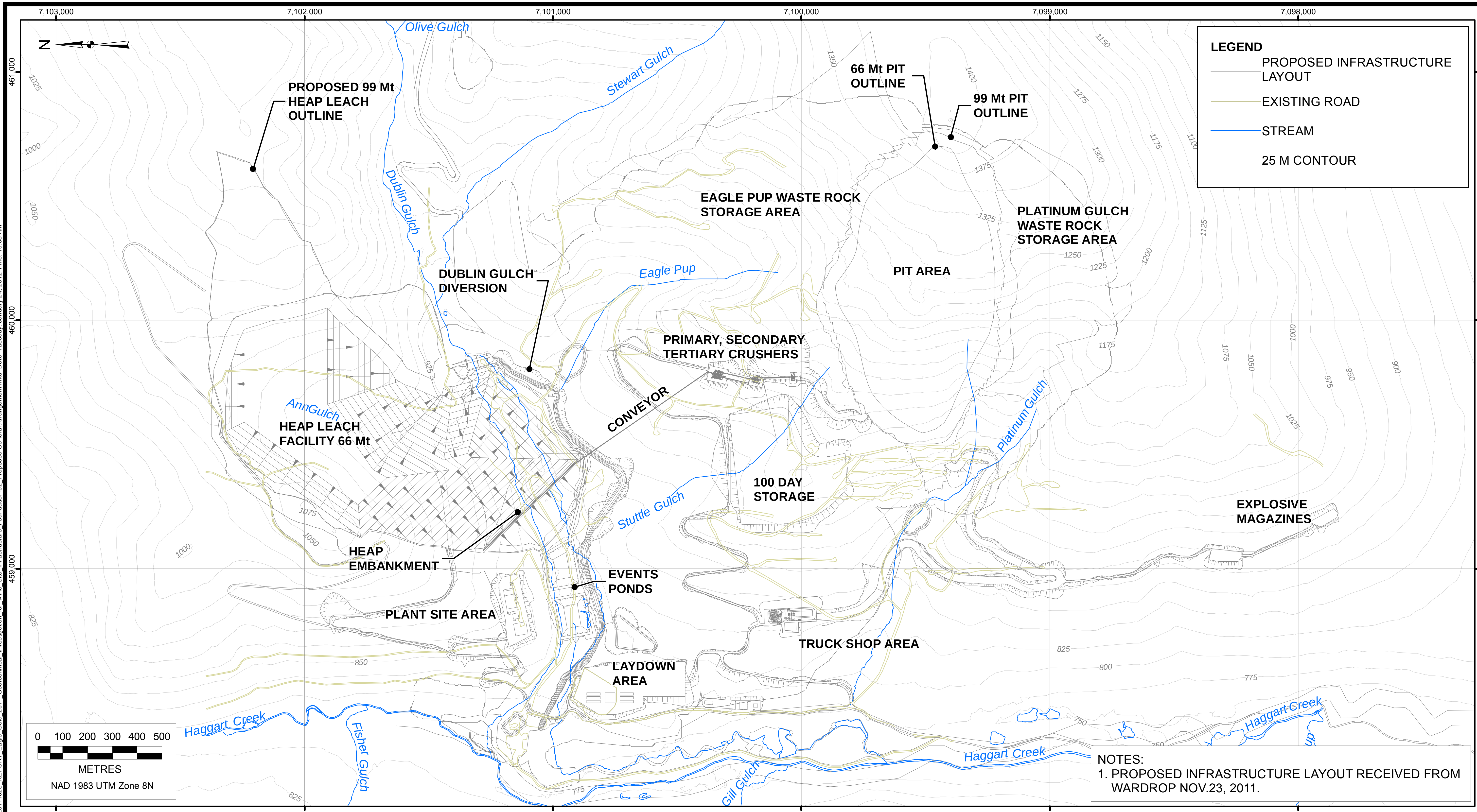

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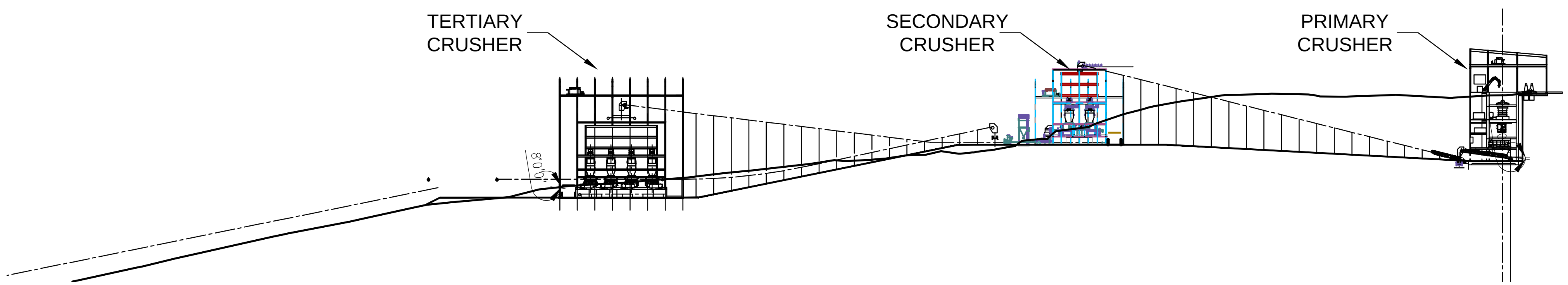

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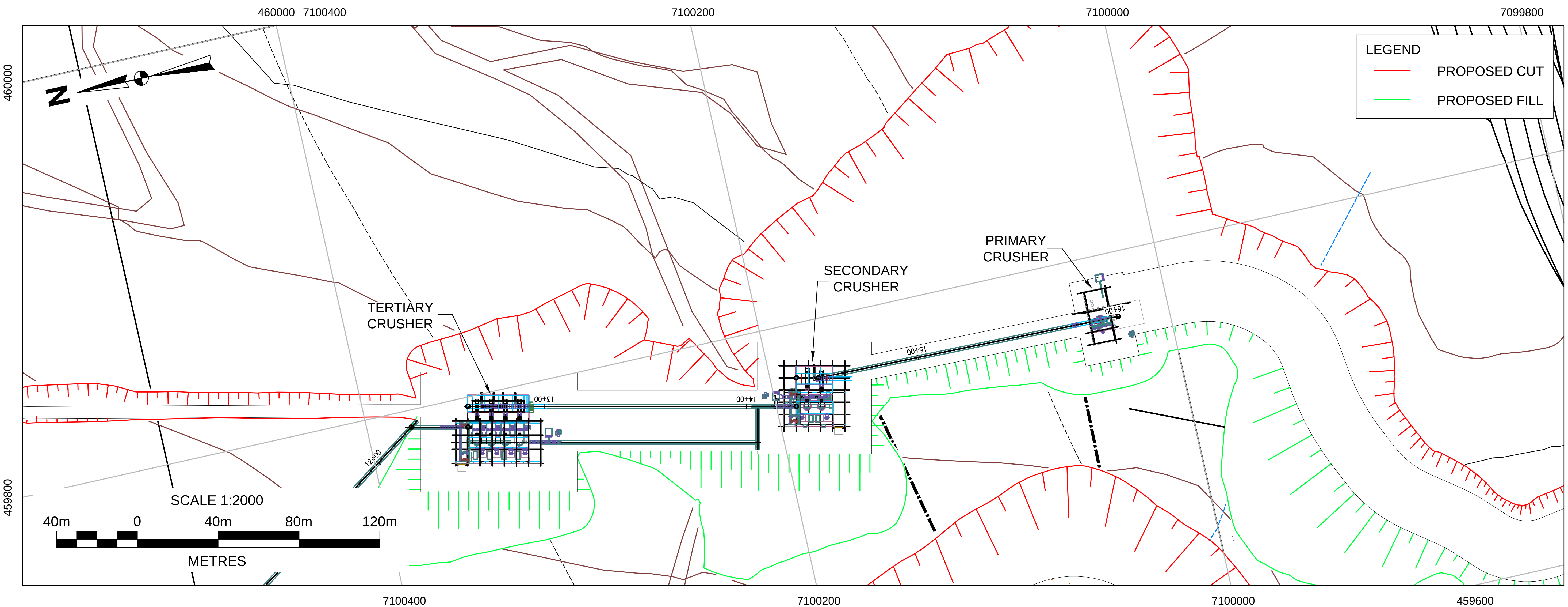

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TITLE: PROPOSED ARRANGEMENT - CRUSHERS

PROJECT No.:	DWG No.	REV.:
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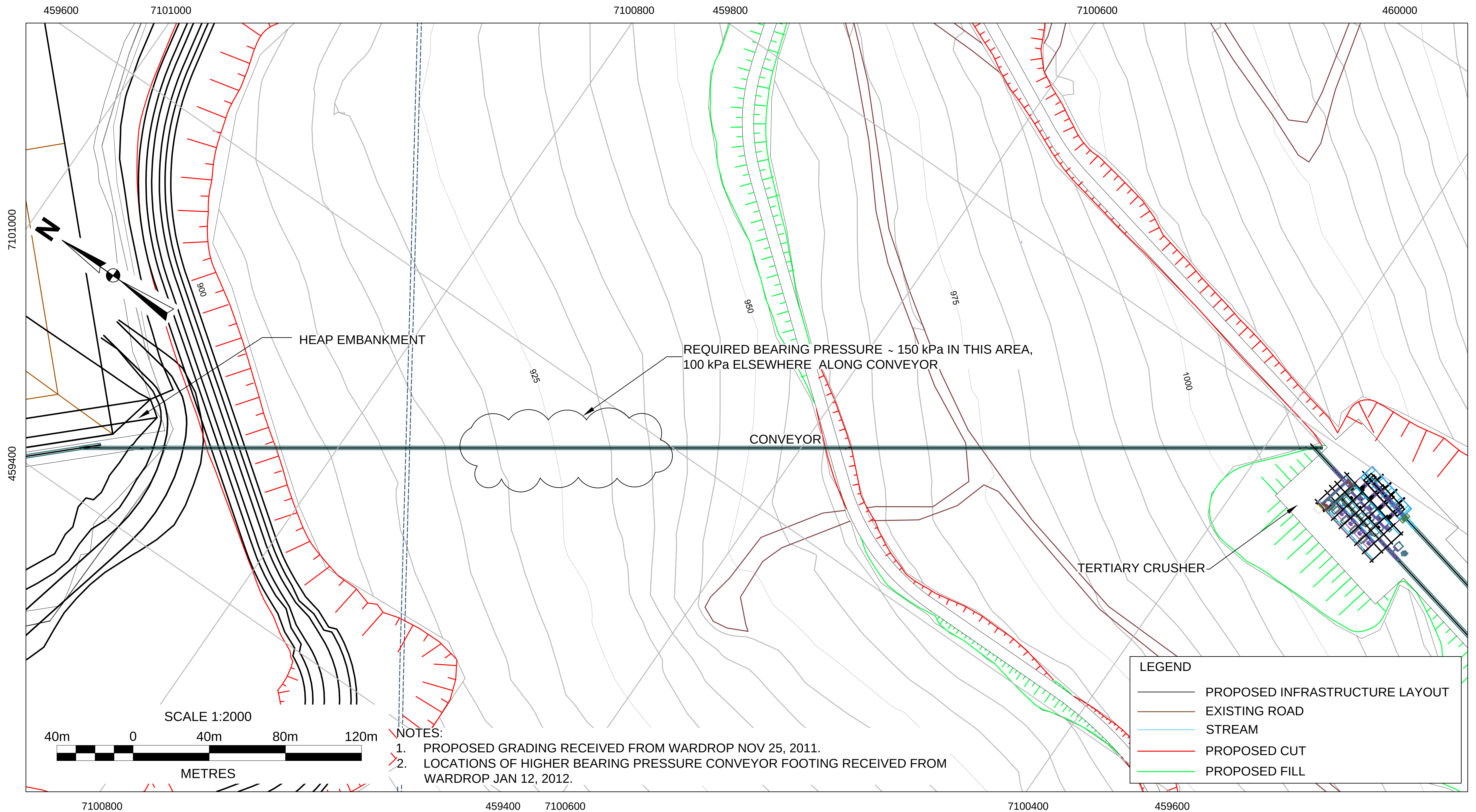

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TITLE: PROPOSED GENERAL ARRANGEMENT CONVEYORS

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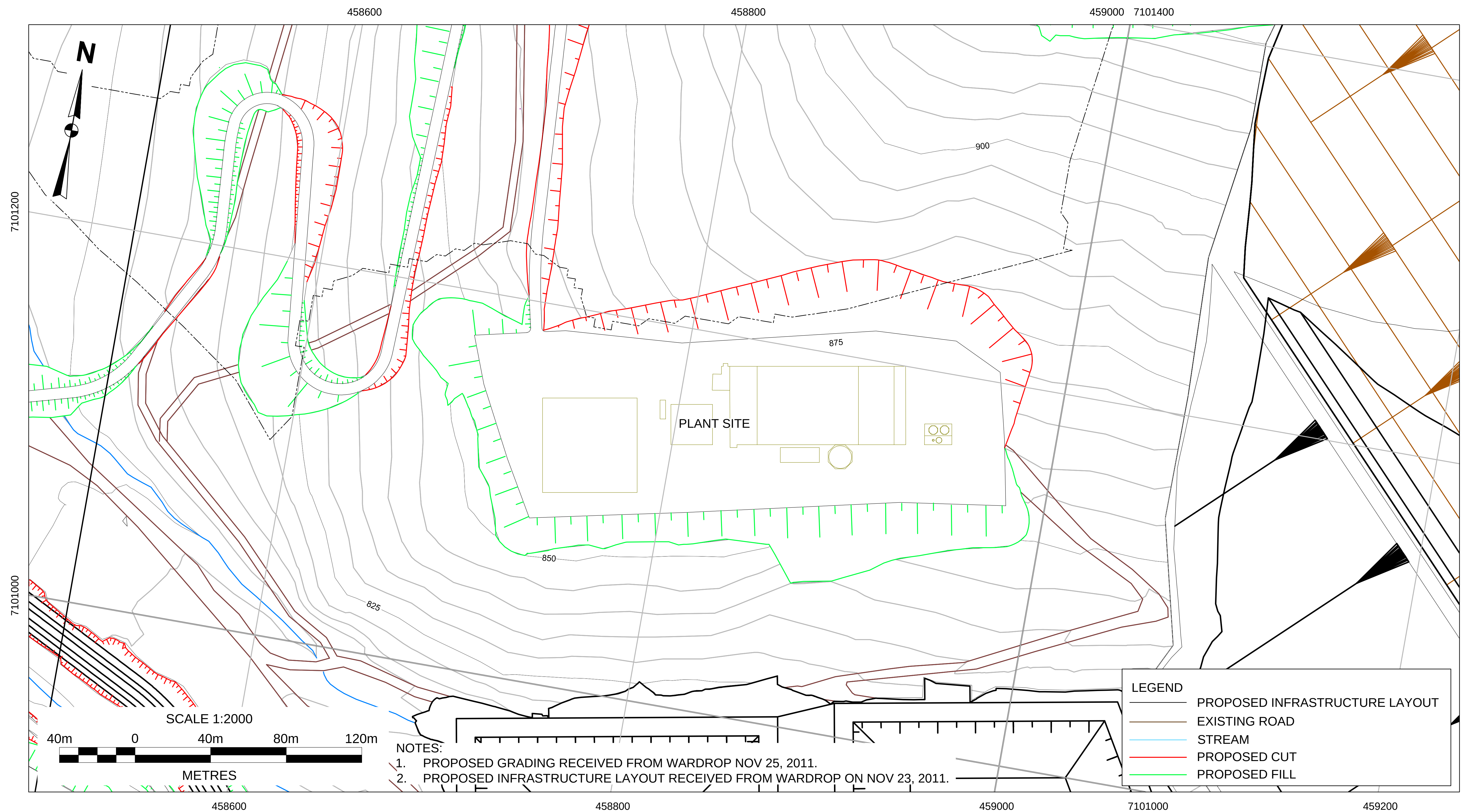

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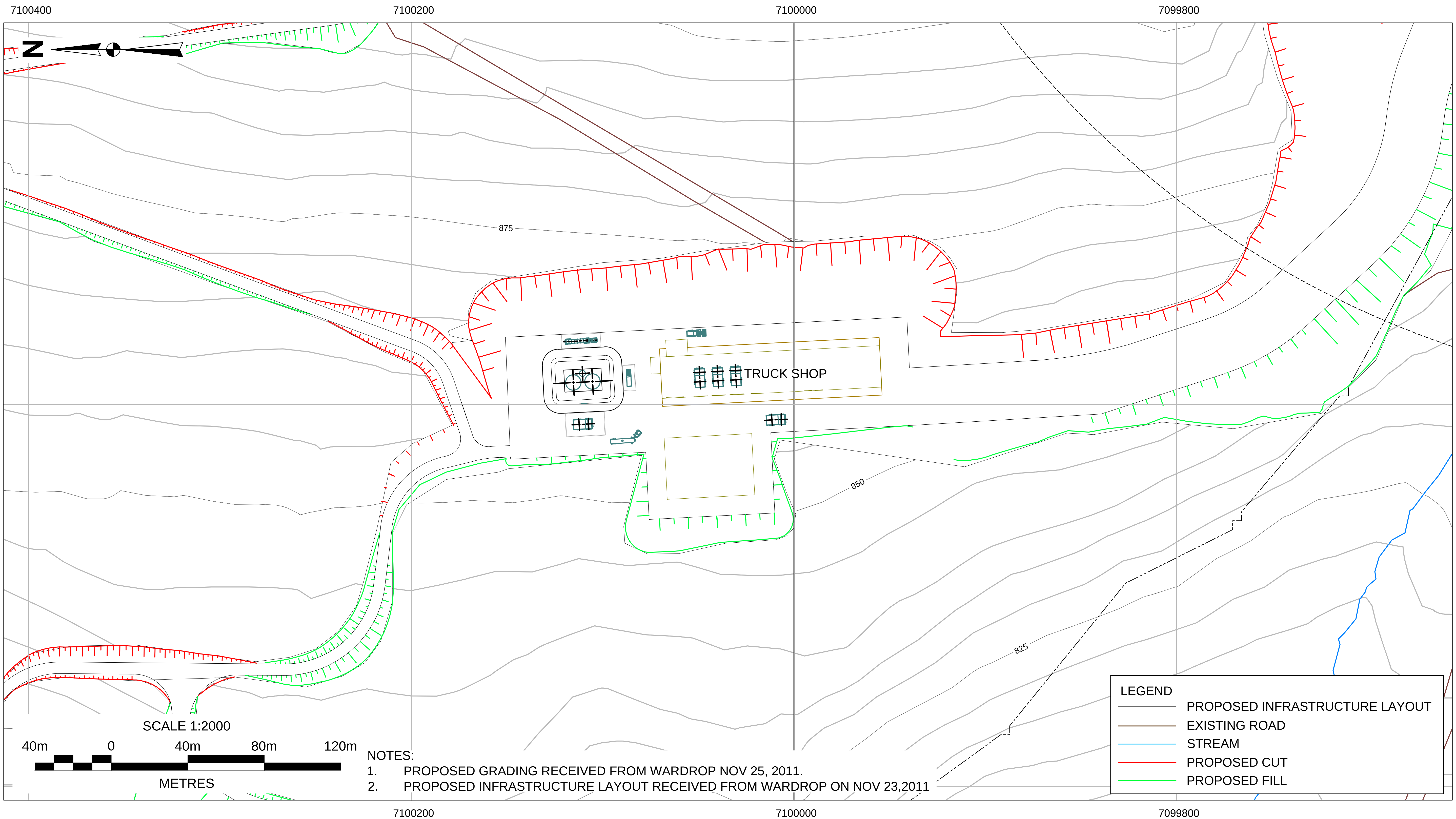

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TITLE:
PROPOSED GENERAL ARRANGEMENT
TRUCK SHOP

PROJECT No.:	DWG No.:	REV.:
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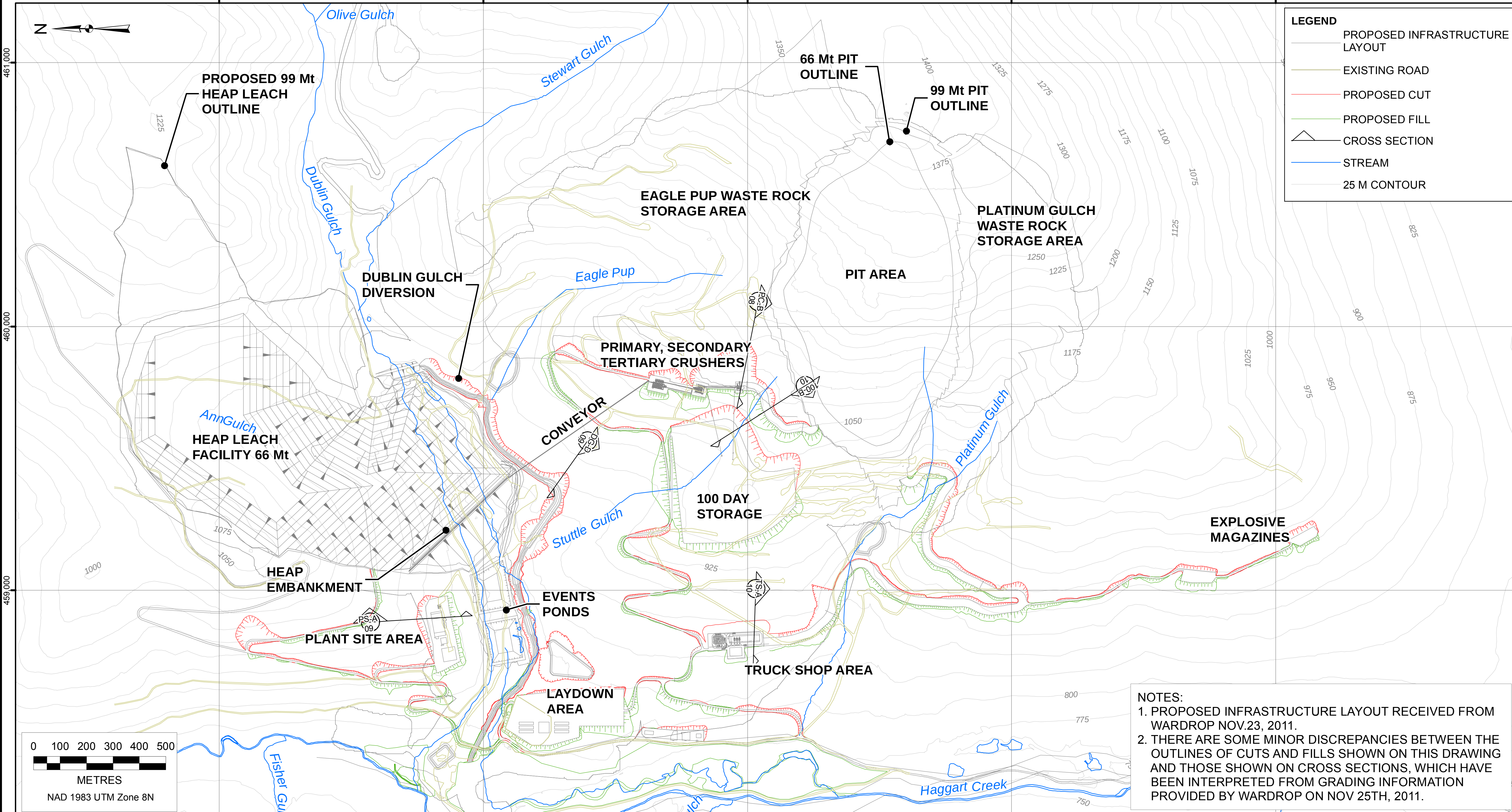

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: PLANNED CUT SLOPES

PROJECT No.: 0792-006
DWG No.: 07
REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

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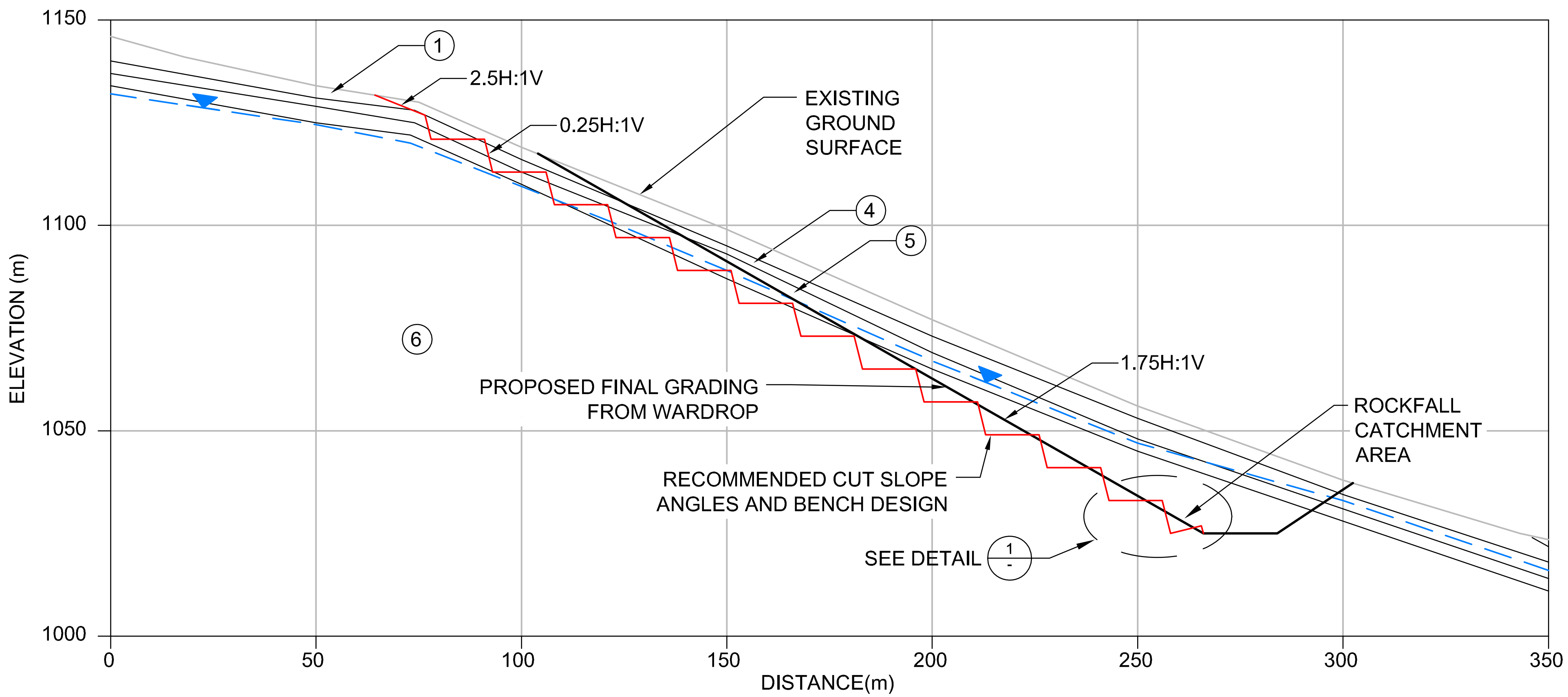
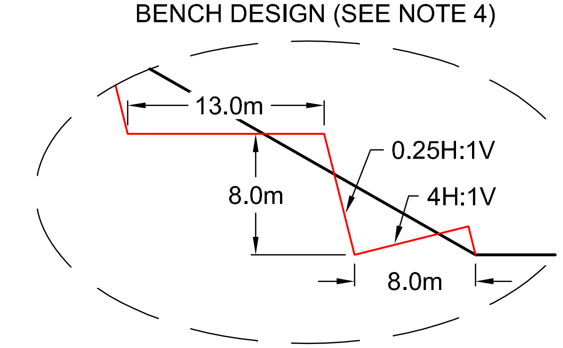


BENCH DESIGN

CROSS SECTION PC-B 07

RECOMMENDED SLOPE ANGLES AND BENCH DESIGN (SEE NOTE 4)



DETAIL 1 -

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METRES

NOTES:

1. PROPOSED FINAL GRADING PLAN RECEIVED FROM WARDROP ON NOV. 25, 2011.
2. INFERRED SUBSURFACE CONDITIONS ARE IDEALIZED FROM AVAILABLE TEST HOLE INFORMATION, AND MAY VARY FROM THAT SHOWN.
3. GROUNDWATER LEVELS ARE INFERRED FROM LIMITED INFORMATION AND CAN BE EXPECTED TO VARY FROM THE LINE SHOWN ON THIS CROSS-SECTION.
4. THE BENCH DESIGN NEEDS TO MAINTAIN THE RECOMMENDED OVERALL SLOPE ANGLE OF 1.75H:1V.

ASSUMED SUBSURFACE CONDITIONS

1	COLLUVIUM
4	TYPE 3 ROCK
5	TYPE 2 ROCK
6	TYPE 1 ROCK
	WATER LEVEL
	GROUND SURFACE
	PROPOSED FINAL GRADING RECEIVED FROM WARDROP
	RECOMMENDED CUT SLOPE ANGLES

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
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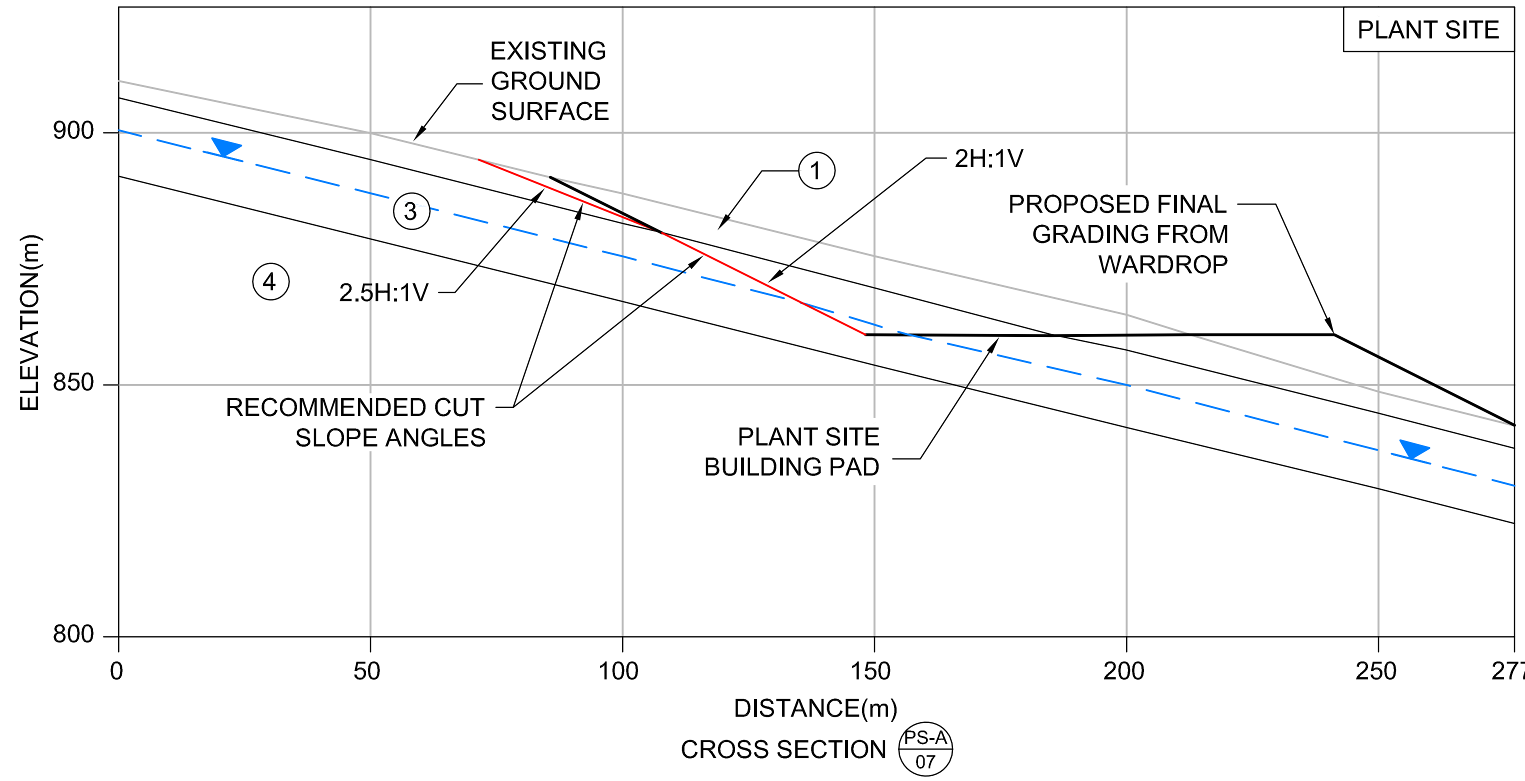
CLIENT: VICTORIA GOLD CORPORATION

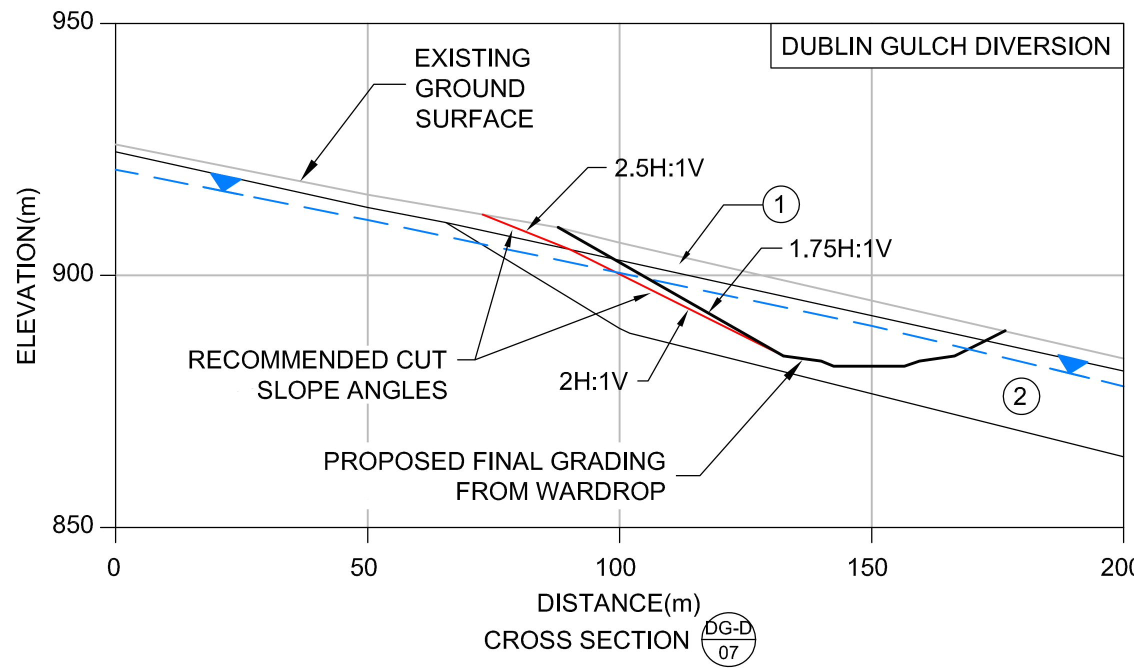
PROJECT: EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: CUT SLOPE CROSS-SECTION - PRIMARY CRUSHER

PROJECT No.:	DWG No.:	REV.:
0792006	08	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012





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1. PROPOSED FINAL GRADING PLAN RECEIVED FROM WARDROP ON NOV. 25, 2011.
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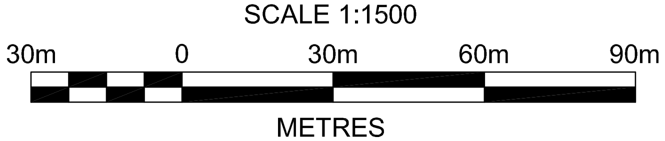

ASSUMED SUBSURFACE CONDITIONS

- 1 COLLUVIUM
- 4 TYPE 3 ROCK
- 5 TYPE 2 ROCK
- 6 TYPE 1 ROCK
- WATER LEVEL
- GROUND SURFACE
- PROPOSED FINAL GRADING RECEIVED FROM WARDROP
- RECOMMENDED CUT SLOPE ANGLES

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PROJECT: EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: CUT SLOPE CROSS-SECTIONS - PLANT SITE AND DUBLIN GULCH DIVERSION

PROJECT No.:	DWG No.:	REV.:
0792006	09	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

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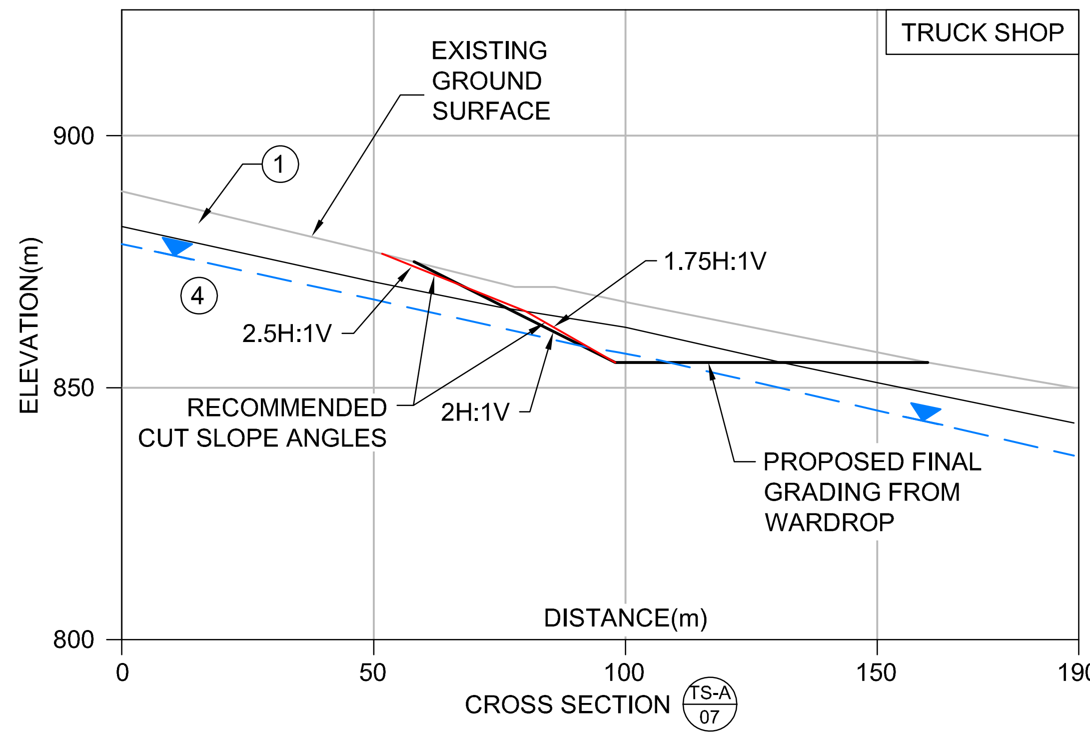

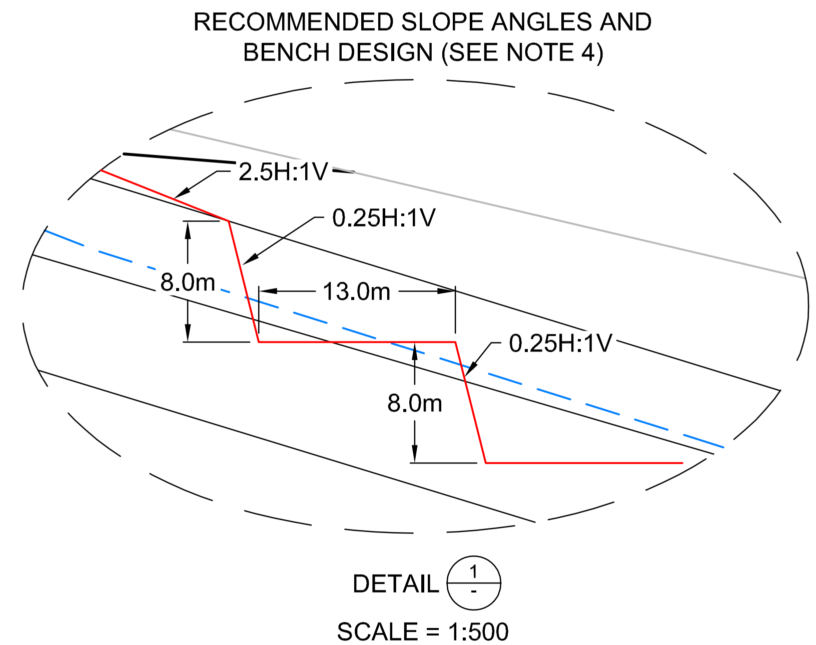

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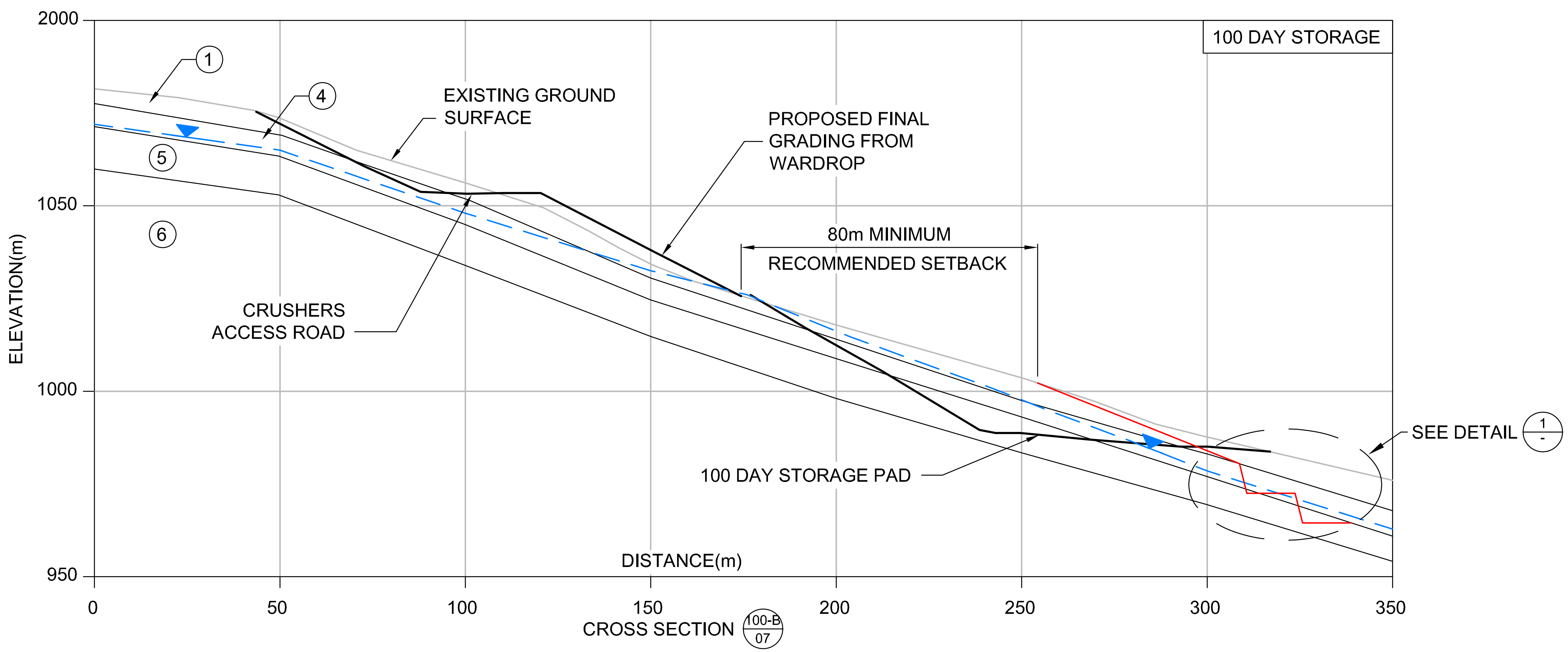
3. GROUNDWATER LEVELS ARE INFERRED FROM LIMITED INFORMATION AND CAN BE EXPECTED TO VARY FROM THE LINE SHOWN ON THIS CROSS-SECTION.
4. THE BENCH DESIGN NEEDS TO MAINTAIN THE RECOMMENDED OVERALL SLOPE ANGLE OF 1.75H:1V.



SCALE 1:1500

30m 0 30m 60m 90m

METRES

ASSUMED SUBSURFACE CONDITIONS

1	COLLUVIUM
4	TYPE 3 ROCK
5	TYPE 2 ROCK
6	TYPE 1 ROCK
(blue triangle)	WATER LEVEL
(grey line)	GROUND SURFACE
(black line)	PROPOSED FINAL GRADING RECEIVED FROM WARDROP
(red line)	RECOMMENDED CUT SLOPE ANGLES

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	01/31/2012	ISSUED WITH FINAL REPORT	GCB	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	PB	DW	PQ

SCALE:	AS SHOWN
DATE:	JAN 2012
DRAWN:	PB/GCB
DESIGNED:	EB
CHECKED:	DW
APPROVED:	PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT-
2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: CUT SLOPE CROSS-SECTIONS -
TRUCK SHOP AND 100 DAY STORAGE

PROJECT No.:	DWG No.:	REV.:
0792006	10	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

NOTES:
1. GENERAL ARRANGEMENT RECEIVED FROM WARDROP NOV.23, 2011.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE:	1:7,500
DATE:	JAN 2012
DRAWN:	LL
DESIGNED:	GH
CHECKED:	DW
APPROVED:	PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: AVAILABLE SUBSURFACE DATA

PROJECT No.:	DWG No.:	REV.:
0792-006	11	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:7,500
DATE: JAN 2012
DRAWN: LL
DESIGNED: GH
CHECKED: PQ
APPROVED: SQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

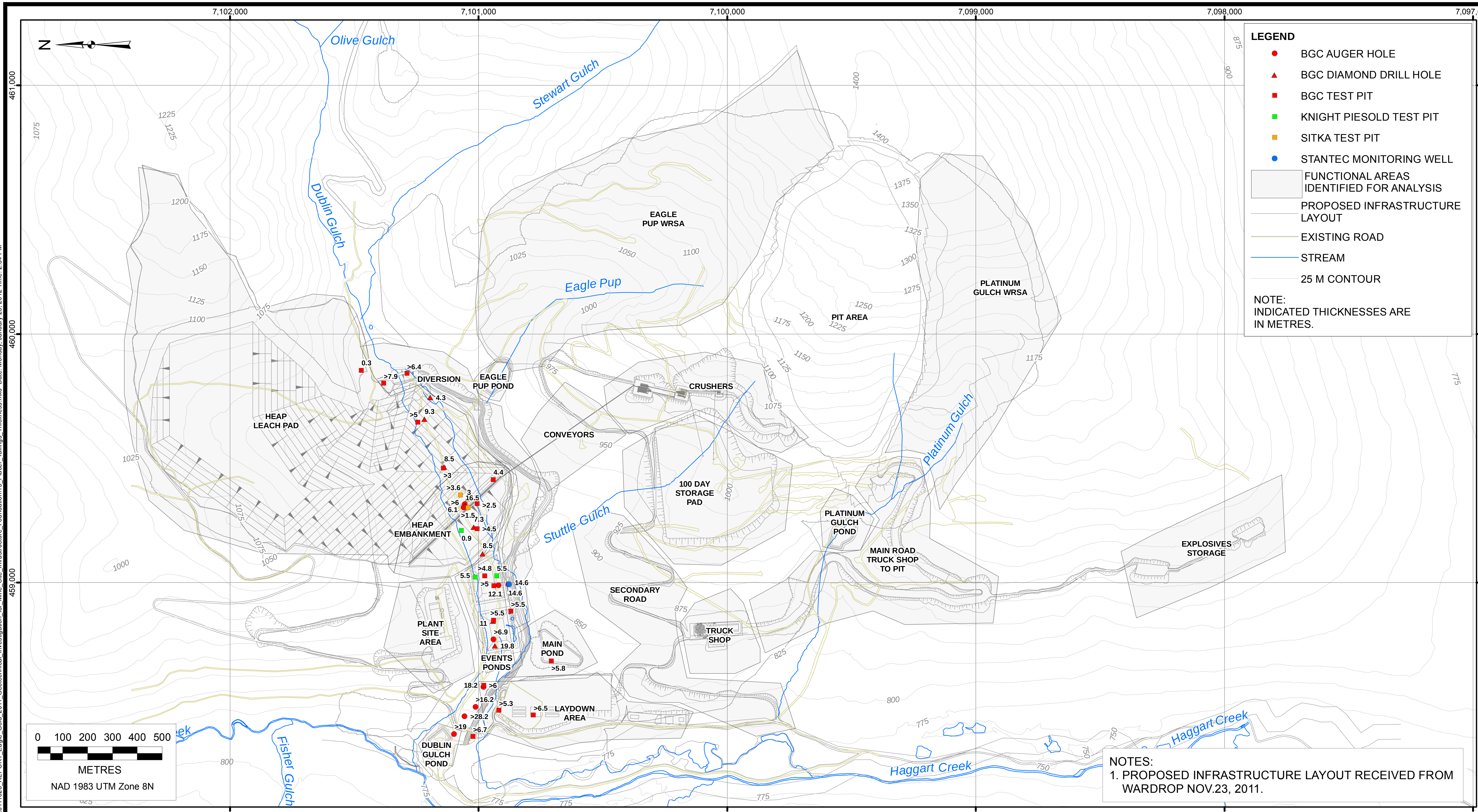
CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: OVERBURDEN THICKNESS

PROJECT No.: 0792-006
DWG No.: 12
REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:15,000
DATE: JAN 2012
DRAWN: LL
DESIGNED: GH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: PLACER TAILINGS THICKNESS

PROJECT No.: 0792-006
DWG No.: 13
REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

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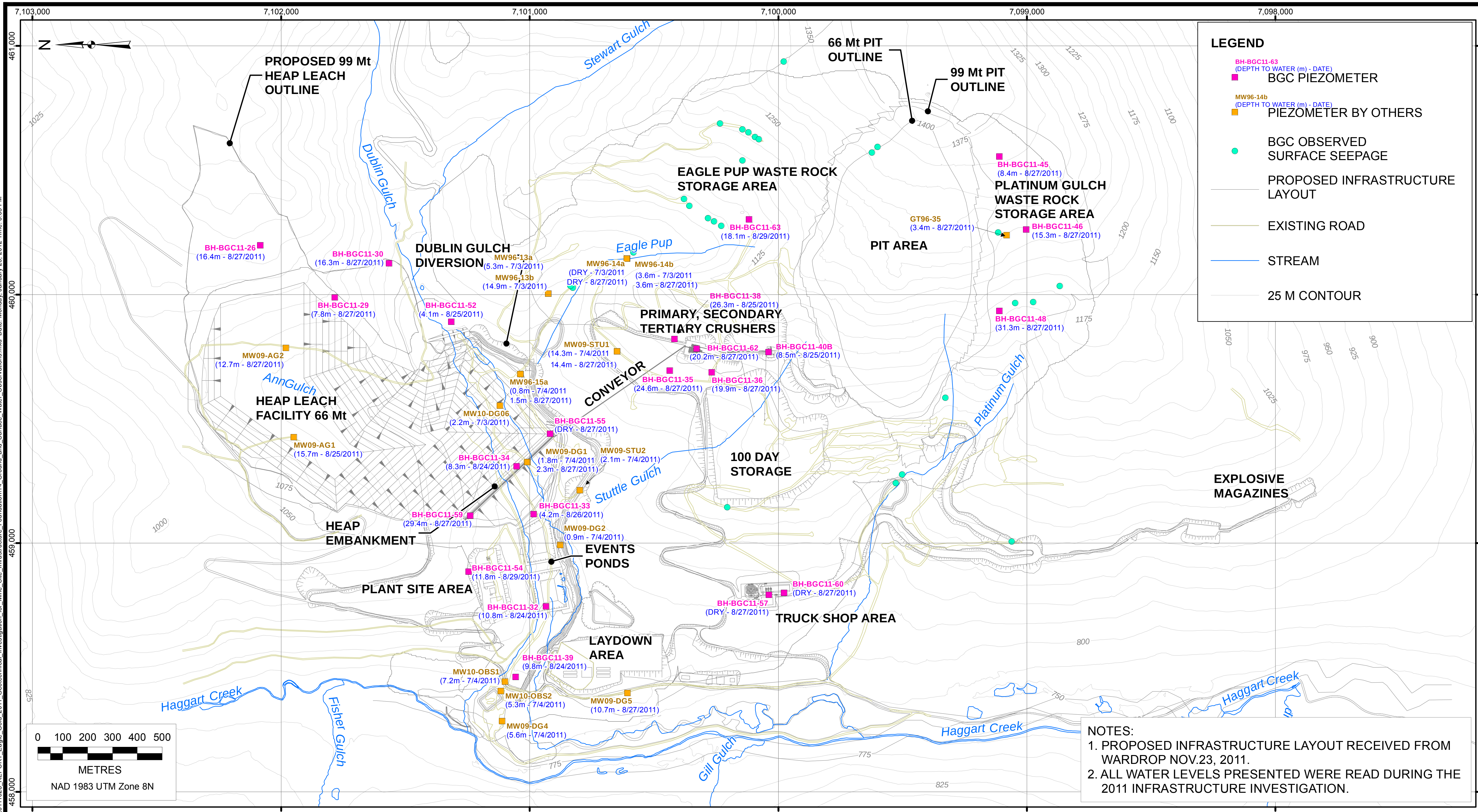

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REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:15,000
DATE: JAN 2012
DRAWN: LL
DESIGNED: KH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: GROUND AND SURFACE WATER OBSERVATIONS

PROJECT No.: 0792-006 | DWG No.: 14 | REV.: 0

X:\Projects\0792 Victoria Gold\006\Workspace\20111028_REPORT_Eagle_Gold_2011_Geotechnical_Investigation_for_Mine_Site_Infrastructure_Foundation\14_Ground_and_Surface_Water_Observations.mxd Date: Monday, January 23, 2012 Time: 3:05 PM

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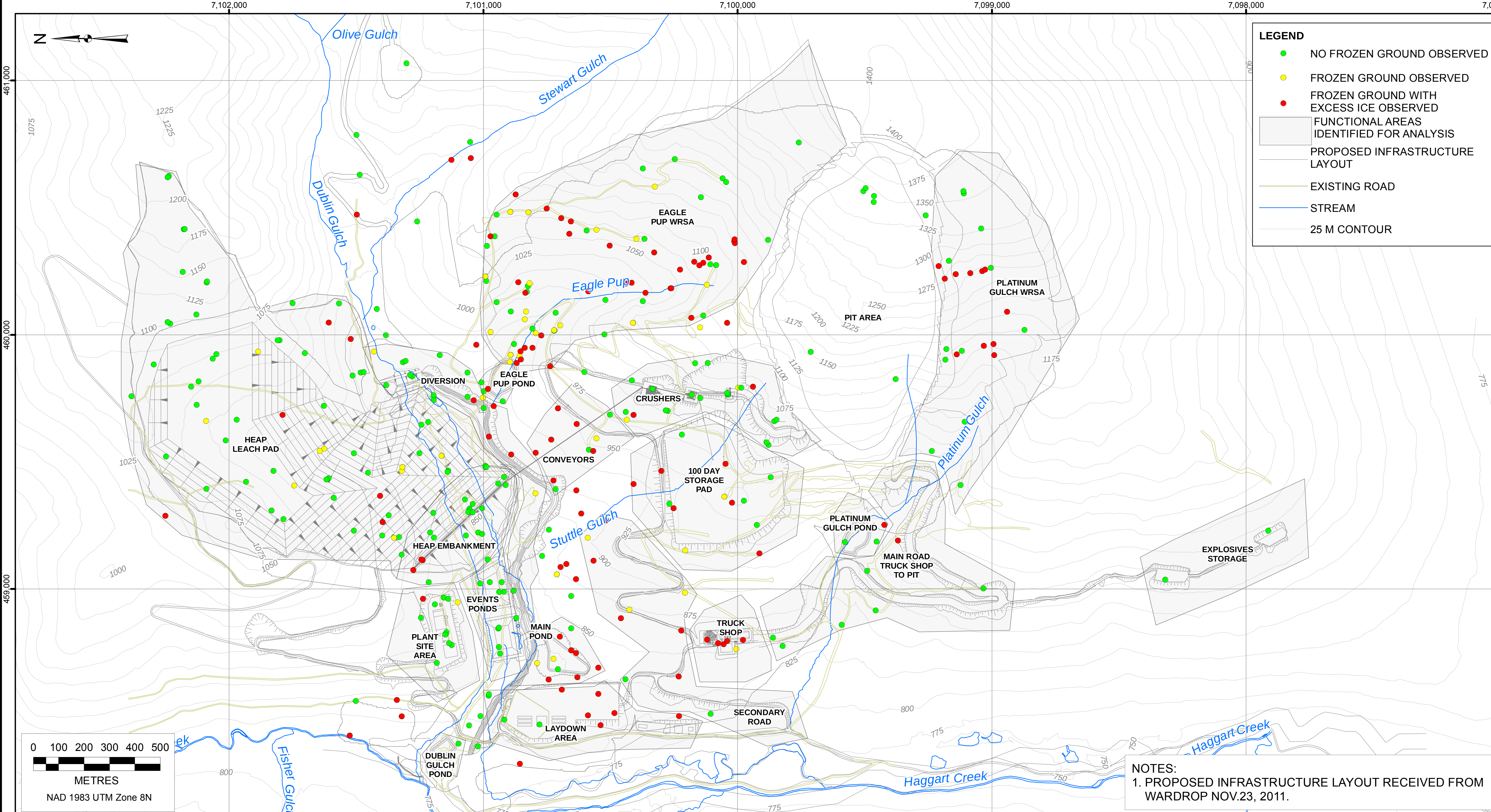

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REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:15,000
DATE: JAN 2012
DRAWN: LL
DESIGNED: GH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

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AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

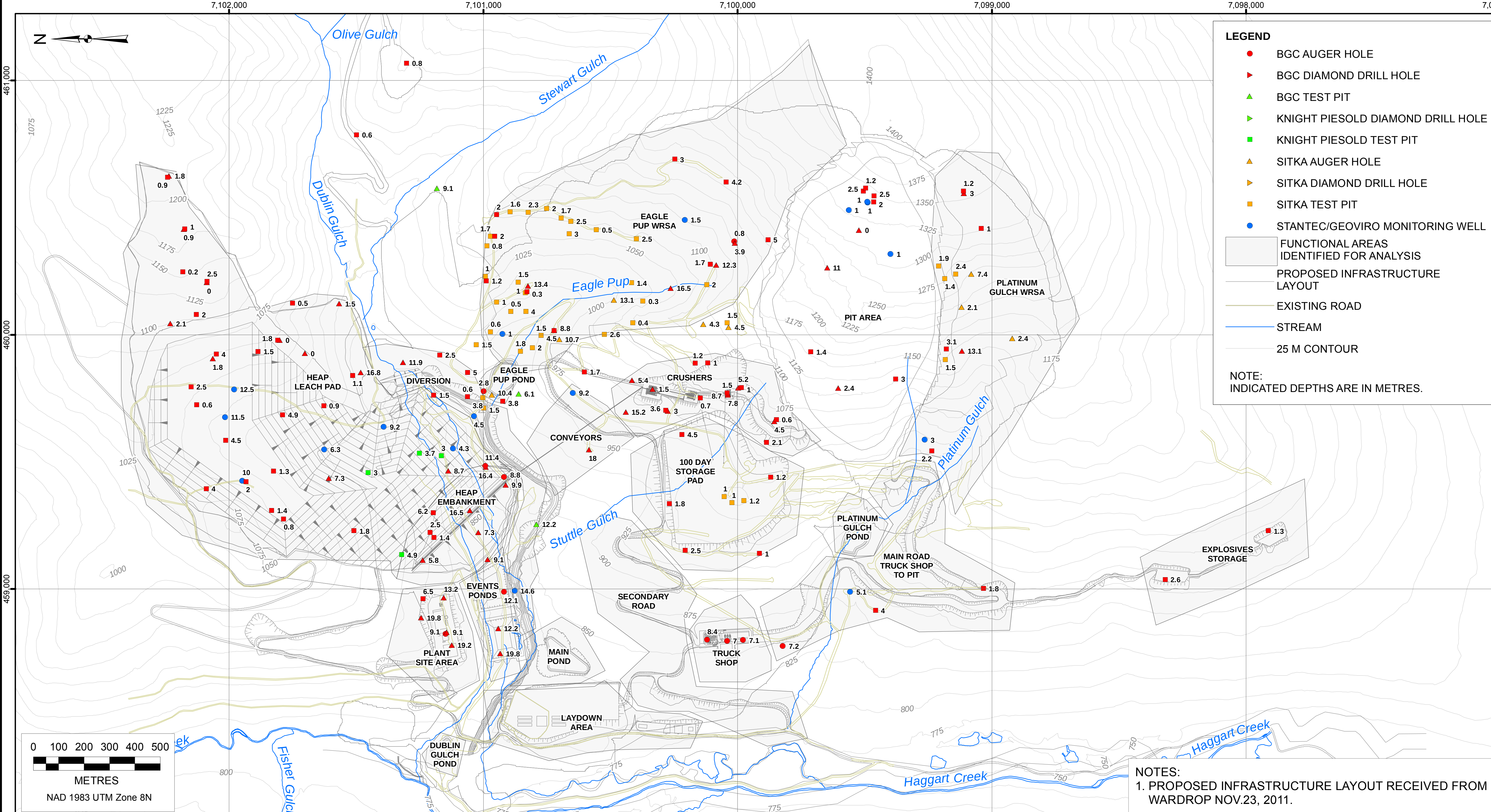
PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: FROZEN GROUND AS OBSERVED IN TEST HOLES

PROJECT No.: 0792-006
DWG No.: 15
REV.: 0

X:\Projects\0792 Victoria Gold\006\Workspace\20111028_REPORT_Eagle_Gold_2011_Geotechnical_Investigation_for_Mine_Site_Infrastructure_Foundation\15_Frozen_Ground_Observations.mxd Date: Monday, January 23, 2012 Time: 2:36 PM

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



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REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE:	1:15,000
DATE:	JAN 2012
DRAWN:	LL
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APPROVED:	PQ

PROFESSIONAL SEAL:

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AN APPLIED EARTH SCIENCES COMPANY

CLIENT:
VICTORIA GOLD CORPORATION

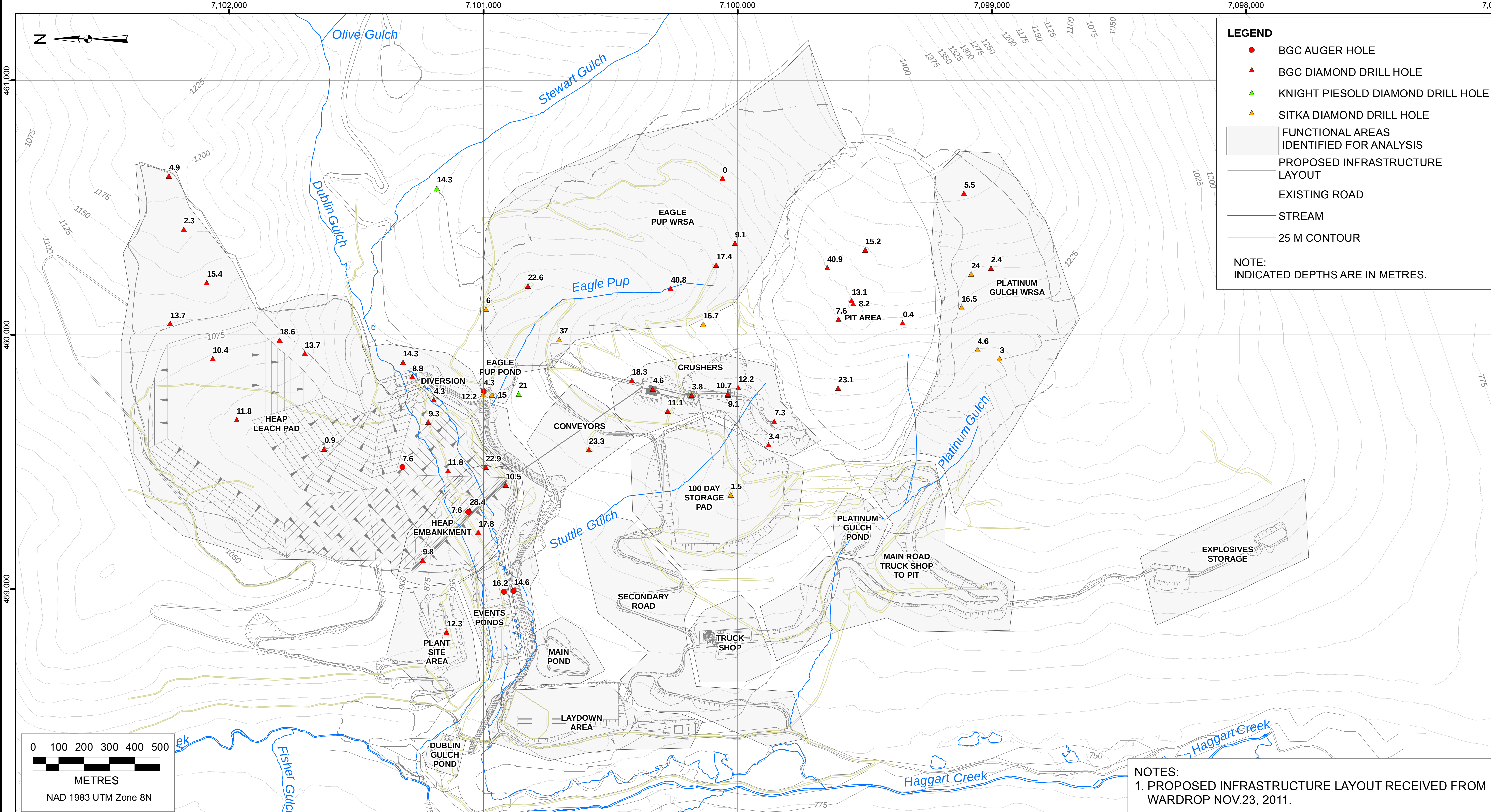
PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
DEPTH TO TYPE 3 ROCK

PROJECT No.:	DWG No.:	REV.:
0792-006	16	0

X:\Projects\0792 Victoria Gold\006\Workspace\20111028_REPORT_Eagle_Gold_2011_Geotechnical_Investigation_for_Mine_Site_Infrastructure_Foundation\16_Depth_to_Type_3_Rock.mxd Date: Monday, January 23, 2012 Time: 2:36 PM

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



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REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:15,000
DATE: JAN 2012
DRAWN: LL
DESIGNED: GH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

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AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: DEPTH TO TYPE 2 ROCK

PROJECT No.: 0792-006
DWG No.: 17
REV.: 0

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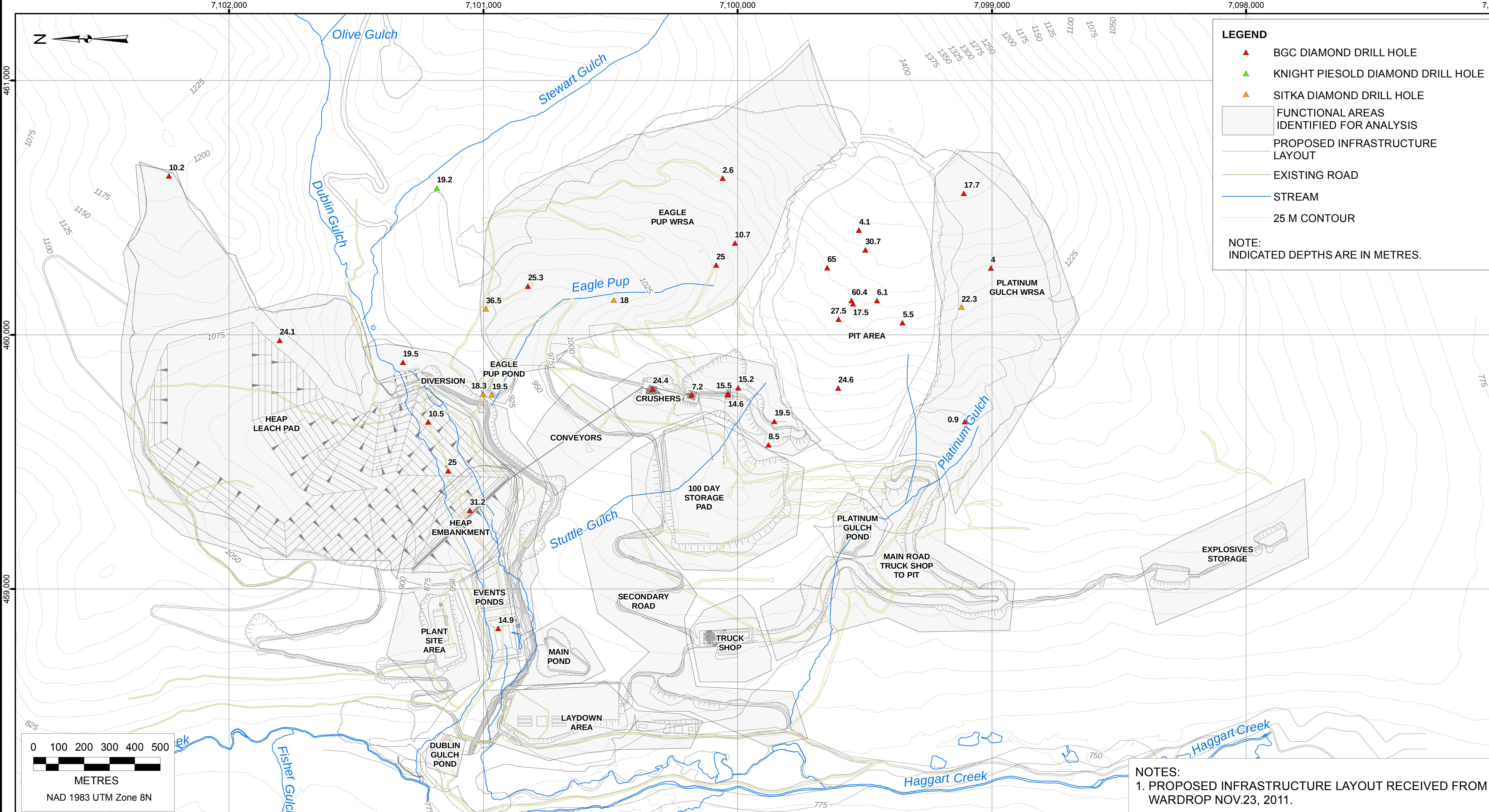
DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ
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SCALE: 1:15,000
DATE: JAN 2012
DRAWN: LL
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CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: DEPTH TO TYPE 1 ROCK

PROJECT No.: 0792-006
DWG No.: 18
REV.: 0

X:\Projects\0792 Victoria Gold\006\Workspace\20111028_REPORT_Eagle_Gold_2011_Geotechnical_Investigation_for_Mine_Site_Infrastructure_Foundation\18_Depth_to_Type_1_Rock.mxd Date: Monday, January 23, 2012 Time: 2:37 PM

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

0 200 400 600 800
METRES

NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:7,500
DATE: JAN 2012
DRAWN: LL
DESIGNED: GH
CHECKED: DW
APPROVED: PQ
PROFESSIONAL SEAL:

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AN APPLIED EARTH SCIENCES COMPANY

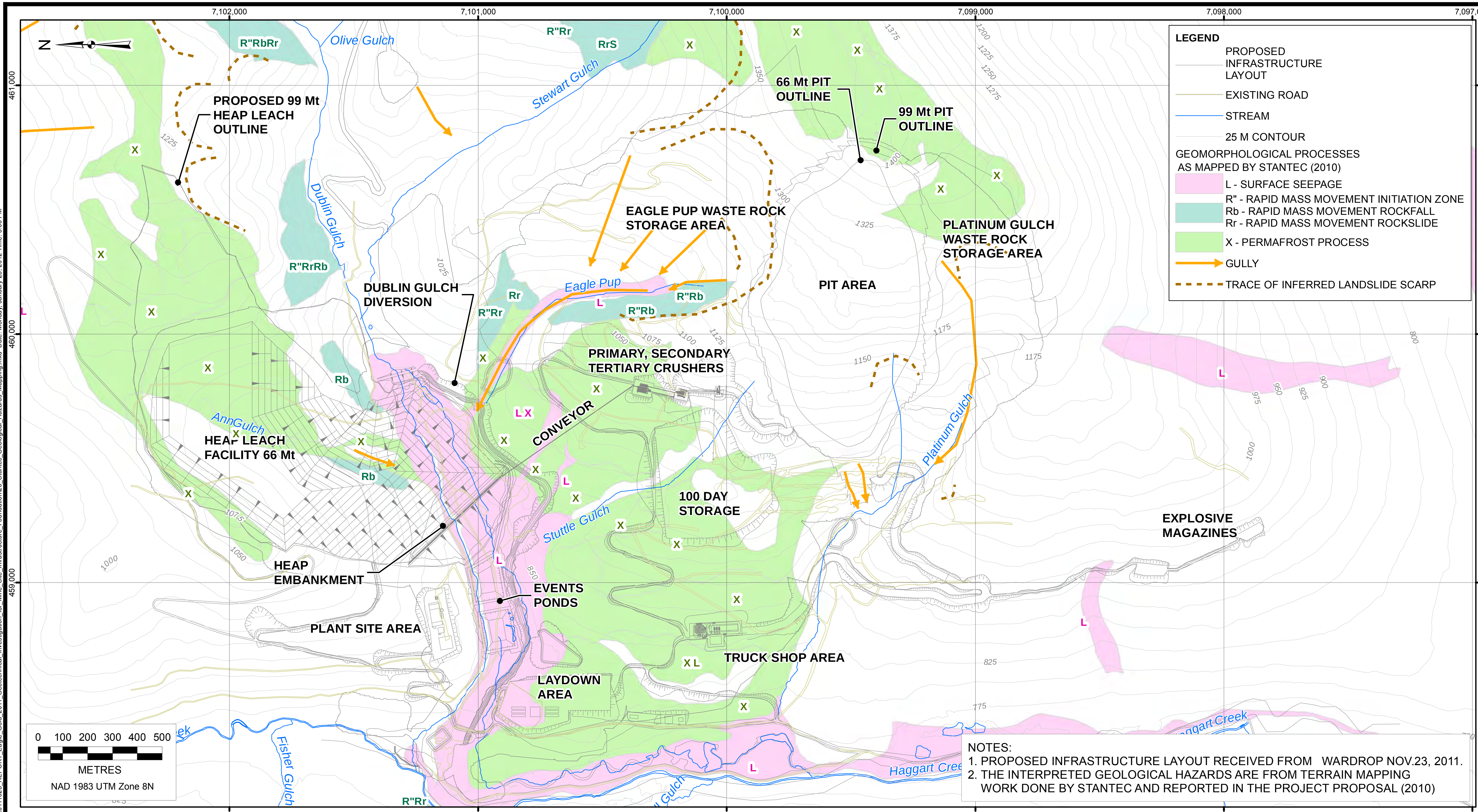
CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: STRUCTURAL DOMAINS

PROJECT No.: 0792-006 | DWG No.: 19 | REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:15,000
DATE: JAN 2012
DRAWN: LL
DESIGNED: KH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

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AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: STANTEC GEOLOGICAL HAZARDS MAPPING

PROJECT No.: 0792-006
DWG No.: 20
REV.: 0

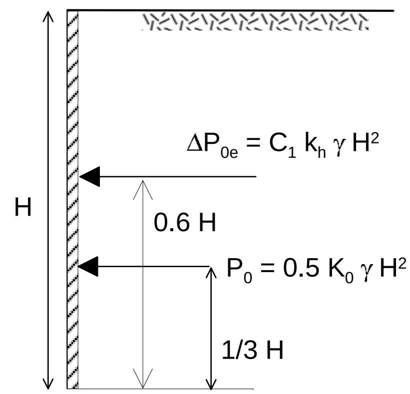
X:\Projects\0792 Victoria Gold\006\Workspace\20111028_REPORT_Eagle_Gold_2011_Geotechnical_Investigation_for_Mine_Site_Infrastructure_Foundation\20_Stantec_Geological_Hazards_Mapping.mxd Date: Monday, January 23, 2012 Time: 3:06 PM

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

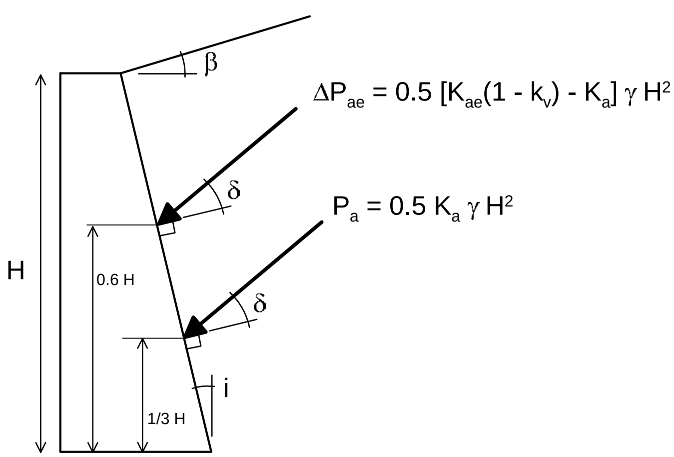
PARAMETERS:

i = angle of back of wall from vertical
β = angle of backfill surface from horizontal
ϕ' = internal friction angle of backfill
δ = interface friction angle at back of wall:
  $2/3\phi'$ soil - concrete contact
  ϕ' soil - soil contact
γ = unit weight of backfill
k_h = horizontal seismic coefficient
k_v = vertical seismic coefficient
$\theta = \tan^{-1}[k_h/(1 - k_v)]$
H = height of backfill behind wall
Q_P = point load (KN)
Q_L = line load (KN/m)
q = surcharge load (KPa)
P = static earth load (KN/m)
K = applicable earth pressure coefficient (K_a, K_p, K_0)
ΔP_e = incremental load induced by earthquake forces (KN/m)
K_e = earth pressure coefficient including effects of earthquake loading

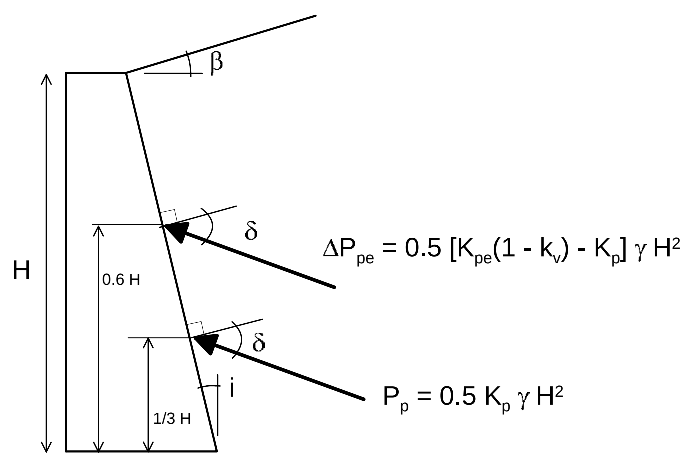
At Rest Case



Active Case



Passive Case



SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT:
VICTORIA GOLD CORP.

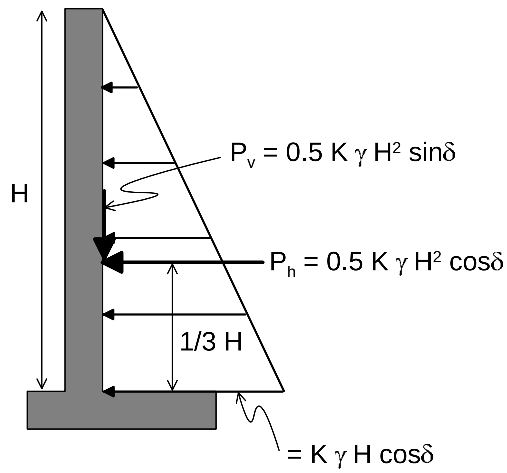
PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC Phone: (604) 684 5900

TITLE:
EARTH PRESSURE
FOR RETAINING WALLS

PROJECT No.	DWG No.	REV.
0792-006	21	0

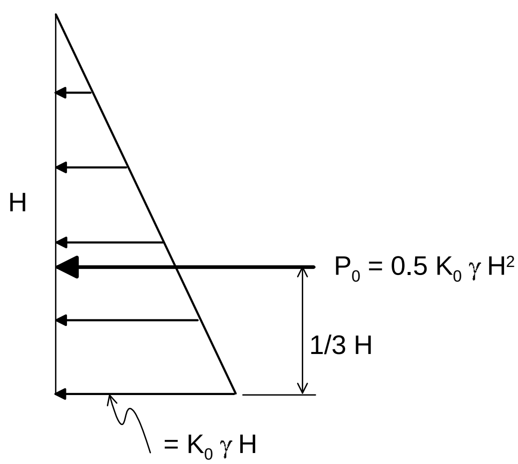
DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT", DATED JAN 2012



$$K_a = \frac{\cos^2(\phi' - i)}{\cos^2 i \cos(\delta + i)\left[1 + \sqrt{\dfrac{\sin(\phi' + \delta)\sin(\phi' - \beta)}{\cos(\delta + i)\cos(\beta - i)}}\right]^2}$$

$$K_p = \frac{\cos^2(\phi' + i)}{\cos^2 i \cos(\delta - i)\left[1 - \sqrt{\dfrac{\sin(\phi' + \delta)\sin(\phi' + \beta)}{\cos(\delta - i)\cos(\beta - i)}}\right]^2}$$

(a) Unrestrained



$$K_0 = 1 - \sin\phi'$$

(b) Restrained

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT:
VICTORIA GOLD CORP.

PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

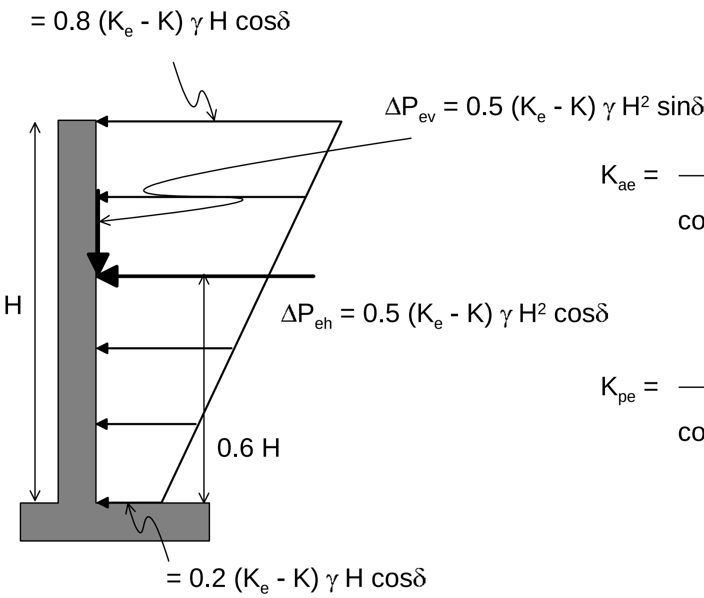
BGC
BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC
Phone: (604) 684 5900

TITLE:
EARTH PRESSURE DISTRIBUTIONS FOR UNRESTRAINED AND RESTRAINED WALLS, STATIC CASE

PROJECT No.	DWG No.	REV.
0792-006	22	0

N:\BGC\Projects\0792 Victoria Gold\006 EG FS Infrastructure\05 Analysis and Design\Retaining Walls

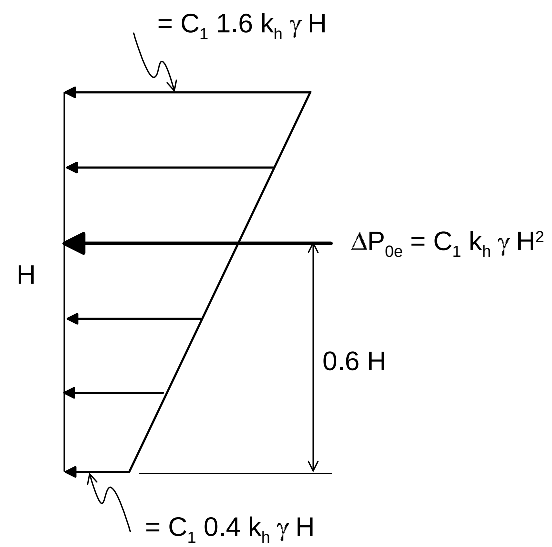
DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT", DATED JAN 2012



$$K_{ae} = \frac{\cos^2(\phi' - \theta - i)}{\cos\theta \cos^2 i \cos(\delta + i + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi' + \delta)\sin(\phi' - \theta - \beta)}{\cos(\delta + i + \theta)\cos(\beta - i)}}\right]^2}$$

$$K_{pe} = \frac{\cos^2(\phi' + i - \theta)}{\cos\theta \cos^2 i \cos(\delta - i + \theta)\left[1 - \sqrt{\dfrac{\sin(\phi' + \delta)\sin(\phi' - \theta + \beta)}{\cos(\delta - i + \theta)\cos(\beta - i)}}\right]^2}$$

(a) Unrestrained



NOTE:
- Refer to Drawing 24 to estimate the dynamic amplification factor, C_1.

(b) Restrained

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT: VICTORIA GOLD CORP.

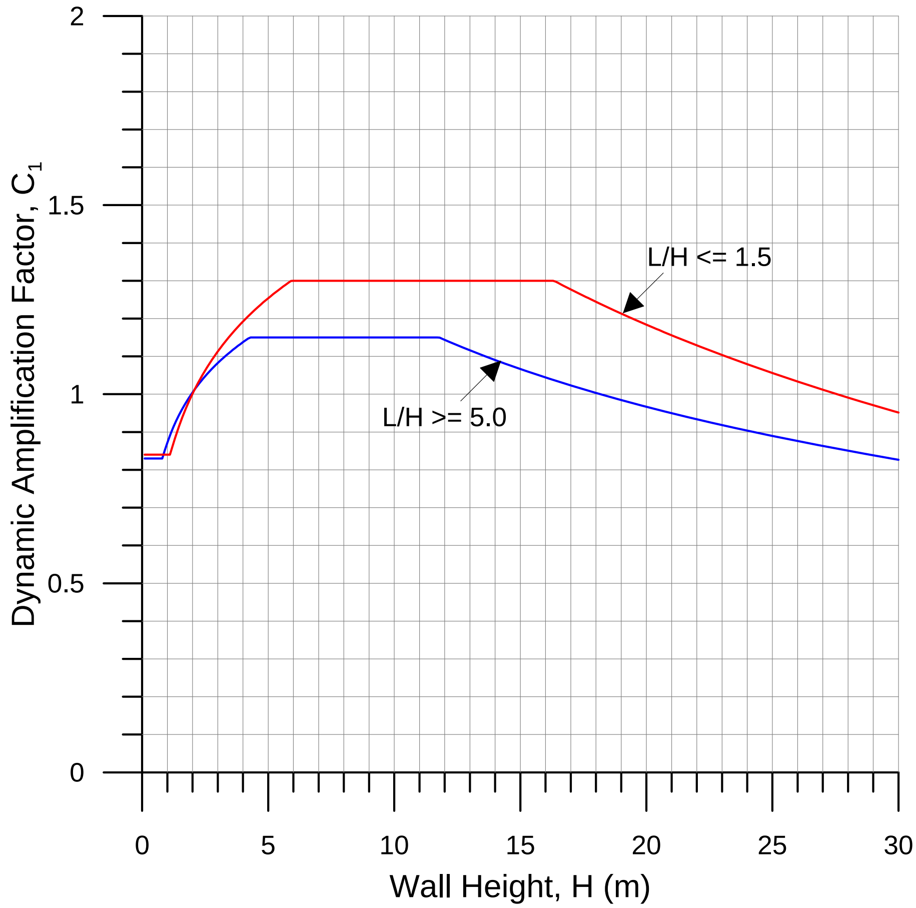
PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC
Phone: (604) 684 5900

TITLE: EARTH PRESSURE DISTRIBUTIONS FOR UNRESTRAINED AND RESTRAINED WALLS, DYNAMIC

PROJECT No.	DWG No.	REV.
0792-006	23	0

N:\BGC\Projects\0792 Victoria Gold\006 EG FS Infrastructure\05 Analysis and Design\Retaining Walls



NOTES:
- Based on 84th percentile curves presented by Wu and Finn (1999).
- Assumptions:
  - Backfill is structural fill compacted to 100% SPMDD
  - unit weight of backfill, $\gamma = 20\ kN/m^3$
  - fundamental frequency of backfill ranges from initial fundamental frequency to degraded frequency equal to the initial fundamental frequency divided by 1.5 (to account for possible strain-softening effects)
  - fundamental frequency of ground motion, f = 5 Hz
- L = average horizontal distance from back of wall to rock
- Use linear interpolation if 1.5 < L/H < 5.0.

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT: VICTORIA GOLD CORP.

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

BGC ENGINEERING INC.
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TITLE: DYNAMIC EARTH PRESSURE AMPLIFICATION FACTORS, C_1, FOR RESTRAINED WALLS

PROJECT No.	DWG No.	REV.
0792-006	24	0

N:\BGC\Projects\0792 Victoria Gold\006 EG FS Infrastructure\05 Analysis and Design\Retaining Walls

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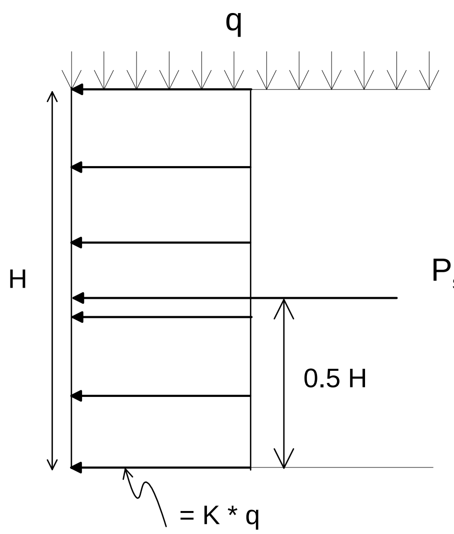


$P_s = K * q * H$

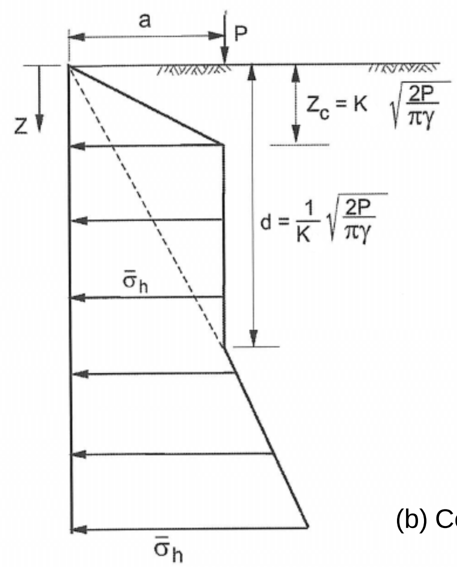
(a) Uniform surcharge

For $Z_c \le Z \le d$

$$\bar{\sigma}_h = \sqrt{\frac{2P\gamma}{\pi}} \frac{L}{a+L}$$

For $Z > d$

$$\bar{\sigma} = K \gamma z$$



(b) Compaction surcharge

$$P \text{ (roller load)} = \frac{\text{dead weight of roller + centrifugal force}}{\text{width of roller}}$$

a = distance of roller from wall

L = length of roller

Source: From Canadian Foundation Engineering Manual 4th Edition, CGS, 2006

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CLIENT:

VICTORIA GOLD CORP.

PROJECT:

EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC
Phone: (604) 684 5900

TITLE:

EARTH PRESSURE DISTRIBUTION DUE TO UNIFORM LOAD SURCHARGE AND COMPACTION

PROJECT No.	DWG No.	REV.
0792-006	25	0

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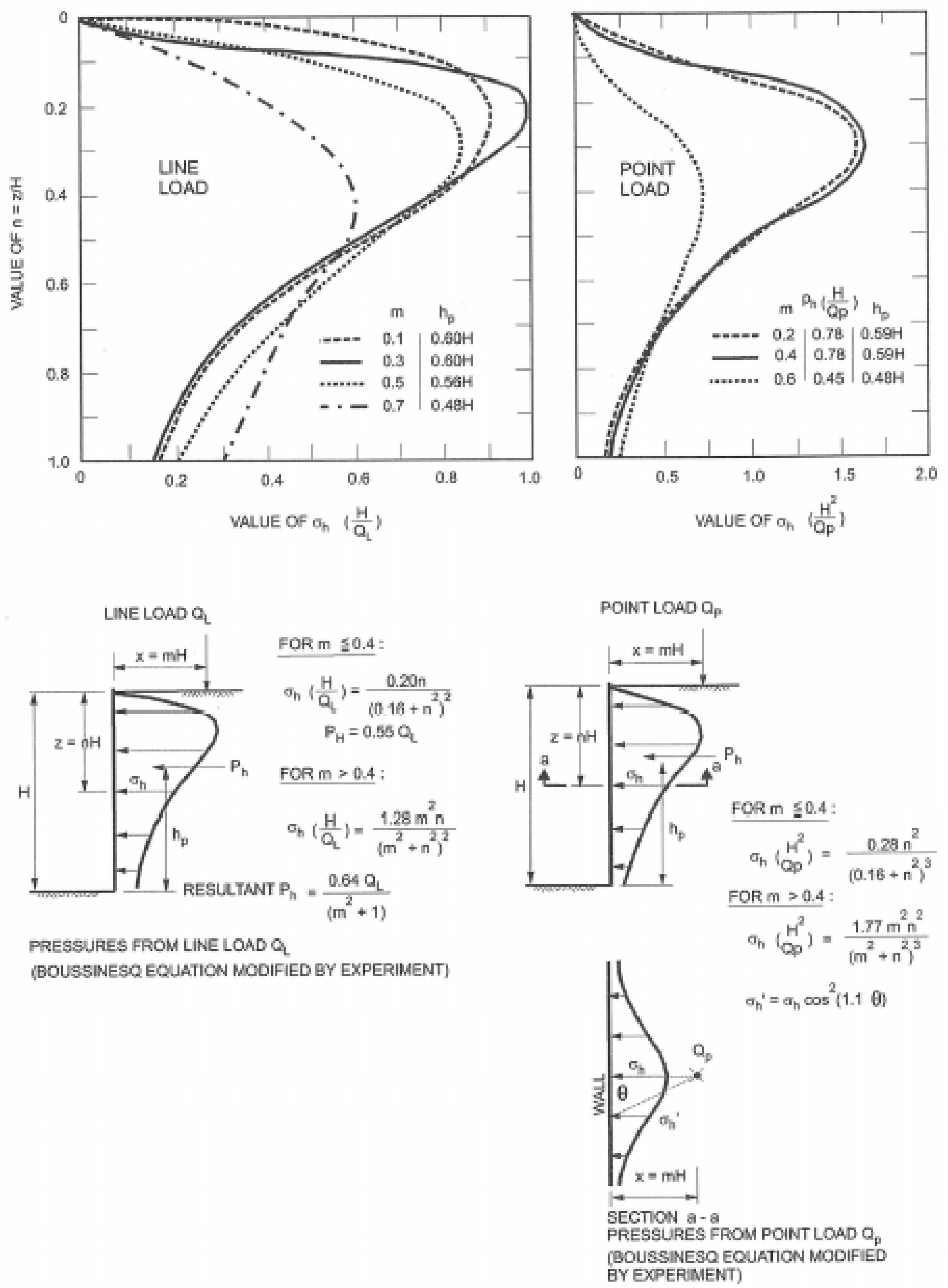

Source: From Canadian Foundation Engineering Manual 4th Edition, CGS, 2006

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DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT:
VICTORIA GOLD CORP.

PROJECT:
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BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC
Phone: (604) 684 5900

TITLE:
EARTH PRESSURE DISTRIBUTION DUE TO POINT AND LINE LOAD SURCHARGES

PROJECT No.	DWG No.	REV.
0792-006	26	0

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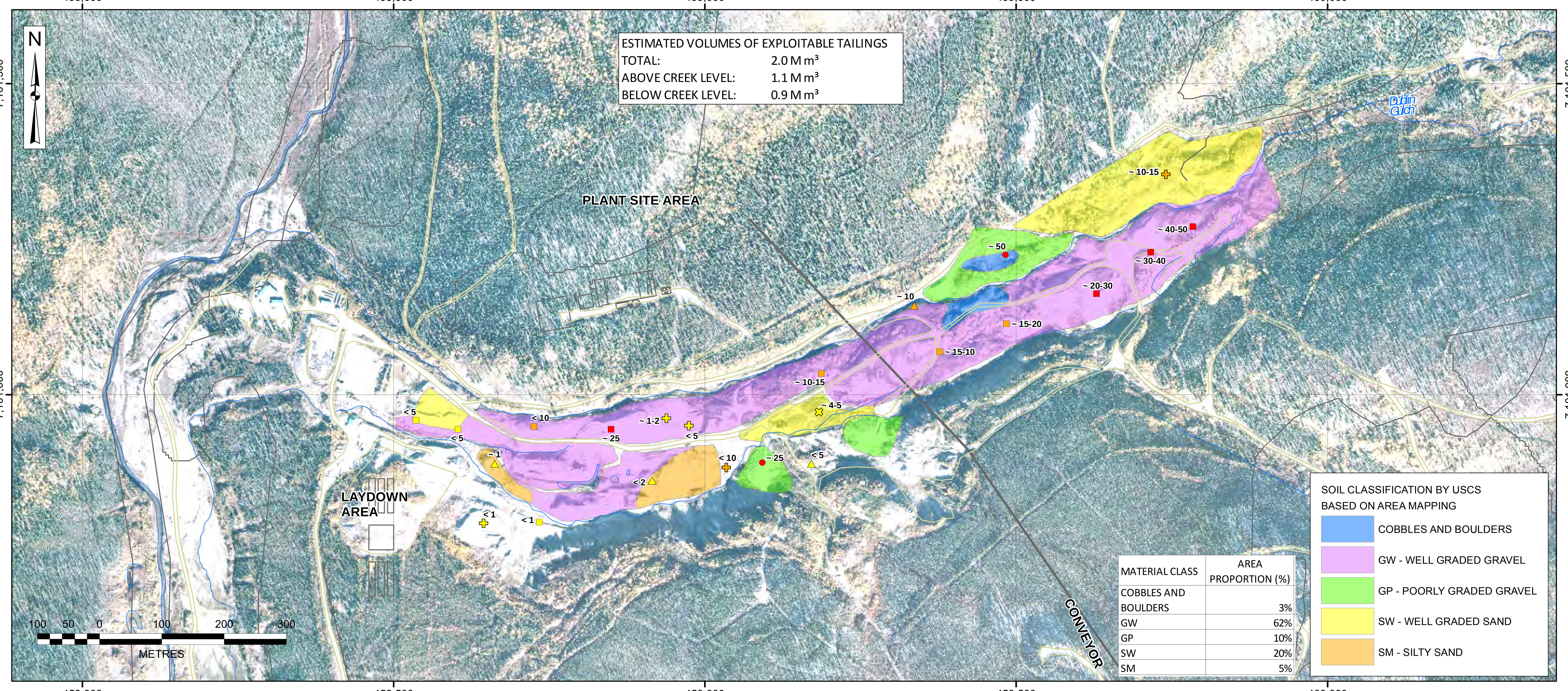
DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT", DATED JAN 2012

DUBLIN GULCH

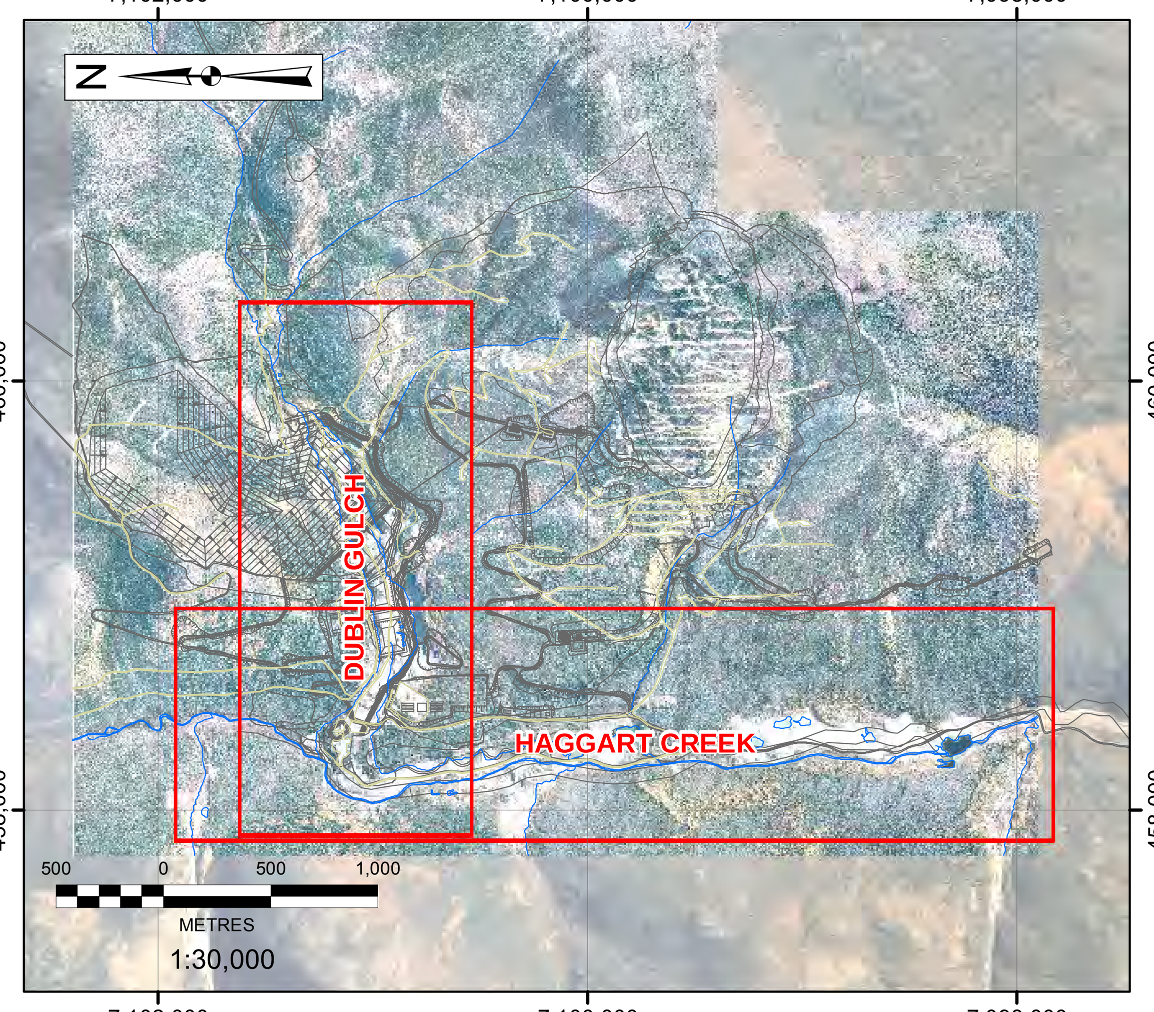


ESTIMATED VOLUMES OF EXPLOITABLE TAILINGS	
TOTAL:	2.0 M m^3
ABOVE CREEK LEVEL:	1.1 M m^3
BELOW CREEK LEVEL:	0.9 M m^3

MATERIAL CLASS	AREA PROPORTION (%)
COBBLES AND BOULDERS	3%
GW	62%
GP	10%
SW	20%
SM	5%

SOIL CLASSIFICATION BY USCS BASED ON AREA MAPPING
- COBBLES AND BOULDERS
- GW - WELL GRADED GRAVEL
- GP - POORLY GRADED GRAVEL
- SW - WELL GRADED SAND
- SM - SILTY SAND

OVERVIEW MAP



LEGEND

PROPOSED INFRASTRUCTURE LAYOUT

EXISTING ROAD

SOIL CLASSIFICATION BY UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) BASED ON ISOLATED SAMPLING
- GP - POORLY GRADED GRAVEL
- GW - WELL GRADED GRAVEL
- SM - SILTY SAND
- SW - WELL GRADED SAND
- NA - NON-SUITABLE MATERIAL

ESTIMATED % OF OVERSIZE PARTICLES (>75 MM)
- N/A
- 0-5 %
- 5-20 %
- >20 %

EXAMPLE:
< 2

WELL GRADED SAND, <2% OF MATERIAL IS GREATER THAN 75MM.

HAGGART CREEK

PILE NAME	MATERIAL DESCRIPTION (USCS)	VOLUME (M^3)
A	GW-SW - well graded sand and gravel	144,000
B	GW - well graded gravel	91,000
C	GW-GP-SW - sand and gravel,well graded and poorly graded	126,000
D	GW - well graded gravel	44,000
E	GW-SW - well graded sand and gravel	76,000
F	SW - well graded sand	150,000
G	GW - well graded sand	19,000
H	GW - well graded sand	101,000
TOTAL		751,000

NAD 1983 UTM Zone 8N

NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.
2. GRAIN SIZE OBSERVATIONS ARE FROM VISUAL OBSERVATION ONLY.
3. VOLUME ESTIMATES FOR DUBLIN GULCH PLACER TAILINGS BASED ON TEST HOLE AND GEOPHYSICAL OBSERVATIONS.
4. VOLUME ESTIMATES FOR HAGGART CREEK PLACER TAILINGS ARE INFERRED FROM SURFACE OBSERVATIONS AND TOPOGRAPHY. NO SUBSURFACE DATA ARE AVAILABLE IN THIS AREA.
5. IMAGERY PROVIDED BY VICTORIA GOLD CORPORATION.

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A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

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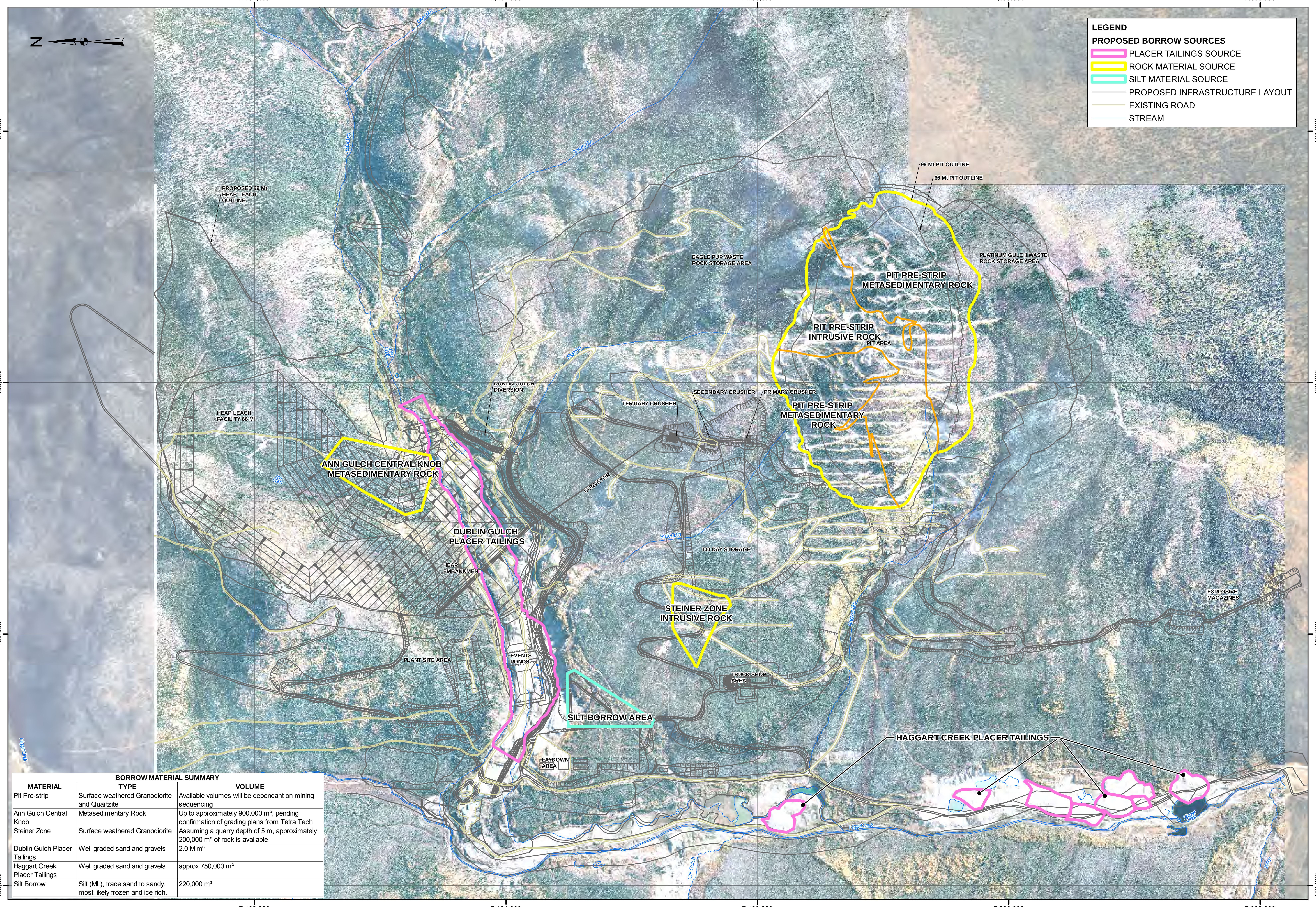
PROJECT: EAGLE GOLD PROJECT 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: BORROW MATERIALS PLACER TAILINGS

PROJECT No.: 0792-006 | DWG No.: 27 | REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



BORROW MATERIAL SUMMARY		
MATERIAL	TYPE	VOLUME
Pit Pre-strip	Surface weathered Granodiorite and Quartzite	Available volumes will be dependant on mining sequencing
Ann Gulch Central Knob	Metasedimentary Rock	Up to approximately 900,000 m³, pending confirmation of grading plans from Tetra Tech
Steiner Zone	Surface weathered Granodiorite	Assuming a quarry depth of 5 m, approximately 200,000 m³ of rock is available
Dublin Gulch Placer Tailings	Well graded sand and gravels	2.0 M m³
Haggart Creek Placer Tailings	Well graded sand and gravels	approx 750,000 m³
Silt Borrow	Silt (ML), trace sand to sandy, most likely frozen and ice rich.	220,000 m³

100 50 0 100 200 300
METRES
NAD 1983 UTM Zone 8N

NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.
2. IMAGERY PROVIDED BY VICTORIA GOLD CORPORATION.

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VICTORIA GOLD CORPORATION

PROJECT:
EAGLE GOLD PROJECT 2011
GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
BORROW MATERIAL - OTHER ENGINEERING MATERIALS

PROJECT No.:	DWG No.:	REV.:
0792-006	28	0

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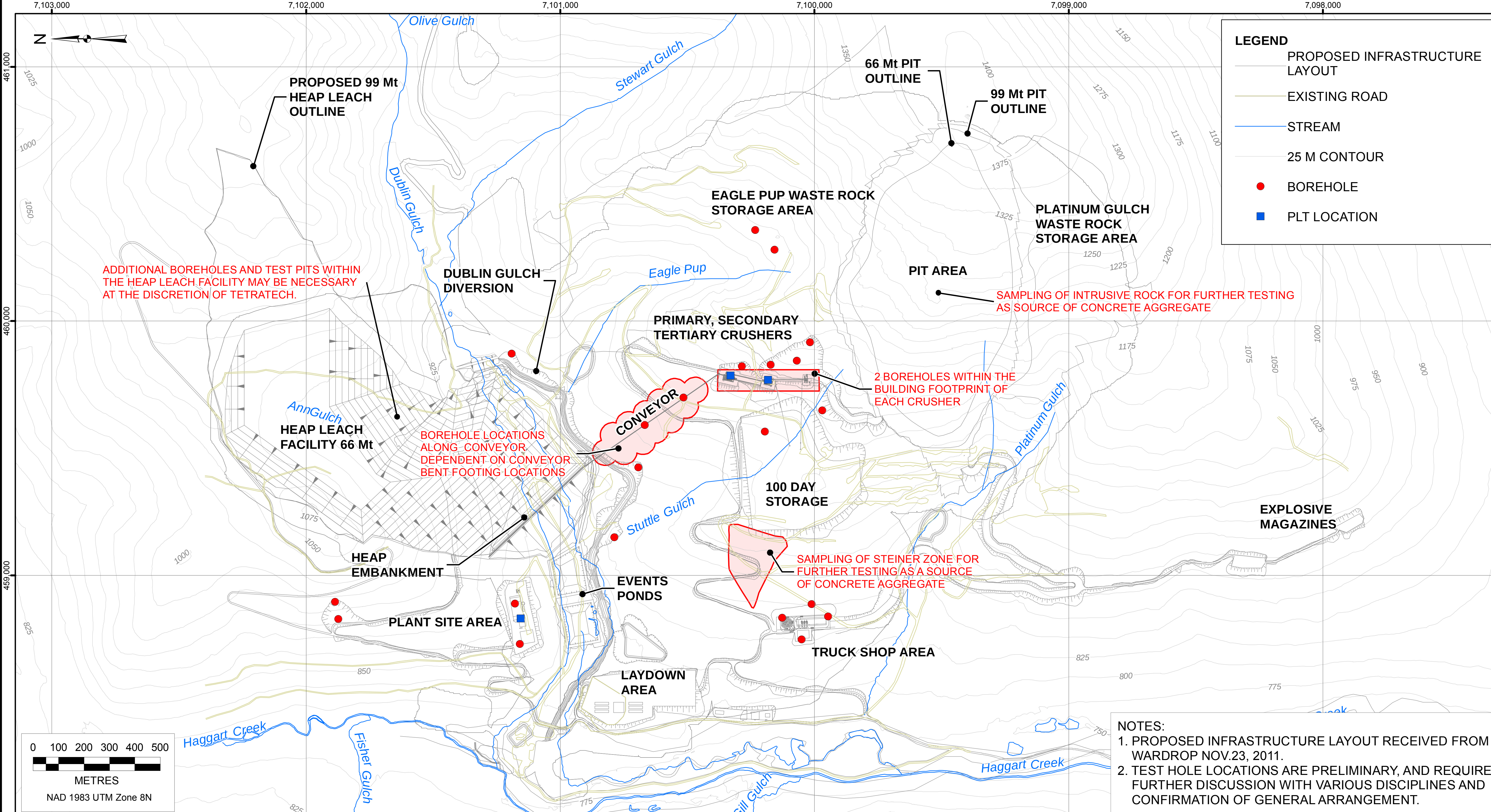

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CLIENT:
VICTORIA GOLD CORPORATION

PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
RECOMMENDATIONS FOR FURTHER INVESTIGATION

PROJECT No.:	DWG No.:	REV.:
0792-006	29	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

APPENDIX A
SUMMARY OF OBSERVED SITE CONDITIONS

1.0 INTRODUCTION

1.1. General

BGC completed site investigations to support design of mine site infrastructure in 2009, 2010 and 2011 (BGC 2010, BGC 2011, BGC 2012). These data, along with data from historic site investigation work completed by Stantec (2010), Knight Piesold (1996a, 1996b), Sitka (1996), GeoViro (1996) and various exploration programs, was used to summarize the site conditions relevant to the development of mine site infrastructure.

This appendix presents a synthesis of all presently available geotechnical data of relevance to the geotechnical design of mine site infrastructure for the Eagle Gold project.

1.2. Generalized Site Conditions in the Mine Site Area

1.2.1. General Site Conditions

The site topography involves moderate to high relief, with ground elevation varying from approximately 800 to 1400 m ASL. Ground conditions are highly variable across the site. Groundwater was observed at varying depths across the site, generally close to the elevation of streams in the valley bottoms, and often below the depth of test pit excavation on the hillsides.

Frozen ground is present in the area, and is relatively warm (typically 0 to -1 degrees Celsius), discontinuous and occasionally contains excess ground ice. Although not specifically controlled by slope aspect, permafrost is found more frequently in the north-facing lower slopes above the south side of Dublin Gulch.

1.2.2. Typical Subsurface Conditions

Subsurface data from BGC geotechnical investigations, and relevant data from prior investigations by others, have been compiled for review in support of this work. The locations of available data are shown on Drawing 11.

Overburden soils encountered on the sloping ground at the mine site typically consist of a veneer of organic soils overlying a blanket of colluvium, which overlies weathered bedrock. The observed thickness of overburden materials is illustrated in Drawing 12.

Glacial till is generally encountered on the lower flanks of the north- and west-facing slopes north and west of the proposed open pit, above Dublin Gulch and Haggart Creek. Placer tailings (fill) cover most of the valley bottom of Dublin Gulch and Haggart Creek. Alluvial soils are occasionally encountered along the undisturbed valley-bottom areas. Surface soils typically consist of organic soil, rootlets, woody debris and plant matter.

Colluvium varies in composition but typically consists of loose to compact subangular to angular gravel and occasional cobbles in a sand and silt matrix, derived from weathered metasedimentary rock. In some zones, the colluvium is gravelly, sandy low plastic silt, also derived from weathered metasedimentary rock. Till is typically firm to stiff sandy silt or compact silty sand with varying proportions of gravel.

Placer tailings (fill) are typically well graded sand and gravel with varying proportions of silt, cobbles and boulders. Particle size distribution and density vary considerably throughout the placer tailings. Drawing 13 shows the distribution of placer tailings thickness, where known. This unit consists of native materials that have been reworked by placer mining activities, and is present in the Dublin Gulch and Haggart Creek valley bottoms.

"Weathered rock" is defined as un-transported bedrock that is completely weathered and weak, with weathering grade of W5 or higher, and intact strength of R0 or less (i.e. UCS less than 1 MPa). "Weathered rock" is expected to behave like a soil, and is therefore included as part of the thickness of overburden illustrated on Drawing 12.

Drawing 14 shows the ground and surface water observations. Drawing 15 shows the distribution of frozen ground, where encountered, which can generally be inferred to be permafrost, but may in some cases be seasonally frozen soils. Frozen ground is more difficult to excavate than unfrozen ground, and can be expected to require ripping. Drawing 15 also shows the distribution of ice rich permafrost, which for the purposes of this report is defined as frozen soils that become very wet and soft when thawed. Ice-rich permafrost soils are unstable as a foundation for an engineering structure when thawed.

The bedrock encountered at the mine site is classified as either intrusive (typically granodiorite, in the uplands) or metamorphosed sedimentary rock (typically schist, phyllite or quartzite), with a variable depth of weathering. Bedrock has been subdivided into three types on the basis of expected engineering characteristics, including, from weakest to strongest: Type 3; Type 2; and, Type 1.

"Type 3" rock is the first "rock-like" material underlying the overburden soil materials, and is defined as being rock that is highly or less weathered (i.e. W4.5 or better), and has intact strength greater than R0 (i.e. minimum UCS strength 1 MPa). It is expected that Type 3 rock can be excavated with normal excavating equipment with some material requiring ripping. Drawing 16 shows the observed depth to "Type 3" rock.

"Type 2" rock is stronger and stiffer than "Type 3" rock. This material is defined as rock with Geological Strength Index (GSI, Hoek and Marinos, 2000) or Rock Mass Rating (RMR, Bieniawski, 1976) of 30 or greater, and core recovery during drilling of 50% or greater. Alternatively, where GSI and RMR data are unavailable, average Rock Quality Designation (RQD) of 10 or greater serves as an approximately equivalent criterion. It is expected that Type 2 rock will require a combination of normal excavation and ripping. Drawing 17 shows the observed depth to "Type 2" rock.

"Type 1" rock is the strongest rock observed during the site investigations. This material is defined as having GSI, RMR or average RQD exceeding 40. It is expected that Type 1 rock will require ripping, and may require local blasting. Drawing 18 shows the observed depth to "Type 1" rock.

2.0 BEDROCK STRUCTURE

Design of selected cut slopes will be controlled by the presence, orientation, persistence and strength properties of discontinuities in bedrock. This section provides a general overview of interpreted structural geology in relation to the design of mine site infrastructure facilities.

2.1. Area overview

Dublin Gulch lies within an area that was deformed and metamorphosed in the late Jurassic to early Cretaceous period by north-directed folding and thrusting. The site sits on the hanging wall to the south of the major thrust faults that accommodated the north-south shortening (Murphy 1997). Intrusive plutons in the area were emplaced by subsequent magmatism associated with this event (Mortensen et al. 2000). Most of the modern rock structure at Dublin Gulch can be attributed to this event, and to a period of north-south extension shortly afterwards – around the time of the gold mineralization at Dublin Gulch – that caused the development of steeply dipping, E- to NE-striking extension veins and NNW-striking strike-slip fault veins (Stephens et al. 2004).

Two main rock types are found within the study area around Dublin Gulch: metasedimentary rocks of the Hyland Group, and a granodiorite intrusive stock belonging to the Tombstone Plutonic Suite (Murphy 1997). The metasedimentary rocks range from quartzite to phyllite, and are contact-metamorphosed to hornfels near the granodiorite intrusion. Their engineering characteristics are primarily determined by their relative content of quartz and mica/phyllite, and by their degree of contact metamorphism (controlled in turn by distance from the granodiorite intrusion). The quartz-rich metasedimentary rocks are strong, blocky, lightly folded, and have a well-jointed structure, whereas the mica-rich phyllite tends to be very weak, friable, intensely folded, and its structure is almost entirely controlled by the closely-spaced foliation planes. The mica-rich phyllite is mainly found north of and within Dublin Gulch, and the rocks south of Dublin Gulch are generally more quartz-rich, with quartz content increasing in proximity to the intrusive body. Where the metasedimentary rocks have been contact metamorphosed, they are much stronger and tend to have rougher, more widely-spaced discontinuities.

2.2. Structural domains

The Dublin Gulch study area was divided into structural domains based mainly on similarity of rock type and structures (see Drawing 19). The intrusive rocks comprise one single domain (B), and the sedimentary rocks were divided into four separate domains (A, C, D, and E). Domains A and C occupy the southern three quarters of the study area and are separated by a major west-plunging anticline that runs from northeast to southwest, with its axis passing just north of the proposed open pit. In domain A on the south limb of the anticline, the average foliation dips shallowly to steeply southwest; in domain C on the north limb of the anticline, the foliation dips moderately northwest. Domain D covers an area around Tin Dome where the bedrock is very phyllitic and intensely folded, resulting in a distribution of somewhat irregular foliation orientations. Domain E, covering the upper eastern side of Ann Gulch, is the southwest corner of an area stretching to the north and east of the study area wherein the foliation dips mostly north.

2.2.1. Domain A – South Limb of Anticline

Much of Domain A is situated around the edges of the granodiorite intrusion, so the metasedimentary rocks observed in the area are almost all contact metamorphosed and are fairly strong. A notable exception is the rocks directly within a few meters of the intrusive contact, metasedimentary and intrusive alike, which tend to be clay-altered and very weak; in places completely disintegrated. The foliation in Domain A is slightly wavy perpendicular to its dip direction at outcrop scale (10s of meters), but noticeably more so at the inter-outcrop scale (100s of meters), with average dips ranging from 25-63 degrees between different outcrops. The poles to foliation planes measured in Domain A fall into a bi-modal distribution on stereonet which suggests the limbs of a series of tilted monoclines (Figure 1). Some two-sided folds occur in this set as well, as indicated by instances of foliation dipping shallowly in the opposite direction of the regional average.

The strongest discontinuity set in Domain A apart from foliation is the set of subvertical, SW-striking extension fractures that host much of the mineralization in the area (JV1). These are highly persistent, planar joints and joint-veins and were observed at most mapped outcrops. A secondary joint set with similar orientation to JV1 but dipping more shallowly and with generally lower persistence was observed sporadically (JS2). A steeply-dipping, NNW-striking set cross-cuts JV1 and was also observed at most mapped outcrops (JS1). Finally, two additional joint sets that dip moderately towards the NE and SE were observed sporadically (JS3 and JS4).The average orientations and properties of the discontinuity sets in Domain A are shown in Table 1.

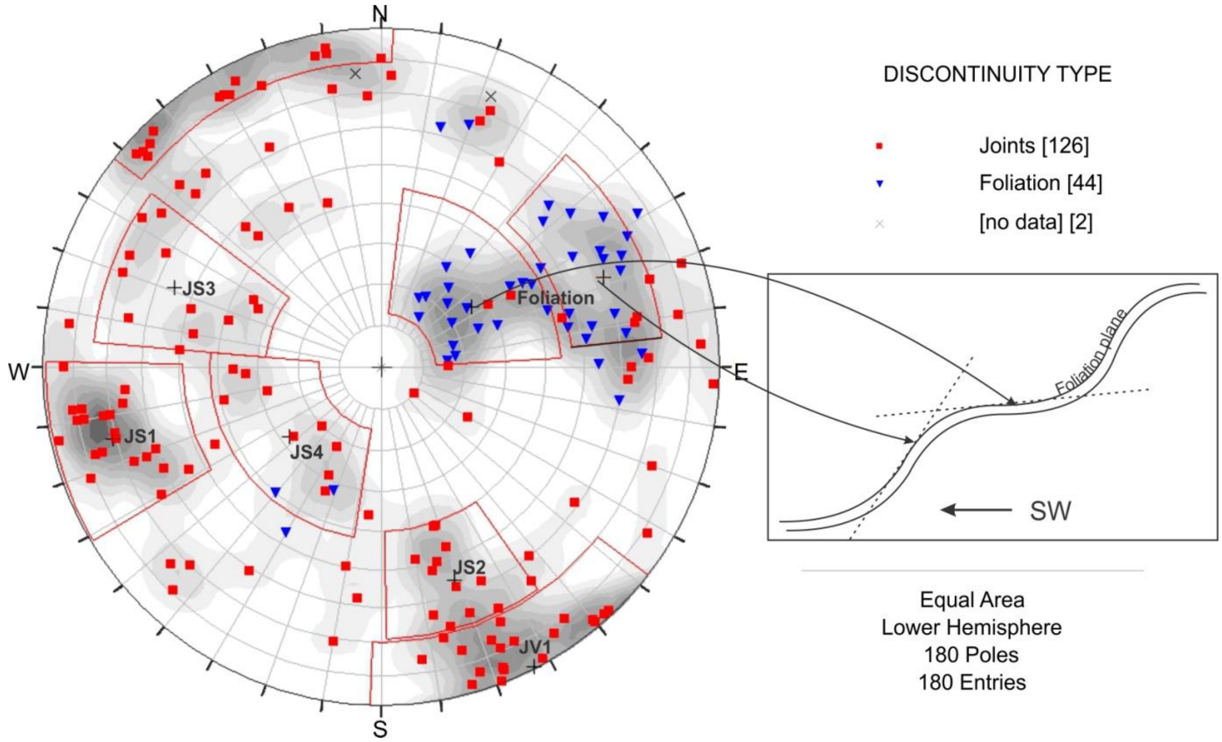

Figure 1. Discontinuities mapped in Domain A, with major sets and conceptual sketch demonstrating potential fold geometry.

Table 1. Discontinuity set average properties in structural domain A.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
Foliation	43	245	11	3.5	2.2	0.08
JV1	87	332	9	4	1.6	0.22
JS1	74	76	10	4	1.8	0.39
JS2	53	343	12	3.5	1.3	0.22
JS3	56	120	9	3.5	1.6	0.47
JS4	29	60	10	3	1.6	0.2

2.2.2. Domain B – Granodiorite Intrusion

Most of the intrusive rocks in Domain B are very strong, with the exception (noted above) of clay-altered rocks often found within a few meters of the contact with the metasedimentary rocks. The sub-vertical ENE-striking, mineral-hosting extension fractures are strongly expressed in Domain B (JV1), as well as an orthogonal sub-vertical set that strikes NNW (JS1; Figure 2). The strike of these two sets varies substantially around Domain B, being rotated clockwise (to the north/east) in the northerly part of the domain. However, their relative orthogonality is consistent everywhere. As in Domain A, these sets tend to be very planar and have high persistence.

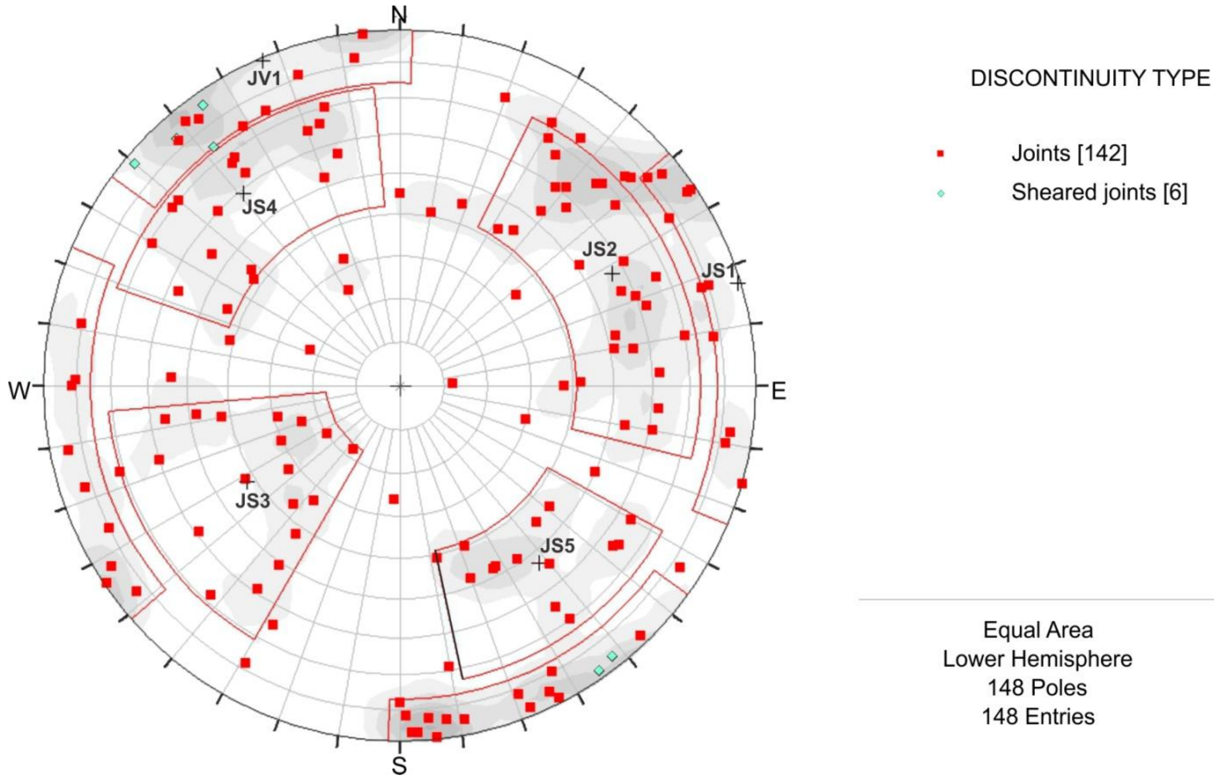

Figure 2. Discontinuities mapped in Domain B, showing major sets.

Four other joint sets were also observed throughout Domain B (JS2-JS5), with moderate average dips from 40-70 degrees. Their average orientations on stereonet suggest two groups of conjugate sets, whose strikes are rotated approximately 15 degrees counter-clockwise from JV1 and JS1. In general, joints in sets JS3 and JS4 tend to be more persistent than those in JS2/JS5, and JS4 joints are smoother than the other three sets. JS2 and JS4 were observed at most outcrops, whereas JS3 and JS5 appear sporadically. Table 2 shows the average orientations and properties of all discontinuity sets in Domain B.

Table 2. Discontinuity set average properties in structural domain B.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
JV1	90	341	10	4.5	4.3	0.41
JS1	89	253	12	4	3.0	0.54
JS2	57	242	14	4.5	2.1	0.56
JS3	58	42	14	4	2.3	0.59
JS4	59	141	10	4	4.0	0.75
JS5	53	322	11	4.5	1.1	0.44
Sheared joints	86	140	5	3.5	2.8	0.5

2.2.3. Domain C – North Limb of Anticline

Domain C represents the northern limb of the major anticline that crosses the study area from east to west. Some hornfels are found in the southeastern corner of the domain, adjacent to the granodiorite intrusion. The rocks in Domain C grade northwards from quartzite into progressively more micaceous phyllitic rock. In general, the part of Domain C south of Dublin Gulch is primarily quartzite, and the part within and north of Dublin Gulch is primarily phyllite. However, these two units are often interbedded in fairly narrow seams (~ 0.1 – 1 m), particularly near Dublin Gulch.

The foliation in Domain C is regularly oriented, dipping shallowly to moderately NW across most of the domain (Figure 3), and relatively planar – the waviness of foliation surfaces perpendicular to the direction of dip is less pronounced than that seen in Domain A. The spacing of foliation planes varies more than an order of magnitude between the quartzite and phyllite rocks.

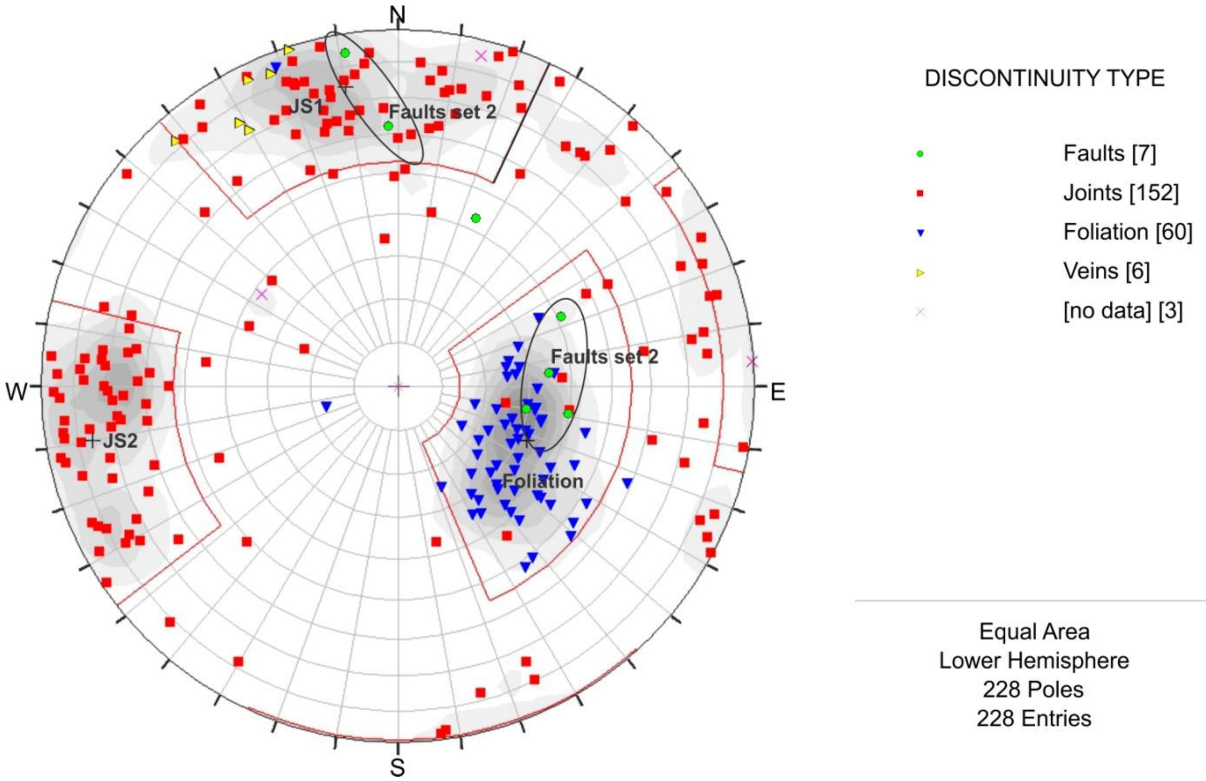

Figure 3. Discontinuities mapped in Domain C, showing major sets.

Two major joint sets cross-cutting foliation were observed at nearly every outcrop in Domain C (JS1 and JS2), one dipping steeply SSE and the other dipping steeply ENE. Their strikes are equivalent to the sub-vertical extension fracture sets JV1 and JS1 in domains A and B, but they dip less steeply and the ENE-striking set only rarely hosts mineralized veins. Two sets of faults were noted in Domain C: one parallel to foliation, and one sub-parallel to JS1. Additional minor joint sets cutting obliquely across foliation exist at most sites, but none stands out as a consistent set on the inter-outcrop scale. Table 3 shows the average orientations and properties of discontinuity sets in Domain C.

Table 3. Discontinuity set average properties in structural domain C.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
Foliation	32	299	13	3	2.1	0.11
JS1	73	173	9	3	2.2	0.29
JS2	76	79	12	3	1.4	0.27
Fault set 1	36	267	19	1	6.0	N/A
Fault set 2	73	174	7	2.5	4.8	N/A

2.2.4. Domain D – Area around Tin Dome

A distribution of irregular foliation orientations was mapped in the well-folded phyllitic rocks around Tin Dome, just north of the bottom of Dublin Gulch. The density of mapping in the area is insufficient to interpret the major geologic structures controlling foliation attitudes, so the rock structure in Domain D cannot be interpolated between mapped sites with good confidence. However, the majority of the bedrock in Domain D is so soft and fractured that its engineering behavior will likely be controlled less by structure than by the intact rock and rock mass properties.

Despite this, some structural interpretation can still be made in Domain D. The two steeply dipping orthogonal extension fracture sets seen elsewhere at Dublin Gulch are present (JS1 and JS2; Figure 4). A widely-spaced minor third joint set dipping moderately southwest is also apparent (JS3). Foliation in the southwest corner of Domain D near the planned plant site dips generally southwest, although at highly variable angles. In general, foliation planes are very wavy due to the high degree of small-scale folding present (1s to 10s of metres-scale).

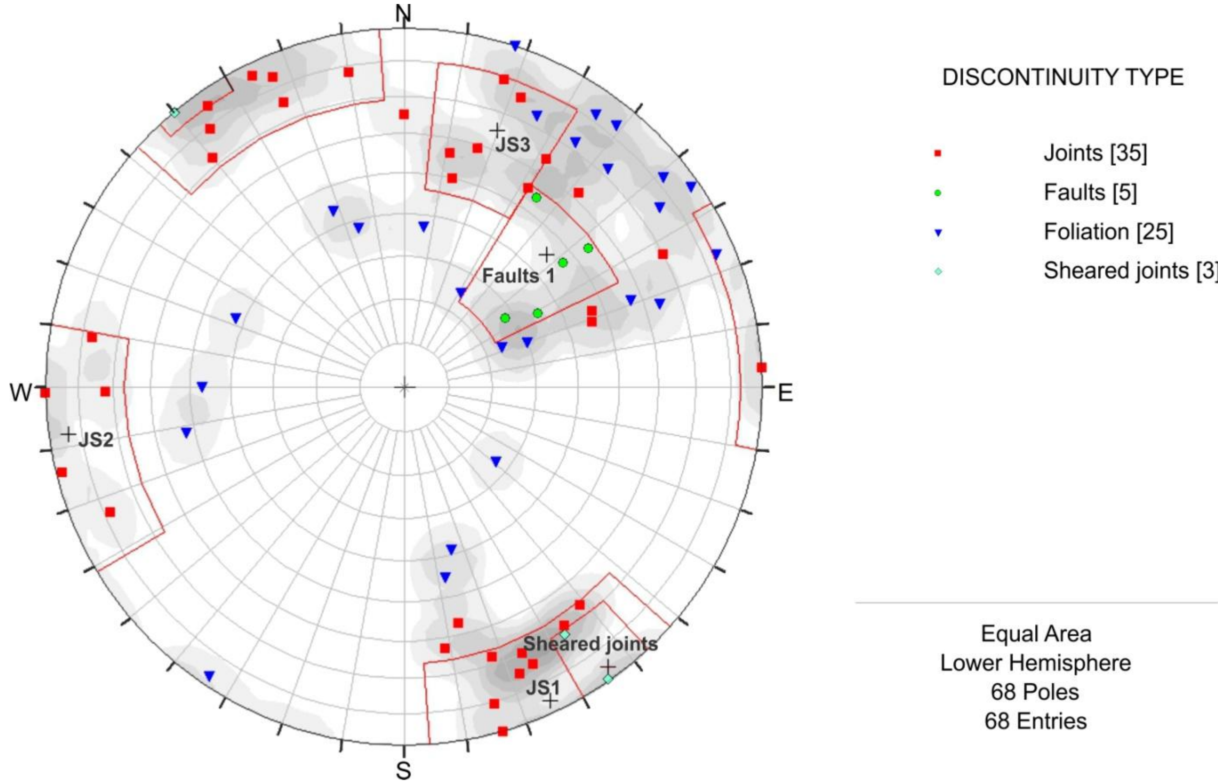

Figure 4. Discontinuities mapped in Domain D, showing major sets.

Table 4. Discontinuity set average properties in structural domain D.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
Foliation	Various	Various	15	2	1.8	0.05
JS1	85	335	8	3	2.2	0.17
JS2	82	84	14	3	1.0	0.13
JS3	62	203	14	3	1.0	0.38
Faults 1	43	230	18	3	9.2	0.5
Sheared joints	83	324	12	3	3	0.1

2.2.5. Domain E – northeast of Ann Gulch

The BGC field program mapped structural data at only one station in Domain E; however, data from others (Stephens, et. al. 2004) in the area north of Dublin Gulch and east of Ann Gulch agrees with the north-dipping foliation observed by BGC (Figure 5).

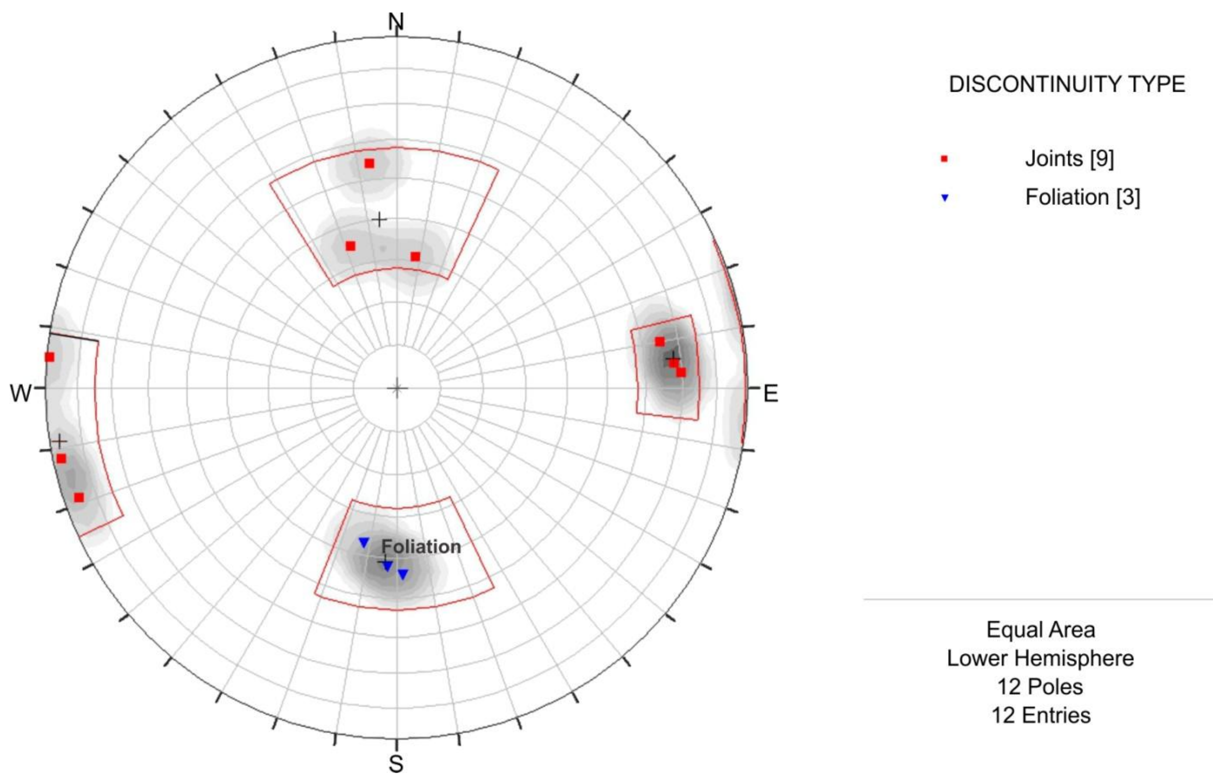

Figure 5. Discontinuities mapped in Domain E.

3.0 AREA-SPECIFIC SUBSURFACE CONDITIONS

3.1. General Overview by Functional Area

The project site has been subdivided into a number of distinct functional areas related to proposed infrastructure elements for the purpose of data synthesis and analysis.

Summary observations for each functional area are presented in Table 5. This table provides an overview of the general conditions within each area, including the observed thickness of overburden, presence or absence of frozen ground and excess ice, and depth, where encountered, to Types 1, 2 or 3 bedrock.

Extensive deposits of placer tailings fill are present in the Dublin Gulch valley bottom in the area of the heap leach pad, heap embankment, a portion of the Dublin Gulch diversion, and ponds or other facilities to be constructed in this area. The observed thickness of placer tailings at 16 test holes had a mean value of about 10 m, with a range between 0.3 m and 19.8 m. Based on shear wave geophysical surveys of the Dublin Gulch valley bottom, the placer tailings likely extend to a depth of up to about 25 m, and possibly deeper toward Haggart Creek.

There is typically a thin cover of organic soils overlying the other overburden units. The observed thickness of this unit varies across the site, ranging between 0 m and 3.7 m, with an average thickness of 0.3 m, and standard deviation of 0.3 m, from 285 observations. All organic materials are unsuitable for re-use as engineering fill materials, but should be suitable for reuse as cover materials for reclamation and should be segregated and separately stockpiled.

The main body of the report includes commentary on earthworks construction in each functional area in relation to these specific observations.

Table 5. Summary Observations of Ground Conditions by Functional Area

Area	Overburden[7] Thickness (m)				Observations of Frozen Ground			Depth to Rock where Encountered (m)					
	Known Thickness[1]		Minimum Thickness[2]		N[3]	Frozen Ground[4], N_f	Excess Ice[5], N_{ei}	Type 3		Type 2		Type 1	
	Median	N[3]	Median	N[3]				Median	N[3]	Median	N[3]	Median	N[3]
100 Day Storage	1.2	7	3	7	14	9	7	1.2	7	1.8	6	N/A	N/A
Conveyors	13.5	2	2.3	9	11	7	6	13.6	2	23.3	1	N/A	N/A
Crushers	3.2	18	4.7	3	21	4	2	2.6	14	4.6	11	17.4	4
Diversion	4.8	10	5.5	12	22	7	4	4.5	9	8.8	7	19.5	1
Dublin Gulch pond	N/A	N/A	16.8	3	3	0	0	N/A	N/A	N/A	N/A	N/A	N/A
Eagle Pup WRSA pond	10.4	3	5.5	3	6	1	1	3.8	3	12.2	3	18.9	2
Eagle Pup WRSA	2.5	39	3.9	38	77	47	29	1.9	36	3	25	21.5	6
Events Ponds	12.2	3	5.5	5	8	0	0	12.2	3	16.2	1	14.9	1
Explosive Storage	2.0	2	N/A	N/A	2	0	0	2.0	2	4.5	1	N/A	N/A
Heap Embankment	8.8	13	5.7	12	25	3	3	9.0	12	14.2	6	31.2	1
Heap Pad	3.5	50	5	21	71	14	6	1.9	42	4.8	29	17.3	4
Laydown Area	N/A	N/A	5.5	11	11	6	6	N/A	N/A	N/A	N/A	N/A	N/A
Main Pond	N/A	N/A	5.8	9	9	7	5	N/A	N/A	N/A	N/A	N/A	N/A
Main truck road	5.1	3	4.9	4	7	1	1	5.1	3	N/A	N/A	N/A	N/A
Plant site	9.1	7	6.4	4	11	2	1	9.1	5	12.3	1	N/A	N/A
Platinum Gulch WRSA pond	N/A	N/A	6.9	4	4	1	1	N/A	N/A	N/A	N/A	N/A	N/A
Platinum Gulch WRSA	2.4	19	3.1	10	29	11	11	2.3	14	3.3	12	10.9	4
Secondary road	N/A	N/A	2.8	10	10	7	4	N/A	N/A	N/A	N/A	N/A	N/A
Truck Shop	7.1	3	2.5	3	6	6	5	7.1	3	N/A	N/A	N/A	N/A

1. "Known thickness" of overburden indicates the full depth is known because bedrock was encountered during drilling or test pitting.
2. "Minimum thickness" of overburden represents observations where the overburden is known to be at least a given thickness, equal to the depth of exploration, but total thickness is not known, since bedrock was not encountered.
3. "N" is the number of observations taken into consideration.
4. N_f is the number of observation locations where frozen ground was noted.
5. N_{ei} is the number of observation locations where excess ice was observed in the frozen ground.
6. "N/A" indicates no data available in that area.
7. Overburden is defined as soil material, including organics, till, colluvium, alluvium, fill (placer tailings) and completely weathered rock.
8. Median values are presented in this table. There is significant variability throughout functional areas. Please see drawings 12 through 18 for detailed illustrations of site conditions.

3.2. Subsurface Conditions at the Proposed Plant Site

3.2.1. General

Drawing 11 shows the distribution of test holes located in the general vicinity of the proposed Plant Site. The subsurface observations from these holes are summarized in Table 6, below.

Table 6. Summary Subsurface Observations in Proposed Plant Site Area

Testhole	Approx. Ground Elevation (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth to Type 3 Rock (m)	Total Depth (m)	Frozen Ground Encountered
TP-BGC09-HL1-1	878	0.3	1.3	4.9	6.5	6.5	Yes
TP-BGC09-HL1-2	845	0.3	5.9	N/A	N/A	6.2	Yes
TP-BGC10-33	868	0.25	4.0	>1.3	N/A	6.5	No
TP-BGC10-34	852	0.2	3.8	>1.5	N/A	6.5	No
TP-BGC11-103[3]	865	0.2	3.3	>3.7	N/A	6.0	No
TP-BGC11-105	878	0.2	2.3	>0.5	N/A	3	No
TP-BGC11-130	865	0.1	>5.9	N/A	N/A	6	No
BH-BGC10-11	857	NR	NR	6.1	13.2	46.6	No
BH-BGC10-12	863	NR	NR	13.2	19.2	28.7	No
BH-BGC11-54	884	NR	NR	15.2	19.8	41.2	No
BH-BGC11-67[4]	867	N/A	N/A	9.1	9.1	9.9	No
BH-BGC11-69[4]	867	N/A	N/A	9.1	9.1	21.34	No

Notes:

1. "NR" = no recovery
2. N/A – not observed or not applicable
3. In TP-BGC11-103, till was present between 3.3 m and 6 m.
4. BH-BGC11-67 and BH-BGC11-69 were drilled in the pit excavated for plate load testing. Base of pit was in completely weathered metasedimentary rock. This pit is logged as TP-BGC11-103.

3.2.2. Overburden

The upper soil unit consists of a horizon of organic soil, rootlets, woody debris and plant matter ranging from 0.1 to 0.3 m thickness and averaging approximately 0.2 m.

The organic cover is immediately underlain by colluvium to depths ranging from 2.0 m to > 5.9 m, with an average depth of approximately 3.4 m. The colluvium consists of loose to compact, subangular to angular gravel and occasional cobbles in a silt and sand matrix, derived from transported weathered metasedimentary bedrock further upslope. Till was present in TP-BGC11-103, and consisted of compact sand and gravel with both granodiorite and metasedimentary clasts.

Weathered rock is present below the colluvium in a number of test pits and boreholes, ranging in thickness from approximately 4 m to 15 m. The material is typically highly to completely weathered, extremely weak to very weak metasedimentary rock.

3.2.3. Bedrock

Bedrock was observed below weathered rock at depths ranging between 6.5 m and 19.8 m below drill collar elevation (average depth to bedrock at 12.8 m) in the test holes shown in Drawing 11. Observed bedrock consisted of slightly to moderately weathered, very weak to weak metasedimentary rock (i.e. Type 3 rock). Type 2 rock was encountered in one borehole (BH-BGC11-69) at 12.3 m depth; it should be noted that BH-BGC11-69 was drilled at the base of an excavation and its collar elevation is approximately 6 m below existing ground surface.

Recovery from drill holes in bedrock was poor in some zones and good in others, ranging from 20% to 100%, with average recoveries of approximately 55%. Rock Mass Rating (RMR, Bieniawski, 1976) values ranging from 15 to 55, with an average of approximately 30, were determined from the observed rock core.

These observations suggest that weathered rock will typically be encountered at foundation grades. The cut for the plant site pad will be primarily in completely weathered rock with a thick blanket of colluvium.

The soft, phyllitic bedrock around the plant site is intensely folded and faulted, resulting in a fairly erratic distribution of foliation attitudes (Figure 6). In large plate load test pits in the footprint of the plant site, two distinct directions of foliation were mapped, dipping 81° towards 213° and 39° towards 169°. On average, foliation in the area dips roughly southwest between 25 and 85 degrees. Sheared fault structures sub-parallel to foliation, some containing clay gouge infill, were observed at outcrops 48A and 51 (BGC 2012). There are also steeply-dipping joint sets that strike NE and NNW; these are consistent with strong regional sets observed at most sites around Dublin Gulch. Although these structures should be considered in engineering analyses of the plant site, the extreme weakness of the bedrock here suggests that its engineering behavior will be controlled primarily by the strength of the intact rock and rock mass.

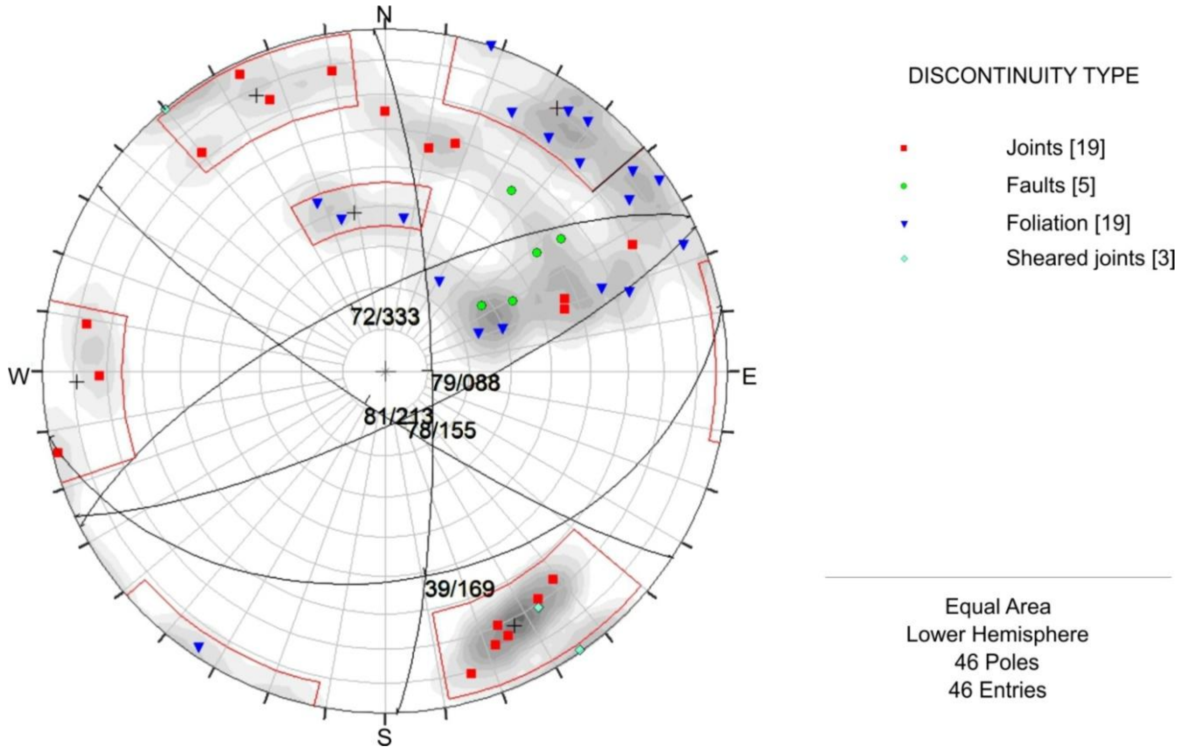

Figure 6. Discontinuities and structural sets mapped around the plant site.

Shear wave velocities were measured in one borehole (BH-BGC11-69) to a depth of 19.5 m. Although this depth is insufficient to calculate a Vs_{30}, a shear wave velocity of 832 m/s was used to approximate the site class as B/Rock (NBCC 2005).

3.2.4. Groundwater

Groundwater seepage was not noted in any of the excavated test pits in proximity to the Plant Site, up to 7.3 m depth below grade. A water level of 11.8 m below ground surface was observed from the vibrating wire piezometer installed in BH-BGC11-54. Groundwater is observed at shallower depths, close to the elevation of Dublin Gulch, in the valley bottom below the proposed plant site location.

3.2.5. Permafrost

Frozen ground was encountered in two (TP-BGC09-HL1-1 and -2) of the seven test pits excavated in this area. This suggests that sporadic permafrost could be encountered during site preparation.

3.2.6. Geological Hazards

No specific geological hazards have been identified by Stantec (2010) in the general area of the proposed plant site. Subsurface investigation (BGC 2010, 2011 and 2012) confirmed the presence of colluvial soils derived from shallow gravitational translational movement of native materials from further upslope, as is typically seen in rugged terrain.

3.3. Subsurface Conditions at the Proposed Crushers and Conveyors

3.3.1. General

The proposed layout of the crushers was changed during BGC's 2011 site investigation program in response to encountering poor rock quality at the locations proposed at the outset of the 2011 site investigation program. Additional holes were drilled at the new proposed location for the secondary and tertiary crushers. The available data from all test holes in the general vicinity of the crushers is summarized below.

Drawing 11 shows the distribution of test holes located in the general vicinity of the proposed crushers and conveyors. The subsurface observations from these holes are summarized in Table 7 and Table 8 below.

Table 7. Summary Subsurface Observations in Proposed Crusher Area

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth to Type 3 Rock (m)	Depth to Type 2 Rock (m)	Depth to Type 1 Rock (m)	Total Depth (m)	Frozen Ground Encountered
BH-BGC10-8	1036	NR[1]	NR	3.4	N/A[2]	3.4	8.5	26.2	No
BH-BGC10-18	1063	NR	4.5	N/A	4.5	7.3	19.5	30.2	No
BH-BGC11-35	986	NR	7.5	7.7	15.2	N/A	N/A	50.3	No
BH-BGC11-36	1002	NR	3.1	N/A	3.1	11.1	N/A	50.3	No
BH-BGC11-37	1034	NR	NR	N/A	N/A	3.8	7.2	43.6	No
BH-BGC11-38	1013	NR	5.2	0.2	5.4	18.3	N/A	50.5	No
BH-BGC11-40A	1050	NR	NR	7.3	7.8	9.1	14.6	33.2	No
BH-BGC11-40B	1050	NR	NR	N/A	8.7	10.7	15.5	45.7	No
BH-BGC11-50	1058	NR	5.2	N/A	5.2	12.2	15.2	41.2	No
BH-BGC11-62	1018	NR	1.5	N/A	1.5	4.6	24.4	35.1	No
TP-BGC09-HL4-5	987	0.2	>6.3	N/A	N/A	N/A	N/A	6.5	Yes
TP-BGC10-05	1064	0.2	0.4	N/A	0.6	4	N/A	4	No
TP-BGC10-06	1038	0.2	1.2	0.7	2.1	4.2	N/A	4.2	No
TP-BGC10-09	1038	0.2	0.5	N/A	0.7	1.0	N/A	1.0	No
TP-BGC10-10	1080	0.2	1.0	N/A	1.2	1.5	N/A	1.5	No
TP-BGC11-50	1011	0.2	3.4	N/A	3.6	N/A	N/A	5.8	No
TP-BGC11-51	972	0.2	>4.5	N/A	N/A	N/A	N/A	4.7	Yes
TP-BGC11-59	1065	0.2	0.9	N/A	1.1	2.6	N/A	2.6	No
TP-BGC11-60	1067	0.2[3]	2	N/A	N/A	N/A	N/A	2.7	Yes

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth to Type 3 Rock (m)	Depth to Type 2 Rock (m)	Depth to Type 1 Rock (m)	Total Depth (m)	Frozen Ground Encountered
TP-BGC11-127	1096	0.1	0.9	N/A	1.1	2.5	N/A	2.5	No
TP-BGC11-138	1050	0.3	1.2	N/A	1.5	N/A	N/A	6.3	No

Notes:
1. “NR” = no recovery
2. N/A – not observed or not applicable

3. A secondary layer of organics was present below the colluvium in TP-BGC11-60.

Table 8. Summary Subsurface Conditions in Proposed Conveyor Area

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth[4] to Type 3 Rock (m)	Depth[4] to Type 2 Rock (m)	Depth[4] to Type 1 Rock (m)	Total Depth[4] (m)	Frozen Ground Encountered
BH-BGC10-7	948	NR	18[1]	N/A	18	23.3	N/A	30	No
MW09-STU1	967	NR	9.2	N/A	9.2	N/A	N/A	14.3	Unknown
TP-BGC09-HL4-1	963	0.3	>1.6	N/A	N/A	N/A	N/A	1.9	Yes
TP-BGC09-HL4-2	910	0.3	>2.0	N/A	N/A	N/A	N/A	2.3	Yes
TP-BGC09-HL4-3	913	0.2	N/A	N/A	N/A	N/A	N/A	5	Yes
TP-BGC10-11	945	0.2	>4.1	N/A	N/A	N/A	N/A	4.3	Yes
TP-BGC11-90	981	0.2	>6.3	N/A	N/A	N/A	N/A	6.5	No
TP-BGC11-91	969	0.2	>1.9	N/A	N/A	N/A	N/A	2.1	Yes
TP-BGC11-92	933	0.2	>1.5	N/A	N/A	N/A	N/A	1.7	Yes
TP-BGC11-93	917	0.4	>1.3	N/A	N/A	N/A	N/A	1.9	Yes
TP95-47	N/A	0.2	>5.3	N/A	N/A	N/A	N/A	5.5	No

Notes:

1. The overburden materials may be completely weathered rock below some thickness of colluvium but poor recovery during drilling prevented confident classification.
2. A layer of ice 0.2 m thick was present below the organics in TP-BGC11-93.
3. Recovery was poor in top 18 m of BH-BGC10-7. This zone may be colluvium or weathered rock.
4. Depths indicated are below existing ground surface.

3.3.2. Overburden

The upper soil unit consists of a thin horizon of organic soil, rootlets, woody debris and plant matter ranging from 0.1 m to 0.4 m thickness and averaging approximately 0.2 m.

In the vicinity of the proposed crushers, the organic cover is underlain by colluvium ranging in thickness from 0.4 m to 7.5 m, averaging 2.6 m. The colluvium typically consists of a loose to compact, subangular to angular gravel and occasional cobbles in a silt and sand matrix, derived from transported weathered metasedimentary rock further upslope.

In the vicinity of the proposed conveyor line extending north from the tertiary crusher to the heap, the organic cover is underlain by colluvium (possibly overlying completely weathered rock) to depths up to 18 m (in BH-BGC10-7, where there was poor recovery up to 18 m). The colluvium consists of loose to compact subangular to angular gravel and occasional cobbles in a silt and sand matrix derived from transported weathered metasedimentary bedrock further upslope. This colluvium is expected to be typically frozen and ice-rich; however, limited subsurface information is available at depth.

Completely weathered rock is present below the colluvium and above bedrock in a number of the holes in the vicinity of the crushers. The highly to completely weathered rock ranges in thickness from 0.7 m to 7.7 m, averaging 4.8 m and consists of cobbley or sandy gravel.

3.3.3. Bedrock

In the vicinity of the proposed crushers, bedrock was observed immediately below colluvium in some holes and below a weathered rock horizon in other holes at depths ranging from 0.6 m to 15.2 m, averaging 4.0 m. Observed bedrock when first encountered consisted of moderately to highly weathered metasedimentary rock, typically Type 3 rock. Type 2 and Type 1 rock were encountered below the Type 3 rock. The contact between the metasedimentary and intrusive (granodiorite) rock was encountered in BH-BGC11-50 and both rock types were observed in this hole.

At the location of the proposed primary crusher and primary crusher haul road, the depth to Type 3 rock ranged from 0.6 m to 8.7 m. The depth to Type 2 rock ranged from 2.6 m to 19.5 m and the depth to Type 1 rock ranged from 8.5 m to 15.5 m. Rock mass quality and characteristics were inferred from five bore holes (BH-BGC10-8, BH-BGC10-18, BH-BGC11-40A, BH-BGC11-40B and BH-BGC11-50). Typical Rock Mass Rating (RMR, Bieniawski, 1976) values of about 40 were determined from the observed rock core and ranged from 20 to 75. RMR values generally increased with depth. Given the founding grades at the primary crusher, Type 1 rock is expected at founding elevation and will comprise the majority of the cut at the proposed primary crusher location.

At the current proposed location of the secondary crusher, Type 2 rock was encountered near surface at depths ranging from 1.0 m to 3.8 m. The depth to Type 1 rock was encountered at 7.2 m in BH-BGC11-37. Rock mass quality and characteristics were inferred from one borehole (BH-BGC11-37), located in the footprint of the secondary crusher. Average RMR values of about 40 were determined from the observed rock core and ranged from 20 to 65. RMR values generally increased with depth. The secondary crusher pad is currently proposed to be constructed from a cut fill balance and is planned to be founded on both Type 2 rock and fill.

At the current proposed location of the tertiary crusher, Type 2 rock was encountered at 4.6 m in BH-BGC11-62. Type 1 rock was encountered at 24.4 m. Rock mass quality and characteristics were inferred from one borehole (BH-BGC11-62). Average RMR values of about 35 were determined from the observed rock core and ranged from 20 to 45. RMR values generally increased with depth. The tertiary crusher pad is currently proposed to be constructed from a cut fill balance and is planned to be founded on both Type 2 rock and fill.

Down slope from the currently proposed locations, where the secondary and tertiary crushers were previously planned, rock quality is poorer. Type 3 rock is encountered at greater depths (ranging from 3 m to 15.2 m), with Type 2 rock at encountered at 11.1 m in BH-BGC11-36 and not encountered through the full depth of BH-BGC11-35, to 50.3 m. Type 1 rock was not encountered in the vicinity of the previously proposed secondary and tertiary crusher locations. Average RMR values of approximately 30 were determined from the observed rock core (in two boreholes, BH-BGC11-35 and BH-BGC11-36) and ranged from 19 to 55, typically increasing with depth. Although rock quality was observed to be better at the currently proposed locations for the secondary and tertiary crushers, given the observations of poor rock quality in the general area, it is possible that poor rock quality may be encountered at the currently proposed locations of the secondary and tertiary crushers.

At the location of the proposed conveyor, rock mass quality and characteristics were inferred from two boreholes (BH-BGC10-7 and MW09-STU1). Average RMR values of about 30 were determined from the observed rock core and ranged from 20 to 40. The depth to Type 3 rock was 18 m in BH-BGC10-7 and 9.2 m in MW09-STU1. Type 2 rock was encountered at a depth of 23.3 m and continued through the full depth of the hole to 30 m in BH-BGC10-7 and was not encountered through the full depth of MW09-STU1 to 14.3 m.

Geological mapping of surface structural features was carried out at several locations within 300 to 400 m of the proposed crushers. Mapped sites included natural outcrops, road cuts, test pits, and boreholes. Figure 7 shows joints, faults, and foliation planes mapped by BGC as well as Victoria Gold field geologists.

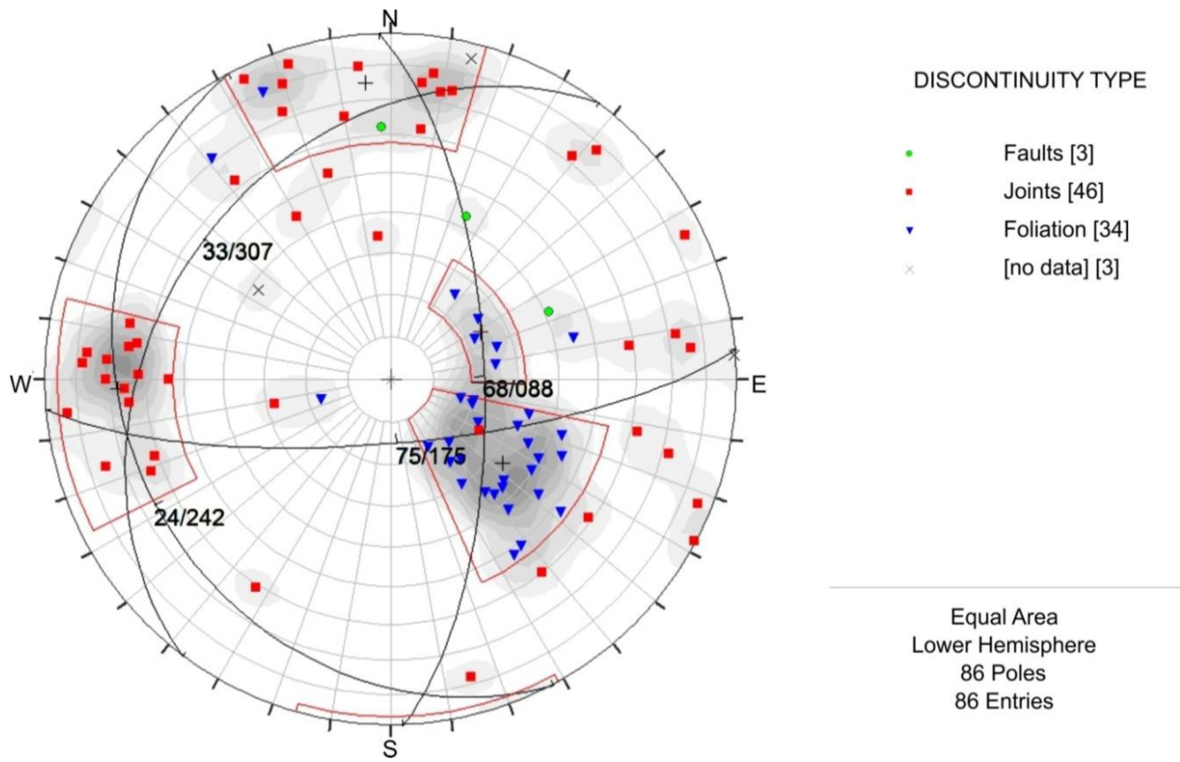

Figure 7. Discontinuities and structural sets mapped around the crushers.

The rock structure near the crushers, particularly the primary crusher, is strongly influenced by the axis of the major east-west anticline shown in Drawing 19 that marks the boundary between structural domains A and C. At the secondary and tertiary crushers on the northern limb of the anticline, the foliation of metasedimentary rocks dips northwest from 20-60 degrees, averaging 33 degrees. Borehole BH-BGC11-50, at the primary crusher, is near the axis of the anticline; here the foliation dips more shallowly west to southwest, averaging 24 degrees. Foliation planes are smooth at a small scale (centimetres to metres), but folded and undulating at the scale of 10s of meters, as shown by the variability of mapped orientations. Foliation in nearby outcrops dips more steeply, with an overall average in the area of approximately 32 degrees.

The angle of foliation planes in relation to the drill core axis in nearby vertical drillholes is generally similar to the field mapping data, with a mean dip of 26 degrees. The core retrieved from these boreholes was not retrieved using oriented core methods, and therefore the dip direction of these discontinuities is unknown. However, based on the relatively tight cluster of surface mapping observations, it can be inferred that many of the discontinuities observed in the core would likely also have similar orientations.

Two steep joint sets that cross-cut the foliation and each other are present near the crushers, dipping an average of 75 degrees towards the south and 68 degrees towards the east. The surfaces of these planes are fairly smooth, with average JRC values between 4 and 12. The bedrock lithology around the crushers is mostly quartzite, with a degree of contact metamorphism that increases towards the intrusive body from north to south.

Shear wave velocities were measured in 3 boreholes (BH-BGC11-36, BH-BGC11-40B and BH-BGC11-62) to a depth of 30 m. The site class (NBCC 2005) varied for this site between B – Rock at BH-BGC11-36 and BH-BGC11-40B and C- very dense soil and soft rock at BH-BGC11-62.

3.3.4. Groundwater

Groundwater seepage, which may represent thawing of seasonally frozen ground, was noted in two test pits in the vicinity of the conveyors (TP-BGC11-92 and TP-BGC11-93) at depths of 0.25 m and 0.4 m respectively. Groundwater seepage was not noted in any of the excavated test pits in the vicinity of the crushers, up to 6.5 m depth below grade. Four standpipe piezometers were installed in the vicinity of the crushers as part of the 2011 site investigation program. Water levels in these piezometers are tabulated in Table 10. Depth to the water table varies from approximately 8.5 m below ground surface near the primary crusher to approximately 20 m to 26 m below ground surface in the vicinity of the secondary and tertiary crushers.

Table 9. Crusher Area Groundwater Observations

Piezometer ID	Groundwater Depth (m below ground surface)	Location of Piezometer
BH-BGC11-35	24.6	Near Tertiary Crusher
BH-BGC11-36	19.9	Near Secondary Crusher
BH-BGC11-38	26.3	Tertiary Crusher
BH-BGC11-40B	8.5	Primary Crusher
MW09-STU1	15.4	Near Conveyor

Given the depth of the proposed cuts for the crushers, groundwater is expected to be encountered in the primary crusher cut. Seepage observations in test pits along the conveyor alignment suggest that groundwater may be encountered during foundation preparation for the conveyor.

3.3.5. Permafrost

Frozen ground was encountered in 11 of the 19 test pits excavated in this area, more commonly near the proposed conveyor line, and less commonly near the crushers. This suggests that sporadic patches of permafrost could be encountered during site preparation.

3.3.6. Geological Hazards

No specific geological hazards have been identified by Stantec (2009) in the general area of the proposed primary crusher (Drawing 20). The secondary crusher and tertiary crusher are located in an area identified as being subject to permafrost processes. Subsurface investigation (BGC 2010, 2011 and 2012) confirmed the presence of colluvial soils, as is typically seen in rugged terrain. The observed frozen ground mentioned in above was observed within the terrain unit identified as subject to permafrost processes, and is therefore consistent with the terrain analysis reported by Stantec (2010).

The conveyor is located in an area identified as being subject to both permafrost processes and surface seepage. Subsurface investigation (BGC 2012) confirmed the presence of colluvial soils, frozen ground and shallow seepage along the proposed conveyor alignment and is therefore consistent with the terrain analysis reported by Stantec (2010).

3.4. Subsurface Conditions at the Proposed Truck Shop

3.4.1. General

Drawing 11 shows the distribution of test holes located in the general vicinity of the proposed truck shop. The subsurface observations from these holes are summarized in Table 10, below.

Table 10. Summary Subsurface Observations in Proposed Truck Shop Area.

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Depth[2] to Type 3 Rock (m)	Total Depth[2] (m)	Frozen Ground Encountered
BH-BGC11-57	859	0.1	7.0	7.1	12.1	Yes
BH-BGC11-58	859	0.1	8.3	8.4	10.8	Yes
BH-BGC11-60	859	0.1	6.9	7.0	9.2	Yes
TP-BGC11-83	863	0.5	>0.8	N/A[1]	1.3	Yes
TP-BGC11-84	863	0.3	>2.3	N/A	2.6	Yes
TP-BGC11-85	865	0.5	>2.0	N/A	2.5	Yes

Notes:

1. N/A – not observed or not applicable.
2. Depths indicated are below existing ground surface.

3.4.2. Overburden

The upper soil unit consists of a horizon of organic soil, rootlets, woody debris and plant matter ranging in thickness from 0.1 m to 0.5 m and averaging approximately 0.3 m.

The organic cover is underlain by colluvium to depths ranging from 7.0 m to 8.4 m with an average depth of approximately 7.4 m. The colluvium consists of low plastic silt with some sand and some gravel derived from transported weathered metasedimentary bedrock further upslope.

Standard Penetration Testing (SPT) was completed in all three boreholes in the truck shop area. Tests in BH-BGC11-58 and BH-BGC11-60 are considered invalid since recovery in the samples was less than six inches. Three valid tests in colluvium were completed in BH-BGC11-57, all with SPT N_{60} greater than 50.

3.4.3. Bedrock

Bedrock in the proposed truck shop location was observed only by auger drilling and therefore rock mass parameters were not measured. Type 3, moderately weathered metasedimentary rock was present at depths ranging from 7.0 m to 8.4 m. Given the founding grades of the truck shop, Type 3 rock or better is expected at founding grades and will likely comprise the majority of the cut at the proposed truck shop location.

A single SPT test was completed was completed in the moderately weathered bedrock in BH-BGC11-57 with SPT N_{60} greater than 50.

3.4.4. Groundwater

Seepage was observed at 0.5 m in TP-BGC11-84. Two piezometers were installed in BH-BGC11-57 and BH-BGC11-58. No groundwater was observed in either set of measurements taken in late August 2011. The proposed founding grade of the truck shop is approximately 25 m below existing ground surface, so although not observed, groundwater could be present at greater depths.

3.4.5. Permafrost

Frozen ground, including excess ice, was observed in all test pits and boreholes completed in the proposed truck shop location, with excess ice confined to a depth of approximately 2-3 m. Frozen ground conditions are anticipated in the upper portions of the cut at the proposed truck shop location.

3.4.6. Geological Hazards

The truck shop is located in an area identified as being subject to permafrost processes (Stantec 2010, Drawing 20). Frozen ground was observed in both test pits and auger holes in the vicinity of the test pit, which is therefore consistent with the terrain analysis reported by Stantec (2010).

3.5. Subsurface Conditions at the Proposed Heap Leach Pad, Water Diversion and Process Management Ponds

3.5.1. Heap Leach Pad

3.5.2. General

Drawing 11 shows the distribution of test holes located in the vicinity of the proposed heap leach pad. The data suggest that this area can be divided into three zones with distinct overburden conditions: Heap Leach Upland, Heap Leach Valley Bottom, and Heap Leach Southern Edge above Valley Bottom (see Figure 8). The test hole observations from these three zones are summarized in Table 11, Table 12 and Table 13, respectively.

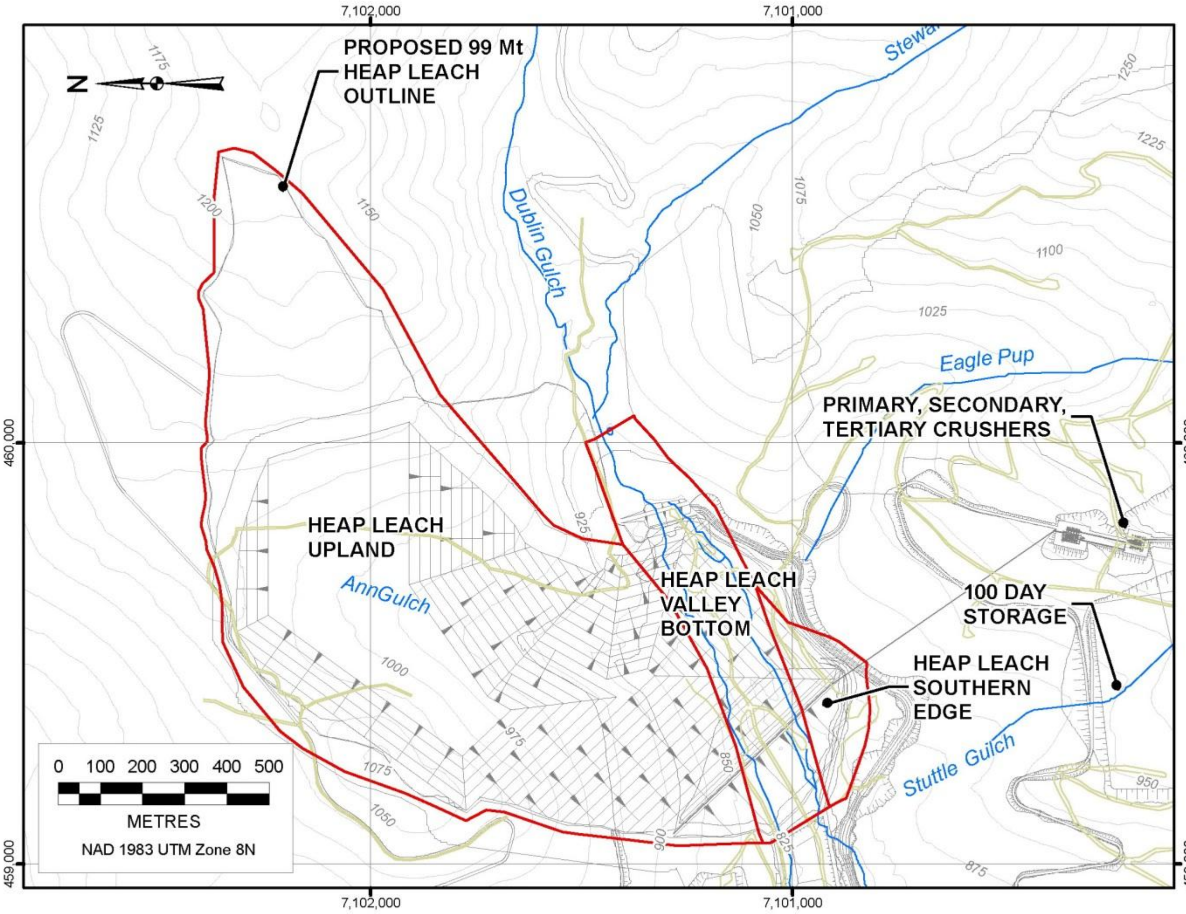

Figure 8. Heap Leach Pad Areas

Table 11. Summary Subsurface Observations in Proposed Heap Leach Pad Area - Upland

Test Hole ID	Appro x.Elev[1]. (m)	Overburden thickness (m)					Depth[8] to Type 3 Rock (m)	Depth[8] to Type 2 Rock (m)	Depth[8] to Type 1 Rock	Total Depth[8] (m)	Frozen Ground
		Organics	Colluvium	Till	Fill	Completely Weathered Rock					
TP-BGC09-A1[6]	884	0.2	1.1	>0.9	-	-	-	-	-	2.2	Yes
TP-BGC09-HL6-1	1038	0.1	2.4	-	-	-	2.5	-		6.5	No
TP-BGC09-HL6-2	1024	0.1	0.6	-	-	-	0.7	4.4	-	4.4	No
TP-BGC09-HL6-3	1010	0.2	3.0	-	-	3.0	-	-		6.2	No
TP-BGC09-HL6-4	981	0.2	3.8	-	-	0.8	-	4.8	-	4.8	Yes
TP-BGC09-HL6-5	1022	0.1	0.6	-	-	0.7	1.4	4.0	-	4.0	No
TP-BGC09-HL6-6	1062	0.2	2.8	-	-	2.5	-	-	-	5.5	No
TP-BGC09-HL6-7	1072	0.2	2.3	-	-	2.9	-	-	-	5.4	No
TP-BGC09-HL6-8	920	0.4	> 2.2	-	-	-	-	-	-	2.6	No
TP-BGC09-HL6-9	1042	0.2	0.6	-	-	0.7	1.5	3.8	-	3.8	Yes
TP-BGC09-HL6-10	939	0.2	1.0	-	-	3.6	-	4.8	-	4.8	Yes
TP-BGC09-HL6-11	976	0.2	0.5	-	-	0.2	0.9	2.8	-	2.8	No
TP-BGC09-HL6-12	957	0.1	> 5.9	-	-	-	-	-	-	6.0	No
TP-BGC09-HL6-13	959	0.1	1.5	-	-	0.2	1.8	2.4	-	2.4	No
TP-BGC09-HL6-14[5]	870	0.2	5.6	-	-	-	6.2	-	-	6.2	No
TP-BGC09-HL6-15	979	0.2	4.7	-	-	-	4.9	-	-	5.3	Yes
TP-BGC09-HL6-16	999	0.1	-	-	-	4.4	4.5	-	-	5.3	No
TP-BGC09-HL6-17	984	0.2	1.1	-	-	-	1.3	3.3	-	3.3	No
TP-BGC10-25	1023	0.1	0.7	-	-	-	0.8	-	-	6.5	No

Test Hole ID	Approx.Elev[1]. (m)	Overburden thickness (m)					Depth[8] to Type 3 Rock (m)	Depth[8] to Type 2 Rock (m)	Depth[8] to Type 1 Rock	Total Depth[8] (m)	Frozen Ground
		Organics	Colluvium	Till	Fill	Completely Weathered Rock					
TP-BGC10-26	1023	0.1	1.6	-	-	0.3	2.0	-	-	5.3	No
TP-BGC10-27	1045	0.2	1.5	-	-	2.3	4.0	4.5	-	5.3	No
TP-BGC10-28	1027	0.3	> 0.2	-	-	-	-	-	-	0.5	Yes
TP-BGC10-29	1049	0.2	1.0	-	-	>1.8	-	-	-	3.0	No
TP-BGC10-30	1060	0.2	3.8	-	-	-	4.0	-	-	5.5	No
TP-BGC10-31	1048	0.2	3.0	-	-	>2.1	-	-	-	5.3	No
TP-BGC10-35	880	0.5	2.0	-	-	-	2.5	-	=	5.5	No
TP-BGC10-41	942	0.3	4.0	-	-	1.8	-	-	-	6.1	No
TP-BGC10-42	917	0.3	> 3.2	-	-	-	-	-	-	3.5	Yes
TP-BGC11-52	1051	0.3	1.5	-	-	-	1.8	5.0	-	5.0	No
TP-BGC11-53	1103	0.3	0.7	-	3.0	-	-	4.0	-	4.0	No
TP-BGC11-54	1178	0.2	0.8	-	-	-	1.0	2.0	-	2.0	No
TP-BGC11-55	1209	0.2	0.7	-	-	-	0.9	2.0	-	2.0	No
TP-BGC11-56	1158	0.2	-	-	-	-	0.2	2.0	-	2.0	No
TP-BGC11-57	1144	0.2	2.3	-	-	-	2.5	5.0	-	5	No
TP-BGC11-58	1118	0.2	1.8	-	-	-	2.0	5.0	-	5	No
TP-BGC11-71	885	0.3	1.9	-	-	-	-	-	-	2.2	Yes
TP-BGC11-72	874	0.3	1.1	-	-	-	1.4	-	-	4.3	No
TP-BGC11-86	894	0.3	0.2	-	-	-	0.5	-	-	7.5	No
TP-BGC11-94	930	0.2	>4.8	-	-	-	-	-	-	5.0	Yes

Test Hole ID	**Approx.Elev[1]. (m)**	**Overburden thickness (m)**					**Depth[8] to Type 3 Rock (m)**	**Depth[8] to Type 2 Rock (m)**	**Depth[8] to Type 1 Rock**	**Total Depth[8] (m)**	**Frozen Ground**
		Organics	**Colluvium**	**Till**	**Fill**	**Completely Weathered Rock**					
TP-BGC11-132	922	0.1	1.0	-	-	-	1.1	4.0	-	5.3	No
TP-BGC11-133	984	0.2	1.0	-	-	3.2	-	-	-	4.4	Yes
TP-BGC11-145[7]	959	0.2	>4.6	-	0.3	-	-	-	-	5.1	No
DH-BGC09-AG3	884	1.2	6.4	-	-	-	-	7.6	-	13.7	No
BH-BGC10-1	1057	NR				-	1.8	10.4	-	20.4	No
BH-BGC10-2	949	NR				-	7.3	-	=	20.4	No
BH-BGC11-24	1208	NR	1.8	-	-	-	1.8	4.9	10.2	20.9	No
BH-BGC11-25	1183	-	-	-	-	0.9	0.9	2.3	-	20.4	No
BH-BGC11-26	1140	-	-	-	-	-	0	15.4	-	30.2	No
BH-BGC11-27	1100	-	-	-	-	2.1	2.1	13.7	-	26.5	No
BH-BGC11-28	1011	-	-	-	-	-	0	13.7		40.8	No
BH-BGC11-29	1045	-	-	-	-	-	0	18.6	24.1	41.2	No
BH-BGC11-30	952	-	1.5	-	-	-	1.5	-	-	35.1	No
BH-BGC11-31	918	-	15.2	-	-	1.6	16.8	-	-	35.1	No
BH-BGC11-59	884	-	4.6	-	-	1.2	5.8	9.8	-	30.2	Yes
MW09-AG1	1017	-	10.0	-	-	-	10.0	-	-	15.9	No
MW09-AG2	1009	0.3	10.3	-	-	1.9	12.5	-	-	15.9	No
MW10-AG3	997	0.1	7.5	-	-	3.9	11.5	-		16.8	No
MW10-AG5	934	0.2	1.0	-	-	5.1	6.3	-	-	20.8	Yes
MW10-AG6	906	0.2	4.4	-	-	4.6	9.2		-	17.7	No

Test Hole ID	Approx.Elev[1]. (m)	Overburden thickness (m)					Depth[8] to Type 3 Rock (m)	Depth[8] to Type 2 Rock (m)	Depth[8] to Type 1 Rock	Total Depth[8] (m)	Frozen Ground
		Organics	Colluvium	Till	Fill	Completely Weathered Rock					
TP95-51	912	-	> 5.5	-	-	-	-	-	-	5.5	No
TP95-52	899	0.2	-	-	-	2.9	3.1		-	3.1	No
TP95-53	917	0.4	> 1.2	-	-	-	-	-	-	1.6	Yes
TP95-54	911	-	6.4	-	-	0.9	-	-		7.3	No
TP95-55	904	0.2	> 5.2	-	-	-	-	-	-	5.5	No
TP95-56	920	-	> 6.0	-	-	-	-	-	-	6.1	No
TP95-57	902	2.7	-	-	-	2.2	4.9		-	5.5	No
TP95-58	889	1.8	>5.5	-	-	-	-	-	-	7.3	Yes
TP95-59	871	1.2	>4.9	-	-	-	-	-	-	6.1	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. “NR” = no recovery
3. N/A – not observed or not applicable
4. Frozen ground observations from older test pits (TP95-XX or TP96-XX) may not reflect current conditions
5. 0.2 m of Alluvium was present below the colluvium in TP-BGC09-HL6-14.
6. Till was observed in TP-BGC09-A1 to a depth greater than 0.9 m.
7. Drill pad fill was observed in TP-BGC11-145 to a depth of 0.3 m.
8. Depths indicated are below existing ground surface.

Table 12. Summary Subsurface Observations in Proposed Heap Leach Pad Area – Valley Bottom

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[7] toType 3 Rock (m)	Depth[7] toType 2 Rock (m)	Depth[7] toType 1 Rock (m)	Total Depth[7] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP-BGC09-DG1	923	-	-	-	> 2.5	-	-	-	-	2.5	No
TP-BGC09-HL6-14	870	0.2	4.7	-	-	-	4.9	-	-	5.3	No
TP-BGC10-17[4]	873	0.1	-	>1.5	4.4	-	-	-	-	6.0	No
TP-BGC10-18[4]	877	0.2	0.3	>7.0	-	-	-	-	-	7.5	No
TP-BGC10-21[3]	895	0.1	-	-	>6.4	-	-	-	-	6.5	No
TP-BGC10-22[4]	884	0.1	0.8	-	-	0.6	1.5	-	-	5.3	No
TP-BGC10-23[4]	880	-	-	-	> 5.0	-	-	-	-	5.0	No
TP-BGC10-24	858	0.1	-	-	>2.9	-	-	-	-	3.0	No
TP-BGC10-32[4]	902	0.1	-	-	>7.9	-	-	-	-	8.0	No
TP-BGC10-35	880	0.5	2.0	-	-	-	2.5	-	-	5.5	No
TP-BGC10-36	837	-	-	-	>4.5	-	-	-	-	4.5	No
DH-BGC09-DG1	923	-	-	-	6.1	1.5	-	7.6	-	12.8	No
BH-BGC10-3	878	-	-	-	9.3	-	-	9.3	10.5	50.7	No
BH-BGC10-4	858	-	-	-	8.5	0.2	8.7	11.8	25.0	31.0	No
BH-BGC10-5[4]	884	-	-	-	4.3	-	-	4.3	-	21.0	No
BH-BGC10-6[4]	876	-	-	16.4	-	-	-	16.4	-	28.9	No
BH-BGC10-17	836	-	-	-	7.3	-	7.3	17.8	-	37.3	No
BH-BGC10-23	849	-	-	-	>6.0	-	-	-	-	6.0	No
BH-BGC11-33	833	-	-	-	8.5	0.6	9.1	-	-	41.4	No
BH-BGC11-34	848	-	-	-	16.5	-	16.5	28.4	31.2	38.1	No
MW10-DG06	859	-	-	-	2.8	1.5	4.3	-	-	11.9	No

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[7] toType 3 Rock (m)	Depth[7] toType 2 Rock (m)	Depth[7] toType 1 Rock (m)	Total Depth[7] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP95-45	838	-	-	-	>5.5	-	-	-	-	5.5	No
TP95-46	867	-	-	-	2.4	0.7	3.1	-	-	3.1	No
TP95-50	872	-	-	-	2.4	1.3	3.7	-	-	3.7	No
TP96-230	845	-	-	-	>1.5	-	-	-	-	1.5	No
TP96-231	843	-	-	-	>3.5	-	-	-	-	3.5	No
TP96-232	851	-	-	-	>3.6	-	-	-	-	3.6	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. “NR” = no recovery
3. N/A – not observed or not applicable
4. Also considered in the proposed velocity reduction pond and rockfill diversion structures analysis
5. Frozen ground observations from older test pits (TP95-XX or TP96-XX) may not reflect current conditions
6. Stantec monitoring wells MW09-DG1 has been excluded from the table since it did not provide any soil information.
7. Depths indicated are below existing ground surface.

Table 13. Summary Subsurface Observations in Proposed Heap Leach Pad Area – Southern Edge of Proposed Heap above Valley Bottom

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[4] to Type 3 Rock (m)	Depth[4] to Type 2 Rock (m)	Depth[4] to Type 1 Rock (m)	Total Depth[4] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP-BGC10-17[3]	873	0.1	-	>1.5	4.4	-	-	-	-	6.0	No
TP-BGC10-18[3]	877	0.2	0.3	>7.0	-	-	-	-	-	7.5	No
BH-BGC10-6[3]	876	-	-	16.4		-	16.4	22.9		28.9	No
BH-BGC10-16[3]	878	NR				1.5	9.9	10.5	-	28.0	No
BH-BGC11-53	876	-	-	11.4	-	-	11.4	-	-	14.5	No
BH-BGC11-55	881	-	8.8	-	-	-	8.8	-	-	14.5	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. "NR" = no recovery
3. Also considered in the proposed velocity reduction pond and rockfill diversion structures analysis
4. Depths indicated are below existing ground surface.

3.5.3. Overburden

Overburden soil conditions are distinctly different in the Dublin Gulch valley bottom from those encountered above the valley bottom in Ann Gulch and south of Dublin Gulch along the southern edge of the proposed heap.

In the Uplands above the valley bottom, the upper soil unit consists of a thin horizon of organic soil, rootlets, woody debris and plant matter ranging from 0.1 to 2.7 m in thickness and averaging approximately 0.3 m (Table 11). The organic cover in the uplands overlies colluvium ranging in thickness from 0.2 m to 15.2 m, and averaging approximately 2.9 m (Table 11). The colluvium consists of loose to compact angular gravel with occasional cobbles in a silt and sand matrix, derived from transported weathered metasedimentary bedrock. The colluvium may also include variable amounts of organics, which are often observed in distinct layers within the colluvium. Highly to completely weathered metasedimentary rock is present below the colluvium in a number of boreholes and test pits. It ranges in thickness from 0.2 m to 5.1 m, averaging 2.2 m.

The overburden soils in the valley bottom have been reworked by historical placer mining activities. Placer tailings (fill) are observed from the ground surface to bedrock, with thicknesses ranging between 2.4 m and 16.5 m, and an average thickness of approximately 6.6 m (Table 12). The material encountered is generally a well graded, loose to dense, silty sand and gravel, ranging to sand and gravel with some silt and occasional cobbles and boulders. Loose zones were encountered within the placer tailings. There is little to no vegetative cover on most of the placer tailings.

The placer tailings in the valley bottom have highly variable particle size distribution and density, and are generally saturated. Recorded Standard Penetrometer (SPT) blowcounts, N, are summarized in Table 17 for the placer tailings within the footprint of the events ponds. No blowcount data are available in the placer tailings portion of the footprint of the heap leach pad, but the placer tailings materials are expected to have a similar variability in penetration resistance and associated strength and stiffness.

Highly to completely weathered metasedimentary rock is present below the placer tailings in some boreholes and test pits in the valley bottom. It ranges in thickness from 0.2 m to 1.5 m, averaging 0.9 m.

The overburden at the southern edge of the proposed HLF includes 4.4 m of placer tailings at TP-BGC10-17, and a variable thickness of till ranging up to 16.4 m (Table 13). The till is a compact to dense sandy silt to silty sand with some gravel. It must be noted that in borehole BH-BGC10-16 there was no soil recovery, so the contact between fill and undisturbed till has been inferred from observations in the adjacent test pit TP-BGC10-17. SPT blow counts recorded in BH-BGC11-53, within the till, have an average SPT N_{60} value of 44 and range from 23 to 55. Upslope from the valley bottom a debris flow deposit is present to a depth of 8.8 m in BH-BGC11-55. This material consists of fine silty sand with some gravel. SPT blow counts recorded in BH-BGC11-55, within the debris flow/colluvium have an average N_{60} value of 20 and range from 14 to 35.

3.5.4. Bedrock

Drawing 11 shows the plan view of the Heap Leach Pad and includes all the existing test holes in the area. Bedrock was observed in the uplands above Dublin Gulch immediately below colluvium at depths ranging between 0.0 and 16.8 m below existing grade (average depth to bedrock at 3.5 m where observed).

Bedrock was observed in the valley bottom at depths ranging between 1.5 and 16.5 m below existing grade, with an average depth to bedrock at 6.2 m where observed.

Bedrock was observed in four boreholes on the southern edge of the HLF and ranged in depth from 8.8 m to 16.4 m, averaging 11.6 m.

Observed bedrock consists primarily of Type 3 and Type 2 metasedimentary rock. Type 1 metasedimentary rock was encountered in a small number of boreholes, ranging in depth from 10.5 m to 31.2 m. The metasediments in general are observed as strongly foliated yellowish brown to dark grey phyllites interbedded with quartzites. The quartzites are variably gritty, micaceous, and massive. Phyllitic metasediments are composed of muscovite-sericite and chlorite.

The rock mass quality and characteristics have been inferred from observations in the boreholes completed by BGC as tabulated above, which were drilled within the heap leach facility footprint. Average RMR values of approximately 25 were determined from the observed rock core for Type 3 rock, approximately 35 for Type 2 rock and approximately 50 for Type 1 rock. A single SPT test was completed in the moderately weathered rock in BH-BGC11-55 with an N_{60} of 86.

Mapping of structural discontinuities was carried out at road cuts, valley cuts and outcrops within and around the heap leach pad footprint by BGC during summer 2011. The mapped discontinuity features, shown on stereonets below, are divided into two groups by area. Figure 8 shows discontinuities mapped in the upper (northern) portion of the HLF, between Tin Dome and the eastern edge of the heap leach pad. Figure 9 shows discontinuities mapped in Dublin Gulch valley bottom, between the proposed diversion berm and velocity reduction pond to the east, and the proposed process management ponds to the west.

The upper portion of the HLF covers three different structural domains; C, D, and E. Structural data in this area show two major joint sets and three distinct foliation orientations. Foliation dipping southeast, opposite of the regional average, was observed at a small outcrop on top of Tin Dome. This orientation probably represents the eastern limb of a small-scale fold with its axis running perpendicular to the average dip direction of foliation, similar to the folds observed in Domain A. Foliation measured on the upper eastern flank of Ann Gulch dips 27 degrees in the opposite direction (northwest). One mapping station, located at the upper northern end of Ann Gulch, showed foliation dipping north at 41 degrees.

While this is anomalous in the context of the BGC study, north-dipping foliation has been observed in the area north of Ann Gulch by Stephens et al. (2004). The two main joint sets in the upper heap leach facility cross-cut the foliation and each other, dipping 52 degrees towards the south and 84 degrees towards the east. Bedrock lithology in this area is mostly phyllite, with interbedded seams of quartzite 10-30 cm thick.

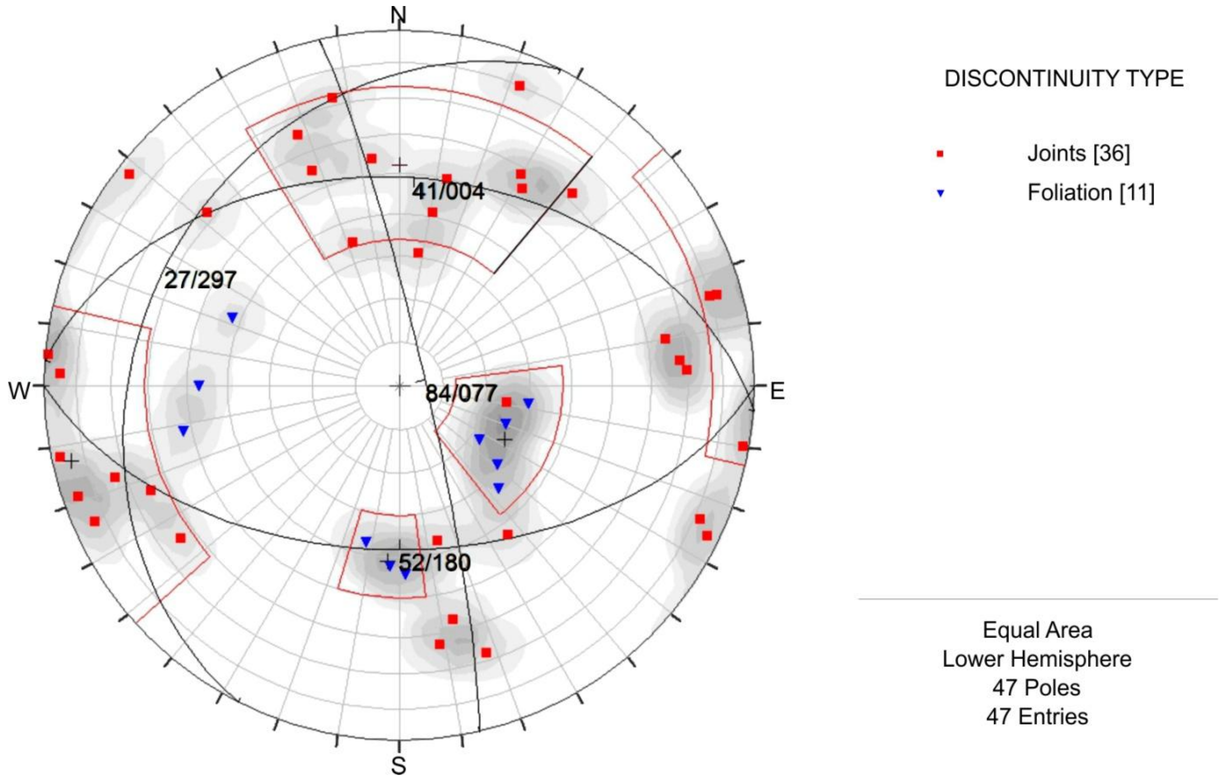

Figure 9. Discontinuities and structural sets mapped in the upper portion of the heap leach facility.

At the lower heap leach facility in Dublin Gulch valley bottom, the foliation dips shallowly (15-30 degrees) at a range of orientations from northwest to south-southwest. The foliation is cross-cut by two major joint sets dipping 81 degrees towards the east-northeast and 84 degrees towards the southeast. A third, minor joint set dips 67 degrees southwest. The surfaces of joints in this area vary from smooth to very rough (JRC 4-20), whereas the wavy foliation surfaces are mostly rough (JRC 16-20). The bedrock lithology in this area is mostly quartzite, with up to 40% phyllite at some outcrops interbedded in seams 10-20 cm thick.

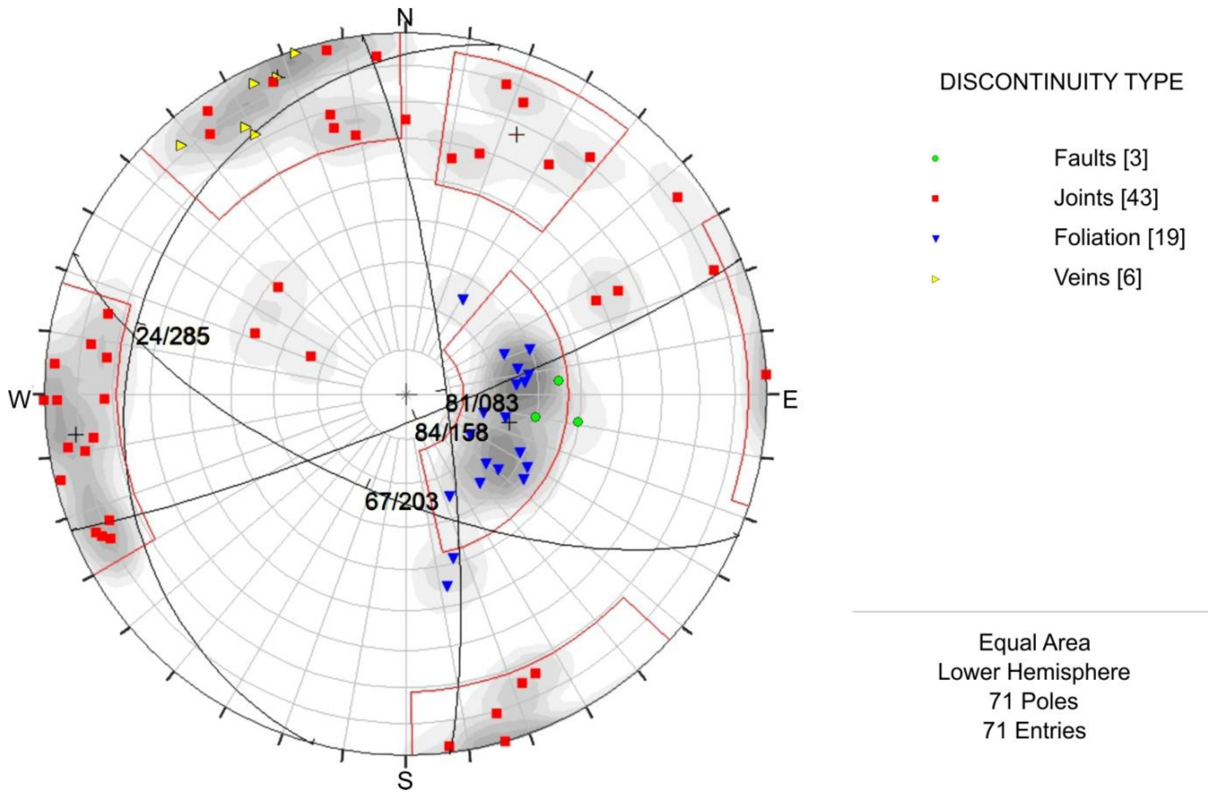

Figure 10. Discontinuities and structural sets mapped in the lower portion of the heap leach facility.

Shear wave velocities were measured in four boreholes within the heap embankment and heap pad (BH-BGC11-28, BH-BGC11-33, BH-BGC11-34 and BH-BGC11-59) to depths of 30 m, 30 m, 30 m and 28 m respectively. Although Vs_{30} values could not be calculated for all boreholes, site class (NBCC 2005) was determined to be C – very dense soil and soft rock.

3.5.5. Groundwater

Groundwater seepage was noted in five test pits, two in the upland areas of the heap leach facility (TP-BGC09-HL6-8 and TP-BGC11-133) and three in the valley bottom (TP-BGC11-131, TP-BGC11-134 and TP-BGC11-135). A number of standpipe piezometers were installed in 2011 in the footprint of the proposed heap leach facility. In addition, a number of standpipe piezometers have been installed in the footprint by Stantec (Stantec 2010). Typical groundwater observations for the area around the HLF are compiled in Table 14.

Table 14. Groundwater Observations in the General Area of the Proposed Heap Leach Pad (data compiled from Stantec 2010, BGC 2012)

Location	Well ID	Typical Groundwater Depth (m below ground surface)
Upland	MW09-AG1[1]	15.4
	MW09-AG2[1]	13.6
	MW10-AG3a[1]	9.9
	MW10-AG5[1]	7.0
	MW10-AG6[1]	12.6
	BH-BGC11-26[2]	16.4
	BH-BGC11-29[2]	7.8
	BH-BGC11-52[2]	4.1
Valley Bottom	MW09-DG1[1]	2.6
	MW09-DG2[1]	2.5
	MW10-DG06[1]	3.4
	BH-BGC11-30[2]	16.3
	BH-BGC11-32[2]	10.8
	BH-BGC11-33[2]	4.2
	BH-BGC11-34[2]	8.3
South Side of Heap Leach	BH-BGC11-55[2]	>12.7

Notes:
1. Stantec water levels are average water level since installation.
2. Water levels in BGC holes were measured in late August 2011.

The observed groundwater depths on the open slopes in the upper Ann Gulch valley range from 4.1 m below grade close to the middle of Ann Gulch to 15.4 m in the headwaters of Ann Gulch. Water levels in the Dublin Gulch valley bottom are variable, but can be expected to be closer to ground surface near streams and deeper below piles of tailings. It is anticipated that these levels will vary seasonally. The groundwater table has not been observed in the south corner of the heap leach facility on the south side of Dublin Gulch.

For preliminary design, it may be assumed that the natural groundwater table will be encountered at approximately 10-15 m depth below grade in the uplands, and at close to the elevation of existing drainage courses in the valley bottom. However, groundwater can be expected to be encountered locally at shallower depths, specifically when approaching the main drainages. This variability should be considered in planning, design and construction.

3.5.6. Permafrost

Frozen ground was encountered in the upper part of the HLF footprint (i.e. Upland area) in test pits TP-BGC09-A1, TP-BGC09-HL6-04, -09, -10, -15, TP-BGC10-28, -42, TP95-53 and -58, and boreholes BH-BGC11-59 and MW10-AG5. When observed in a plan view, many of the test pits are located on the eastern slope of Ann Gulch, and all except for TP-BGC10-28 align in a NE trend, covering the entire HLF footprint, from its most eastern edge to its western end at the heap leach containment dike. The reason for this connection between the frozen ground observations is unknown and might simply correspond to sporadic disconnected patches; nevertheless the continuity of the linear feature may deserve to be studied in more detail and accounted for during site preparation and construction. Frozen ground was typically encountered within gravels and gravels and sands with depths varying between 0.6 m to 2.8 m, and occasionally included excess ice. Test pit TP95-58 encountered visible ice encountered between 6.7 m to 7.3 m depth.

Frozen ground was not encountered in the valley bottom or on the southern edge of the proposed heap leach pad, but localized pockets of frozen ground may be present in these areas, particularly in areas where natural vegetative cover has not been disturbed by prior mining activities.

3.5.7. Geological Hazards

Around the HLF, geological hazards as determined by Stantec (2010) mainly include permafrost processes in the west-facing slopes at the upper part of the valley and surface seepage at the bottom of the valley between the rockfill diversion berm and rockfill embankment Drawing 20. Some of the south-facing lower and steeper slopes above Dublin Gulch are affected by rockfall and rockslide hazards (Drawing 20).

3.6. Water Diversion Structure

3.6.1. General

The water diversion system consists of a rockfill diversion berm and velocity reduction pond which will divert water coming from Dublin Gulch into a diversion channel. The channel carries water on the south side of Dublin Gulch, adjacent to the events ponds prior to discharging into Haggart Creek.

Overburden conditions encountered along the proposed diversion channel alignment, east of Stuttle Gulch, are generally different than those encountered further west in the valley bottom. The first segment is located at a higher elevation containing primarily colluvium and till; whereas, the second segment is underlain by placer tailings (fill, see Drawing 11 and Drawing 13). The ground conditions for the second (lower) segment of the diversion channel and sediment ponds are discussed in Section 3.7.

Ground conditions at the proposed Dublin Gulch diversion berm and velocity reduction pond are similar to those encountered at the valley bottom component of the heap leach pad.

Subsurface conditions in the area of the proposed Dublin Gulch diversion to the Stuttle Gulch energy dissipation structure are summarized below in Table 15.

Table 15. Summary Subsurface Observations in Proposed Dublin Gulch Diversion Area.

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[6] to Type 3 Rock (m)	Depth[6] to Type 2 Rock (m)	Depth[6] to Type 1 Rock	Total Depth[6] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP-BGC09-HL4-2[4]	910	0.3	>2.0	-	-	-	-	-	-	2.3	Yes
TP-BGC10-17[4]	873	0.1	-	>1.5	4.4	-	-	-	-	6.0	No
TP-BGC10-18[4]	877	0.2	0.3	>7.0	-	-	-	-	-	7.5	No
TP-BGC10-19[4]	899	0.2	>7.3	-	-	-	-	-	-	7.5	Yes
TP-BGC10-20[4]	905	0.2	0.4	-	-	-	0.6	-	-	3.2	No
TP-BGC10-21[3]	895	0.1	-	-	>6.4	-	-	-	-	6.5	No
TP-BGC10-22[3]	884	0.1	0.8	-	-	0.6	1.5	-	-	5.3	No
TP-BGC10-32[3]	902	0.1	-	-	>7.9	-	-	-	-	8.0	No
TP-BGC10-40[4]	816	-	-	-	>5.5	-	-	-	-	5.5	No
TP-BGC11-88[4]	922	0.2	>5.8	-	-	-	-	-	-	6.0	No
TP-BGC11-92[4]	933	0.2	>1.5			N/A	N/A	N/A	N/A	1.7	Yes
TP-BGC11-93[4]	917	0.4	>1.3			N/A	N/A	N/A	N/A	1.9	Yes
TP-BGC11-104[4]	832	0.1	0.6	2.8	0.3	3.5	-	-	-	5.6	No
TP-BGC11-110[4]	942	0.1	4.9	-	-	-	5.0	-	-	5.0	No
TP-BGC11-131[3]	921	0.1	>3.4	-	-	-	-	-	-	3.5	No
TP-BGC11-136[3]	910	0.1	>4.7	-	-	-	-	-	-	4.8	No
TP-BGC11-137[4]	943	0.1	1.0	-	-	1.4	2.5	5.0	-	5.0	No
DH-BGC09-DG-2[4]	828	-	-	-	14.6	-	-	14.6	-	16.3	No
BH-BGC10-5[3]	884	-	-	-	4.3	-	-	4.3	-	21.0	No
BH-BGC10-6[4]	876	-	-	16.4	-	-	16.4	22.9	-	28.9	No

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[6] to Type 3 Rock (m)	Depth[6] to Type 2 Rock (m)	Depth[6] to Type 1 Rock	Total Depth[6] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
BH-BGC10-15[3]	893	NR				-	-	8.8	-	21.0	No
BH-BGC10-16[4]	878	NR				1.5	9.9	10.5		28.0	No
BH-BGC11-41[4]	914	0.3	2.5	-	-	-	2.8	4.3	-	5.0	No
BH-BGC11-52[3]	909	-	11.3	-	-	0.6	11.9	14.3	19.5	22.6	No
BH-BGC11-53[4]	876	-	-	11.4	-	-	11.4	-	-	14.5	No
BH-BGC11-55[4]	881	-	8.8	-	-	-	8.8	-	-	14.5	No
TP96-127[4]	909	0.4	>5.1	-	-	-	-	-	-	5.5	Yes
TP96-129[4]	904	0.2	-	-	>4.9	-	-	-	-	5.1	Yes
TP96-130[4]	893	-	-	-	1.5	-	1.5	1.8	-	1.8	No
TP96-131[4]	901	-	2.3	-	1.5	-	3.8	4.2	-	4.2	Yes
TP95-49[4]	886	-	>4.9	-	-	-	-	-	-	4.9	Yes
DH95-152[4]	865	-				-	12.2	-	-	30.2	No
GT96-13[4]	904	-				-	-	12.2	18.3	36.3	No
MW96-15a[4]	943	-	4.5	-	-	-	4.5	-	-	9.2	No
MW09-STU2[4]	857	-	4.0	>6.1	-	-	-	-	-	10.1	No

Notes:

1. "NR" = no recovery
2. not observed or not applicable
3. Test holes relevant to the proposed rockfill diversion structure
4. Test holes relevant to the proposed diversion channel
5. Frozen ground observations from older test pits (TP95-XX or TP96-XX) may not reflect current conditions.
6. Depths indicated are below existing ground surface.

3.6.2. Overburden

The current diversion berm arrangement, as shown in Drawing 11, is located entirely within the approximate extent of placer tailings. There is a thin organic layer approximately 0.1 m thick underlain by placer tailings with thickness varying between 4.3 m to greater than 7.9 m. The tailings are generally loose to compact silty sands and gravels and soft to firm sandy silts. Recorded Standard Penetration Test (SPT) blowcounts, N, are summarized in Table 17 for the placer tailings within the footprint of the proposed process management ponds.

The first segment of the diversion channel runs along the north facing slope, south of Dublin Gulch, at an elevation of approximately 900 m, and is generally outside the extent of placer tailings. The overburden consists of a thin horizon of organic soil ranging from 0.1 to 0.4 m thick and averaging approximately 0.2 m. The organic cover is underlain by colluvium ranging in thickness from 0.3 m to 8.8 m, with an average thickness of approximately 3.5 m. Colluvium is described as a loose to compact gravelly sand with some silt to gravelly silt with some sand with occasional cobbles and boulders. Glacial till is observed locally west of Eagle Pup and east of Stuttle Gulch along the proposed alignment of the diversion channel. The observed thickness of the till unit varied between 2.8 m to 16.4 m. In this area till is described as being a firm to stiff (or compact to dense) silt and sand with some gravel.

3.6.3. Bedrock

The bedrock near the diversion berm was observed at a maximum depth of 11.9 m in borehole BH-BGC11-52 and a minimum depth of 1.5 m in TP-BGC10-22. The rock is described as slightly to moderately weathered metasedimentary rock (W2 – W3), weak to medium strong (R2 – R3), and with very closely spaced discontinuities and is Type 3 and Type 2 rock. The rock mass rating (RMR ’76) ranges from 20 to 50 with an average rating of about 40. For the mapped geological structures in this area refer to Drawing 19.

In the proposed diversion channel footprint, metasedimentary bedrock was encountered at depths ranging from 0.6 m to 16.4 m, averaging 13.1 m. Type 3 rock is present up to depths ranging from 0.6 m to 22.9 m with Type 2 rock below. Type 1 rock was encountered in two boreholes at depths of approximately 19 m. An average RMR value of approximately 35, ranging from 19 to 54, was determined from the observed rock core (BH-BGC10-6 and BH-BGC10-16).

Shear wave velocities were measured in one borehole (BH-BGC11-52) to a depth of 21 m. Although this depth is insufficient to calculate a Vs30 value, a shear wave velocity of 439 m/s was used to approximate a site class (NBCC 2005) of C – very dense soil/soft rock.

3.6.4. Groundwater

Within the diversion berm footprint, seepage was observed in two test pits at a depth of 3.0 m. Groundwater is expected to be close to the existing grade in the valley bottom near existing drainages and deeper further upslope on either side of the valley.

Within the proposed diversion channel footprint, seepage was observed in six test pits at depths ranging from 0.1 m to 5.5 m. A standpipe piezometer installed in BH-BGC11-55 downslope of the diversion channel alignment was dry to a depth of 12.7 m in August 2011.

3.6.5. Permafrost

Frozen ground was not encountered in test pits and boreholes within the footprint of the proposed diversion berm. Frozen ground was encountered in nine test pits along the proposed diversion channel alignment.

3.6.6. Geological Hazards

As shown in Drawing 20, the geological hazards identified by Stantec (2010) that might affect the construction of the diversion berm include surface seepage within the footprint of the placer tailings.

For the upper segment of the diversion channel, the presence of permafrost may affect construction and operation, while surface seepage in creek crossings will need to be considered also.

3.7. Events Ponds

3.7.1. General

The proposed event ponds are located immediately downstream (west) of the heap leach pad and below (south of) the process plant (Drawing 11), and are to be constructed in the Dublin Gulch valley bottom, between Stuttle Gulch in the east and Haggart Creek to the west.

The overburden soil encountered in the vicinity of the proposed process management ponds area mainly comprises placer tailings and occasional colluvium or till. Subsurface conditions in the area are summarized below in Table 16.

Table 16. Summary Subsurface Observations in Proposed Process Management Ponds Area

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[3] to Type 3 Rock (m)	Depth[3] to Type 2 Rock (m)	Depth[3] to Type 1 Rock (m)	Total Depth[3] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
BH-BGC11-32	820	-	-	-	19.8	-	19.8	-	-	24.4	No
BH-BGC11-65	820	-	-	-	>6.9	-	-	-	-	6.9	No
BH-BGC10-13	824	-	1.1	-	11.1	1.1	12.2	-	14.9	19.5	No
DH-BGC09-DG3	844	-	-	-	12.1	-	12.1	16.2	-	20.7	No
TP-BGC09-DG3	837	-	-	-	>5.0	-	-	-	-	5.0	No
TP-BGC10-38	830	-	-	-	>4.8	-	-	-	-	4.8	No
TP-BGC10-39	825	-	-	-	>5.5	-	-	-	-	5.5	No
TP95-43	822	-	-	-	>5.5	-	-	-	-	5.5	No
TP95-44	828	-	-	-	>5.5	-	-	-	-	5.5	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. N/A – not observed or not applicable.
3. Depths indicated are below existing ground surface.

3.7.2. Overburden

The placer tailings within the footprint of the events ponds and lower segment of the proposed diversion channel, above Dublin Gulch, have a variable thickness up to 19.8 m.

The placer tailings encountered within the footprint of the events ponds are generally a well graded, loose to compact, sand and gravel with some fines and some cobbles. Table 17 below summarizes the available SPT N-value for the boreholes within the area of the proposed events ponds. Detailed records of recorded N values can be found on the borehole logs in BGC's site investigation data reports (BGC 2011, 2012).

Table 17. Summary of Standard Penetration Test N-values for the placer tailings within the Process Management Ponds Footprint

Borehole ID	Depth Interval tested (m)	USCS	Number of Tests Meeting Refusal	N-value (raw blowcount, blows / 300 mm)	
				Average	Standard Deviation
BH-BGC10-13	0.8 – 5.0	GW, trace SW	1	30	8
BH-BGC11-65	0.8 – 6.9	SW/GW	2	21	6

3.7.3. Bedrock

Bedrock was encountered underlying the placer tailings within the footprint of the proposed events ponds in boreholes BH-BGC09-DG3, BH-BGC10-13 and BH-BGC11-32 (Drawing 11). Depth to bedrock ranged between 12.1 m and 19.8 m below existing grade. The placer tailings surface is highly variable and the majority of holes were completed on top of piles of placer tailings. Based on shear wave geophysical surveys, the typical thickness of placer tailings within the events ponds is approximately 10 m.

Observed bedrock consisted of moderately to highly weathered metasedimentary rock (i.e. Type 3 rock as described above (Section 1.2). Type 2 rock was encountered at 16.2 m in DH-BGC09-DG-3. Type 1 rock was encountered in a depth of 14.9 m in BH-BGC10-13. The metasediments are moderately to strongly foliated highly fractured.

Shear wave velocities were measured in one borehole (BH-BGC11-32) to a depth of 21 m. Although this depth is insufficient to calculate a Vs30 value, a shear wave velocity of 367 m/s was used to approximate a site class (NBCC 2005) of C – very dense soil and soft rock.

3.7.4. Groundwater

Groundwater was observed at approximately 3 m depth in two test pits within the valley bottom (TP-BGC09-DG3 and TP95-44).

A standpipe piezometer was installed in BH-BGC11-32; the groundwater level in this hole observed at 10.8 m below existing grade. BH-BGC11-32 is located near the crest of a placer tailings pile. The groundwater table is expected to be at or near the elevation of the Dublin Gulch surface water course in the vicinity of the proposed events ponds.

3.7.5. Permafrost

While frozen ground was not observed within the placer tailings in the valley bottom, isolated patches of permafrost may be encountered.

3.7.6. Geological Hazards

The geological hazards identified by Stantec (2010) that might affect the construction of the process management ponds are limited to surface seepage within the footprint of the placer tailings (Drawing 20).

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BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

VICTORIA GOLD CORPORATION

EAGLE GOLD PROJECT
DUBLIN GULCH, YUKON

2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

FINAL

PROJECT NO:	0792-006	DISTRIBUTION:	
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January 31, 2012
Project No: 0792-006

Mike Padula, Project Manager
Victoria Gold Corporation
584 – Bentall #4
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Dear Mr. Padula,

Re: Eagle Gold Project: 2011 Geotechnical Investigation For Mine Site Infrastructure Foundation Report - Final

Please find attached the final version of the aforementioned report. Should you have any questions or comments, please do not hesitate to contact the undersigned.

Yours sincerely,

BGC ENGINEERING INC.
per:

Pete Quinn, Ph.D., P.Eng.
Senior Geotechnical Engineer

Att.

PQ

EXECUTIVE SUMMARY

Introduction

Victoria Gold Corporation (Victoria), with assistance from Wardrop a Tetra Tech Company (Wardrop) is completing a feasibility study (FS) for development of the proposed Eagle Gold mine at Dublin Gulch, Yukon. BGC Engineering Inc. (BGC) was contracted by Victoria to complete geotechnical investigation work in support of FS design for mine site infrastructure. This report presents geotechnical engineering recommendations for selected mine site infrastructure resulting from the 2011 site investigation program performed between June and August 2011. The results of the site investigation have been published in a data report under separate cover.

The Eagle Gold property is located approximately 40 km north of Mayo, and 15 km northwest of Elsa (Drawing 01). The mine will comprise an open pit and heap leach pad; haul roads; waste rock storage areas; process plant; crushers and conveyors; truck shop; camp; water diversion structure; process water ponds; drainage ditches; sediment control structures and various other ancillary facilities. The current layout for the proposed mine facilities was received from Wardrop on November 23, 2011 (Drawing 02).

In the summer of 2011, BGC completed field investigations in support of geotechnical recommendations for mine site infrastructure. That work involved the excavation of ninety-six test pits, advancement of forty-six drillholes (29 Diamond holes and 17 Auger holes), and mapping of fifty-nine outcrops (natural exposures, existing road cuts and drill pads cuts) to characterize subsurface conditions relevant for foundation and earthworks design. Samples were taken from selected test pits and drillholes for index testing of soil and rock. Bulk samples of rock and placer tailings were also analyzed for a range of parameters related to the potential for re-use as select fill or aggregate. Downhole and surface geophysical investigations were completed, and plate load tests were conducted at selected locations of proposed building and machine foundations.

Several engineering reports were issued in draft by BGC in early 2011, with preliminary foundation and earthworks recommendations for a number of key facilities based on site investigation data from 2010, and in relation to the layout available at that time. Those reports are superseded by the recommendations contained herein.

This report provides geotechnical engineering recommendations for selected mine site infrastructure. This report does not provide recommendations for the open pit, the waste rock storage areas (WRSAs) or the heap leach facility (HLF), with the exception of cut slope recommendations for the Dublin Gulch diversion. Recommendations for the open pit and WRSAs will be provided by BGC under separate cover. Geotechnical design of the HLF, including the heap embankment, heap leach pad, Dublin Gulch diversion and events ponds, is the responsibility of Tetra Tech.

The report is organized as follows:

- Section 1.0 Introduction – general introductory material;
- Section 2.0 Proposed Facilities – general description of facilities under consideration in this report, along with design criteria used in the geotechnical analysis;
- Section 3.0 Site Conditions – a high level summary of generalized site conditions to provide basic context;
- Section 4.0 Material Properties – this section summarizes the assumed engineering properties of the in-situ foundation materials and processed engineering materials expected to be encountered or used in site development and earthworks construction;
- Section 5.0 Foundation Recommendations – this section provides recommendations of primary interest to the Structural designers, and includes recommendations for foundations and retaining walls;
- Section 6.0 Earthworks Recommendations – this section provides recommendations of primary interest to the civil designers, and includes recommendations for bulk earthworks, including cutting and filling to provide design grades for building pads, roads and other required surfaces;
- Section 7.0 Construction Materials – this section includes descriptions of different material types for use in earthworks construction, with discussion of quantities and schedule of required engineering materials, and quality and quantity of different material sources; and
- Section 8.0 Recommendations for Further Investigation – this section highlights areas of uncertainty where additional data will be required to support further development of geotechnical design, and presents recommendations for further work.
- Section 9.0 Closure.

Proposed Facilities

The proposed layout provided by Wardrop includes a number of buildings; including those containing the crushers, and those at the process plant site, truck shop and explosives storage areas (Drawing 02). Anticipated foundation dimensions and loads, and tolerable foundation deformations were provided by Wardrop on November 18, 2011.

The crushers will include three separate facilities – the primary, secondary and tertiary crushers – connected by conveyors. These facilities will include heavy vibratory machinery (i.e. the crushers) and associated machine foundations, in addition to the building foundations. The primary crusher will be accessed at the top by trucks from the pit, and will be founded some 25 to 30 m lower. Thus the primary crusher building will also function as a large retaining wall.

The line of crushers will be built on steeply sloping terrain, thus requiring cutting and filling to allow development of building pads. The cut above the lower platform below the primary crusher is shown on the general arrangement as approximately 90-95 m high at its highest point. This cut is shown as being lower above the secondary and tertiary crushers.

A line of conveyors will connect the tertiary crusher with the heap leach facility in the valley bottom below. The conveyors will be supported on sleepers where appropriate, and on elevated bents where necessary.

The process plant facilities will be developed on a cut/fill pad constructed on the hillside below Tin Dome, above and to the north of the Dublin Gulch valley bottom. The truck shop will be developed on a cut/fill pad constructed on the hillside below and to the west of the planned open pit, above and to the east of Haggart Creek.

A number of significant earth and rock cuts will be required for development of roads and building pads on the sloping ground at the project site, including the following:

- Main cut above primary crusher, currently shown in Wardrop grading information to be about 90-95 m in height at 1.75H:1V;
- Main cut at plant site building pad, currently shown by Wardrop to be 31 m in height at 2H:1V;
- Cuts along the Dublin Gulch diversion channel, currently shown by Wardrop to be up to about 26 m in height at 1.75H:1V to 2H:1V;
- Main cut at truck shop building pad, currently shown by Wardrop to be 20 m in height at 2H:1V;
- Main cut at upper edge of 100 day storage pad, currently shown by Wardrop to be 36 m in height at 1.75H:1V;
- Numerous other cuts of up to 15-20 m in height.

Foundation Recommendations

Recommendations have been provided for building foundations allowing for a minimum factor of safety of 3 against bearing capacity failure, and minimizing settlements within the objectives specified by Wardrop. Summary recommendations are:

Recommended Allowable Bearing Pressures for Ancillary Facilities

Bearing Stratum	Allowable Bearing Capacity (kPa)	
	Up to 2 m x 2 m Pad Footing	Up to 2 m x 20 m Strip Footing
Structural Fill[1]	250	150
Highly to Completely Weathered Rock	250	150
Type 3 Rock	500	300
Type 2 Rock	1000	600
Type 1 Rock	1500	1000

Notes:

1. Footings founded on structural fill require a minimum of 1.5 m of embedment (depth of bottom of footing below surrounding grade) to obtain the indicated allowable bearing capacity. Separate consideration of frost protection may be necessary.

Recommended Allowable Bearing Pressures for Specific Facilities

Facility	Expected Pad Elevation (m ASL)	Foundation Dimensions[1]	Expected Subgrade Conditions	Allowable Bearing Pressure[2] (kPa)
Primary Crusher	1026/1050[3]	Up to 12 m x 18 m mat	Type 1 rock	1000
Secondary Crusher[4]	1032	Up to 16 m x 16 m mat, 12 m x 5 m spread footing	Type 2 rock[4]	400
Tertiary Crusher[4]	1014	Up to 14 m x 9 m mat	Type 2 rock[4]	400
Conveyors from Tertiary Crusher to Heap Leach Facility	Varies along conveyor	Bents on 1.5 m x 6 m footing	Type 3 rock at ~ 5 m to 20 m depth below grade, typically 10 m expected	200 mm concrete-filled steel pipe piles socketed 2 m into Type 3 rock at ~ 10 m depth below grade can support 700 kN
		Sleepers at grade, on timber cribbing, where possible	Colluvium below stripped topsoil	N/A – adjustable foundations
Plant Site	860	3.5 m x 12 m	Highly to Completely weathered rock or structural fill	200
Truck Shop	855	3 m x 8 m	Type 3 rock	300

Notes:

1. Provided by Wardrop on 18 Nov 2011.
2. Based on factor of safety of 3 against bearing capacity failure and limiting settlements to those specified by Wardrop on 18 Nov 2011.
3. The lower portion of the primary crusher is at 1026 m. The elevation of the top of the primary crusher, where trucks will deposit ore is at 1050 m.
4. Crushers cannot be supported on regular structural fill. If the secondary and tertiary crushers must be built at planned grades well above the suitable bearing stratum, the gap between the bearing stratum and foundation grades can be made up by lean concrete or some other form of stiff fill material.

Earthworks Recommendations

There are a number of ground-related challenges to construction of earthworks and buildings at the proposed mine site. These include, generally:

- Presence of discontinuous permafrost, including some areas with excess ground ice;
- Relatively short "traditional" (i.e. spring/summer/fall) construction season, with specific challenges and limitations during other parts of the year (e.g. poor trafficability and material workability on hillsides before mid-summer; and long, harsh winter);
- Uncertain quality and quantity of required borrow materials;
- Presence of significant quantities of existing random fill (placer tailings); and
- Presence of steep slopes and geological hazards.

Each of these specific challenges requires consideration in the planning, design and construction of mine site infrastructure, as discussed in the report.

Engineered slopes constructed of structural fill or rock fill may be made at 2H:1V or flatter. Buildings should be set back a minimum of 10 m from the crest of fill slopes.

Where a structural fill is to be constructed on an existing natural slope, the fill should be keyed into the natural slope by excavating steps into the slope at the edge of successive lifts of structural fill.

Selected high fills, including those below the pit-crushers haul road and at the lower (north) end of the 100 day storage pad, may encroach into seasonal drainage areas or depressions with shallow groundwater. Particular care should be taken in these potentially wet areas to choose free draining, coarse granular fill materials, preferably angular durable rockfill, to prevent buildup of excess pore pressures in the fills.

Recommended slope geometry for cut slopes follows:

Recommended Permanent Cut Slope Angles – Area Specific

Area	Overburden		Slope below Overburden		Notes
	Thick-ness (m)	Steepest Cut Angle	Material	Steepest Cut Angle [1]	
Primary Crusher	2 - 4	2.5H:1V	Type 1, 2, 3 Rock	1.75H:1V	Design FS = 1.5; maximum slope height ~107 m; slope angle controlled by dip of foliation at about 30-32 degrees; benched slope design recommended; 8 m maximum bench height; 13 m minimum bench width; 0.25H:1V bench face angle.
100 Day Storage	3 - 4	2.5H:1V	Type 2, 3 rock	1.75H:1V	Design FS = 1.5[2]; slope angle controlled by dip of foliation at about 30-32 degrees; minimum distance of 80-100 m required between slope crest and toe of haul road / crusher platform fill slopes. Benched slope design is recommended as detailed above for primary crusher.
Truck Shop	5 - 8	2.5H:1V	Type 3 rock	1.75H:1V	Design FS = 1.5; maximum slope height = ~22 m; slope angle controlled by dip of foliation. Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Plant Site	3 - 7	2.5H:1V	Highly to completely weathered rock	2H:1V	Design FS = 1.5; maximum slope height ~35 m; Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Dublin Gulch Diversion	2 - 5	2.5H:1V	Till	2H:1V[3]	Design FS = 1.5; maximum slope height ~28 m; maximum cut angle assumes that the cut slope is dry.

Notes:

1. Maximum overall slope angle in the slope materials below the overburden depth. Overall slope angle defined by the line that connects the toe of the slope with the slope crest at the rock-overburden contact.
2. Recommended FS for the 100 day storage cut is 1.5 due to proximity to crushers and potential to undermine them in case of failure. FS = 1.3 could be considered when the cut is moved 80-100 m further from the crushers, however, the overall slope angle will still be controlled by the dip of the foliation and cannot be steepened significantly.
3. Assumed groundwater level is greater than 6 m below existing ground surface, which is inferred but not confirmed and requires further investigation.

At the primary crusher, 100-day storage, and truck shop areas, the cut slope design is controlled by the potential for failure of the rock along discontinuities defined by foliation in the metasedimentary rock. The foliation is expected to dip out of the slope at angles ranging from about 20° to 40°. The potential failure wedge that could form on slopes of this size is large enough to make mechanical support of the slopes impractical. Therefore a relatively shallow overall slope angle has been recommended. This overall slope angle is approximately parallel to the observed dip of the foliation, which essentially eliminates the potential for a planar failure at the slope-scale.

Bench scale failures are expected, including minor raveling and slumping, where the foliation is undercut; however, failures occurring on upper bench faces are not expected to adversely affect the infrastructure at the base of the slope due to the presence of the 13 m wide rockfall catchment benches. However, an allowance should be made in the design for spot bolting of loose blocks of rock on the bench faces in case specific weak structures are encountered. Mesh may also be required to contain poor quality rock that could ravel, should it be encountered, particularly on the bottom bench where service vehicles may be entering. Additionally, an 8 m wide rockfall catchment area should be included in the design at the upper and lower platform elevations. A 1 m high barrier (concrete or earth, or permanent fence) is recommended to be placed at the outer edge of the rockfall catchment area to deter encroachment into the catchment area by vehicles or personnel.

At the primary crusher, it is expected that blasting will be required to excavate the rock; therefore a benched slope design has been recommended. The recommended bench face angle is 0.25H:1V, which has been selected to facilitate controlled blasting. The maximum recommended bench height is 8 m. The minimum recommended bench width is 13 m to facilitate installation of a safety berm and to allow access for bench clean up. The bench width may need to be adjusted at detailed design to maintain the recommended overall slope angle of 1.75H:1V.

The recommendations provided for the primary crusher cut are based on assumed water levels and ground conditions, which are based on relatively sparse site characterization data. The consequences of a slope-scale failure at the primary crusher cut are perceived to be very high. Additional site investigations are recommended to reduce the current level of uncertainty in the understanding of ground conditions. The recommendations provided in this report assume that the design is controlled by the foliation of the metasedimentary rock. Future site investigation should verify that additional unfavorable conditions are not present and should be designed to characterize the orientation and condition of the contact between the meta-sedimentary and igneous rock, which is expected to daylight near the base of the cut.

At the 100-day storage area, the crest of the cut slope may daylight near the toe of the fill slope from the haul road and crusher platform. A minimum distance of 80-100 m between the cut slope crest and toe of fill is recommended to reduce the possibility of a slope failure at the 100-day storage area which could affect the crusher or haul road.

The recommended cut angle at the Dublin Gulch diversion assumes that the slope materials are unsaturated. If the slope materials are saturated, the recommend cut angle would decrease to 2.5H:1V. Current information regarding the depth to groundwater along the diversion is sparse. Future site investigation programs should be designed to characterize the groundwater depth along the diversion, and update the cut slope design, if appropriate.

The following Table provides general cut slope angle recommendations based on material type, for general application across the site for cut slopes less than 10 m high. It is assumed these cuts will be unsaturated and without adverse geologic structure. Cut slopes that do not meet these conditions should be reviewed on a case-by-case basis by the geotechnical engineer.

A rockfall catchment area should be provided at the base of all cut slopes. For soil slopes, the catchment area should be sloped back toward the cut slope at an angle of 4H:1V. The recommended minimum width of the rockfall catchment is 2.5 m below soil cuts, and 8 m below rock cuts.

Recommended Permanent Cut Slope Angles – General

Slope Material	Maximum Cut Slope Angle[1]	Maximum Cut slope Height	Notes
Colluvium	2.5H:1V	10 m	
Till	2H:1V	10 m	
Highly to completely weathered rock (excavatable)	2H:1V	10 m	
Type 3 rock (generally excavatable)	1.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 2 rock (generally rippable)	1H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 1 rock (may require blasting)	0.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered

Note:

1. Maximum cut slope angles assume the slope is < 10 m high, unsaturated, and without adverse geologic structure.

Construction and Waste Materials

Material take offs (MTOs) with earthworks quantity estimates were provided by Merit Consultants International on January 6, 2012. The MTOs include numerous line items for quantities of earth or rock materials and various types of granular borrow required for construction of the mine site facilities, including the following approximate quantities of specific materials:

- Approximately 2.9 million m^3 of engineered fill, which includes approximately 2.1 million m^3 of engineered fill for the heap containment dyke and diversion embankment, selected from a variety of sources, including processed placer tailings, non-durable rock obtained during bulk earthworks activities, and possibly durable waste rock from mining. This engineered fill includes the following general categories of materials:
  - General fill,
  - Structural fill,
  - Durable rock fill, and
  - Non-durable rock fill;
- 298,000 m^3 of crushed durable rock to produce a well-graded material for the heap overliner;
- Various minor quantities of miscellaneous engineering materials, including silt/fines for liner construction, transition/filter materials, drainage materials, rip rap, concrete aggregate, and road pavement structure materials.

The report includes suggested specifications for various materials to be used in earthworks construction.

This project will involve the movement of large quantities of earth and rock fill in a relatively short construction period (currently understood to be about three years) and within a limited footprint in rugged terrain. It will be challenging to manage material movement to meet construction schedule requirements. An effort has been made to understand the temporal nature of planned material movement, with consideration of MTOs provided by Wardrop, Tetra Tech, and Knight Piésold, as compiled by Merit Consultants and received by BGC on January 06, 2012.

The report presents a breakdown of material quantities over time, based on an analysis of quarterly supply and demand, as listed in the following table. Cut quantities are shown as positive numbers, being quantities available for use (or intended for disposal). Fill quantities are shown in brackets to represent negative numbers, being deficit quantities required for construction.

Quarterly Demand for Cut and Fill Quantities, as inferred from MTOs from Merit

Used for Material Balance	Category	Material Quantity (m^3)												
		Q4 2012	**Q1 2013**	**Q2 2013**	**Q3 2013**	**Q4 2013**	**Q1 2014**	**Q2 2014**	**Q3 2014**	**Q4 2014**	**Q1 2015**	**Q2 2015**	**Q3 2015**	**Total**
No	Strip and stockpile topsoil	50,738	26,026	147,437	0	0	0	37,015	7,701	0	0	44,485	0	**313,402**
	Excavate and dispose waste rock in waste dump	77,319	0	261,201	0	0	0	54,344	35,234	0	0	67,842	0	**495,940**
	Excavate colluvium	35,050	0	168,300	0	0	0	18,000	5,700	0	0	0	0	**227,050**
	Excavate rock	10,758	0	375,555	0	0	0	4,278	0	0	0	0	271,369	**661,960**
	Excavate permafrost	3,500	0	34,900	0	0	0	0	1,200	0	0	0	0	**39,600**
	Local cut and fill	76,791	0	239,749	0	0	0	45,210	13,251	0	0	0	839,463	**1,214,464**
Yes	Excavate and stockpile suitable materials	208,271	0	185,885	0	0	0	0	24,632	0	0	0	133,699	**552,487**
	General excavation	333,280	0	1,182,390	0	0	0	75,400	120,000	0	0	0	0	**1,711,070**
	Excavate placer tailings	0	0	876,000	0	0	0	0	0	0	0	0	0	**876,000**
	Subgrade preparation	0	0	(18,300)	(104,600)	0	0	0	(3,500)	0	0	0	0	**(126,400)**
	Other materials	(3,520)	0	(58,823)	(3,100)	0	0	(12,000)	(298,000)	0	0	0	0	**(375,443)**
	Fill from stockpile	(18,110)	0	(355,643)	0	0	0	(149,191)	(17,461)	0	0	0	(7,430)	**(547,835)**
	Fill	(70)	0	(126,518)	(1,119,000)	0	0	(743,000)	0	0	0	0	0	**(1,988,588)**
Material balance - each quarter		519,851	0	1,684,991	(1,226,700)	0	0	(828,791)	(174,329)	0	0	0	126,269	**101,291**
Material balance - cumulative		519,851	519,851	2,204,842	978,142	978,142	978,142	149,351	(24,978)	(24,978)	(24,978)	(24,978)	101,291	**101,291**

Note:

1. Quantities (in brackets) indicate deficit quantities, or fill to be derived from elsewhere. The material categories have been modified slightly from those received in information provided by Merit.

Summary of Borrow Material Availability

Borrow Source	Material Types	Estimated Volumes (in situ volumes, except where noted)	Comments
Pit Pre-Strip	Durable rock fill Non-durable rock fill Concrete aggregate Heap overliner Rip rap	Very large. Available volumes depend on the sequence of mining activities, although materials can be developed prior to mining activities by developing a quarry prior to pre-strip.	Source consists of weathered granodiorite and weathered silicified metasedimentary rock, typically quartzite. Suitable concrete aggregate has not yet been identified, and requires further study. Testing of material for use as heap overliner was commissioned by Tetra Tech, and the results are not available to BGC at the time of writing. Availability of rip rap in desired block size of 500-600 mm will require further input from mine plan, and careful selection. Most near surface weathered rock suggests excavated block size of approximately 100-300 mm.
Ann Gulch Central Knob	Non-durable rock fill	Up to approximately 900,000 m^3, subject to further input from Tetra Tech.	Grading plans showing the volumes of anticipated rock excavation are not available to BGC at the time of writing.
Steiner Zone	Same as for Pit Pre-strip	Up to approximately 200,000 m^3, assuming quarry depth of 5 m	Very little information is known about this area. Further subsurface investigation is required to confirm quality and quantity of available materials.
Dublin Gulch Placer Tailings	General Fill Structural Fill Concrete aggregate Heap overliner Rip rap	Approximately 2.0 million m^3, of which about 1.1 million m^3 is above the groundwater level	Materials are highly variable, and will require processing through screening, crushing and/or washing to develop the required material specifications. Oversized materials (> 75 mm) screened from the tailings may be suitable for use, after crushing, as heap overliner or concrete aggregate pending further analysis. Some rip rap can be developed from the screened oversize material; however, the quantity of 500-600 mm particles is expected to be small and would require careful selection.

Borrow Source	Material Types	Estimated Volumes (in situ volumes, except where noted)	Comments
Haggart Creek Placer Tailings	General Fill Structural Fill	Approximately 750,000 m^3 available above the elevation of Haggart Creek	No subsurface information is available to support the quantity estimate. Available volume of suitable material is estimated from visual classification of surficial materials present in several distinct piles.
Silt Borrow	Silt liner	Approximately 220,000 m^3	Available silt materials are frozen and potentially ice-rich, and will require thawing and drying prior to use.

The current analysis shows a peak excess of approximately 2.2 million cubic metres of excavated material which will require storage during the first year of the project. This excess supply will be drawn down over the following year, leaving a small excess of available fill at the end of construction.

The material categories listed in the previous table have been modified slightly from categories provided in the MTOs from Merit Consultants on January 6, 2012.

Bulk earthworks activities will generate several types of material that are unsuitable for immediate use, or may not be suitable for any use, thus necessitating temporary storage or permanent disposal. Decisions on ultimate disposition may require further consideration of the need for soil for reclamation. Preliminary information suggests the development of the following materials requiring storage or disposal:

- Topsoil – these materials will be required for reclamation. It will be necessary to develop stockpiles to store these materials during construction and mine operation. The current estimate of 313,000 m^3 does not yet include open pit pre-stripping;
- Ice-rich permafrost – these materials will be unsuitable for immediate re-use in any application. They may be suitable for re-use in reclamation after thawing and draining of excess water. These materials will require careful storage after excavation and prior to use, as they will be weak and unstable when thawed. It may be necessary to develop specific storage areas with containment structures and water management infrastructure. Current estimates indicate approximately 40,000 m^3 of ice-rich permafrost will be removed during development of the heap leach facility, and with additional volumes from other areas on site (quantity currently unknown), all requiring management during construction and mining operations;
- Colluvium – some of the shallow colluvial soils removed during bulk excavation work will contain excessive amounts of deleterious materials, such as organic inclusions or excess proportions of fines. Current estimates suggest approximately 227,000 m^3 of colluvium requiring permanent disposal or storage for re-use in reclamation.
- Waste rock - these materials are indicated by Merit and Wardrop as unsuitable for re-use as construction fills and are intended to be permanently disposed in designated disposal areas. In general they correspond to soils or rock with deleterious materials and may include excess fines or excess ice. Current estimates indicate approximately 500,000 m^3 of unsuitable material that needs to be excavated, removed and disposed, either in the waste rock storage areas, or other disposal areas to be determined.

Work was done to explore potential borrow sources, including effort to determine the characteristics of the placer tailings; investigation of potential silt borrow near the proposed laydown area, evaluation of various rock sources for use as engineered fill; and, evaluation of placer tailings and rock near the proposed open pit for potential use as concrete aggregate. Summary information for various borrow sources is presented in the Table "Summary of Borrow Material Availability."

Recommendations for Further Work

This report has provided feasibility study level geotechnical recommendations for mine site infrastructure. There are several areas of uncertainty that should be further explored as part of detailed design. The following list provides recommendations for further investigation.

- Diamond drillholes:
  - Vertical holes at all three crushers to better establish depth to suitable bearing stratum across the facilities' footprints;
  - Inclined holes in the area of proposed rock cuts at the crushers and 100 day storage pad;
  - Vertical holes at the plant site to better determine depth to suitable bearing stratum within the extent of the building pad;
  - Allowance for additional holes within the footprint of the heap leach facility, in the event Tetra Tech consider additional data warranted;
  - Allowance for additional holes at major cuts such as that along the phase 1 heap access road;
  - Allowance for holes for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Auger holes (with CRREL barrel available):
  - Conveyor bent foundation locations between tertiary crusher and heap leach facility;
  - Along the alignment of the proposed Dublin Gulch diversion channel;
  - In Eagle Pup to confirm the extent of the ice-rich lobate feature in the valley bottom;
  - At the revised truck shop buildings and cut locations;
  - Allowance for holes for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Plate load tests at plant site and all three crushers;
- Design and construction of a test fill embankment to determine whether high quality structural fill would be suitable to support the secondary and tertiary crushers;
- Sampling and strength testing of materials selected for heap embankment fill, if considered necessary by Tetra Tech;
- Additional sampling and testing of granodiorite from the pit area and Steiner zone for possible use as concrete aggregate. Obtain materials engineering advice to guide this process, and including trial mix designs possibly with additives to make use of local aggregates, and trial design mix for lean concrete for use in raising grades at crushers;
- Sample mixes for low strength concrete as stabilized fill

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LIMITATIONS

BGC Engineering Inc. (BGC) prepared this document for the account of Victoria Gold Corporation (Victoria). The material in it reflects the judgment of BGC staff in light of the information available to BGC at the time of document preparation. Any use which a third party makes of this document or any reliance on decisions to be based on it is the responsibility of such third parties. BGC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this document.

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1.0 INTRODUCTION

1.1. General

Victoria Gold Corporation (Victoria), with assistance from Wardrop a Tetra Tech Company (Wardrop) is completing a feasibility study (FS) for development of the proposed Eagle Gold mine at Dublin Gulch, Yukon. BGC Engineering Inc. (BGC) was contracted by Victoria to complete geotechnical investigations for mine site infrastructure. This report presents geotechnical engineering recommendations for selected mine site infrastructure resulting from geotechnical investigations performed between June and August 2011.

1.2. Project description

The Eagle Gold property is located approximately 40 km north of Mayo, and 15 km northwest of Elsa, as illustrated in Drawing 01. The mine will comprise an open pit and heap leach pad; haul roads; waste rock storage areas; process plant; crushers and conveyors; truck shop; camp; water diversion structure; process water ponds; drainage ditches; sediment control structures and various other ancillary facilities. The current layout for the proposed mine facilities, as received from Wardrop on November 23, 2011, is illustrated in Drawing 02.

1.3. Previous Investigations

Site conditions at the Eagle Gold site have been partially described in several reports as follows:

- Report on 1995 Geotechnical Investigations for Four Potential Heap Leach Facility Site Alternatives, First Dynasty Mines, Dublin Gulch Property. (Knight Piésold, 1996a).
- Report on Feasibility Design of the Mine Waste Rock Storage Area, First Dynasty Mines, Dublin Gulch Property. (Knight Piésold, 1996b).
- Field Investigation Data Report, Dublin Gulch Project, New Millennium Mining. (Sitka Corp, 1996).
- Hydrogeological Characterization and Assessment, Dublin Gulch Project, New Millennium Mining. (GeoEnviro Engineering, 1996).
- BGC Engineering Inc. 2010. Site Facilities Geotechnical Investigation Factual Data Report. Eagle Gold Project, Victoria Gold Corporation.
- Stantec. 2010. Project Proposal for Executive Committee Review. Pursuant to the Yukon Environmental and Socio-Economic Assessment Act. Eagle Gold Project, Victoria Gold Corporation.
- BGC Engineering Inc. 2011a. 2010 Geotechnical Investigation for Mine Site Infrastructure, Factual Data Report. Eagle Gold Project, Victoria Gold Corporation.
- BGC Engineering Inc. 2011b. 2011 Geotechnical Investigation for Mine Site Infrastructure, Factual Data Report. Eagle Gold Project, Victoria Gold Corporation.

In 1996, Knight Piésold completed a feasibility level geotechnical study to evaluate the surficial materials and bedrock conditions at four potential heap leach pad locations, two potential waste rock storage areas, and the open pit. Groundwater wells and two thermistors were installed in selected drillholes at that time. Test pitting and diamond drilling were completed from June to September 1995 at upper Bawn Bay Gulch, lower Dublin Gulch, the north side of Lynx Creek, and at the confluence of Haggart and Lynx Creeks.

In 1996, Sitka Corp completed test pits and diamond drillholes in Bawn Bay Gulch, Eagle Pup, Stewart Gulch, and Platinum Gulch for preliminary design of the heap leach and waste rock facilities. Auger holes were drilled in Gill Gulch to evaluate it as a potential borrow source of silt material for use as a liner for the heap leach facility. Monitoring wells were installed in Bawn Bay Gulch and Eagle Pup. Eight thermistor strings were installed.

In 2009, BGC was engaged to gather factual data describing subsurface conditions at the proposed heap leach and waste rock storage facility locations. The work involved the excavation of sixty-nine test pits and advancement of seven boreholes. Thermistor strings were installed in three boreholes to gather temperature data. Dynamic cone penetration profiles were obtained at two borehole locations to obtain information about material density. Dynamic cone soundings were attempted in two other holes and met with refusal. Groundwater monitoring wells were installed by Stantec in two of the seven BGC boreholes.

In 2010, Stantec presented a Project Proposal which included general site conditions such as regional geology, physiography, drainage, climate and seismicity. Air-photo based terrain mapping and an evaluation of geological hazards affecting the project area were both also described in this report.

In 2010, BGC was engaged to develop a geotechnical site investigation program in support of FS for proposed mine site infrastructure. A total of forty-nine test pits and twenty-five drill holes were completed to characterize the overburden material and bedrock conditions. Additionally, three cut slopes were logged for exposed soil and rock conditions, and core from one client-drilled condemnation hole was logged for geotechnical purposes. Laboratory testing was completed on selected samples for moisture content, and representative samples were also tested for Atterberg Limits and grain size analysis. Various other lab tests were also completed on bulk samples of placer tailings being considered for potential use as select fill or aggregate.

In the summer of 2011, BGC completed additional field investigations in support of geotechnical recommendations for mine site infrastructure. That work involved the excavation of ninety-six test pits, advancement of forty-six drill holes (29 Diamond holes and 17 Auger holes), and mapping of fifty-nine outcrops (natural exposures, existing road cuts and drill pads cuts) to characterize subsurface conditions relevant for foundation and earthworks design. Samples were taken from select test pits and boreholes for index testing of soil and rock. Bulk samples of rock and placer tailings were also tested to evaluate their potential for re-use as select fill or aggregate. Downhole and surface geophysical investigations were completed, and plate load tests were conducted at selected locations of proposed building and machine foundations.

BGC issued a series of draft reports in March and April 2011, with preliminary foundation and earthworks recommendations for a number of key facilities. The recommendations presented in those reports are superseded by those contained herein

1.4. Scope of Work

BGC was engaged to provide further geotechnical investigation work to address gaps in the data required in support of FS design for mine site infrastructure. The 2011 site investigation program was conducted between June and August 2011 and the results have been published under separate cover (BGC 2011b). BGC was also engaged to provide geotechnical engineering recommendations in support of the FS-level design of mine site infrastructure.

This foundation design report provides geotechnical engineering recommendations for selected mine site infrastructure as noted in Section 2.0. This report does not provide recommendations for the open pit, the waste rock storage areas (WRSAs), or the heap leach facility (HLF), with the exception of cut slope recommendations for the Dublin Gulch diversion. Recommendations for the open pit and WRSAs will be provided by BGC under separate cover. Geotechnical design of the HLF, including the heap embankment, heap leach pad, Dublin Gulch diversion structures and events ponds, is the responsibility of Tetra Tech.

1.5. Report Outline

The report is organized as follows:

- Section 1.0 Introduction – general introductory material;
- Section 2.0 Proposed Facilities – general description of facilities under consideration in this report, along with design criteria used in the geotechnical analysis;
- Section 3.0 Site Conditions – a high level summary of generalized site conditions to provide basic context. Readers are referred to BGC (2011b) for greater detail as required, and a more detailed summary of site conditions is provided in Appendix A of this report;
- Section 4.0 Material Properties – this section summarizes the assumed engineering properties of the in-situ foundation materials and processed engineering materials expected to be encountered or used in site development and earthworks construction;
- Section 5.0 Foundation Recommendations – this section provides recommendations of primary interest to the Structural designers, and includes recommendations for foundations and retaining walls;

- Section 6.0 Earthworks Recommendations – this section provides recommendations of primary interest to the civil designers, and includes recommendations for bulk earthworks, including cutting and filling to provide design grades for building pads, roads and other required surfaces;
- Section 7.0 Construction Materials – this section includes descriptions of different material types for use in earthworks construction, with discussion of quantities and schedule of required engineering materials, and quality and quantity of different material sources; and
- Section 8.0 Recommendations for Further Investigation – this section highlights areas of uncertainty where additional data will be required to support further development of geotechnical design, and presents recommendations for further work.
- Section 9.0 Closure.

2.0 PROPOSED FACILITIES AND DESIGN OBJECTIVES

2.1. General

The geotechnical recommendations contained in this report rely on information from several key sources, including:

- General Arrangement, Revision J, received from Wardrop, 23 November, 2011;
- Topographic contours and aerial imagery provided by Victoria, February, 2011;
- Grading information provided by Wardrop November and December, 2011; and
- Anticipated foundation dimensions, loads and settlement tolerances provided by Wardrop, 18 November, 2011.

The following subsections present brief overviews of anticipated building foundations, major earthworks, and geotechnical design parameters used for design.

2.2. Buildings

The proposed layout illustrated in Drawing 02 shows a number of buildings including those containing the crushers, the process plant site, truck shop and explosives storage areas. Anticipated foundation dimensions and loads, and tolerable foundation deformations have been provided by Wardrop on 18 November, 2011, and are summarized in Table 2-1.

The crushers will include three separate facilities – the primary, secondary and tertiary crushers – connected by conveyors, as illustrated in Drawing 03. These facilities will include heavy vibratory machinery (i.e. the crushers) and associated machine foundations, in addition to the building foundations. The primary crusher will be accessed at the top by trucks from the pit, and will be founded some 25 to 30 m lower. Thus the primary crusher building will also function as a large retaining wall.

The line of crushers will be built on steeply sloping terrain, thus requiring cutting and filling to allow development of building pads. The cut above the lower platform below the primary crusher is shown on the general arrangement as approximately 90-95 m high at its highest point. This cut is expected to be lower above the secondary and tertiary crushers.

A line of conveyors will connect the tertiary crusher with the heap leach facility in the valley bottom below. The conveyors will be supported on sleepers where appropriate, and on elevated bents where necessary. The conveyor layout is illustrated on Drawing 04.

The process plant facilities will be developed on a cut/fill pad constructed on the hillside below Tin Dome, above and to the north of the Dublin Gulch valley bottom. The proposed layout is illustrated on Drawing 05.

The truck shop will be developed on a cut/fill pad constructed on the hillside below and to the west of the planned open pit, above and to the east of Haggart Creek. The proposed layout is illustrated on Drawing 06.

Specific foundation dimensions and loads have not been provided for the camp facilities or explosives storage area. It is presently assumed that the camp will consist of settlement-tolerant structures (e.g. portable structures on timber cribbing that can be jacked and shimmed as required) that do not require specific foundation recommendations. It is also assumed that the explosives storage will consist of portable containers placed a grade on level pads, rather than permanent structures on concrete foundations. Therefore, specific foundation recommendations are not provided for either the camp site or the explosives storage facilities.

Table 2-1. Foundation Loads and Settlement Tolerances Provided by Wardrop[1]

Area And Facility	Equipment	Presumed Foundation Type And Footprints	Type Of Loadings/Presumed Bearing Pressure	Maximum Allowable Settlement
Primary Crushing	Crusher	Mat – 18m x 12m	Vibrating – 350 to 400 kPa	10 mm in 6 m.
Tertiary/Secondary Crushing/Silos	Tertiary/ Secondary Crushing/Silos	Building - spread footings – 2m x 2m to 12m x 5m Silos/Crushers – mat. – 14m x 9m to 16m x 16m	Building - 250 to 300 kPa Crushers – Vibrating - 350 to 400 kPa	Building- • 20 mm individual footings • 10 mm in 7 m bays differential • 12 mm across crane aisle between crane rails. Crushers/silos – 5 mm in 8 m differential.
Conveyors[2]	Gallery and Bents	Spread footings – 1.5m x 6m (typ.)	Static - 100 to 150 kPa[2]	25 mm – individual footings[2].
Reagent/Refinery[3]	Cranes	Building - Spread footings – 1.5m x 1.5m to 12m x $3.5m^3$	Static - 250 kPa Static - 200 kPa for 12 m x 3.5 m^3	20 mm individual footings 10 mm in 7 m bays differential 12 mm across crane aisle between crane rails.
Process shop/Truck shop	Cranes	Spread footings - 2m x 2 m Spread footings - 8m x 3m	Static - 250 kPa	20 mm individual footings 10 mm in 7 m bays differential 12 mm across crane aisle between crane rails.
Ancillary Buildings		Spread footings – 1.5m x 1.5 m to 2m x 2 m	Static - 150 to 200 kPa	20 mm individual footings 15 mm in 6 m bay differential.

Notes:

1. As provided by Wardrop on 18 Nov 2011, except where noted otherwise.
2. Per email from Wardrop 12 January 2012, conveyor footing loads are expected to be limited to 100-150 kPa, with maximum tolerable settlement 25 mm.
3. Per email from Wardrop 2 December 2011.

2.3. Major Earthworks

The proposed mine will be located in rugged terrain, necessitating large cuts and fills in some areas, including the following:

- Main cut above primary crusher, currently shown in Wardrop grading information to be about 90-95 m in height at a slope of 1.75H:1V;
- Main cut at plant site building pad, currently shown by Wardrop to be 31 m in height at a slope of 2H:1V;
- Cuts along the Dublin Gulch diversion channel, currently shown by Wardrop to be up to about 26 m in height at a slope of 1.75H:1V to 2H:1V;
- Main cut at truck shop building pad, currently shown by Wardrop to be 20 m in height at 2H:1V;
- Main cut at upper edge of 100 day storage pad, currently shown by Wardrop to be 36 m in height at a slope of 1.75H:1V;
- Numerous other cuts of up to 15-20 m in height, at typical slopes of 1.5H:1V to 2H:1V.

Drawing 07 shows areas of planned cutting and filling associated with the bulk earthworks for infrastructure development. Several of the larger planned cut slopes are identified on that drawing, and illustrated in cross section in subsequent drawings. Drawing 08 shows the planned cut near the primary crusher. Drawing 09 shows the planned cuts near the plant site and at the Dublin Gulch diversion channel. Drawing 10 shows planned cuts at the truck shop and at the 100 day storage pad.

It is noted that the cut slope angles shown on these cross sections, and described above, are from the grading plan received from Wardrop on 23 November, 2011. Recommended slope angles are presented in Section 6.3 and may differ from those listed above and shown on these drawings.

2.4. Design Criteria

2.4.1. Allowable Bearing Pressures for Foundations

Allowable bearing pressures for the static performance of foundations must consider allowable settlements, and must also consider the potential for bearing capacity (shear) failure. Settlement tolerance criteria have been presented previously in Table 2-1. A minimum factor of safety of 3 against bearing capacity failure has been included in all bearing pressure recommendations presented later in the report.

Machine foundations must also be designed to limit vibrations to acceptable levels. Vibratory loads and vibration tolerances are equipment-specific, and therefore further analysis will be required during detailed design once equipment suppliers have been identified. Based on input from Wardrop, for preliminary planning purposes, it has been assumed that machine foundations can be designed and constructed economically if the bearing strata can provide at least 400 kPa allowable bearing pressure for static loads on the large mat foundations indicated in Table 2-1.

2.4.2. Slope Stability

Cut and fill slopes associated with the civil earthworks discussed in this report are designed to meet specific criteria for static and pseudo-static earthquake loading. The recommended safety factors are summarized in Table 2-2 below.

Table 2-2. Recommended Factors of Safety for Slope Design

Consequence of Failure	Cut/Fill Location	Minimum Static Factor of Safety (FS)	Minimum Pseudo-Static FS; Slope Displacement-Based (Seed, 1979)	Minimum Pseudo-Static FS; Slope Displacement-Based (Bray, 2007)
High	Plant Site	1.5	≥1.15 for k_h = 0.1g M = 6.5 Maximum slope displacement of 100 cm	≥1.0 for k_{15} = (0.006+0.038M)*S(0.5)-0.026; S<1.5g and 2% in 50-year ground motion Maximum slope displacement of 15 cm
	Truck Shop			
	100-Day Storage SE Section (Close To Crushers)			
	Crushers			
	Crushers Haul Road			
	Substation			
	Diversion Channel			
Moderate to Low	Laydown Area	1.3		
	Explosive Magazines			
	100-Day Storage (Distant From Crushers)			
	Main Pond			
	Truck Shop - Pit Road			
	General Site Roads			

2.4.3. Seismic Design

Site specific seismic hazard information was obtained from Natural Resources Canada at www.EarthquakesCanada.ca. The National Building Code of Canada (NBCC 2005) design ground motions, corresponding to a 2 % probability of exceedence in 50 years (0.000404 per annum) are detailed in Table 2-3 below.

Table 2-3. National Building Code of Canada Recommended Design Motions

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.513	0.312	0.155	0.086	0.245

Ground motions for other return periods are provided in Table 2-4 below.

Table 2-4. Ground Motions for other Probabilities

Probability of exceedence per annum	0.010	0.0021	0.001
Probability of exceedence in 50 years	**40 %**	**10 %**	**5 %**
Sa(0.2)[1]	0.131	0.272	0.368
Sa(0.5)	0.076	0.160	0.219
Sa(1.0)	0.037	0.077	0.107
Sa(2.0)	0.020	0.043	0.059
PGA[2]	0.072	0.139	0.182

Notes:

1. S_a is spectral acceleration at the selected period (e.g. 0.2 seconds), in units of acceleration due to gravity, g.
2. Peak ground acceleration, in units of acceleration due to gravity, g.

The seismic hazard described above can be re-stated in terms of a representative earthquake event. An earthquake of M5.65 located at a distance of 17 km from the site would yield ground motions similar to those reported above. This de-aggregation of the seismic hazard was provided by the Geological Survey of Canada (GSC) on the basis of site coordinates. They were requested to do the de-aggregation for peak ground acceleration, and using the return period/annual probability specified in the National Building Code (therefore applicable to buildings). Slightly different values may apply for other structures to which the NBCC does not apply, and for which other components of the hazard (specific spectral acceleration values, rather than PGA) may be more important.

The information provided by the GSC (email to BGC dated July 2009) was accompanied by the following qualifying notes:

> *De-aggregations of the NBCC Robust seismic hazard generate a suite of files, one for each period, for each site.*
>
> *"Robust" hazard values are the ones used in the NBCC and are the higher of the H, R, C, and F model values at each site. Where any of the three other models give hazard values "sub-equal" to that from the highest model for any period, for that period the de-aggregations for those other models should also be considered for engineering purposes. This is because certain hazard and risk contributions of those other models may exceed those of the Robust model.*
>
> *A hazard example might be for liquefaction, where nearby, small-magnitude sources from the H model may give the Robust value of PGA (suitable for structural design of short-period buildings), but the liquefaction hazard may come from mid-distance large-magnitude earthquakes in the R model (because of the longer duration of ground motions from those sources).*
>
> *A risk example might be for structural damage, to the degree that it is influenced by duration effects not captured by the 5%-damped spectral values.*
>
> *"Sub-equal" can be generally taken as 70% or greater of the Robust value for any period, but there is no certainty that this is the correct value for all cases. The user needs to decide.*

3.0 SITE CONDITIONS

3.1. General

A detailed presentation of the findings of the 2011 site investigations has been provided in BGC (2011b). The following sections provide a brief overview of site conditions relevant to the development of mine site infrastructure, based on the available data. A more detailed synthesis of subsurface data relevant to geotechnical design is presented in Appendix A.

3.2. Generalized Site Conditions in the Mine Site Area

3.2.1. General Site Conditions

The site topography involves moderate to high relief, with ground elevation varying from approximately 800 to 1400 m ASL.

Ground conditions are highly variable across the site. Subsurface data are available from various sources in most areas of proposed development, as shown on Drawing 11. This drawing also subdivides the project area into a number of distinct functional areas for grouping data in relation to key facilities.

Overburden thickness varies substantially across the site as illustrated in Drawing 12. Overburden soils in the Dublin Gulch valley bottom are predominantly placer tailings (fill). Observed thickness of placer tailings is illustrated in Drawing 13.

Groundwater was observed at varying depths across the site, generally close to the elevation of streams in the valley bottoms, and often below the depth of test pit excavation (typically 5 m or greater) on the hillsides (Drawing 14).

Permafrost is present in the area, and is relatively warm (typically 0 to -1 degrees Celsius), discontinuous and occasionally contains excess ground ice. Although not dominantly controlled by slope aspect, permafrost is found more frequently in the north-facing lower slopes above the south side of Dublin Gulch. The distribution of frozen ground (including ice-rich frozen soils) observed in the testholes to date is illustrated in Drawing 15.

Bedrock at the site has been classified in three broad categories on the basis of expected engineering properties: Types 3, 2 and 1. The observed depths to Type 3, 2 and 1 rock are illustrated in Drawings 16, 17 and 18, respectively. These rock types are described in Section 4.0.

Bedrock strength may be controlled in some cases by structures such as joints, faults, bedding and foliation. A compilation of structural data relevant to mine site infrastructure is presented on Drawing 19, which also divides the site into five broad structural domains. This includes one domain for the granodiorite intrusion that hosts the ore body, and four domains in the surrounding metasediments, with domain boundaries determined largely on the basis of orientation of foliation and its relationship with regional bedrock structure.

Geological hazards were mapped by Stantec (2010). Inferred geological hazards within the areas of proposed mine site infrastructure development are illustrated in Drawing 20.

Appendix A provides a detailed compilation of subsurface data relevant to the geotechnical design of specific facilities considered in this report

3.2.2. Site Class

Seismic design parameters (i.e. uniform hazard spectra) applicable for buildings were presented in Table 2-3. A peak ground acceleration value of 0.245 g corresponds to the 1/2475 year design motion (2 % probability of exceedence in 50 years), and has been used for analysis in this report.

Seismic design parameters may require local modification for ground conditions. Site classes and soil profile names inferred based on downhole shear wave velocity profiles from each borehole tested are presented in Table 3-1. These site class designations may be used to modify the design ground motions listed in Table 2-3 for site specific conditions, where appropriate.

Table 3-1. Site Classes for Seismic Design of Buildings from Shear Wave Velocities

Facility/Area	Borehole ID	Depth[1] Analyzed (m)	Average Shear Wave Velocity, Vs_{30} (m/s)	Site Class and Soil Profile Name[2]
Crushers	BH-BGC11-36	30	825	"B" - Rock
Crushers	BH-BGC11-40B	30	800	"B" - Rock
Crushers	BH-BGC11-62	30	655	"C" - Very Dense Soil and Soft Rock
Events Ponds	BH-BGC11-32	21	365	"C" - Very Dense Soil and Soft Rock
Heap Embankment	BH-BGC11-33	30	690	"C" - Very Dense Soil and Soft Rock
Heap Embankment	BH-BGC11-34	30	540	"C" - Very Dense Soil and Soft Rock
Heap Embankment	BH-BGC11-59	28	650	"C" - Very Dense Soil and Soft Rock
Heap Pad	BH-BGC11-28	30	655	"C" - Very Dense Soil and Soft Rock
West end of Dublin Gulch	BH-BGC11-39	18	305-325	"D" - Stiff Soil
Diversion Channel	BH-BGC11-52	21	440	"C" - Very Dense Soil and Soft Rock
Plant Site	BH-BGC11-69	19.5	8302	"B" - Rock

1. Site classifications for depths analyzed less than 30 m do not meet the Vs_{30} criteria and thus should be considered as guidance only.
2. National Building Code of Canada 2005 Volume 1, pp.4-22 Division B, tables 4.1.8.4.A and 4.1.8.4.B National Building Code of Canada 2005 - User's Guide- Structural Commentaries (Part 4 of Division B) - Commentary J, pp. J-30-31.

3.2.3. Anticipated Site Conditions Relevant for Design of Cut Slopes

The proposed major cuts are shown in cross section in Drawings 08, 09 and 10, which also present the interpreted subsurface conditions including lithology and groundwater depth. The orientation, persistence and character of structural discontinuities in the rock are described in Appendix A.

3.2.4. Anticipated Site Conditions Relevant for Foundation Design

Expected subgrade conditions at planned foundation grades are presented for various facilities in Table 3-2. Given the topographic variability at the proposed mine site, the pads for building foundations are to be constructed as cuts or as balanced cut-fill. The subgrade conditions presented in Table 3-2 have been generalized; these conditions are expected to vary within the facility footprints. In particular, suitable bearing strata should be expected to have highly variable and likely sloping surfaces within the footprints of planned facilities.

Table 3-2. Anticipated Subsurface Conditions at Selected Building Foundations

Facility	Expected Pad Elevation (m ASL)	Expected Groundwater Conditions	Expected Subgrade Material at Foundation Grade
Primary Crusher	1026/1050[1]	Pad excavation is expected to encounter groundwater	Type 1 Rock above elevation of lower pad
Secondary Crusher	1032	Groundwater is expected below pad elevation	Type 2 rock at 4 m below existing ground surface[2]
Tertiary Crusher	1014	Groundwater is expected below pad elevation	Type 2 rock at 4 m below existing ground surface[2]
Conveyors from Tertiary Crusher to Heap Leach Facility	Varies along conveyor alignment	Groundwater is expected at rock/colluvium contact	Ice rich colluvium from ground surface to Type 3 rock at 10-20 m below ground surface
Plant Site	860	Pad excavation is expected to encounter groundwater	Varies, completely weathered rock at north end, fill at south end
Truck Shop	855	Pad excavation may encounter groundwater	Varies, Type 3 rock or better at east end to fill at west end

Notes:

1. According to grading information provided by Wardrop, the lower pad adjacent to the primary crusher is at 1026 m. The elevation of the top of the primary crusher, where trucks will deposit ore, is shown at 1050 m.
2. The current grading plan from Wardop shows the pads for the secondary and tertiary crushers constructed as a cut and fill balance with the crusher buildings spanning the cut and fill, and founded close to existing grade, some few metres above the type 2 rock subgrade

4.0 MATERIAL PROPERTIES

Material properties assumed for geotechnical design for in-situ soils, in-situ rock, and imported engineered materials are summarized in Table 4-1, Table 4-2 and Table 4-3, respectively. Additional descriptive information about material definitions, quantities and sources is provided in Section 7.0.

Material properties have been derived from information available from a variety of sources, including visual classification, index testing, field and laboratory shear strength testing, in-situ penetration testing, downhole and surface geophysical investigations, and plate load testing.

Rock has been classified as Type 1, 2 or 3. “Type 3” rock is usually the first “rock-like” material underlying the overburden soil materials, however sharp contacts between overburden and type 2 or type 1 rock have been observed occasionally. Type 3 rock is defined as being rock that is highly or less weathered (i.e. W4 or better), and has intact strength greater than R0 (i.e. minimum UCS strength 1 MPa). It is expected that Type 3 rock can generally be excavated with normal excavating equipment, with approximately 40 % requiring ripping.

“Type 2” rock is defined as rock with Geological Strength Index (GSI, Hoek and Marinos, 2000) or Rock Mass Rating (RMR, Bieniawski, 1976) of 30 or greater, and core recovery during drilling of 50 % or greater. Alternatively, where GSI and RMR data are unavailable, average Rock Quality Designation (RQD) of 10 or greater serves as an equivalent criterion. It is expected that Type 2 rock will generally require ripping, with approximately 35 % that can be excavated with normal excavation equipment.

“Type 1” rock is defined as having GSI, RMR or average RQD exceeding 40. It is expected that Type 1 rock will mostly require ripping, potentially hard ripping, with approximately 10-20 % requiring blasting.

The estimated shear strength of foliation for use in the cut slope design for the metasedimentary unit was determined using lab testing results from the open pit design work. Base friction values were determined through small scale direct shear testing. An increase in shear strength for large-scale roughness was applied based on the variability of the orientated discontinuity measurements, the direct shear results, and field and core observations of joint roughness. As a result, the design foliation strength was assumed to be 35°.

Table 4-1. Recommended Material Properties for Design – In Situ Natural Soils

Material	Bulk Density (kN/m^3)	Mohr-Coulomb Shear Strength			Stiffness[3]	
		Friction Angle (Deg)	Cohesion (kPa)	Concrete-Soil Friction (Degrees)	Deformation Modulus, E_s (MPa)	[1]Modulus Of Subgrade Reaction, Kv_1 (KPa/mm)
Colluvium, Debris Flow	18	34	0	N/A	N/A[2]	
Till	19	35	25	23	N/A	
Completely weathered rock	20	35	50	23	60	210
Placer Tailings	19	30 - 35	0	N/A	N/A	

Notes:

1. Modulus of subgrade reaction has been provided for a standard 1 foot plate diameter. Values need to be scaled to footing size, and will be lower for larger footings. BGC can provide further advice on request.
2. N/A: Not applicable.
3. Poisson's Ratio estimated to be 0.3 (Bowles, 1996).

Table 4-2. Recommended Material Properties for Design – Rock Mass

		Hoek-Brown input parameters[1]			Hoek-Brown strength properties				Rock Mass stiffness[4]	Dynamic Properties	
UNIT	Bulk Density[3] (kN/m^3)	GSI[2]	UCS[2] (MPa)	m_i	m_b	s	a	Rock Mass, sig_c (MPa)	E_{rm} (MPa)	G_{max} (MPa)	ν[5]
Type 1 Rock	25	51	54[6]	11	1.912	0.0043	0.505	3.4	2000 - 3000	3000 - 5000	0.2
Type 2 Rock	25	36	33[6]	6	0.610	0.0008	0.515	0.8	1000 - 2000	2000 - 4000	0.2
Type 3 Rock	25	28	25[7]	6	0.459	0.0003	0.526	0.4	100 - 500	N/A	0.2

Notes:

1. The Hoek-Brown failure criteria have been estimated using a disturbance factor ('D') of 0 for all units.
2. Median RMR'76 parameters are used for each geotechnical unit.
3. Unit Weights are based on average results of specific gravity testing.
4. Rock mass stiffness ranges are estimated considering Plate Load Test results, lab data and results of downhole geophysics
5. Poisson's ratio from average lab test results where failure mode was not along foliation
6. UCS for Type 1 and Type 2 rock are taken from median Is_{50}, multiplied by the corresponding k value (20 and 28, respectively).
7. UCS for Type 3 Rock is estimated from median strength grade (R2.5 = 25 MPa)
8. Shear modulus G_{max} obtained from V_s values from downhole geophysics

Table 4-3. Recommended Material Properties for Design – Construction Fill Materials

Material	Bulk Density (kN/m^3)	Mohr-Coulomb Shear Strength			Stiffness		Dynamic Properties	
		Friction Angle (Degrees)	Cohesion (kPa)	Concrete-Soil Friction Angle (Degrees)	Deformation Modulus, E_s (MPa)	Modulus[1] Of Subgrade Reaction, Kv_1 (kPa/mm)	Shear Modulus[2], G_{max} (MPa)	Poisson's Ratio, ν
General Fill	20	35	0	23	N/A			
[3]Structural Fill	21	40	0	27	50-100	150-300	100 - 200	0.3
Rock fill – durable[4]	18	45	0	30	100-150	300-400	200 - 300	
Rock fill - non-durable[4]	19	38	0	25	50-100	150-300	100 - 200	

Notes:

1. Modulus of subgrade reaction has been provided for a standard 1 foot plate diameter. Values need to be scaled to footing size.
2. Shear modulus is presented for very low strains of 10-6 to 10-5, and have been estimated from available shear wave velocities (Vs). Modulus should be reduced for larger strains, and BGC can provide further assistance in selection of appropriate moduli on request. Poisson's ratio inferred from Bowles, 1982.
3. It is assumed that the selected structural fill material will consist of high quality well graded sand and gravel with low fines content and durable particles, as described in section 7.3.
4. Non-linear strength envelopes may be derived for rock fill from Leps (1970) for applications under high loads, for example in the heap embankment.

5.0 FOUNDATION RECOMMENDATIONS

5.1. General

The most recent General Arrangement provided by Wardrop on 23 November 2011 shows the following key facilities considered in this report:

- Crushers and conveyors;
- Plant Site; and
- Truck Shop.

There are other ancillary facilities that have not been given specific consideration because their final locations have not been set, or due to expected light loads and/or high settlement tolerances. The camp facilities and explosives storage areas have not been considered explicitly in this report.

5.2. Foundations

5.2.1. General

Anticipated foundation conditions described herein should be verified in the field by a qualified geotechnical engineer during construction, and must be confirmed through additional site-specific subsurface investigation prior to final design. If conditions vary significantly from those presented, modifications to the foundation design parameters may be required, and BGC should be given the opportunity to review its recommendations in light of actual conditions.

It is expected that all buildings will be founded on conventional spread footings or other mass concrete foundation elements. Spread footings should be founded on Structural Fill or an approved subgrade of highly to completely weathered rock, or Type 1, 2 or 3 rock. All organics and colluvium must be removed to expose a subgrade of undisturbed rock. In areas where the required subgrade is lower than the proposed design grade, the difference may be made by placing structural fill, except where noted due to high anticipated loads.

It is recommended that foundations be designed to not straddle dissimilar subgrade materials, for example structural fill and type 3 rock. In cases where structural fill is required to make grades below part of a building, a minimum of 1 m of structural fill should be placed below foundation elements above the stiffer subgrade to minimize differential settlements.

It is presently understood that selected conveyor bent foundations will need to be founded on concrete-filled steel pipe piles socketed into bedrock.

Buildings should be set back a minimum of 10 m from the crest of fill slopes. A minimum embedment depth of 1.5 m below surrounding grade is required for adequate bearing capacity of foundations unless indicated otherwise; however, greater embedment may be required for frost protection.

5.2.2. Allowable Bearing Pressures

Anticipated foundation dimensions and loads, and deformation tolerances, were summarized in Table 2-1 in Section 2.1.

Allowable bearing pressures are a function of both the settlement tolerance of the supported facilities, and shearing resistance of the subgrade soil or rock (i.e. safe bearing capacity). The allowable bearing pressure is the minimum value satisfying both factored resistance against shear failure and tolerable settlement. These calculations depend on the foundation loads, dimensions and depths, in addition to groundwater levels and soil/rock strength and stiffness.

Allowable bearing pressures for machine foundations must also consider the vibration loads and tolerances. These were not available at the time that this report was prepared but should be considered at detailed design.

Allowable bearing pressures for small foundations (up to 2 m by 2 m and a strip footing 2 m wide and up to 20 m long) are provided in Table 5-1. These allowable bearing pressures are valid for up to 20 mm of total settlement. These bearing pressures should be considered only where facility specific recommendations are not provided and where footings are up to or smaller than the stated sizes. Facility specific recommendations are provided in Table 5-2.

Table 5-1. Recommended Allowable Bearing Pressures for Ancillary Facilities

Bearing Stratum	Allowable Bearing Capacity (kPa)	
	Up to 2 m x 2 m Pad Footing	Up to 2 m x 20 m Strip Footing
Structural Fill[1]	250	150
Weathered Rock	250	150
Type 3 Rock	500	300
Type 2 Rock	1000	600
Type 1 Rock	1500	1000

Notes:

1. Footings founded on structural fill require a minimum of 1.5 m of embedment (depth of bottom of footing below surrounding grade) to obtain the indicated allowable bearing capacity. Separate consideration of frost protection may be necessary.

Recommended allowable bearing pressures for key facilities are summarized in Table 5-2. In the current grading plan prepared by Wardrop, the secondary and tertiary crushers are planned to be founded on a pad constructed of a cut and fill balance with the crusher building spanning the cut and fill, and foundations at close to existing grade, with the suitable bearing stratum of Type 2 rock present some metres below grade. The crushers cannot be supported on structural fill. The available alternatives to bridge the distance between expected foundation grades and elevation of suitable type 2 rock bearing stratum include some form of stabilized fill material, such as lean concrete, or heavy deep foundations, such as caissons. It is recommended that further consideration be given to adjusting grades to put the foundations on type 2 rock to avoid either of these costly alternatives. Note, however, that it is expected that adjusting grades will require a higher permanent cut slope adjacent to the crushers, so this additional rock excavation should be balanced against the increased foundation costs. BGC can provide more detailed recommendations after further input from Victoria and Wardrop.

Table 5-2. Recommended Allowable Bearing Pressures for Specific Facilities

Facility	Expected Pad Elevation (m ASL)	Foundation Dimensions[1]	Expected Subgrade Conditions	Allowable Bearing Pressure[2] (kPa)
Primary Crusher	1026/1050[3]	Up to 12 m x 18 m mat	Type 1 rock	1000
Secondary Crusher[4]	1032[4]	Up to 16 m x 16 m mat, 12 m x 5 m spread footing	Type 2 rock[4]	400[4]
Tertiary Crusher[4]	1014[4]	Up to 14 m x 9 m mat	Type 2 rock[4]	400[4]
Conveyors from Tertiary Crusher to Heap Leach Facility	Varies along conveyor	Bents on 1.5 m x 6 m footing	Type 3 rock at ~ 5 m to 20 m depth below grade, typically 10 m expected	200 mm concrete-filled steel pipe piles socketed 2 m into Type 3 rock at ~ 10 m depth below grade can support 700 kN
		Sleepers at grade, on timber cribbing where possible	Colluvium below stripped topsoil	N/A – adjustable foundations
Plant Site	860	3.5 m x 12 m	Highly to completely weathered rock or structural fill	200
Truck Shop	855	3 m x 8 m	Type 3 rock	300

Notes:

1. Foundation dimensions provided by Wardrop on 18 Nov 2011.
2. Based on factor of safety of 3 against bearing capacity failure and limiting settlements to those specified by Wardrop on 18 Nov 2011.
3. The lower portion of the primary crusher is at 1026 m. The elevation of the top of the primary crusher, where trucks will deposit ore is at 1050 m.
4. Crushers cannot be supported on regular structural fill. If the secondary and tertiary crushers must be built at planned grades well above the suitable bearing stratum, the gap between the bearing stratum and foundation grades can be made up by lean concrete or some other form of stiff fill material.

5.2.3. Subgrade Preparation

Care should be taken to avoid disturbing subgrade materials that will remain in place. Areas of weathered rock subgrade that become softened or loosened during construction should be removed and replaced with compacted engineered fill (structural fill) or lean concrete. The base of all excavations should be dry and free of loose materials at the time of concrete placement. A layer of lean concrete can be placed on the subgrade for protection to allow work to continue in wet weather prior to pouring of footing concrete.

Subgrades should be reviewed by a qualified geotechnical engineer prior to placement of structural fill, protective blanket, or forms for foundations.

5.2.4. Foundations on Sloping Bearing Strata

Several proposed facilities will be constructed on sloping ground, notably the plant site, truck shop, explosives storage areas and crushers, and in these areas the acceptable bearing strata are also expected to be sloping. Foundation subgrades should be prepared so that foundation elements will be placed on horizontal surfaces, which may require excavation of notches or benches within the suitable bearing strata. BGC can provide further advice with respect to specific foundations on sloping ground during detailed design and construction.

5.2.5. Water Control

Final site grading should maintain positive drainage in the direction of natural drainage and should direct water away from the structures. Improper drainage and ponding of water near or under the structures can be detrimental to the foundation performance. The final grades should be sloped down, away from the structure, at a slope of 4% within 3 m of the structure and at a slope of 2% beyond.

It is recommended that permanent surface water control be provided at the base of all excavation slopes to direct water away from the proposed facilities and to allow the slopes to drain effectively. In addition, temporary surface water control during foundation excavations should be provided by the contractor so that foundation excavations and subgrade remain essentially dry when the foundation is being constructed.

Based on available groundwater level information, it is expected that the pad development for the proposed crushers, plant site and truck shop area will intercept the water table. Due to the fractured nature of the shallow bedrock, dewatering of excavations, if necessary, should be feasible with conventional sumps and pumps.

Development of the silt borrow area near the laydown area may encounter groundwater. Upward seepage gradients, if encountered, may result in softening and/or heaving of the subgrade soils in this area.

5.2.6. Minimum Foundation Depth for Frost Protection

Critical foundations, water lines, and other important infrastructure should be protected from frost. The maximum depth of frost penetration for the project site is estimated to be 3.0 m.

Exterior building foundations should be founded below the anticipated depth of frost penetration or will need to be properly insulated if founded above the maximum depth of frost penetration. Exterior footings at 1.0 m depth below finished grade should be insulated by 50 mm thick Dow Styrofoam SM or equivalent extruded polystyrene insulation buried 0.3 m below final grade and extending horizontally 1.8 m. The vertical portion along the foundation element should also be insulated with 50 mm thick insulation. The insulation should be sandwiched between two layers of bedding sand, 75 mm in thickness, and should be sloped down away from the structure at 1 percent grade. If exterior footings are raised to 1.0 m depth, allowable bearing pressures may need to be reduced. BGC can provide further comment if requested.

5.2.7. Concrete Slabs

A minimum of 150 mm thick layer of compacted free-draining sand and gravel, consisting of 19 mm minus durable material with less than 8 % fines (passing No. 200 sieve), should be placed beneath all slabs-on-grade as a leveling course.

5.2.8. Temporary Excavations

Construction may require temporary excavations into native soil and weathered bedrock. Safe, stable construction slopes should be made the responsibility of the contractor and will depend on the ground and site conditions encountered at the time of construction.

5.3. Retaining Walls

Retaining Walls must be designed to sustain various loads, and should be checked for satisfactory performance with respect to overturning, sliding, bearing capacity and global stability. These checks are generally the responsibility of the wall designer.

5.3.1. Design Basis

BGC provides the following guidance to aid in the design of retaining walls. It is assumed that all retaining walls are intended to be designed as rigid reinforced concrete walls. However, if requested, recommendations can be provided for flexible reinforced soil walls, tied-back walls or other retaining structures, and BGC can provide further advice and assistance in wall design if required.

The recommendations in this report are based on the assumption that the water table can be kept below the base of the wall, and therefore do not account for hydrostatic water pressures which could increase lateral loads and produce uplift on the foundation. In order to achieve this condition in practice, the following measures are suggested:

- Infiltration and seepage behind walls should be minimized. The upper surface of the backfill should be covered with a low permeability material, and the site should be graded away from the structure to prevent surface water from accumulating against the back of the wall.
- Retaining wall backfill should be free-draining granular soil (e.g. 19 mm minus sand and gravel with less than 8 % fines).
- Backfill should be drained using a perforated drain tile set at footing level and draining to a free outlet. In addition, where possible operationally, weep holes should be provided through the face of the wall.

Walls should be designed for internal, external and overall stability. Appropriate shear strength properties for geotechnical materials and concrete-subgrade or concrete-backfill contact are provided in Table 4-1 and Table 4-3.

For seismic design, inertial forces due to the mass of the wall (including the mass of the reinforced soil column in the case of reinforced earth walls) should be considered. For cantilever retaining walls, the additional inertial force due to the mass of the soil column above the heel section of the wall should also be considered.

Lateral earth pressures acting on retaining walls depend to a large degree on how much a wall is allowed to rotate under normal operating conditions. Walls are often defined as either “unrestrained” or “restrained,” where this distinction depends on the allowable wall rotation. Restrained walls, which are sometimes referred to as “non-yielding” walls, are subject to higher loads, assumed to be represented by the “at rest” (i.e. K_o) condition. Unrestrained walls, sometimes called “yielding” walls, are subject to the “active” condition (i.e. K_a) or “passive” condition (i.e. K_p), depending on direction of wall movement.

The distinction between “unrestrained” (or “yielding”) and “restrained” (or “non-yielding”) walls depends on the wall configuration and properties of the retained backfill. “Unrestrained” walls are free to move sufficiently to allow active earth pressures to develop behind the wall in the limiting condition. “Restrained” walls are those that are prevented from moving sufficiently for active pressures to develop behind the wall in the limiting condition, when bearing or sliding failure is occurring.

When a retaining wall is backfilled with compacted granular fill, the transition from the “at rest” to “active” condition occurs at an angular rotation of about 0.001 m/m (i.e. 1 mm of deflection per 1 m of wall height). The transition from the “at rest” to “passive” conditions, for a wall moving inward, toward the backfill, occurs at an angular rotation of about 0.02.

The “active” and “passive” conditions may be assumed to apply for analysis of sliding and overturning. Structural design of the wall should consider “at rest” conditions, plus any compaction pressures and additional line loads or surcharges. Minimum factors of safety of 1.5 may be considered for both sliding and overturning.

5.3.1.1. Static Earth Pressures

The calculation of static earth pressures depends on several factors, including: type and geometry of the wall; strength (friction angle) of the backfill, backfill/wall interface and wall/foundation interface; wall height; bulk weight of backfill; and, inclination of ground surface behind (and above) the top of the wall.

The method for calculating static earth pressures for "unrestrained" and "restrained" walls are shown in Drawings 21 and 22.

5.3.1.2. Dynamic Earth Pressures

The method for calculation of dynamic earth pressures depends on whether the wall is "unrestrained" or "restrained." Both methods are illustrated in Drawings 23 and 24.

5.3.1.3. Compaction Pressures

To minimize compaction-induced earth pressures, use of small vibratory or hand-operated ram compaction equipment is recommended for the area within 2 m of the back of the wall.
If using small compaction equipment behind the wall is not practical, an additional load should be included to account for additional stresses due to compaction. The method for calculating and applying this additional load is detailed in Drawing 25.

5.3.1.4. Lateral Pressures from Surcharge Loads

The presence of vertical loads behind the top of the wall will increase lateral pressures acting on the wall. Three different types of vertical surcharge loads may be considered: point loads, acting at a single point behind the wall; line loads, acting along the full length of the wall at some specified distance behind the wall; and, area loads, which may be uniform pressures acting on the ground surface adjacent to the wall.

The effect of area loads may be determined by assuming a uniform horizontal pressure against the full height of the wall as detailed in Drawing 25, where "K" is the lateral earth pressure coefficient applicable for the wall type (i.e. "unrestrained" or "restrained"). The method for estimating the effect of point loads or line loads is illustrated in Drawing 26.

6.0 EARTHWORKS RECOMMENDATIONS

6.1. General

This section presents general recommendations for bulk civil earthworks necessary to obtain the required grades for building pads, roads and other required platforms.

There are a number of ground-related challenges to construction of earthworks and buildings at the proposed mine site. These include:

- Presence of discontinuous permafrost, including some areas with excess ground ice;
- Relatively short "traditional" (i.e. spring/summer/fall) construction season, with specific challenges and limitations during other parts of the year (e.g. poor trafficability and material workability on hillsides before mid-summer; and long, harsh winter);
- Uncertain quality and quantity of required borrow materials;
- Presence of significant quantities of existing placer tailings; and
- Presence of steep slopes and geological hazards.

Excavation of frozen ground, particularly ice rich permafrost, requires additional effort and care. Well-bonded, ice-rich frozen ground will be difficult to excavate, and as discussed previously, will require ripping. Further consideration needs to be given to the thaw behavior of this material, and allowances made for adequate drainage and associated erosion control, as well as additional time and effort for the work. Exposure of ice-rich permafrost and the associated thaw can result in wet, muddy, soft ground, and poor trafficability, along with local slumping and other nuisance effects.

Each of these specific challenges requires consideration in the planning, design and construction of mine site infrastructure, as discussed in the following sections.

6.2. Area-Specific Earthworks Commentary

6.2.1. General

The project area has been subdivided into a number of functional areas, as shown in Drawing 11. Summary observations for each functional area were presented in Table A-5 in Appendix A. That table provides an overview of the general conditions within each area, including the observed thickness of overburden, presence or absence of frozen ground and excess ice, and depth, where encountered, to Types 1, 2 or 3 bedrock.

The presence of placer tailings (fill) is an issue primarily in the Dublin Gulch valley bottom, and will affect the development of the heap leach pad, heap embankment, a portion of the Dublin Gulch diversion, and ponds or other facilities constructed in this area. The observed thickness of placer tailings at 16 test holes had a mean value of about 10 m, with a range between 0.3 m and 19.8 m.

There is typically a thin cover of organic soils overlying the other overburden units. The observed thickness of this unit varies across the site, ranging between 0 m and 3.7 m, with an average thickness of 0.3 m (285 observations). All organic materials are unsuitable for re-use as engineering fill materials, but should be suitable for reuse as cover materials for reclamation and should thus be segregated and separately stockpiled.

The following sub-sections present commentary related to earthworks construction in each functional area, based on the summary observations just presented. The need to remove surficial organic materials is not repeated in these sections. These general comments are intended to be interpreted in relation to gross earthworks within each identified functional area, and may not apply precisely for specific sites or facilities.

6.2.2. Area-Specific Commentary

The following subsections provide area-specific earthworks commentary. Where bedrock is encountered, it can generally be assumed that common excavation, ripping and blasting may be expected in Types 3, 2, and 1 rock, respectively. Excavated rock can generally be expected to be suitable for reuse as general fill, and potentially suitable for use as structural fill with due care in selection, placement and compaction control. Excavated rock used as structural fill will not be suitable for use in applications where a free-draining material is required, such as at shallow depths below buildings, or behind retaining walls.

Frozen ground will be most efficiently excavated by ripping where it contains excess ice or is otherwise well-bonded, and for planning purposes, all frozen ground may be assumed to require ripping. Excavated frozen ground will generally be unsuitable for reuse without substantial effort to thaw and drain, and may be suitable for reuse only for limited applications, depending on the moisture and fines contents.

It will be necessary to plan for temporary or permanent stockpiling of the wasted ice-rich frozen soil. These materials will be unstable when thawed and will not stand at steep angles or significant height, so a large footprint or containment berm may be required to store relatively small volumes. It may be possible to dispose of ice-rich spoil in areas developed for borrow, including the placer tailings piles in the Haggart Creek area as suggested by Victoria. Such disposal would require further study.

6.2.2.1. 100 Day Storage Pad

The overburden in this area is relatively thin, and is commonly frozen, with excess ice encountered in nearly half the test holes where frozen ground observations were made. Excavated overburden materials will not generally be suitable for re-use as a construction material. The shallow bedrock will be relatively easy to excavate to depths of 5-10 m below grade, and will be suitable for re-use as general fill. Excavations deeper than about 10 m, if required, may require ripping or blasting.

6.2.2.2. Conveyors

This area contains thick, frozen overburden, typically containing excess ground ice. Excavation of frozen ground will likely require ripping, and excavated materials will be unsuitable for re-use. Rock excavation is not anticipated in this area.

6.2.2.3. Crushers

This area contains moderately thick (typically 0 to 7.5 m) overburden, most of which is weathered rock, and is sporadically frozen. It should be assumed that about half of the overburden may be re-used as general fill. Shallow bedrock to approximately 5-10 m below grade will be Type 3. Deeper rock at 10-15 m or > 15 m depth can be expected to be Type 2 and Type 1, respectively. All excavated rock is expected to be suitable for re-use as general fill.

6.2.2.4. Dublin Gulch Diversion

In the portion west of Eagle Pup valley, there is widespread frozen ground with excess ice which may require ripping to excavate and will be unsuitable for re-use. The thickness of ice-rich permafrost, where present, was not delineated in the geotechnical investigations to date, but is expected to be up to about 5 m. Depth to rock is highly variable in this area, and borehole data are limited.

6.2.2.5. Dublin Gulch Pond

Very little subsurface information is available in this area. It should be assumed that loose, variable fill materials (placer tailings) will be present, including wet, silty materials that will likely be unsuitable for reuse.

6.2.2.6. Eagle Pup WRSA Pond

Overburden is relatively thick (typically 3 to 12 m), with locally shallower Type 3 or Type 2 bedrock. Ice-rich frozen ground was observed in one of four testholes probed in this area. An estimated half of excavated overburden materials may be suitable for re-use as general fill. Bedrock, where encountered, can be excavated but may require local ripping. Excavated bedrock will be suitable for re-use as general fill.

6.2.2.7. Eagle Pup WRSA

Overburden is moderately thick (0 to 10 m), but highly variable. Frozen ground is widespread (47 of 77 observations) and frequently contains excess ice (29 of 77 observations). Stripping of ice-rich materials, where required for WRSA foundation preparation, will require ripping, and excavated materials will not be suitable for re-use. Excavation of rock is not expected to be necessary for foundation preparation in the WRSA. There is a lobate feature approximately 100 m x 100 m in plan, with ice-rich colluvium to > 25 m depth, located in the valley bottom. This feature will be discussed in greater detail in the WRSA design report under separate cover.

6.2.2.8. Heap Leach Events Ponds

Overburden is thick (typically 10 m to 20 m) and comprised of placer tailings, which are expected to be generally suitable for reuse as general fill without processing, or for use as select fill (structural fill, and potentially concrete aggregate or heap overliner) with crushing and screening. Excavation of rock is not expected to be necessary in this area, unless pond grades intersect bedrock.

6.2.2.9. Explosives Storage

Overburden is relatively thin (typically 2-3 m). Some ice-rich frozen ground should be anticipated. It may be assumed that roughly half of excavated overburden will be suitable for re-use as general fill. Bedrock to about 5 m depth can be expected to be Type 3, and deeper rock will be Type 2 and will require ripping. If excavations deeper than about 10 m are required, blasting of Type 1 rock should be anticipated.

6.2.2.10. Heap Leach Embankment

Overburden in the valley bottom is thick (typically 4 to 14 m) and comprised of placer tailings, which are expected to be generally suitable for reuse as general fill without processing, or for use as select fill (structural fill, and potentially concrete aggregate or heap overliner) with crushing and screening. No rock excavation is expected to be necessary in this area, based on the current heap facility layout by Tetra Tech.

Overburden materials are more variable at the north and south ends of this area, where the abutments will be constructed. No general commentary can be provided for those areas in this report. Foundation preparation recommendations for the heap embankment and abutments are being undertaken by Tetra Tech.

6.2.2.11. Heap Leach Pad

The overburden within the proposed heap leach pad footprint is typically of moderate thickness (0 to 9 m), but highly variable. Frozen ground is present in some areas (14 of 71 testhole observations) and contains excess ice in isolated areas (6 of 71 observations). Non-frozen overburden will generally be granular colluvium that is expected to be easily excavated and generally suitable for reuse as grading fill for the heap subgrade. Bedrock depth is variable, and shallow bedrock to 5 m depth is generally Type 3. Type 2 rock can be expected at depths below 5 m, and Type 1 rock may be encountered at depths greater than about 10 m, but is locally shallower in the upper part of the heap.

6.2.2.12. Laydown Area

This area includes the area intended to be developed for silt borrow for pond liner material, as well as the proposed construction laydown area and permanent camp.

The proposed laydown area straddles thick (estimated to be 10 to 20 m, no data available) placer tailings in the Dublin Gulch valley bottom, and thick (up to 25 m thick), ice rich permafrost in the undisturbed area further south.

The ice rich permafrost will require ripping to excavate, and the silt borrow material will need to be thawed and dried before it can be re-compacted as liner material.

The placer tailings in this area have been recently re-worked to construct a pad for the 100-man exploration camp. The materials in this pad are silt, sand and gravel in varying proportions.

6.2.2.13. Main Site Water Management Pond

The proposed pond area straddles thick (estimated to be 10 m or greater, no data available) placer tailings in the Dublin Gulch valley bottom, and thick (up to 25 m thick), ice rich permafrost in the undisturbed area further south.

The placer tailings in this area are expected to be generally suitable for re-use as general fill. Ripping will be required to excavate frozen ice rich overburden in the undisturbed part of this area, which comprises roughly the southern three quarters. No rock excavation is expected to be necessary in this area.

6.2.2.14. Main Truck Road

The overburden in this area is of moderate thickness (approximately 1.5 to 7 m), with limited presence of frozen ground (1 of 7 observations). Most of the unfrozen excavated overburden is expected to be suitable for re-use as road grading fill. Excavations deeper than about 5 m may encounter Type 3 rock. Excavations deeper than 10 m and 15 m should be expected to encounter Type 2 and Type 1 rock, respectively.

6.2.2.15. Plant Site

This area has thick overburden, most of which is either till or completely weathered rock. Roughly two thirds of the excavated overburden materials in this area are expected to be suitable for re-use as general fill, assuming a deep cut for the plant site pad. It is expected that excavations in this area can be completed with conventional excavation equipment to at least 30 m depth. The Type 3 rock encountered below about 10 m depth may be suitable for re-use as structural fill with due care in quality control of material selection (possibly including screening), placement and compaction control.

6.2.2.16. Platinum Gulch WRSA Pond

There is very little information available for this area, however the distribution of permafrost may be limited, and bedrock may be locally shallow (i.e. 0 m to 6 m). Type 1 rock should be anticipated for excavations deeper than about 5 m.

6.2.2.17. Platinum Gulch WRSA

The overburden in this area is moderately thick (typically 0 m to 6 m), with significant variability in observed thickness. Frozen ground is locally present and occasionally contains excess ice. Stripping of ice-rich materials, where required for foundation preparation, will require ripping, and such excavated materials will not be suitable for re-use. Rock excavation is not expected to be necessary for foundation preparation in the WRSA.

6.2.2.18. Secondary Road

This functional area contains secondary roads from the main access road along Haggart Creek between the substation and truck shop to the bottom of the 100 day storage pad. Limited information suggests that overburden is thick and likely frozen and ice rich in this area. Ripping may be required for excavation of frozen overburden for road grade preparation. It should be expected that excavated spoil materials will not be suitable for immediate re-use as road grading fill, but may become suitable given adequate time to thaw and drain (perhaps after a minimum of one full summer, but will depend on seasonal weather).

6.2.2.19. Truck Shop

Overburden is moderately thick (typically 7 to 8 m) and consists of frozen silty colluvium with excess ice in the upper 2 to 4 m. The underlying bedrock is Type 3. The shallow frozen overburden will require ripping. The frozen colluvium and bedrock below about 4 to 5 m depth can be excavated with normal excavating equipment. Excavated overburden materials will not be suitable for immediate reuse, but excavated bedrock will be suitable for use as general fill, or for use as structural fill with due care in quality control of material selection, placement and compaction control.

6.3. Site Preparation

The shallow overburden materials, including organic soils and colluvium, should be removed below all building foundations or below pads for building development to expose undisturbed native subgrades of highly to completely weathered rock or type 1, 2 or 3 rock. Organic soils should be stockpiled for re-use in reclamation work. The excavated colluvium materials may be suitable for re-use as general grading fill (General Fill), provided they do not contain deleterious materials, such as organic inclusions or excess ice. Stripped materials should be segregated under the direction of a qualified geotechnical engineer. Selected poor-quality colluvial soils may need to be wasted, at the discretion of the Engineer.

The overburden soils contain a significant percentage of fines (materials passing the No. 200 sieve) and fine sand such that their consistency may be sensitive to moisture and freezing temperatures. These soils may also degrade to slurry-like consistency when subjected to construction traffic loads or otherwise disturbed in wet conditions. It is recommended that defined construction roads be used for repetitive construction traffic to minimize disturbance at prepared areas. Trafficability will be poor on recently thawed ground or in areas of poor drainage.

Permafrost is present in patches, and seasonally-thawed soils may remain frozen late into the summer. Some of these materials may contain excess ice and will therefore become wet when thawed. Care should be taken to segregate frozen materials removed during site grading activities.

Where construction activities are to be conducted during periods of freezing weather, fill should not be placed upon frozen material, snow or ice. Earth fill placement, including non-durable rock fill placement, should be temporarily suspended if freezing conditions exist. It is recommended that if the ambient air temperature is less than zero degrees Celsius for more than four (4) hours over the preceding twenty-four (24) hours, the temperature of the fill should be measured to determine if the fill is frozen. If frozen, the fill should be removed and replaced. To help protect the fill surface from freezing during periods of shutdown it is recommended that placed fills be covered with loose (sacrificial) fill, or blankets, to help insulate the fill from freezing temperatures. Placement of coarse durable rock fill, which does not require water for compaction, can proceed in freezing conditions.

6.4. Site Grading - Fills

6.4.1. General

Site grading, as described in this section, includes all major excavations and fills necessary to bring the site to the proposed design elevations, including fill to support buildings, foundations, floor slabs, and backfill of foundations.

6.4.2. Engineered Fill Slopes

Engineered fill slopes constructed of structural fill or rock fill may be made at 2H:1V or flatter. Buildings should be set back a minimum of 10 m from the crest of fill slopes.

Where a structural fill embankment is to be constructed on an existing natural slope, the fill should be keyed into the natural slope by excavating steps into the slope at the edge of successive lifts of structural fill.

Selected high fills, including those below the pit-crushers haul road and at the lower (north) end of the 100 day storage pad, may encroach into seasonal drainage areas or depressions with shallow groundwater. Particular care should be taken in these potentially wet areas to choose free draining, coarse granular fill materials, preferably angular durable rockfill, to prevent buildup of excess pore pressures in the fills.

6.5. Site Grading - Cuts

6.5.1. Excavation Effort

Bulk excavation activities will encounter various materials. All overburden soil materials, including organics, colluvium, till, debris flow material, alluvium and highly to completely weathered rock are expected to be excavatable by normal excavating equipment when encountered in an unfrozen state. These same materials will likely require ripping when frozen, and ice-rich frozen materials in particular will require hard ripping.

It is expected that Type 3 rock can generally be excavated with normal excavating equipment, with approximately 40 % requiring ripping. It is expected that Type 2 rock will generally require ripping, with approximately 35 % that can be excavated with normal excavation equipment. It is expected that Type 1 rock will mostly require ripping, potentially hard ripping, with approximately 10-20 % requiring blasting.

6.5.2. Permanent Cut Slopes

6.5.2.1. General

Area specific cut slope angle recommendations are provided for the highest and most critical of the proposed excavations (Table 6-1). General cut slope angle recommendations are provided for all other slopes that are less than 10 m high (Table 6-2). Except where noted, the recommendations are applicable to unsupported slopes, where no slope support, reinforcement, or extensive rockfall prevention is used. All constructed slopes should be reviewed in the field during construction to check that design assumptions remain valid. It may be necessary to revise slope design recommendations for specific structures following future site investigation or during construction as ground conditions are exposed.

6.5.2.2. Area Specific Cut Slope Recommendations

Table 6-1 provides area specific cut slope recommendations, which are also illustrated in Drawings 08 to 10. These recommendations assume the stratigraphy and water levels illustrated in Drawings 08 to 10, and the material strength properties listed in Table 4-1, Table 4-2 and Table 4-3. The location of each cross section is illustrated in Drawing 07. A two-dimensional limit equilibrium stability analysis was completed to evaluate the long term stability of each slope. Static and pseudo-static analyses were completed using Slope/W (Geo-Slope, 2007), a commercially available limit equilibrium slope stability analysis software.

Typically, the slopes are composed of variable thickness of overburden over bedrock that is weathered to varying degrees. Table 6-1 provides the estimated overburden thickness and recommended cut angles for both the overburden and the underlying rock. In design, the overburden cut angle should be used in the zone between the ground surface and a depth equal to the overburden thickness.

Table 6-1. Recommended Permanent Cut Slope Angles – Area Specific

Area	Overburden		Slope below Overburden		Notes (refer to Drawings 08 to 10)
	Thick-ness (m)	Steepest Cut Angle	Material	Steepest Cut Angle[1]	
Primary Crusher	2 - 4	2.5H:1V	Type 1, 2, 3 Rock	1.75H:1V	Design FS = 1.5; maximum slope height ~107 m; slope angle controlled by dip of foliation at about 30-32 degrees; benched slope design recommended; 8 m maximum bench height; 13 m minimum bench width; 0.25H:1V bench face angle.
100 Day Storage	3 - 4	2.5H:1V	Type 2, 3 rock	1.75H:1V	Design FS = 1.5[2]; slope angle controlled by dip of foliation at about 30-32 degrees; minimum distance of 80-100 m required between slope crest and toe of haul road / crusher platform fill slopes. Benched slope design is recommended as detailed above for primary crusher.
Truck Shop	5 - 8	2.5H:1V	Type 3 rock	1.75H:1V	Design FS = 1.5; maximum slope height = ~22 m; slope angle controlled by dip of foliation. Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Plant Site	3 - 7	2.5H:1V	Highly to completely weathered rock	2H:1V	Design FS = 1.5; maximum slope height ~35 m; Recommend 5 m wide bench at rock-overburden contact to contain potential slumping of ice-rich overburden and slope maintenance.
Dublin Gulch Diversion	2 - 5	2.5H:1V	Till	2H:1V[3]	Design FS = 1.5; maximum slope height ~28 m; maximum cut angle assumes that the cut slope is dry.

Notes:

1. Maximum overall slope angle in the slope materials below the overburden depth. Overall slope angle defined by the line that connects the toe of the slope with the slope crest at the rock-overburden contact.
2. Recommended FS for the 100 day storage cut is 1.5 due to proximity to crushers and potential to undermine them in case of failure. FS = 1.3 could be considered when the cut is moved 80-100 m further from the crushers, however, the overall slope angle will still be controlled by the dip of the foliation and cannot be steepened significantly.
3. Assumed groundwater level is greater than 6 m below existing ground surface, which is inferred but not confirmed and requires further study.

At the primary crusher, 100-day storage, and truck shop areas, the cut slope design is controlled by the potential for failure of the rock along discontinuities defined by foliation in the metasedimentary rock. The foliation is expected to dip out of the slope at angles ranging from about 20° to 40°, typical values observed in Structural Domain C (Drawing 19). The potential failure wedge that could form on slopes of this size is large enough to make mechanical support of the slopes impractical. Therefore a relatively shallow overall slope angle has been recommended. This overall slope angle is approximately parallel to the observed dip of the foliation, which essentially eliminates the potential for a planar failure at the slope-scale.

Bench scale failures are expected, including minor raveling and slumping, where the foliation is undercut; however, failures occurring on upper bench faces are not expected to adversely affect the infrastructure at the base of the slope due to the presence of the 13 m wide rockfall catchment benches. However, an allowance should be made in the design for spot bolting of loose blocks of rock on the bench faces in case specific weak structures are encountered. Mesh may also be required to contain poor quality rock that could ravel, should it be encountered, particularly on the bottom bench where service vehicles may be entering. Additionally, an 8 m wide rockfall catchment area should be included in the design at the upper and lower platform elevations. A 1 m high barrier (concrete or earth, or permanent fence) is recommended to be placed at the outer edge of the rockfall catchment area to deter encroachment into the catchment area by vehicles or personnel.

At the primary crusher, it is expected that blasting will be required to excavate the rock; therefore a benched slope design has been recommended. The recommended bench face angle is 0.25H:1V, which has been selected to facilitate controlled blasting. The maximum recommended bench height is 8 m. The minimum recommended bench width is 13 m to facilitate installation of a safety berm and to allow access for bench clean up. The bench width may need to be adjusted at detailed design to maintain the recommended overall slope angle of 1.75H:1V.

The recommendations provided for the primary crusher cut are based on assumed water levels and ground conditions, which are based on relatively sparse site characterization data. The consequences of a slope-scale failure at the primary crusher cut are perceived to be very high. Additional site investigations are recommended to reduce the current level of uncertainty in the understanding of ground conditions. The recommendations provided in this report assume that the design is controlled by the foliation of the meta-sedimentary rock. Future site investigation should verify that additional unfavorable conditions are not present and should be designed to characterize the orientation and condition of the contact between the meta-sedimentary and igneous rock, which is expected to daylight near the base of the cut.

At the 100-day storage area, the crest of the cut slope may daylight near the toe of the fill slopes from the haul roads and crusher platform. A minimum distance of 80-100 m between the cut slope crest and toe of fill is recommended to reduce the possibility of a slope failure at the 100-day storage area which could affect the crusher or haul roads.

The recommended cut angle at the Dublin Gulch diversion assumes that the slope materials are unsaturated. If the slope materials are saturated, the recommend cut angle would decrease to 2.5H:1V. Current information regarding the depth to groundwater along the diversion is sparse. Future site investigation programs should be designed to characterize the groundwater depth along the diversion, and update the cut slope design, if appropriate.

A rockfall catchment area should be provided at the base of all cut slopes. The catchment area should be sloped back towards the cut slope at an angle of 4H:1V. The recommended minimum width of the rockfall catchment is 2.5 m below soil cuts, and 8 m below rock cuts.

6.5.2.3. General Cut Slope Recommendations

Table 6-2 provides general cut slope angle recommendations based on material type, for general application across the site for cut slopes less than 10 m high. It is assumed these cuts will be unsaturated and without adverse geologic structure. Cut slopes that do not meet these conditions should be reviewed on a case-by-case basis by the geotechnical engineer.

Table 6-2. Recommended Permanent Cut Slope Angles – General

Slope Material	Maximum Cut Slope Angle[1]	Maximum Cut slope Height	Notes[1]
Colluvium	2.5H:1V	10 m	
Till	2H:1V	10 m	
Highly to completely weathered rock (excavatable)	2H:1V	10 m	
Type 3 rock (generally excavatable)	1.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 2 rock (generally rippable)	1H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered
Type 1 rock (may require blasting)	0.5H:1V	10 m	May have to decrease to as flat as 1.75H:1V to avoid undercutting adverse geologic structure, if it is encountered

Notes:

1. Maximum cut slope angles assume the slope is < 10 m high, unsaturated, and without adverse geologic structure

7.0 CONSTRUCTION MATERIALS

7.1. General

This section discusses the demand for specific engineering fill materials, and provides some comment on quantities of excess materials requiring permanent disposal or temporary storage.

7.2. Borrow Requirements

7.2.1. Mine Site Area

Development of the proposed mine will involve excavation, stockpiling, processing, hauling, placing and compaction of a variety of earth and rock materials. Material take offs (MTOs) with earthworks quantity estimates were provided by Merit Consultants International on January 6, 2012. These MTOs include numerous line items for various types of excavation or granular borrow required for construction of the mine site facilities, including the following approximate quantities of specific materials:

- Approximately 2.9 million m^3 of engineered fill, which includes approximately 2.1 million m^3 of engineered fill for the heap containment dyke and diversion embankment, selected from a variety of sources, including processed placer tailings, non-durable rock obtained during bulk earthworks activities, and possibly durable waste rock from mining. This "engineered fill" includes the following general categories of materials:
  - General fill,
  - Structural fill,
  - Durable rock fill, and
  - Non-durable rock fill;
- 298,000 m^3 of crushed durable rock to produce a well-graded material for the heap overliner;
- Various minor quantities of miscellaneous engineering materials, including silt/fines for liner construction, transition/filter materials, drainage materials, rip rap, concrete aggregate, and road pavement structure materials.

7.3. Suggested Borrow Material Classifications

Construction fill materials at the project site have been identified by other disciplines without specific technical specifications. The following definitions are proposed for consideration by other disciplines responsible for earthworks construction.

<u>Silt/Clay Liners</u>

These are fine-grained fills used as a barrier for chemical and physical migration of fluids. The prefeasibility study report (SWRPA 2010) suggests a target hydraulic conductivity for compacted fine grained liner materials of no more than 1 x 10^{-5} cm/s, or 1 x 10^{-6} cm/s in the absence of a leachate detection and removal system.

Silt liner materials should contain a minimum of 35% passing the No. 200 sieve and be free of all deleterious materials including oversize clasts of 75 mm or greater, frozen soils, and organics. This material should be placed with uniform moisture content, typically within 2% (above) optimum moisture content (ASTM D698) and a USCS classification of CL, ML, CH or MH.

Rock Fill

Rock fill can be classified as one of two types: 1) that derived from strong rock, yielding durable rock fragments larger than gravel size and containing sand and gravel with less than 15% fines when excavated/blasted; and, 2) that derived from weak, fissile rock, generating non-durable rock fragments. The first type may be placed and compacted as a rock fill in 1 m lifts, whereas the second type should be placed and compacted in thinner lifts, with watering and compaction similar to that required for an earth fill.

Additional detail on construction of rock fills derived from strong rock or weaker rock may be found in Cooke (1990) and US Army Corps of Engineers (2005).

For the purpose of this report, rock fill is divided into two categories - durable rock fill; and, non-durable rock fill - each with different anticipated engineering properties, sources, and placement and compaction requirements. Most of the metasedimentary rock excavated at the site will yield non-durable rock fill. Relatively unweathered granodiorite from the pit area, and quartzite from the hornfels aureole around the granodiorite intrusion, are expected to yield durable rock fill.

Structural Fill

Structural Fill is an engineered soil material placed and compacted for use beneath lightly to moderately loaded structures to provide a uniform bearing surface with tolerable movements under load through the life of the structure.

Structural Fill should consist of well graded sand and gravel having a maximum size of 75 mm and less than 8% fines (materials passing the No. 200 sieve) and be free of all deleterious materials including frozen soils, clay lumps and organics. All structural fill should be placed and compacted to at least 95% Modified Proctor Maximum Dry Density (MPMDD). Placement and compaction should be performed in moisture-conditioned lifts less than 300 mm of loose thickness with equipment suitable to obtain the specified density.

Materials that do not satisfy the specifications for structural fill may be used as structural fill in specific applications, at the discretion of a qualified geotechnical engineer. For example, locally excavated weathered rock that contain more than 8 % fines may serve as structural fill provided compaction objectives can be met and drainage/frost susceptibility issues are less important, e.g., used only at depth in thick fills.

General Fill

General Fill is an inorganic granular material used for general site grading, thermal insulation cover and/or protection of pipes, or similar applications. Materials should be limited to maximum 200 mm particle size, and contain no more than 20% fines. General fill should be compacted to yield a stiff surface as determined acceptable to the geotechnical engineer by proof-rolling with fully loaded dump trucks. General Fill should not be used for support of settlement-sensitive structures.

Grading Fill

This is a soil material used as an intermediate layer between in-situ soil or rock subgrade and higher quality engineering materials above, such as road base, for example. Any granular material that can be placed and compacted to 95 % MPMDD to provide a uniform bearing surface may be suitable for this purpose. Selected materials should have a maximum particle size of 150 mm. Oversize materials may be screened out, or can be removed from the surface of placed materials by hand. Suitable materials would include and materials identified as suitable for structural fill or general fill, and may include local colluvium.

Rip rap

Riprap consists of cobble and boulder size rock fragments, typically angular or subangular as derived from blasting or crushing, and is used as a protective barrier from erosion and scour due to water currents and/or ice. Material should consist of hard, durable rock fragments free from splits, seams or defects that could impair its soundness. Thicknesses of riprap layers typically vary from 1.0 to 1.5 times the maximum rock size. Riprap is typically specified by the median particle size, D_{50}. Additional grain size criteria may be presented if the riprap needs to be either well graded or uniformly graded, depending on the specific application. Preliminary information from Tetra Tech suggests there will be a need for riprap with D_{50} of about 500 to 600 mm.

Drainage Material

This is an open or gap-graded granular material intended for allowing free drainage of fluids to pipes and/or seepage collection systems. Drainage material should consist of crushed or uncrushed screened rock or gravel free of fines and flat, elongated particles. Grain size requirements depend on the specific drainage application.

Filter/transition Material

Filters are a transition zone material used for preventing soil migration due to fluid flow between granular materials, and/or between rock fill and finer silt and clay layers. Filter material gradations are generally designed based on the specific material gradations that they will transition. Filter materials can be derived from rock excavations or gravel borrow areas, and may require crushing, screening and/or washing to attain the necessary gradations.

Concrete Aggregate

Concrete aggregate includes fine and coarse aggregate meeting CSA A23.1 specifications for designing and proportioning concrete mix. Aggregates can be derived from crushed durable rock or gravel.

Road Base

This is an engineered material, consisting of a well-graded, hard, durable, very clean (less than 5% fines), screened and crushed sand and gravel or rock, with a maximum particle size of 38 mm. Material should be free of flat and elongated pieces and have a minimum of 50 % fractured particle faces. Road base gravel should also have less than 25% loss by Micro-Deval. Road base materials should be placed and compacted to a minimum of 98% MPMDD.

Road Surfacing Material

Road surfacing material should consist of well-graded hard, durable, angular screened and crushed sand and gravel or rock with less than 15% fines, and maximum particle size of 25 mm. Granular material should have less than 25% loss by Micro-Deval and greater than 50% fractured faces.

Heap Overliner Material

The heap leach pad will include a protective layer of crushed gravel over the primary liner and solution collection piping system, known as the heap overliner. As specified by Tetra Tech on an email dated in November 23, 2011, the overliner drain fill shall consist of free-draining granular material with 38 mm maximum particle size and a maximum of 5 percent fines passing the No. 200 ASTM sieve size (0.075-mm). The material shall be free of organic matter and soft, friable particles in quantities objectionable to the geotechnical engineer. The drain fill shall have a hydraulic conductivity of $2x10^{-4}$ m/sec or higher when tested in accordance with the constant-head method described in ASTM D 2434, using a hydraulic gradient of 1.

7.4. Temporal Material Demand and Material Balance

7.4.1. General

This project will involve the movement of large quantities of earth and rock fill in a relatively short construction period (currently understood to be about three years) and within a limited footprint in rugged terrain. It will be challenging to manage material movement to meet the construction schedule. An effort has been made to understand the temporal nature of planned material movement, drawing from material take offs (MTOs) provided by Wardrop, Tetra Tech, BGC and Knight Piésold, as compiled by Merit Consultants and received on January 06, 2012.

Table 7-1 presents a breakdown of material quantities over time, based on an analysis of quarterly supply and demand. Cut quantities are shown as positive numbers, being quantities available for use (or intended for disposal). Fill quantities are shown, in brackets, as negative numbers, being quantities required for construction. Material supply/demand and total cut/fill balance are also illustrated graphically in Figure 1.

The current analysis shows a peak excess of approximately 2.2 million cubic metres of excavated material which will require storage during the first year of the project, as shown in Table 7-1. This excess supply will be drawn down over the following year, leaving a small excess of available fill at the end of construction.

The material categories listed in Table 7-1 correspond to categories provided in the MTOs from Merit Consultants and Wardrop received on January 06, 2012.

7.4.2. Excess Materials Requiring Storage or Disposal

Bulk earthworks activities will generate several types of material that are unsuitable for immediate use, or may not be suitable for any use, thus necessitating temporary storage or permanent disposal. Decisions on ultimate disposition may require further consideration of the need for soil for reclamation. Preliminary information suggests the development of the following materials requiring storage or disposal:

- Topsoil – these materials will be required for reclamation. It will be necessary to develop stockpiles to store these materials during construction and mine operation. The current estimate of 313,000 m^3 does not yet include open pit pre-stripping;
- Ice-rich permafrost – these materials will be unsuitable for immediate re-use in any application. They may be suitable for re-use in reclamation after thawing and draining of excess water. These materials will require careful storage after excavation and prior to use, as they will be weak and unstable when thawed. It may be necessary to develop specific storage areas with containment structures and water management infrastructure. Current estimates indicate approximately 40,000 m^3 of ice-rich permafrost will be removed during development of the heap leach facility, and with additional volumes from other areas on site (quantity currently unknown), all requiring management during construction and mining operations;
- Colluvium – some of the shallow colluvial soils removed during bulk excavation work will contain excessive amounts of deleterious materials, such as organic inclusions or excess proportions of fines. Current estimates suggest approximately 227,000 m^3 of colluvium requiring permanent disposal or storage for re-use in reclamation.
- Waste rock - these materials are indicated by Merit and Wardrop as unsuitable for re-use as construction fills and are intended to be permanently disposed in designated disposal areas. In general they correspond to soils or rock with deleterious materials and may include excess fines or excess ice. Current estimates indicate approximately 500,000 m^3 of unsuitable material that needs to be excavated, removed and disposed, either in the waste rock storage areas, or other disposal areas to be determined.

Table 7-1. Quarterly Demand for Cut and Fill Quantities, as inferred from MTOs from Merit, Wardrop and Tetra Tech

Used for Material Balance	Category	Material Quantity (m^3)												
		Q4 2012	**Q1 2013**	**Q2 2013**	**Q3 2013**	**Q4 2013**	**Q1 2014**	**Q2 2014**	**Q3 2014**	**Q4 2014**	**Q1 2015**	**Q2 2015**	**Q3 2015**	**Total**
No	Strip and stockpile topsoil	50,738	26,026	147,437	0	0	0	37,015	7,701	0	0	44,485	0	**313,402**
	Excavate and dispose waste rock in waste dump	77,319	0	261,201	0	0	0	54,344	35,234	0	0	67,842	0	**495,940**
	Excavate colluvium	35,050	0	168,300	0	0	0	18,000	5,700	0	0	0	0	**227,050**
	Excavate rock	10,758	0	375,555	0	0	0	4,278	0	0	0	0	271,369	**661,960**
	Excavate permafrost	3,500	0	34,900	0	0	0	0	1,200	0	0	0	0	**39,600**
	Local cut and fill	76,791	0	239,749	0	0	0	45,210	13,251	0	0	0	839,463	**1,214,464**
Yes	Excavate and stockpile suitable materials	208,271	0	185,885	0	0	0	0	24,632	0	0	0	133,699	**552,487**
	General excavation	333,280	0	1,182,390	0	0	0	75,400	120,000	0	0	0	0	**1,711,070**
	Excavate placer tailings	0	0	876,000	0	0	0	0	0	0	0	0	0	**876,000**
	Subgrade preparation	0	0	(18,300)	(104,600)	0	0	0	(3,500)	0	0	0	0	**(126,400)**
	Other materials	(3,520)	0	(58,823)	(3,100)	0	0	(12,000)	(298,000)	0	0	0	0	**(375,443)**
	Fill from stockpile	(18,110)	0	(355,643)	0	0	0	(149,191)	(17,461)	0	0	0	(7,430)	**(547,835)**
	Fill	(70)	0	(126,518)	(1,119,000)	0	0	(743,000)	0	0	0	0	0	**(1,988,588)**
Material balance - each quarter		519,851	0	1,684,991	(1,226,700)	0	0	(828,791)	(174,329)	0	0	0	126,269	**101,291**
Material balance - cumulative		519,851	519,851	2,204,842	978,142	978,142	978,142	149,351	(24,978)	(24,978)	(24,978)	(24,978)	101,291	**101,291**

Note:

1. Quantities (in brackets) indicate deficit quantities, or fill to be derived from elsewhere.

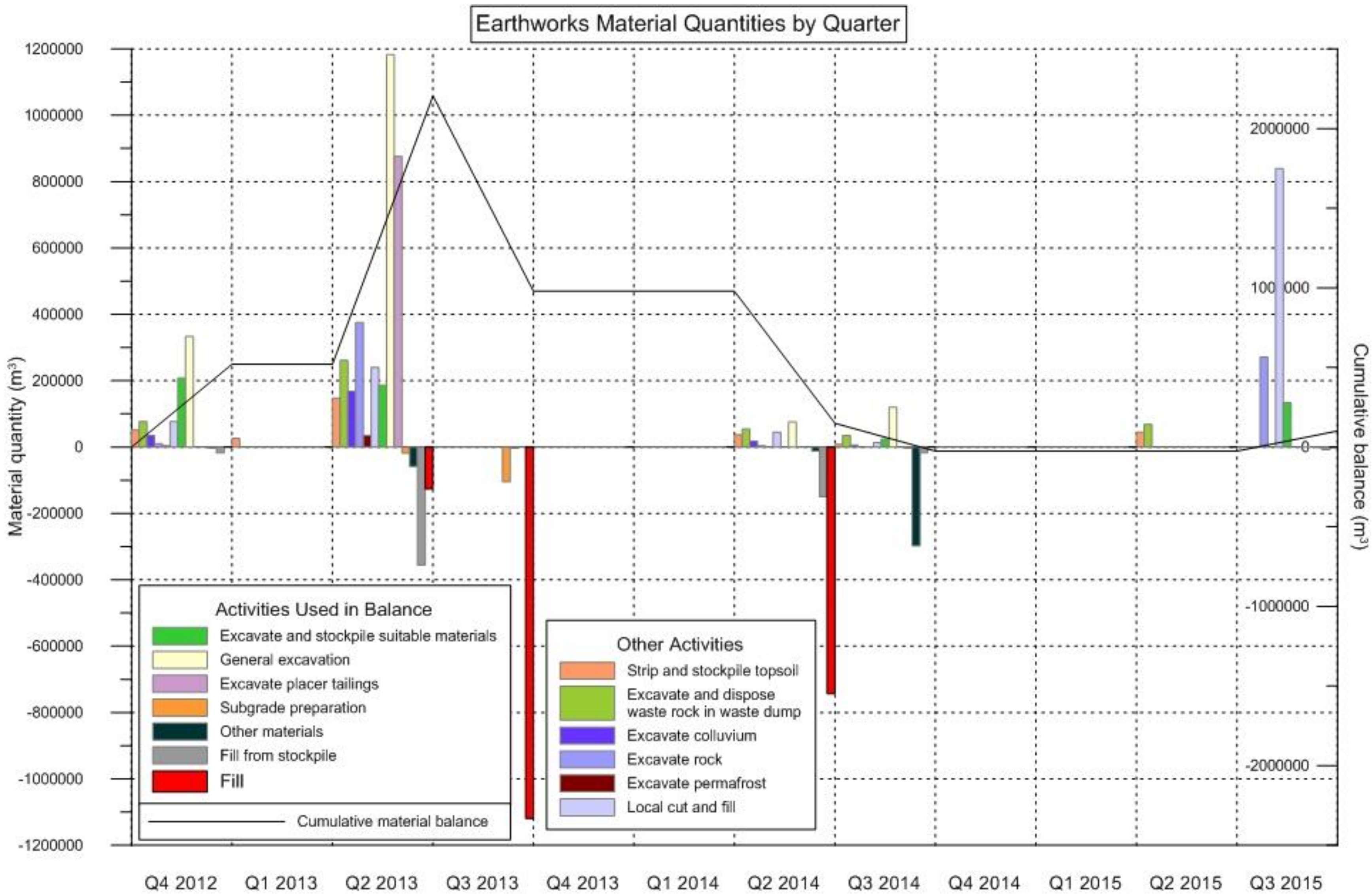

Figure 1. Quarterly Material Demand and Balance

7.5. Available Borrow Materials

7.5.1. General

Several sources of borrow material were identified in the BGC Borrow Evaluation Report (BGC 2011c). This included two potential silt borrow pits near the proposed laydown area and near the confluence of Platinum Gulch and Haggart Creek; the existing placer tailings in the Dublin Gulch valley bottom; and, proposed platform cuts into bedrock along sloping ground. Additional work was conducted in 2011, including investigation of the placer tailings; investigation of potential silt borrow near the proposed laydown area, evaluation of various rock sources for use as engineered fill; and, evaluation of placer tailings and rock near the proposed open pit for potential use as concrete aggregate.

The distribution of placer tailings in Dublin Gulch and Haggart Creek valley bottoms is illustrated in Drawing 27. The locations of potential borrow sources, including the placer tailings, rockfill sources and the proposed silt borrow, are illustrated in Drawing 28. Summary information for these various borrow sources is presented in Table 7-2.

7.5.2. Rock Sources

Rock will be required for use as rock fill (durable and non-durable), rip rap, and for crushing to produce concrete aggregate and heap overliner material.

Most of the rock encountered at the project site consists of weak, friable metasedimentary rock, suitable only for use as non-durable rock fill, or general fill. Such rock will be available in moderate quantities for local re-use from most major cuts for building pad development. A larger quantity is expected to be available from the “Ann Gulch central knob,” an area of extensive cutting within the first phase of heap pad subgrade development.

Durable rock may be available in very small quantities (i.e. several hundred to a few thousand cubic metres) from the oversize materials screened out of the placer tailings in Dublin Gulch valley bottom, and in larger quantities from weathered granodiorite in the open pit pre-strip or the Steiner zone. Suitability for use of these materials as concrete aggregate requires further testing and analysis, as preliminary data suggest that the rock will not meet normal standards for concrete aggregate. Expert advice will be required to determine whether the local rock materials can be used as concrete aggregate with admixtures to counteract the known limitations. It may be possible to use local rock as concrete aggregate without admixtures with more careful selection, e.g. using only granodiorite from the pit pre-strip; however, this alternative also requires tighter controls and further testing.

7.5.3. Placer Tailings

The placer tailings are found within the Dublin Gulch and Haggart Creek valley bottoms and consist of reworked materials from historical placer mining operations.

The distribution of materials within the placer tailings was examined by field reconnaissance, and the spatial distribution of typical material types is illustrated in Drawing 27. The visual observations of surficial materials are supported by test hole observations and associated lab testing in Dublin Gulch area. Interpretations in the Haggart Creek area are based solely on visual observations of surficial materials.

Examination of the surface topography of the tailings and the approximate bedrock surface, as inferred from test hole locations, suggests that approximately 2 million m^3 of fill materials are present in the Dublin Gulch valley bottom and potentially exploitable for use elsewhere as an engineering material. Note that if all of these materials are exploited to expose bedrock, it may be necessary to replace a significant quantity of material to restore grades in the pond development area to a level above the existing valley bottom drainage system. The net quantity of potentially available exploitable materials, currently present above the elevation of the creeks, is roughly 1.1 million m^3. An additional 750,000 m^3 of placer tailings is available in the Haggart Creek area above the level of Haggart Creek.

Producing engineered fills from the placer tailings will require targeted selection combined with crushing, screening and/or washing.

Removal of placer tailings down to bedrock, which may be necessary to provide an adequate foundation subgrade, will require significant efforts for dewatering.

7.5.4. Silt Borrow

Exploration for potential silt borrow was conducted in the general vicinity of the proposed laydown area, near the location of the existing exploration camp. The 2011 investigation work included four auger holes and six test pits.

Compacted samples of silt obtained from the vicinity of the proposed silt borrow area yielded a mean permeability of 4.5×10^{-8} cm/s at 95 % MPMDD, based on four tests. Note that these results are lower than anticipated and should be checked through further testing.

It should be noted that ice-rich permafrost is present to at least 25 m depth in the proposed silt borrow area, and the thickness of suitable silty material is, on average, approximately 4-5 m, ranging from about 2 m to more than 15 m thickness. The excavated silt material will need to be thawed and dried before use. Screening may be required to remove oversize particles.

It is estimated that up to about 220,000 m^3 of silty materials may be obtained from the indicated silt borrow area.

Table 7-2. Summary of Borrow Material Availability.

Borrow Source	Material Types	Estimated Volumes (in situ volumes, except where noted)	Comments
Pit Pre-Strip	Durable rock fill Non-durable rock fill Concrete aggregate Heap overliner Rip rap	Very large. Available volumes depend on the sequence of mining activities, although materials can be developed prior to mining activities by developing a quarry prior to pre-strip.	Source consists of weathered granodiorite and weathered silicified metasedimentary rock (i.e. typically quartzite). Suitable concrete aggregate has not yet been identified, and requires further study focussing on the granodiorite. Testing of material for use as heap overliner was commissioned by Tetra Tech, and the results are not available to BGC at the time of writing. Availability of rip rap in desired block size of 500-600 mm will require further input from mining, and careful selection. Most near surface weathered rock suggests excavated block size of approximately 100-300 mm.
Ann Gulch Central Knob	Non-durable rock fill	Up to approximately 900,000 m^3, subject to further input from Tetra Tech.	Grading plans showing the volumes of anticipated rock excavation are not available to BGC at the time of writing.
Steiner Zone	Same as for Pit Pre-strip	Up to approximately 200,000 m^3, assuming quarry depth of 5 m	Very little information is known about this area. Further subsurface investigation is required to confirm quality and quantity of available materials.
Dublin Gulch Placer Tailings	General Fill Structural Fill Concrete aggregate Heap overliner Rip rap	Approximately 2.0 million m^3, of which about 1.1 million m^3 is above the groundwater level	Materials are highly variable, and will require processing through screening, crushing and/or washing to develop the required material specifications. Oversized materials (> 75 mm) screened from the tailings may be suitable for use, after crushing, as heap overliner or concrete aggregate pending further analysis. Some rip rap (perhaps up to 2-3,000 m^3) can be developed from the screened oversize material, however the quantity of 500-600 mm particles is expected to be small and would require careful selection.
Haggart Creek Placer Tailings	General Fill Structural Fill	Approximately 750,000 m^3 available above the elevation of Haggart Creek	No subsurface information is available to support the quantity estimate. Available volume of suitable material is estimated from visual classification of surficial materials present in several distinct piles.
Silt Borrow	Silt liner	Approximately 220,000 m^3	Available silt materials are frozen and ice-rich, and will require thawing and drying prior to use.

8.0 RECOMMENDATIONS FOR FURTHER INVESTIGATION

This report has provided feasibility study level geotechnical recommendations for mine site infrastructure. There are several areas where additional investigation is recommended to provide sufficient data for subsequent detailed design. The following list provides recommendations for further investigation. This list should be read in conjunction with Drawing 29.

- Diamond drill holes:
  - Vertical holes at all three crushers to better establish depth to suitable bearing stratum across the facilities' footprints;
  - Inclined holes in the area of proposed rock cuts at the crushers and 100 day storage pad;
  - Vertical holes at the plant site to better determine depth to suitable bearing stratum within the extent of the building pad;
  - Allowance for additional holes within the footprint of the heap leach facility, in the event Tetra Tech considers additional data warranted;
  - Allowance for additional holes at major cuts such as that along the phase 1 heap access road;
  - Allowance for holes for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Auger holes (with CRREL barrel available):
  - Conveyor bent foundation locations between tertiary crusher and heap leach facility;
  - Along the alignment of the proposed Dublin Gulch diversion channel;
  - In Eagle Pup to confirm the extent of the ice-rich lobate feature in the valley bottom;
  - At the revised truck shop buildings and cut locations;
  - Allowance for holes in areas being considered for retaining structure(s) for ice-rich overburden storage;
  - Allowance for holes in the Eagle Pup and Platinum Gulch WRSAs, with details to be addressed under separate cover in the WRSA engineering report.
- Plate load tests at plant site and all three crushers;
- Design and construction of a test fill embankment to determine whether high quality structural fill would be suitable to support the secondary and tertiary crushers;
- Sampling and strength testing of materials selected for heap embankment fill, if considered necessary by Tetra Tech;

- Additional sampling and testing of granodiorite from pit area and Steiner zone for possible use as concrete aggregate. Obtain materials engineering advice to guide this process, including trial mix designs possibly with additives to make use of local aggregates and trial design mixes for lean concrete for use in raising grades at crushers;
- Sample mixes for low strength concrete as stabilized fill

9.0 CLOSURE

We trust the above satisfies your requirements at this time. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

BGC ENGINEERING INC.
per:

Daniela Welkner, M.Sc.
Senior Engineering Geologist

Pete Quinn, Ph.D., P.Eng.
Senior Geotechnical Engineer

Reviewed by:

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Senior Geotechnical Engineer

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Senior Civil Engineer

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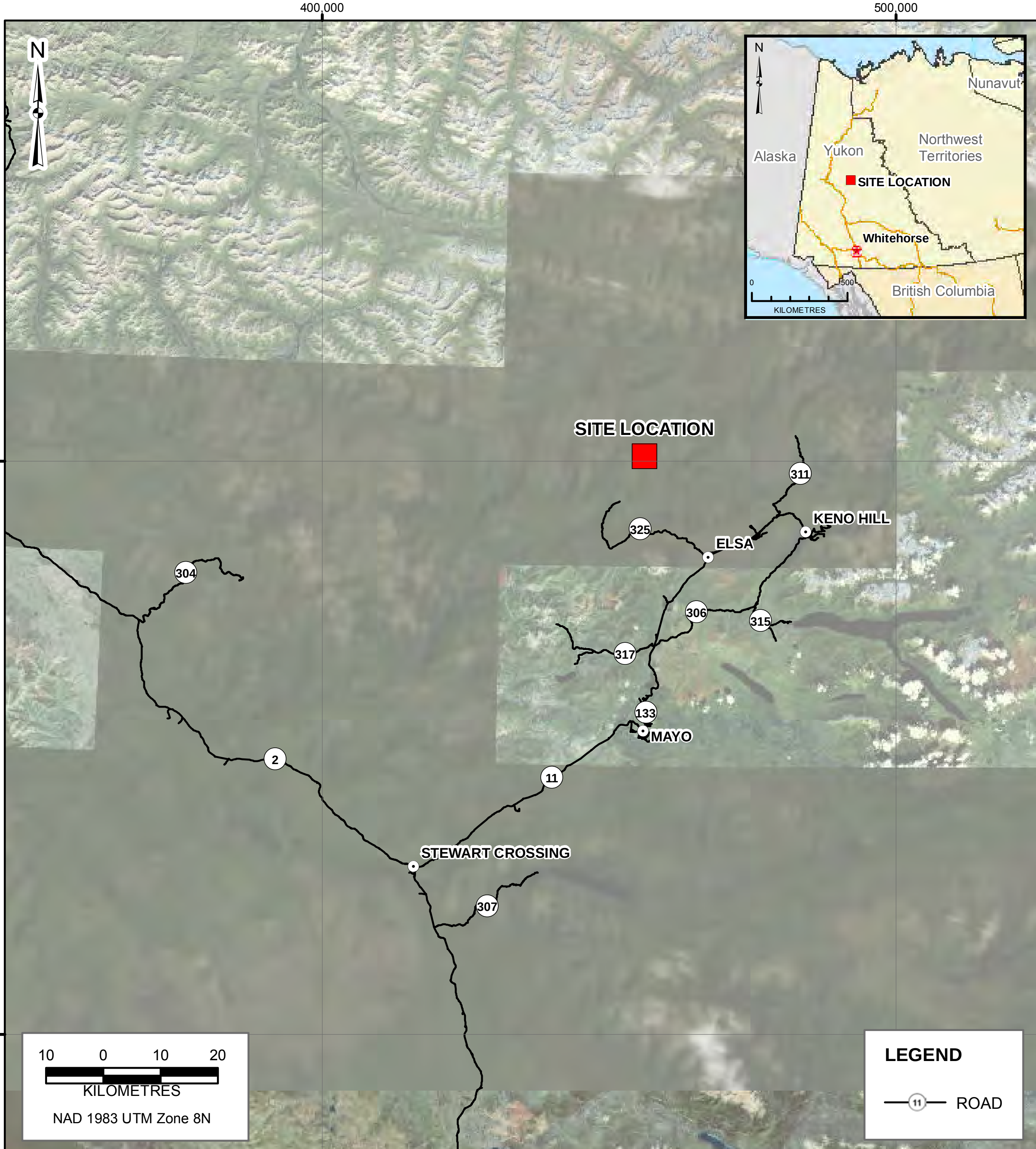
DRAWINGS



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DATE:	JAN 2012	CHECKED:	DW
DRAWN:	LL	APPROVED:	PQ

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
GENERAL SITE LOCATION

CLIENT:
VICTORIA GOLD CORPORATION

PROJECT No.:	DWG No.:	REV.:
0792-006	01	0

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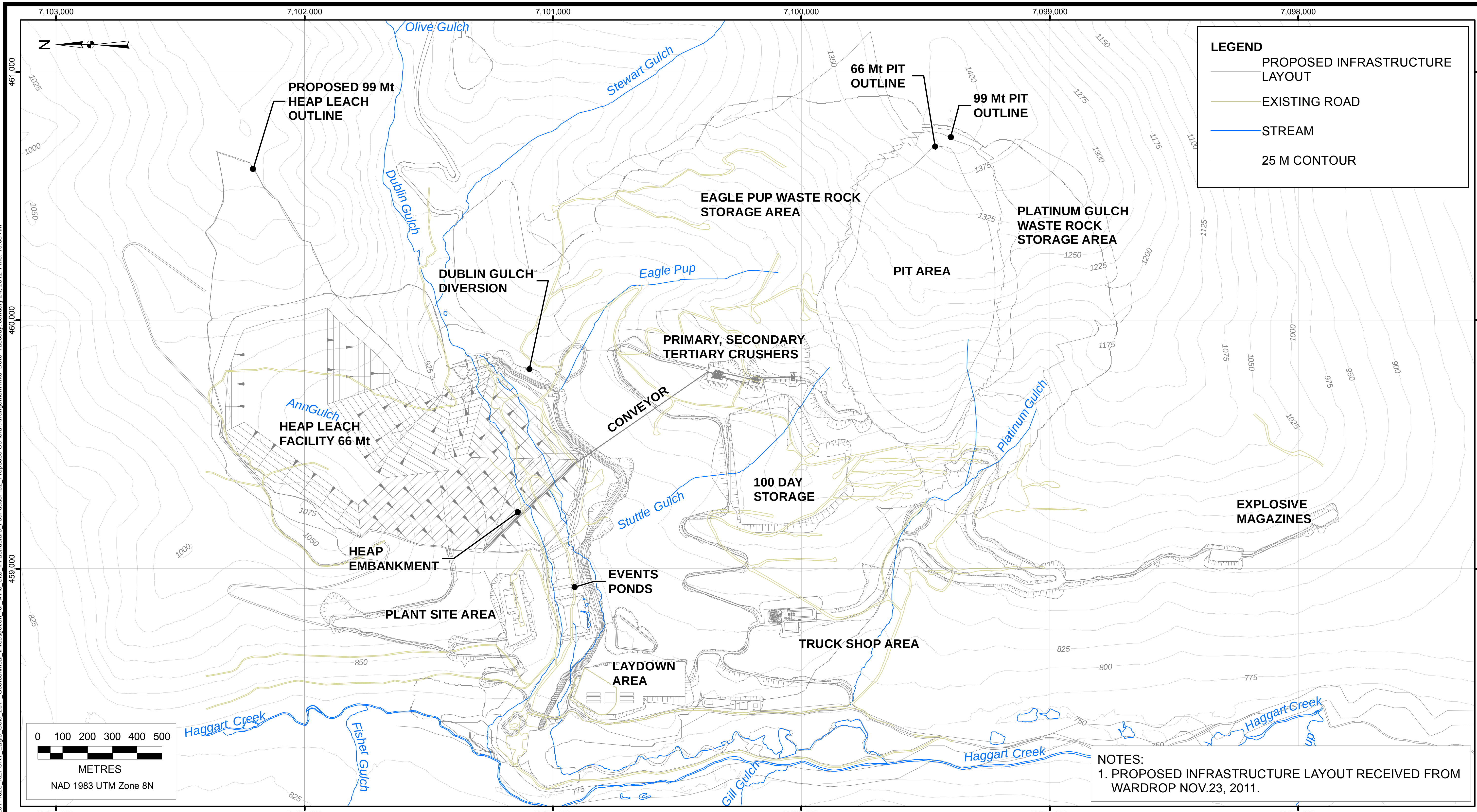

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DATE: JAN 2012
DRAWN: LL
DESIGNED: KH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: PROPOSED GENERAL ARRANGEMENT

PROJECT No.: 0792-006
DWG No.: 02
REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

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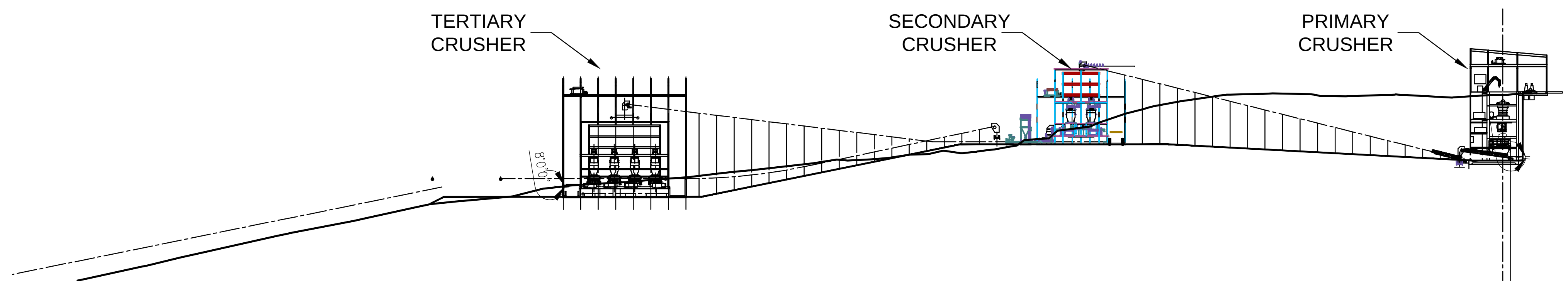

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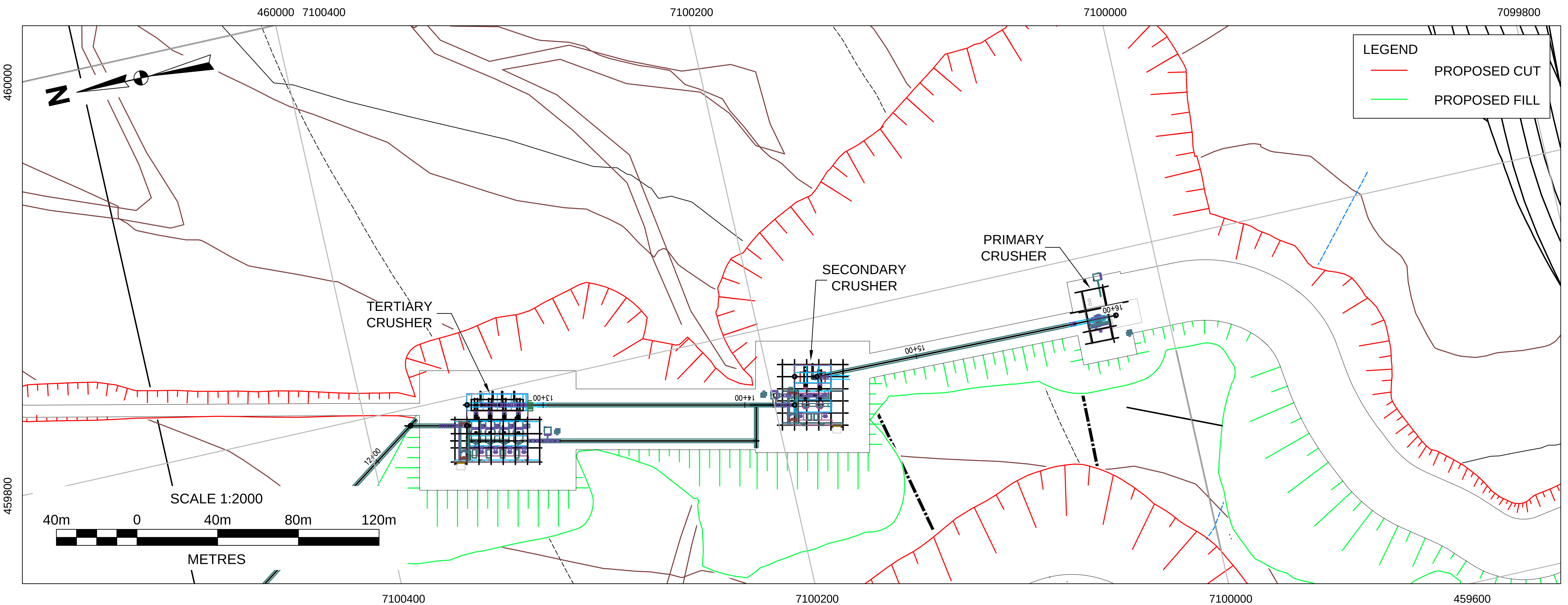

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DESIGNED:	DW
CHECKED:	DW
APPROVED:	PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: PROPOSED ARRANGEMENT - CRUSHERS

PROJECT No.:	DWG No.	REV.:
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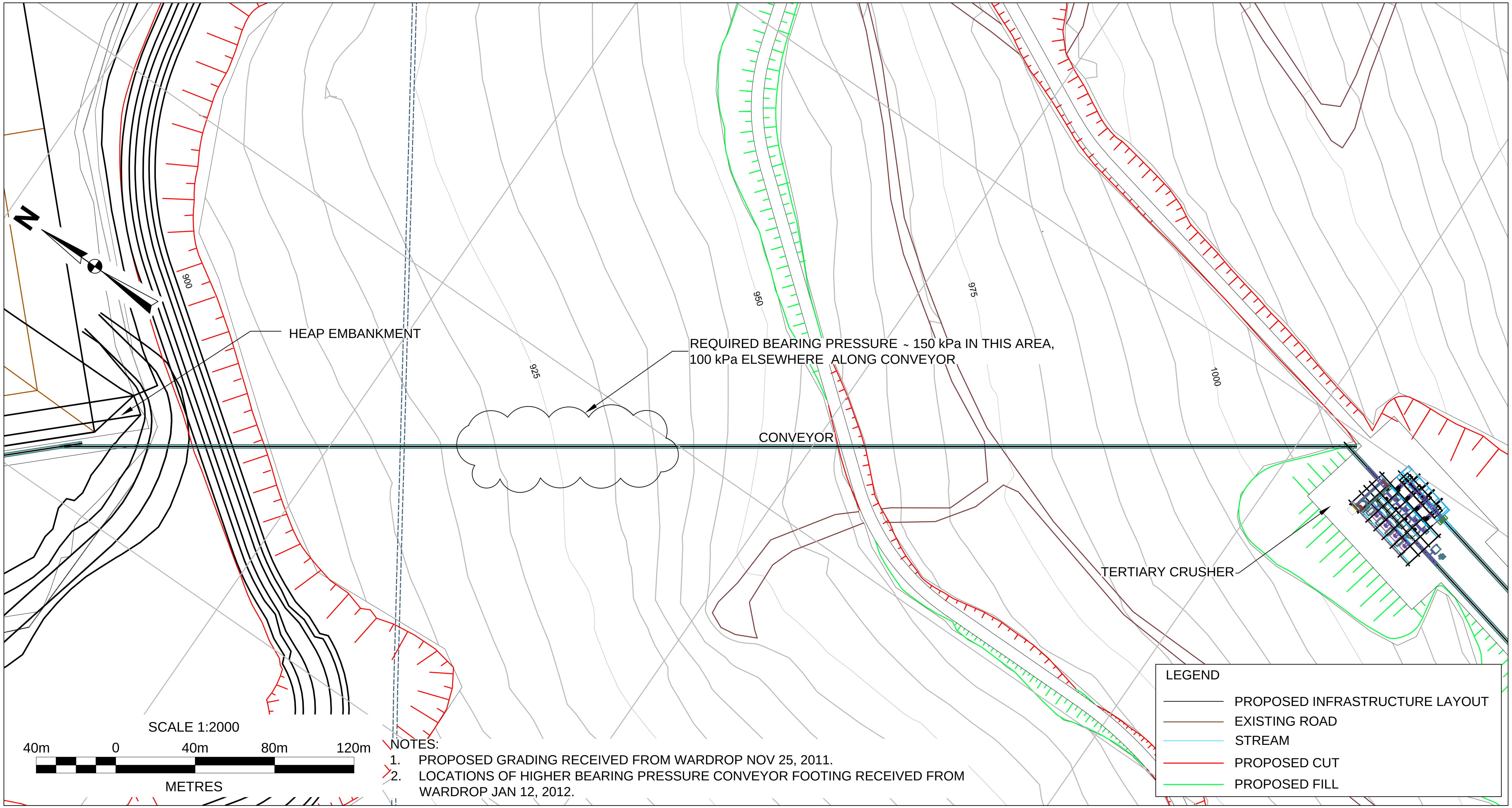

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DATE:	JAN 2012
DRAWN:	COC
DESIGNED:	KH
CHECKED:	DW
APPROVED:	PQ

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: PROPOSED GENERAL ARRANGEMENT CONVEYORS

PROJECT No.:	DWG No.:	REV.:
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DWG TO BE READ WITH BGC REPORT TITLED " EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT " DATED JANUARY 2012

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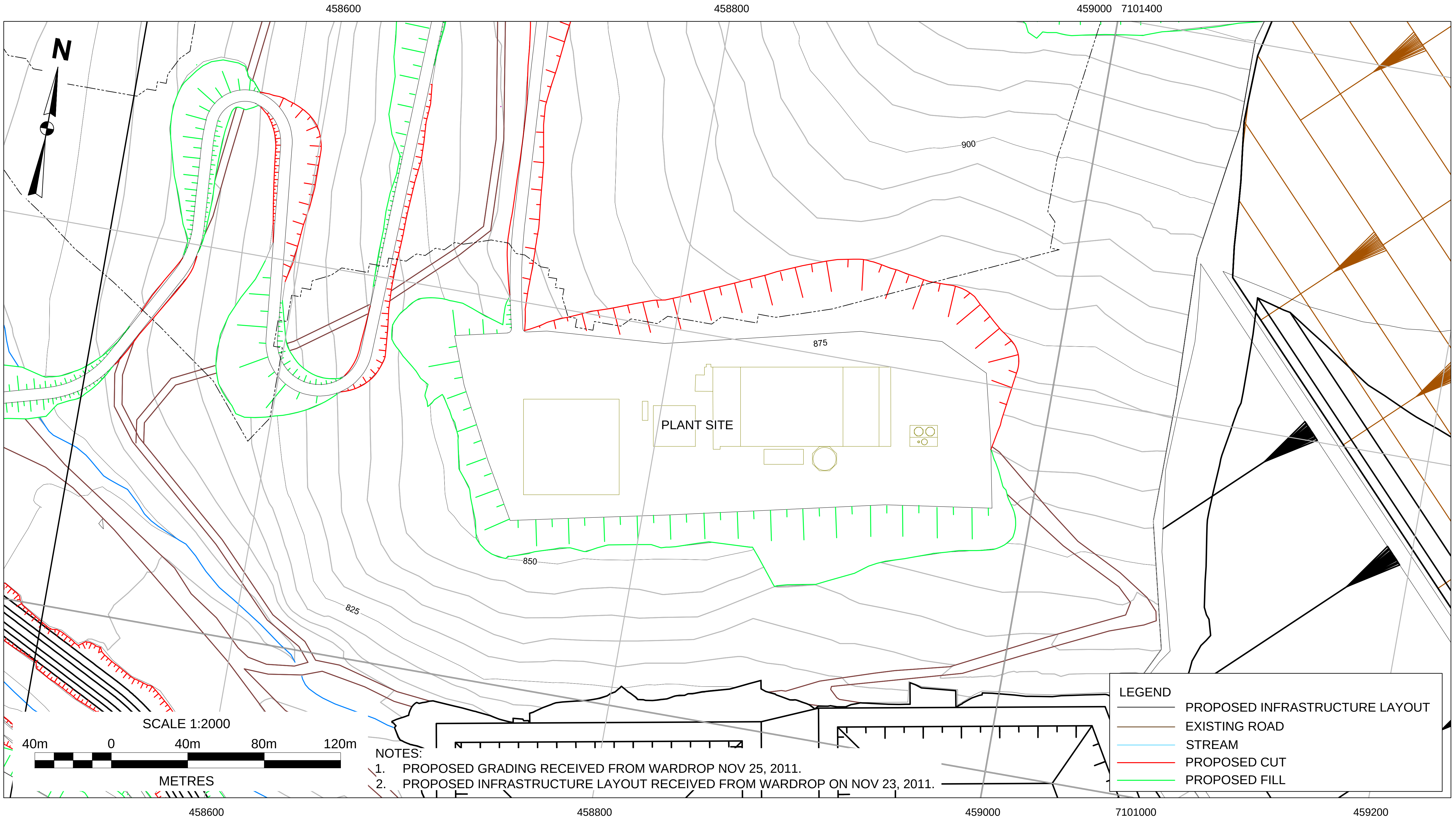

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DESIGNED:	KH
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APPROVED:	PQ

PROFESSIONAL SEAL:

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: PROPOSED GENERAL ARRANGEMENT PLANT SITE

PROJECT No.:	DWG No.:	REV.:
0792-006	05	0

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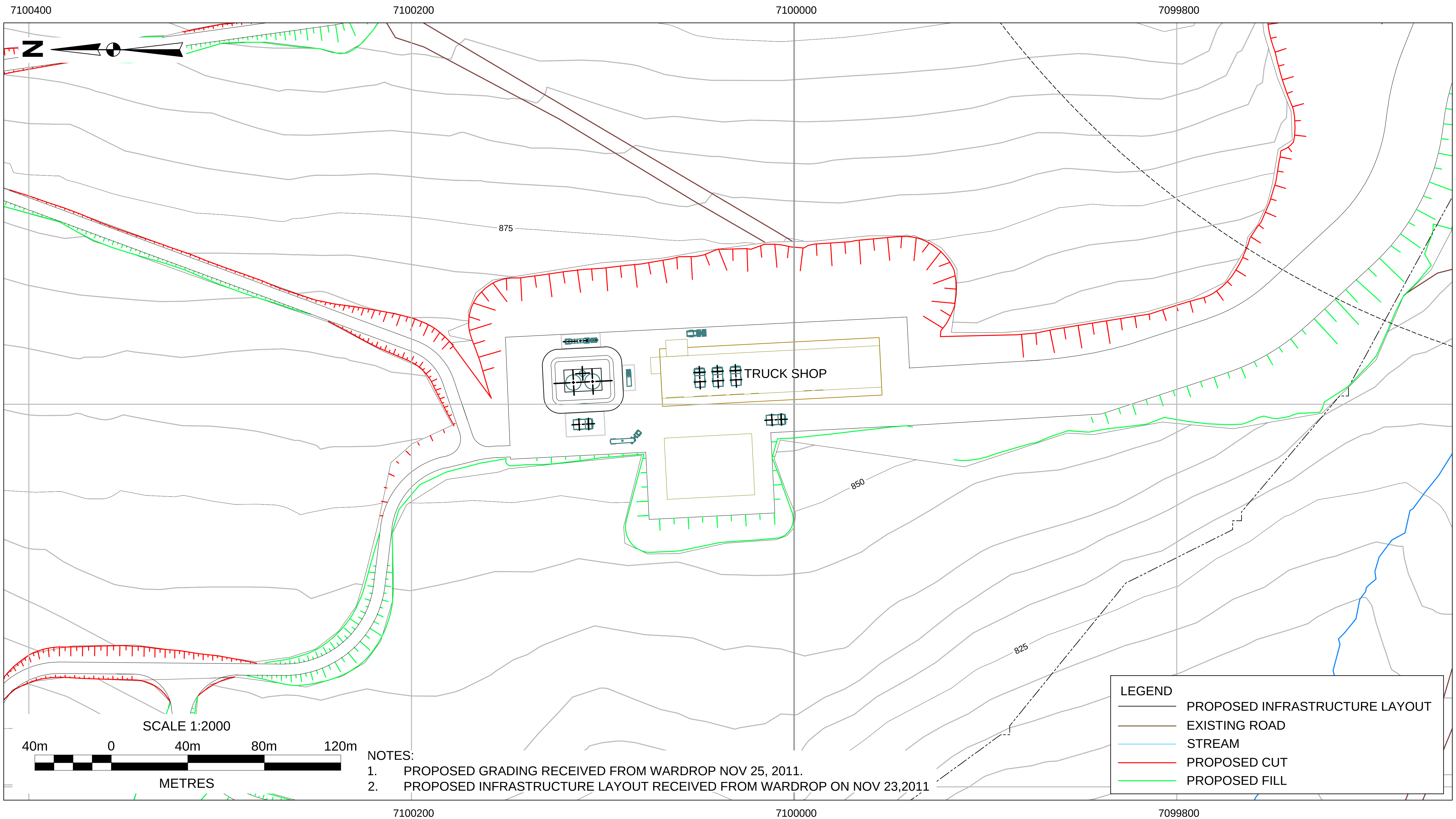

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APPROVED:	PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
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CLIENT:
VICTORIA GOLD CORPORATION

PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
PROPOSED GENERAL ARRANGEMENT
TRUCK SHOP

PROJECT No.:	DWG No.:	REV.:
0792-006	06	0

DWG TO BE READ WITH BGC REPORT TITLED " EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT " DATED JANUARY 2012

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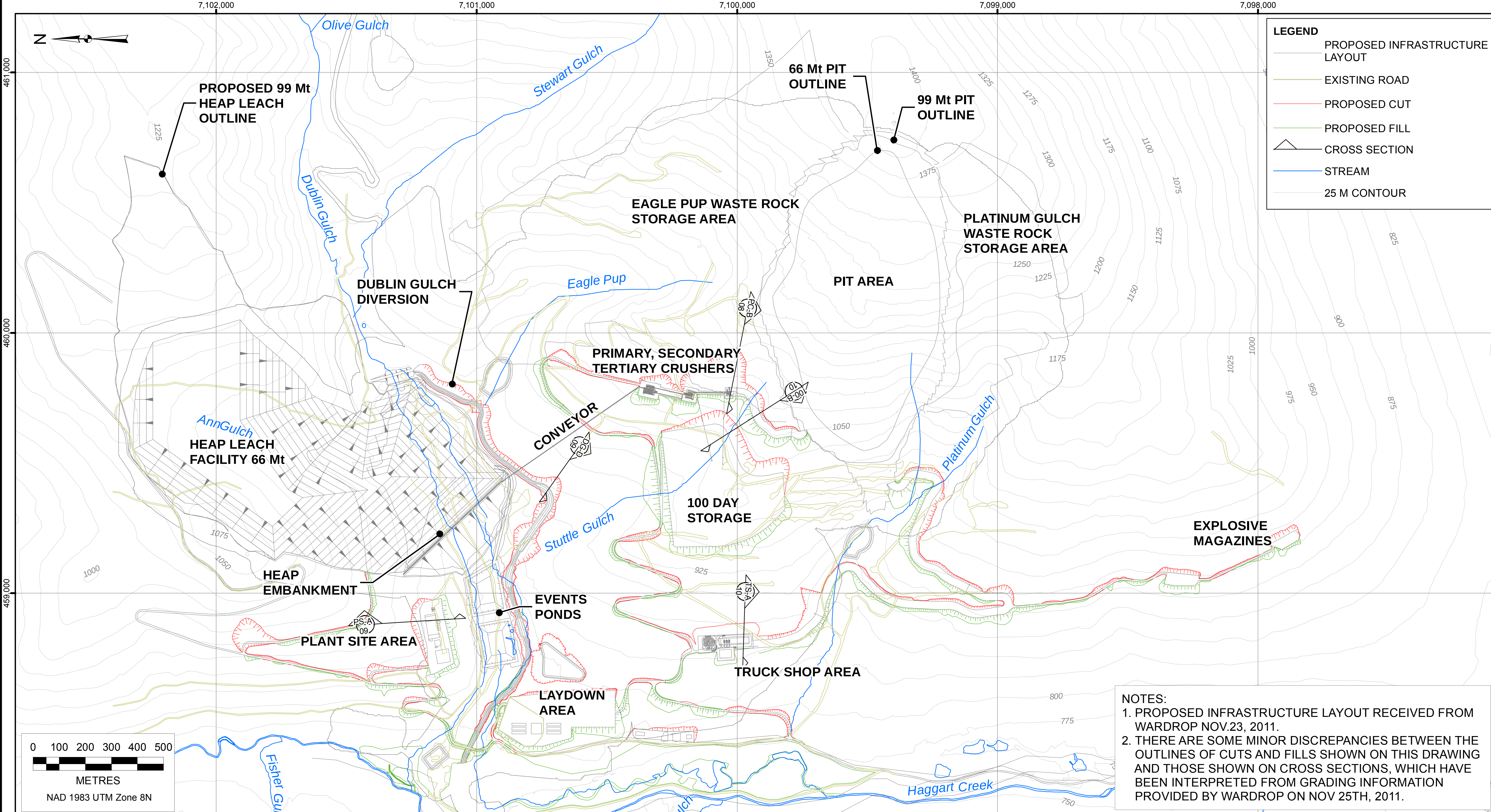

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DESIGNED: GH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

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CLIENT:
VICTORIA GOLD CORPORATION

PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
PLANNED CUT SLOPES

PROJECT No.: 0792-006
DWG No.: 07
REV.: 0

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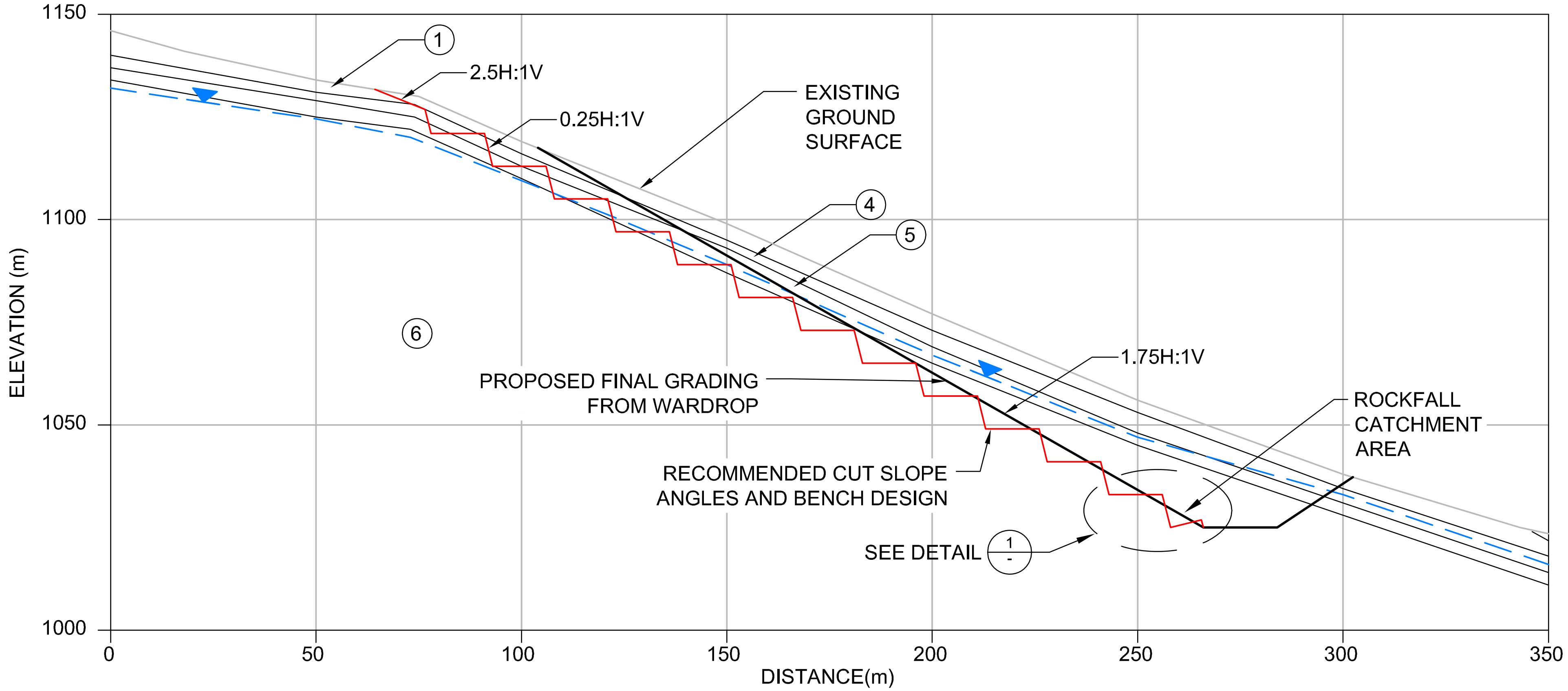
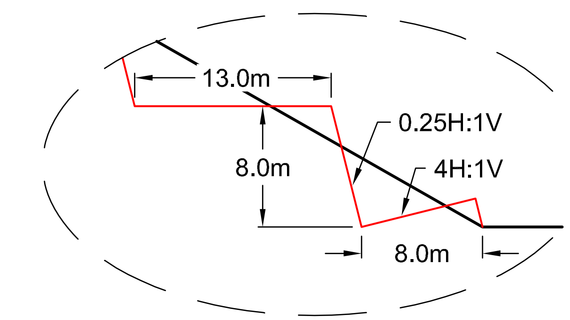


BENCH DESIGN
CROSS SECTION PC-B 07

RECOMMENDED SLOPE ANGLES AND BENCH DESIGN (SEE NOTE 4)



DETAIL 1 -
SCALE = 1:500

SCALE 1:1500
30m 0 30m 60m 90m
METRES

ASSUMED SUBSURFACE CONDITIONS

1	COLLUVIUM
4	TYPE 3 ROCK
5	TYPE 2 ROCK
6	TYPE 1 ROCK
	WATER LEVEL
	GROUND SURFACE
	PROPOSED FINAL GRADING RECEIVED FROM WARDROP
	RECOMMENDED CUT SLOPE ANGLES

NOTES:
1. PROPOSED FINAL GRADING PLAN RECEIVED FROM WARDROP ON NOV. 25, 2011.
2. INFERRED SUBSURFACE CONDITIONS ARE IDEALIZED FROM AVAILABLE TEST HOLE INFORMATION, AND MAY VARY FROM THAT SHOWN.
3. GROUNDWATER LEVELS ARE INFERRED FROM LIMITED INFORMATION AND CAN BE EXPECTED TO VARY FROM THE LINE SHOWN ON THIS CROSS-SECTION.
4. THE BENCH DESIGN NEEDS TO MAINTAIN THE RECOMMENDED OVERALL SLOPE ANGLE OF 1.75H:1V.

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SCALE:	AS SHOWN
DATE:	JAN 2012
DRAWN:	PB/GCB
DESIGNED:	EB
CHECKED:	DW
APPROVED:	PQ

PROFESSIONAL SEAL:

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AN APPLIED EARTH SCIENCES COMPANY

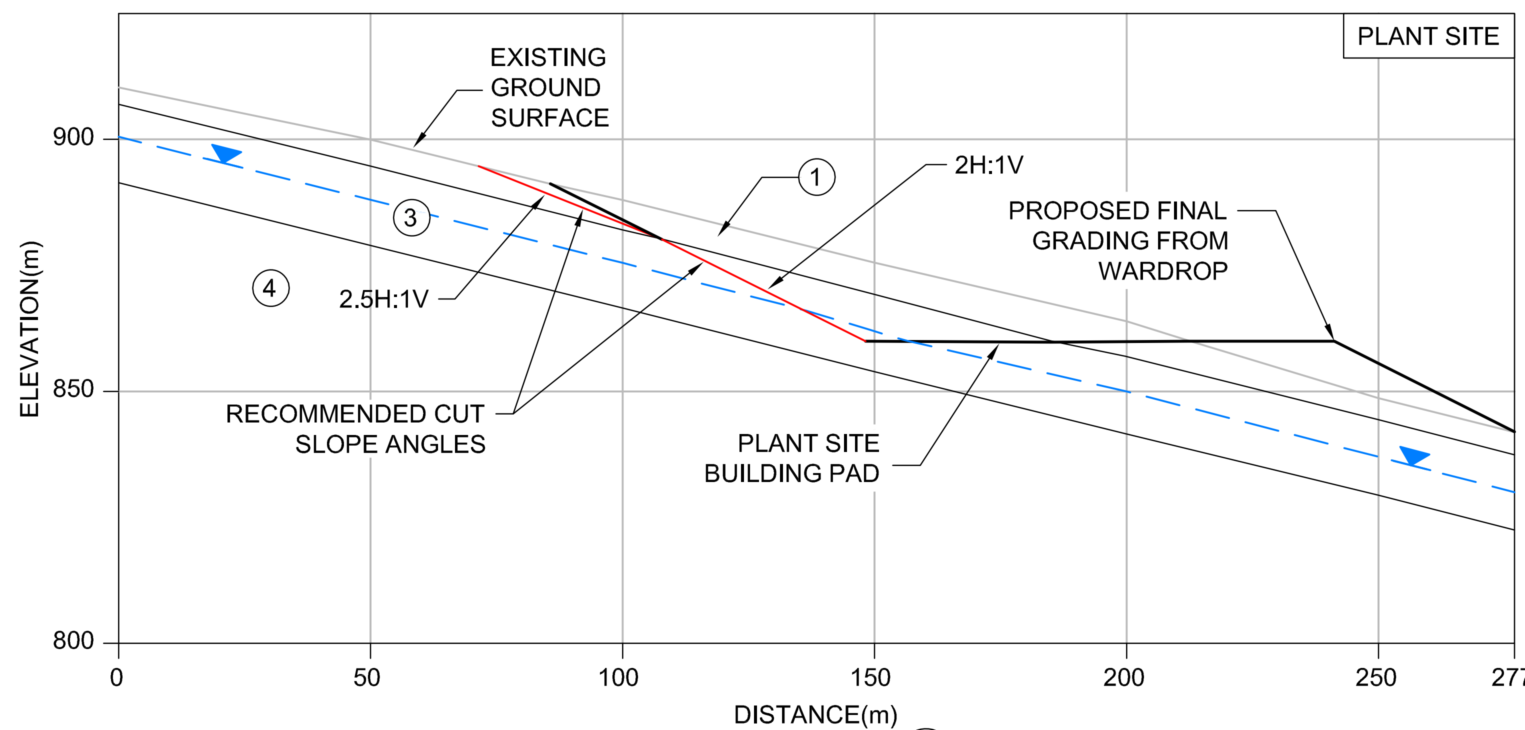
CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: CUT SLOPE CROSS-SECTION - PRIMARY CRUSHER

PROJECT No.:	DWG No.:	REV.:
0792006	08	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



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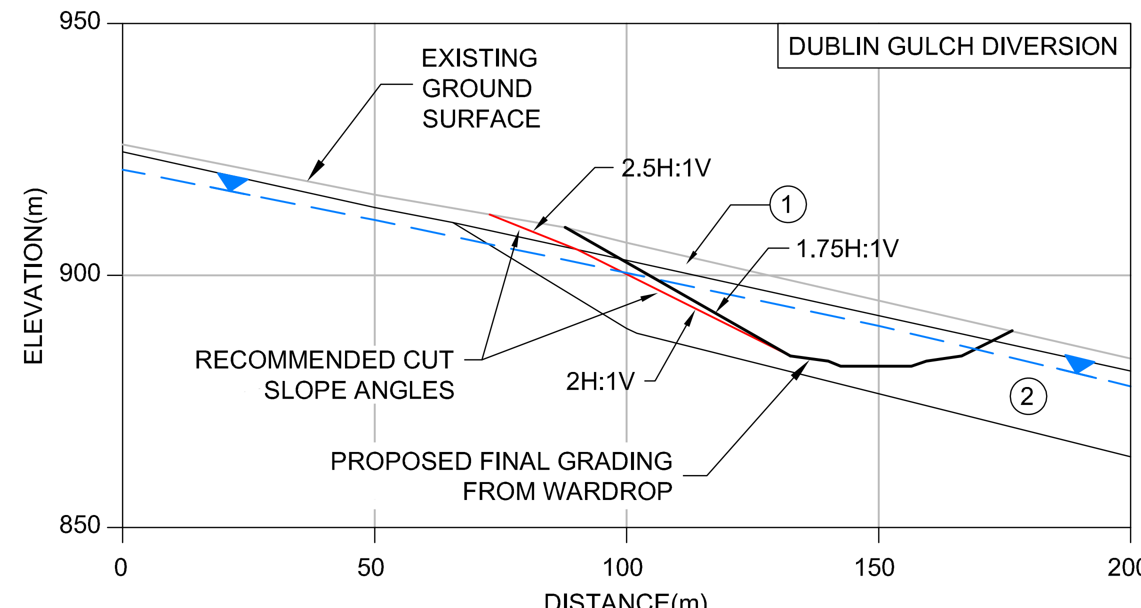

CROSS SECTION DG-D 07

NOTES:
1. PROPOSED FINAL GRADING PLAN RECEIVED FROM WARDROP ON NOV. 25, 2011.
2. INFERRED SUBSURFACE CONDITIONS ARE IDEALIZED FROM AVAILABLE TEST HOLE INFORMATION, AND MAY VARY FROM THAT SHOWN.
3. GROUNDWATER LEVELS ARE INFERRED FROM LIMITED INFORMATION AND CAN BE EXPECTED TO VARY FROM THE LINE SHOWN ON THIS CROSS-SECTION.

SCALE 1:1500

30m 0 30m 60m 90m

METRES

ASSUMED SUBSURFACE CONDITIONS

1 COLLUVIUM
4 TYPE 3 ROCK
5 TYPE 2 ROCK
6 TYPE 1 ROCK
WATER LEVEL
GROUND SURFACE
PROPOSED FINAL GRADING RECEIVED FROM WARDROP
RECOMMENDED CUT SLOPE ANGLES

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
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A	DEC 2011	DRAFT ISSUED FOR REVIEW	PB	DW	PQ

SCALE: 1:1500
DATE: JAN 2012
DRAWN: PB/GCB
DESIGNED: EB
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

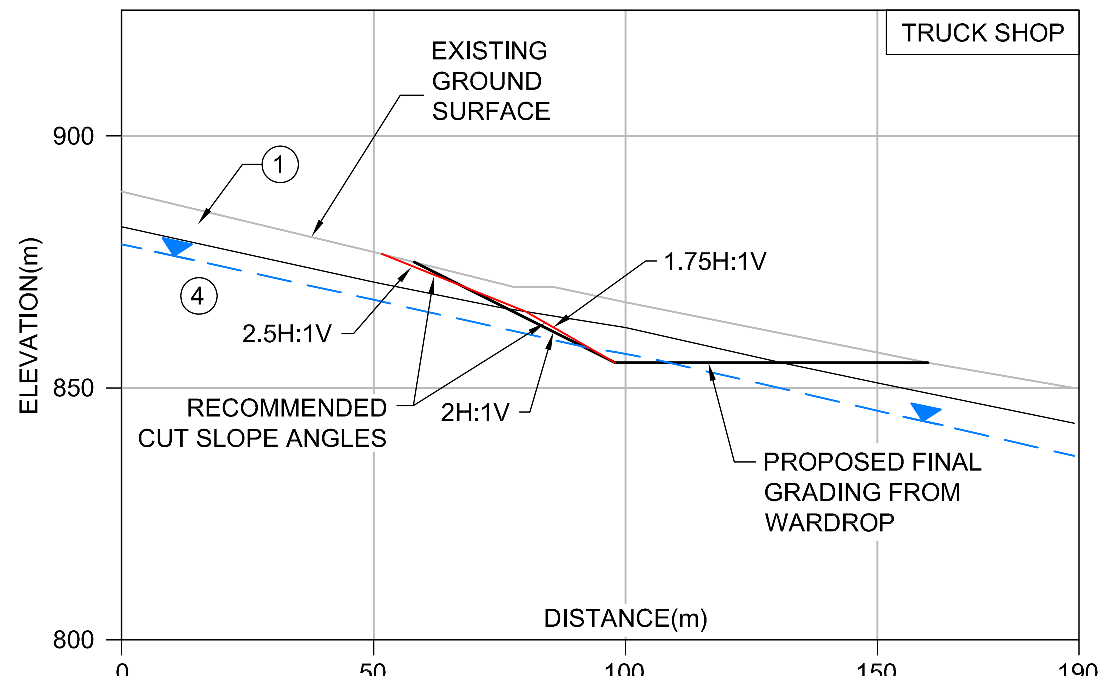
CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: CUT SLOPE CROSS-SECTIONS - PLANT SITE AND DUBLIN GULCH DIVERSION

PROJECT No.: 0792006
DWG No.: 09
REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



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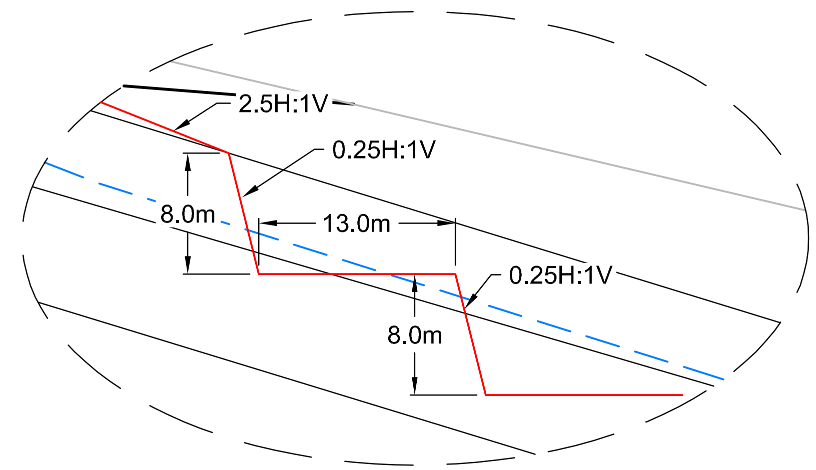

DETAIL 1 / -

SCALE = 1:500

NOTES:

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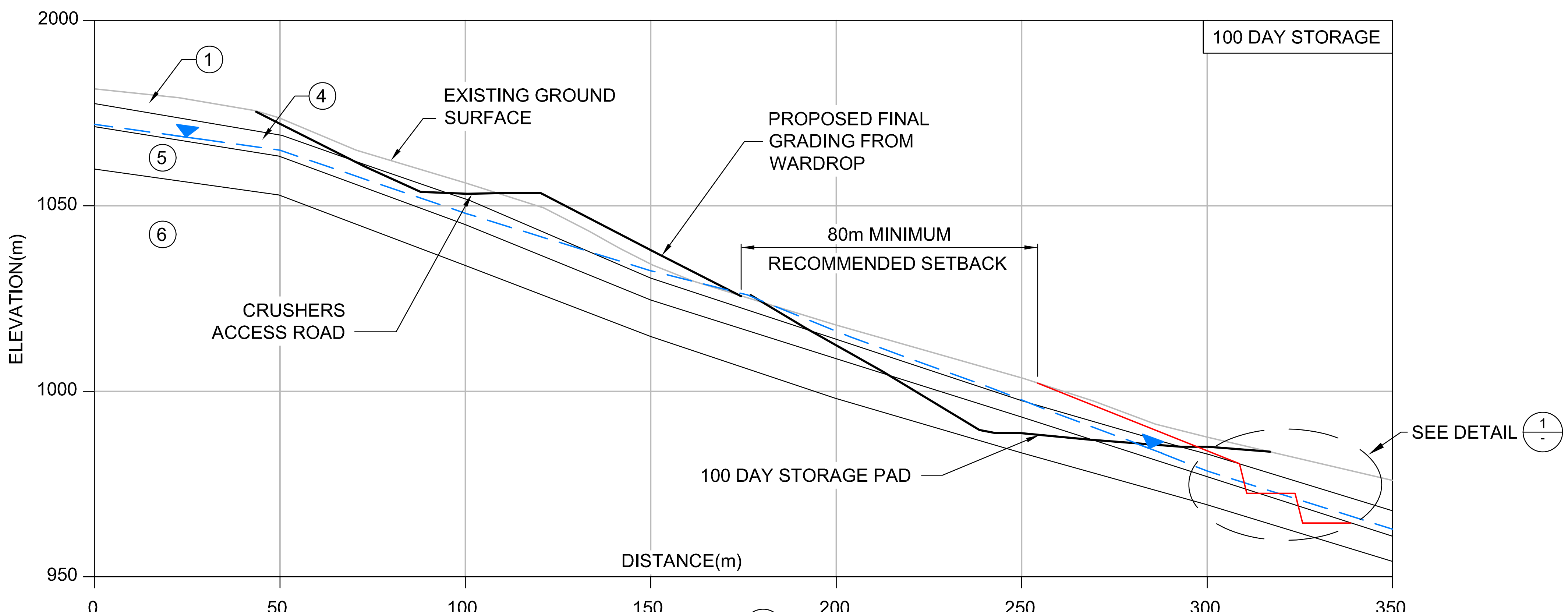
2. INFERRED SUBSURFACE CONDITIONS ARE IDEALIZED FROM AVAILABLE TEST HOLE INFORMATION, AND MAY VARY FROM THAT SHOWN.
3. GROUNDWATER LEVELS ARE INFERRED FROM LIMITED INFORMATION AND CAN BE EXPECTED TO VARY FROM THE LINE SHOWN ON THIS CROSS-SECTION.
4. THE BENCH DESIGN NEEDS TO MAINTAIN THE RECOMMENDED OVERALL SLOPE ANGLE OF 1.75H:1V.



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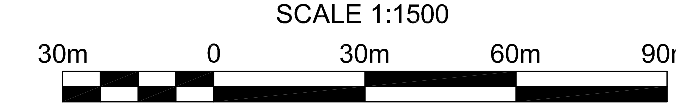

ASSUMED SUBSURFACE CONDITIONS

- 1 COLLUVIUM
- 4 TYPE 3 ROCK
- 5 TYPE 2 ROCK
- 6 TYPE 1 ROCK
- WATER LEVEL
- GROUND SURFACE
- PROPOSED FINAL GRADING RECEIVED FROM WARDROP
- RECOMMENDED CUT SLOPE ANGLES

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
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SCALE:	AS SHOWN
DATE:	JAN 2012
DRAWN:	PB/GCB
DESIGNED:	EB
CHECKED:	DW
APPROVED:	PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT- 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: CUT SLOPE CROSS-SECTIONS - TRUCK SHOP AND 100 DAY STORAGE

PROJECT No.:	DWG No.:	REV.:
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REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
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DESIGNED:	GH
CHECKED:	DW
APPROVED:	PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: AVAILABLE SUBSURFACE DATA

PROJECT No.:	DWG No.:	REV.:
0792-006	11	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE:	1:7,500
DATE:	JAN 2012
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DESIGNED:	GH
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APPROVED:	SQ

PROFESSIONAL SEAL:

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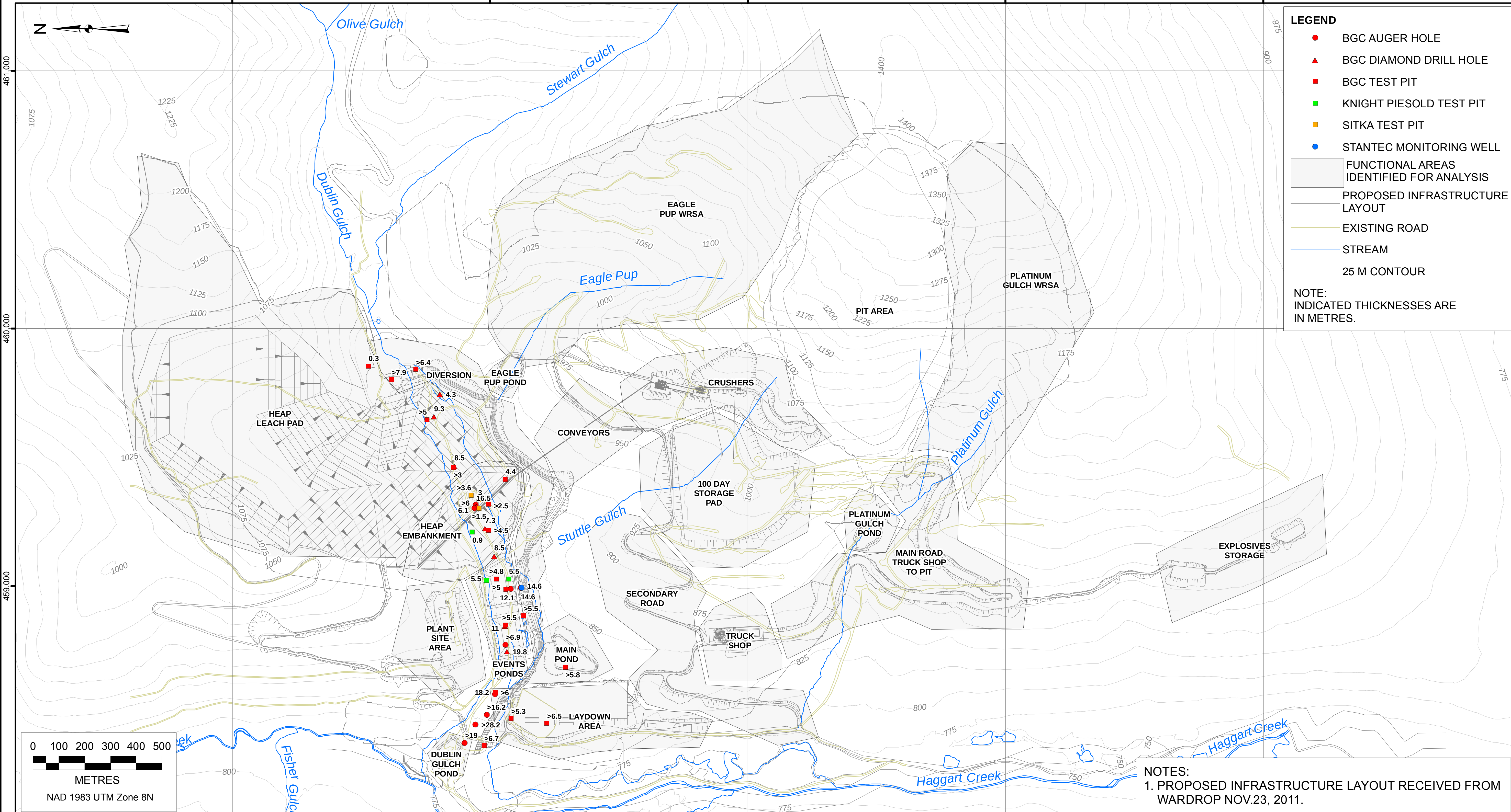
CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: OVERBURDEN THICKNESS

PROJECT No.:	DWG No.:	REV.:
0792-006	12	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: PLACER TAILINGS THICKNESS

PROJECT No.: 0792-006
DWG No.: 13
REV.: 0

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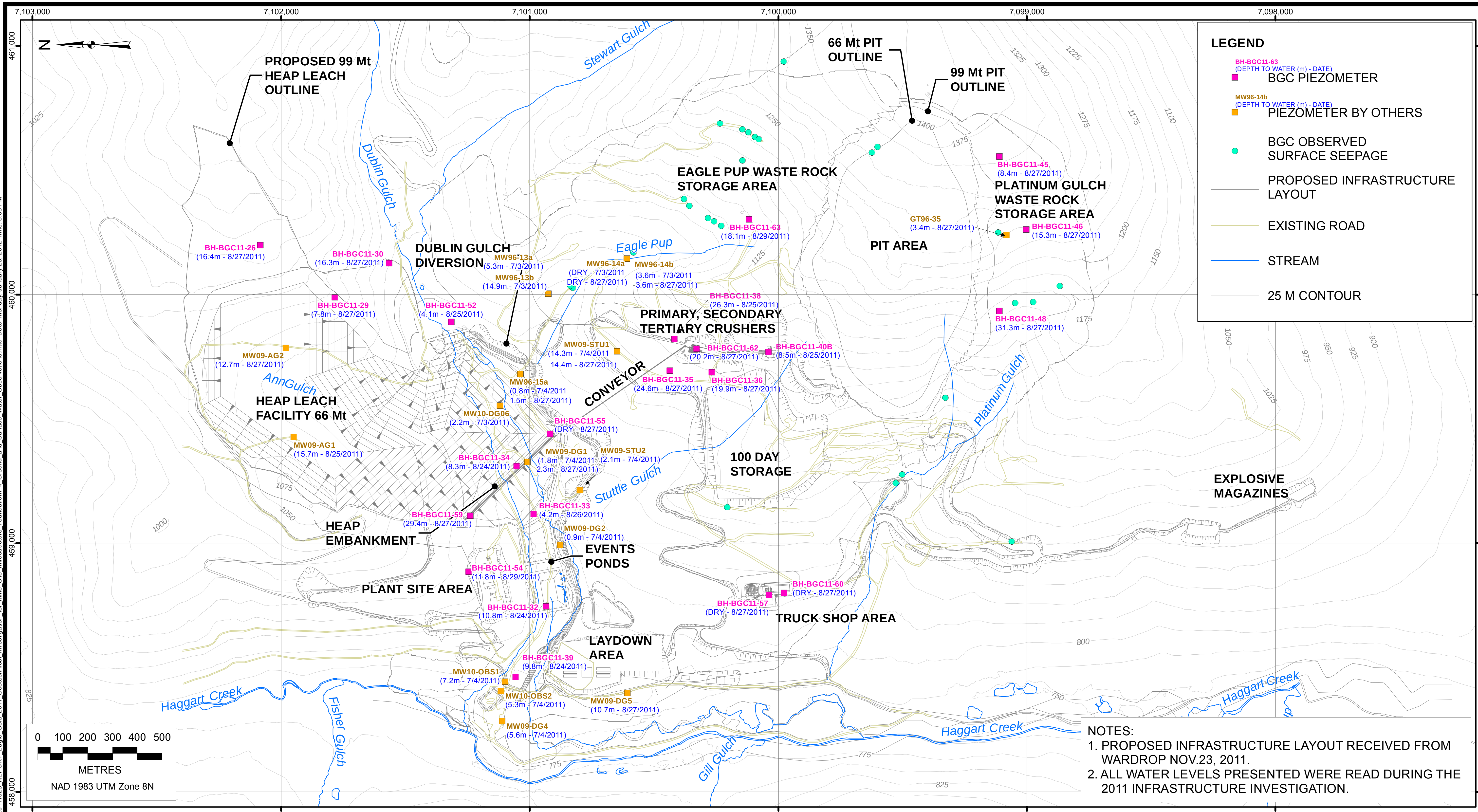

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REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
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APPROVED:	PQ

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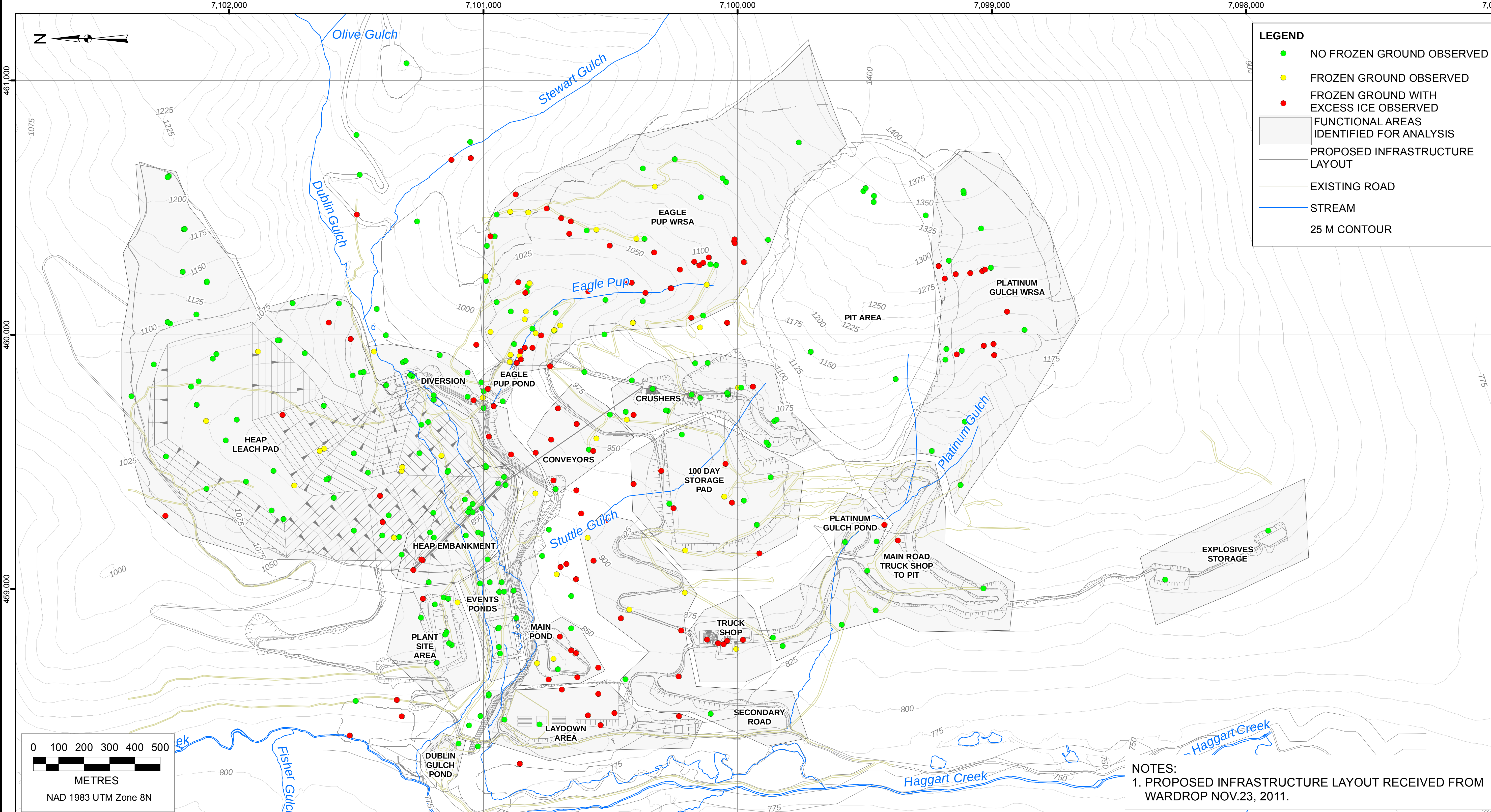
CLIENT:
VICTORIA GOLD CORPORATION

PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
GROUND AND SURFACE WATER OBSERVATIONS

PROJECT No.:	DWG No.:	REV.:
0792-006	14	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



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CLIENT:
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PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
FROZEN GROUND AS OBSERVED IN TEST HOLES

PROJECT No.:	DWG No.:	REV.:
0792-006	15	0

X:\Projects\0792 Victoria Gold\006\Workspace\20111028_REPORT_Eagle_Gold_2011_Geotechnical_Investigation_for_Mine_Site_Infrastructure_Foundation\15_Frozen_Ground_Observations.mxd Date: Monday, January 23, 2012 Time: 2:36 PM

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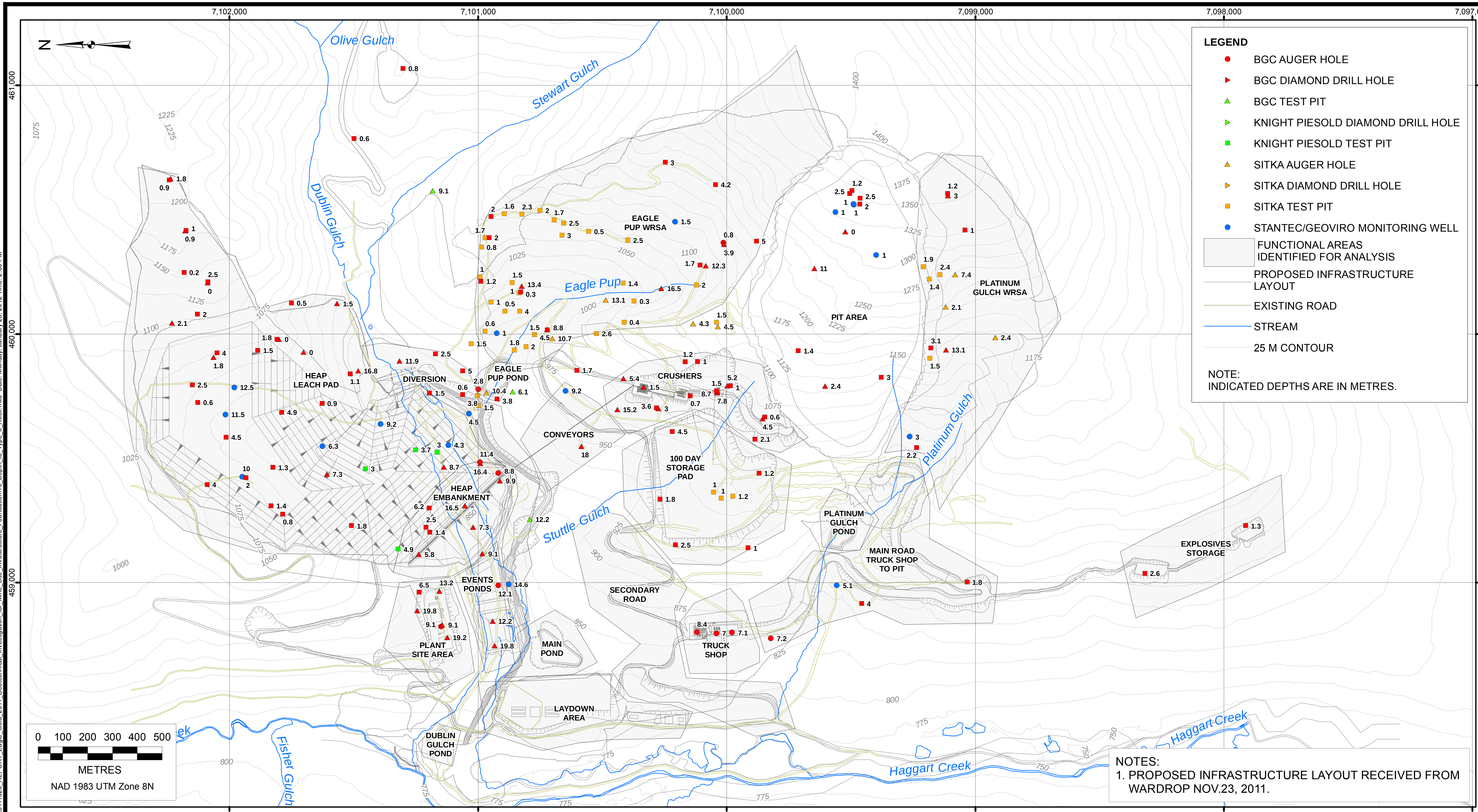

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: DEPTH TO TYPE 3 ROCK

PROJECT No.: 0792-006
DWG No.: 16
REV.: 0

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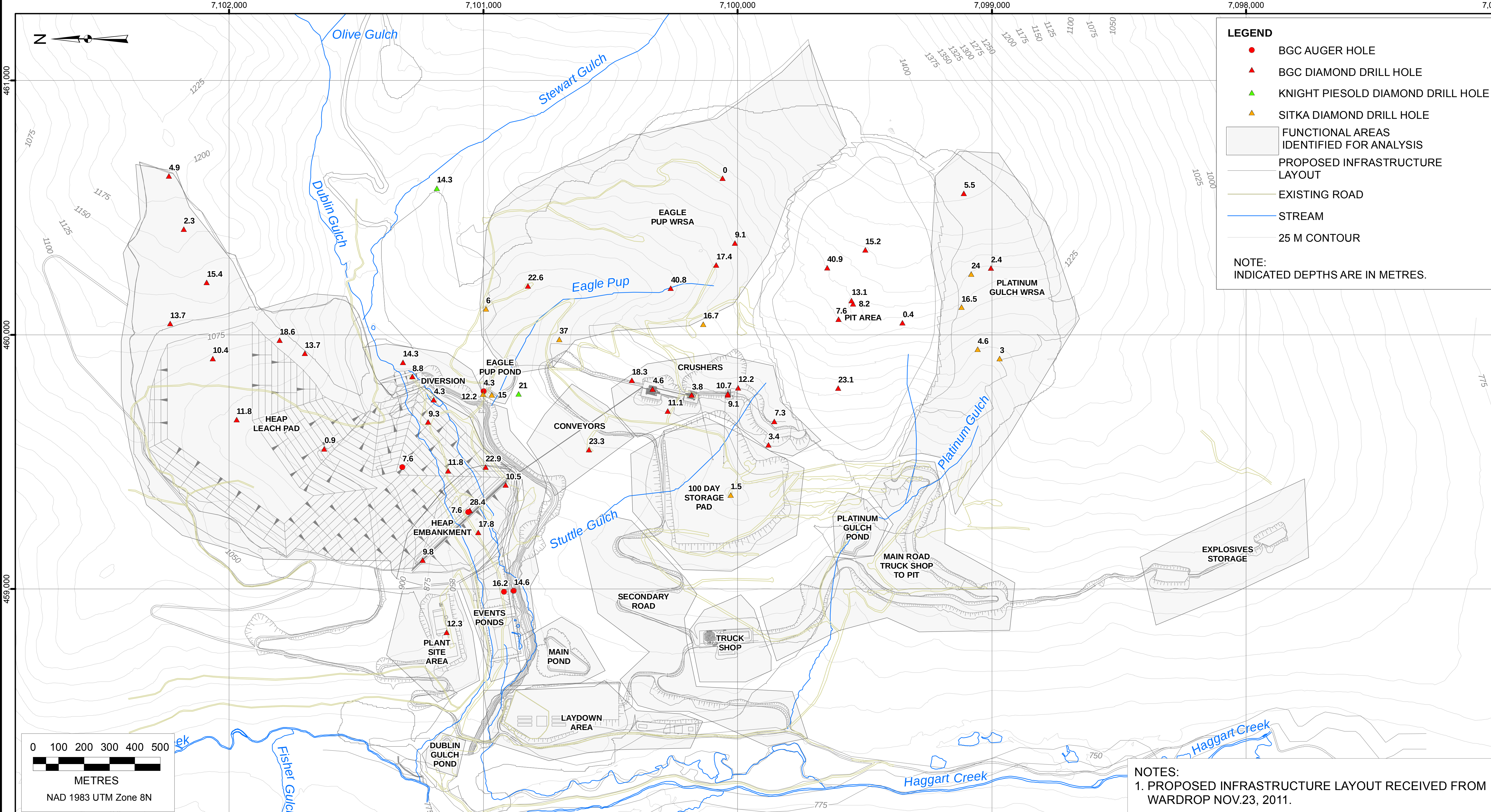

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: DEPTH TO TYPE 2 ROCK

PROJECT No.: 0792-006
DWG No.: 17
REV.: 0

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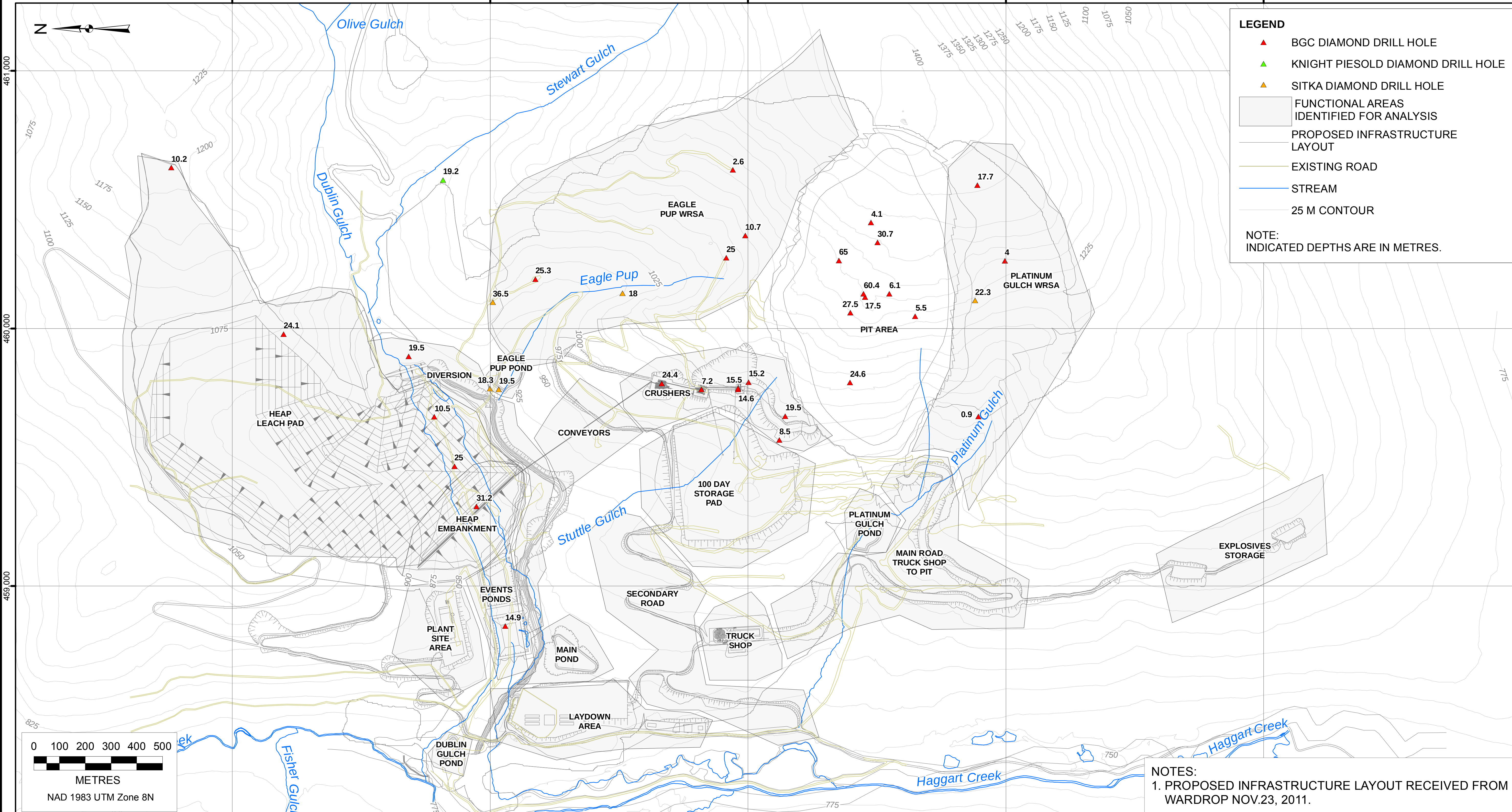

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: DEPTH TO TYPE 1 ROCK

PROJECT No.: 0792-006
DWG No.: 18
REV.: 0

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NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
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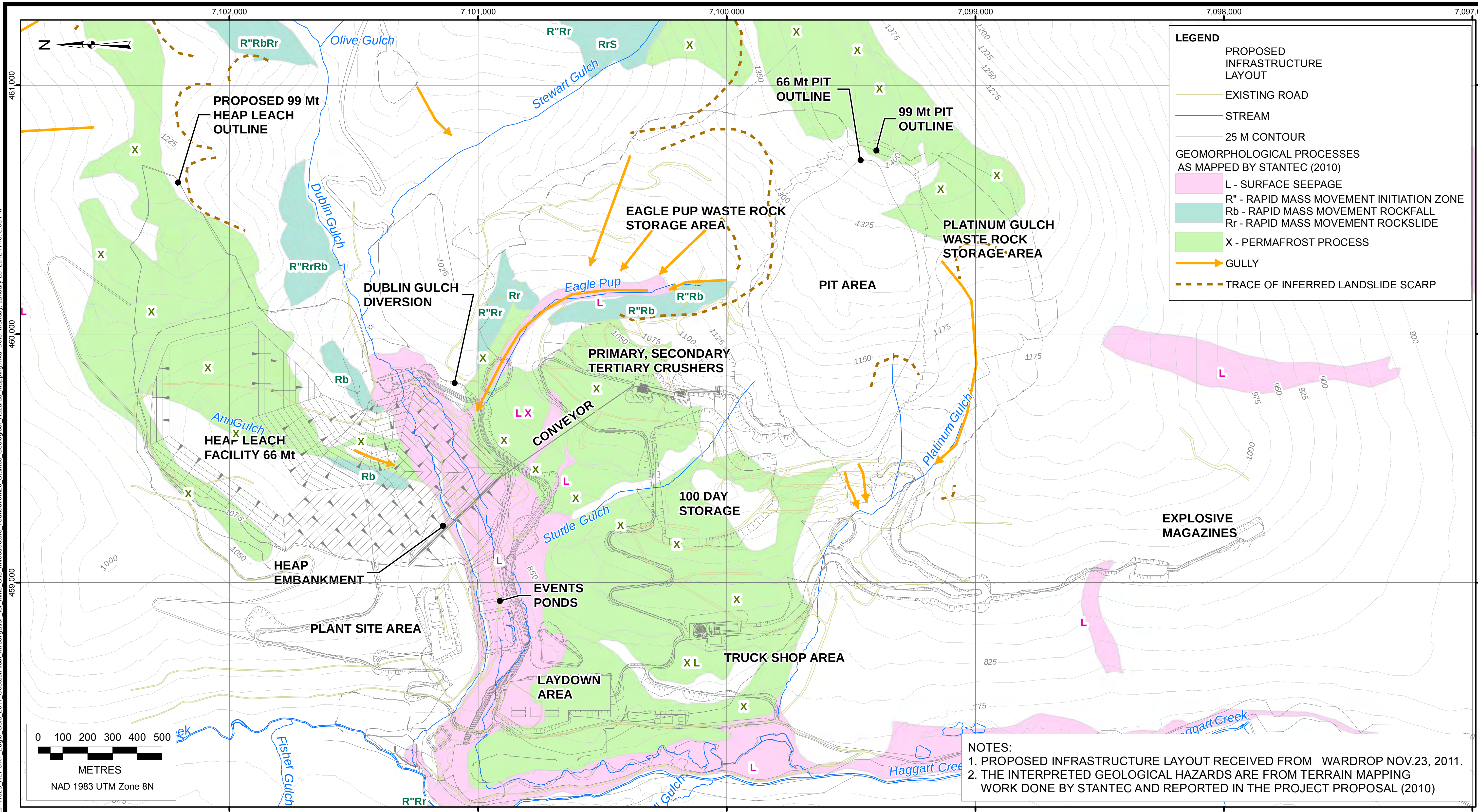
CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: STRUCTURAL DOMAINS

PROJECT No.: 0792-006 | DWG No.: 19 | REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
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CLIENT:
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PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE:
STANTEC GEOLOGICAL HAZARDS MAPPING

PROJECT No.:	DWG No.:	REV.:
0792-006	20	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

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PARAMETERS:

i = angle of back of wall from vertical

β = angle of backfill surface from horizontal

ϕ' = internal friction angle of backfill

δ = interface friction angle at back of wall:

  $2/3\phi'$ soil - concrete contact

  ϕ' soil - soil contact

γ = unit weight of backfill

k_h = horizontal seismic coefficient

k_v = vertical seismic coefficient

$\theta = \tan^{-1}[k_h/(1 - k_v)]$

H = height of backfill behind wall

Q_P = point load (KN)

Q_L = line load (KN/m)

q = surcharge load (KPa)

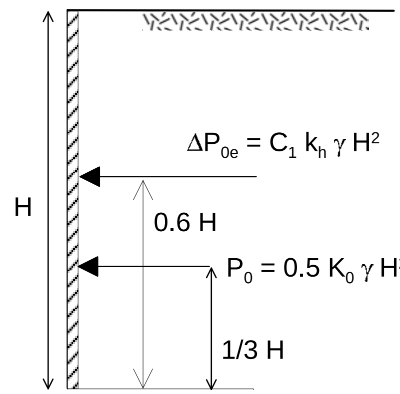
P = static earth load (KN/m)

K = applicable earth pressure coefficient (K_a, K_p, K_0)

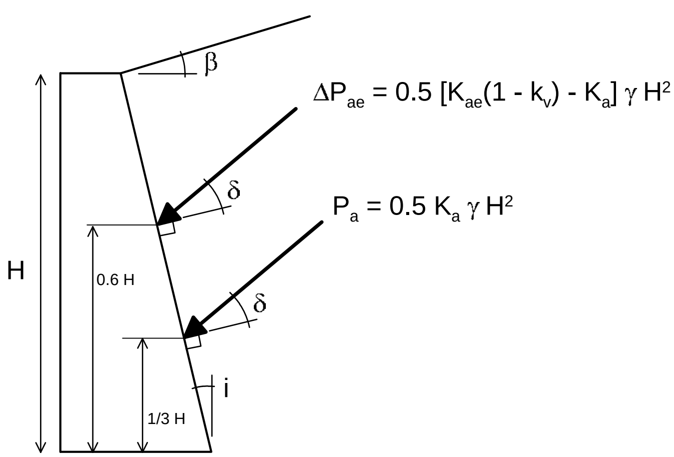
ΔP_e = incremental load induced by earthquake forces (KN/m)

K_e = earth pressure coefficient including effects of earthquake loading

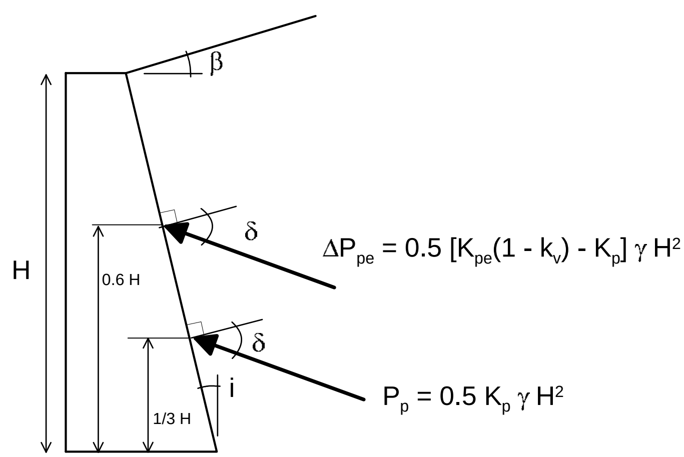
At Rest Case



Active Case



Passive Case



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DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT: VICTORIA GOLD CORP.

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

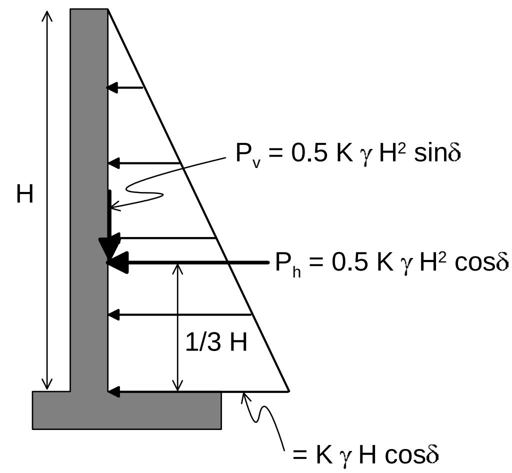
BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
BGC
Vancouver, BC
Phone: (604) 684 5900

TITLE: EARTH PRESSURE FOR RETAINING WALLS

PROJECT No.	DWG No.	REV.
0792-006	21	0

N:\BGC\Projects\0792 Victoria Gold\006 EG FS Infrastructure\05 Analysis and Design\Retaining Walls

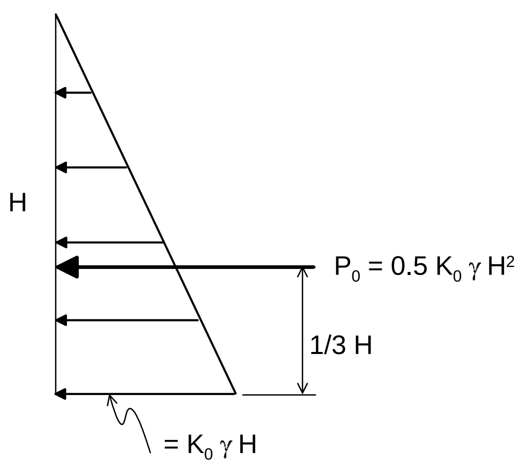
DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT", DATED JAN 2012



$$K_a = \frac{\cos^2(\phi' - i)}{\cos^2 i \cos(\delta + i)\left[1 + \sqrt{\dfrac{\sin(\phi' + \delta)\sin(\phi' - \beta)}{\cos(\delta + i)\cos(\beta - i)}}\right]^2}$$

$$K_p = \frac{\cos^2(\phi' + i)}{\cos^2 i \cos(\delta - i)\left[1 - \sqrt{\dfrac{\sin(\phi' + \delta)\sin(\phi' + \beta)}{\cos(\delta - i)\cos(\beta - i)}}\right]^2}$$

(a) Unrestrained



$$K_0 = 1 - \sin\phi'$$

(b) Restrained

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT:
VICTORIA GOLD CORP.

PROJECT:
EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

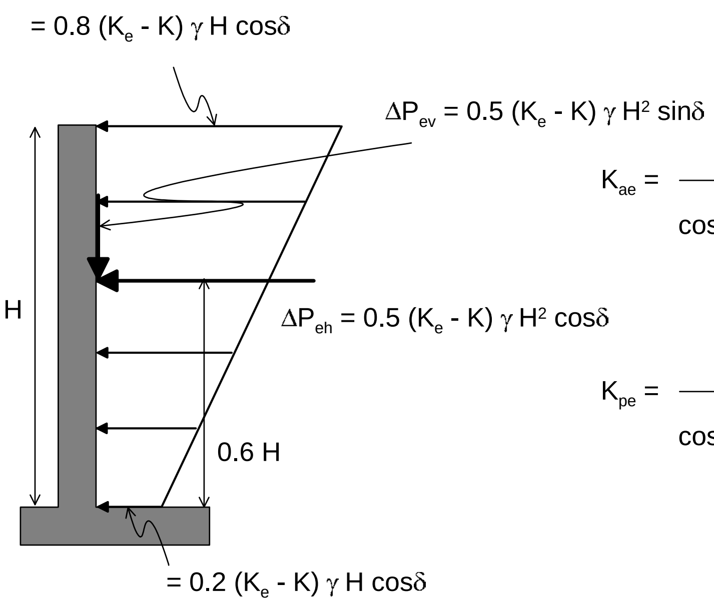
BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC
Phone: (604) 684 5900

TITLE:
EARTH PRESSURE DISTRIBUTIONS FOR UNRESTRAINED AND RESTRAINED WALLS, STATIC CASE

PROJECT No.	DWG No.	REV.
0792-006	22	0

N:\BGC\Projects\0792 Victoria Gold\006 EG FS Infrastructure\05 Analysis and Design\Retaining Walls

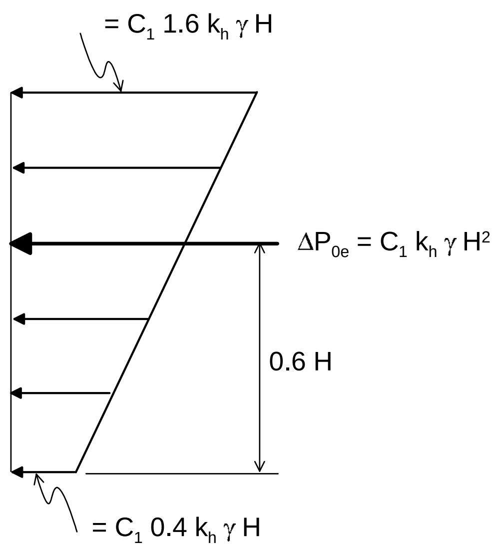
DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT", DATED JAN 2012



$$K_{ae} = \frac{\cos^2(\phi' - \theta - i)}{\cos\theta\,\cos^2 i\,\cos(\delta + i + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi' + \delta)\,\sin(\phi' - \theta - \beta)}{\cos(\delta + i + \theta)\,\cos(\beta - i)}}\right]^2}$$

$$K_{pe} = \frac{\cos^2(\phi' + i - \theta)}{\cos\theta\,\cos^2 i\,\cos(\delta - i + \theta)\left[1 - \sqrt{\dfrac{\sin(\phi' + \delta)\,\sin(\phi' - \theta + \beta)}{\cos(\delta - i + \theta)\,\cos(\beta - i)}}\right]^2}$$

(a) Unrestrained



NOTE:
- Refer to Drawing 24 to estimate the dynamic amplification factor, C_1.

(b) Restrained

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT: VICTORIA GOLD CORP.

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

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AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC
Phone: (604) 684 5900

TITLE: EARTH PRESSURE DISTRIBUTIONS FOR UNRESTRAINED AND RESTRAINED WALLS, DYNAMIC

PROJECT No.	DWG No.	REV.
0792-006	23	0

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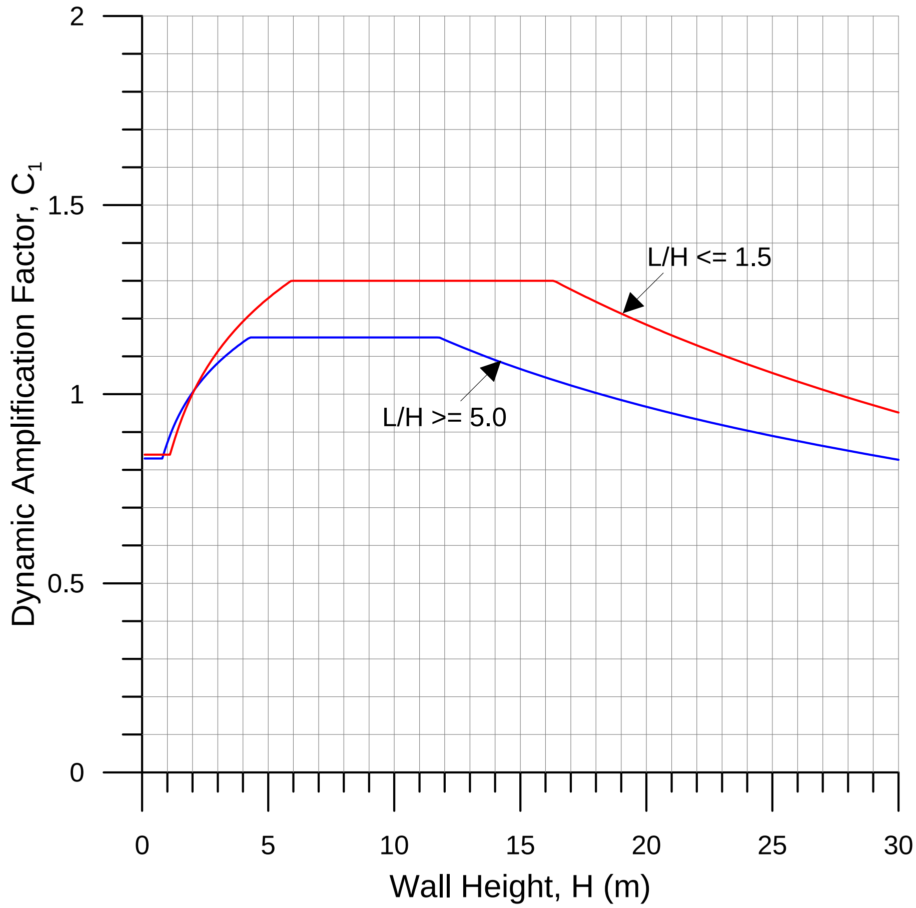

NOTES:
- Based on 84th percentile curves presented by Wu and Finn (1999).
- Assumptions:
    - Backfill is structural fill compacted to 100% SPMDD
    - unit weight of backfill, γ = 20 kN/m^3
    - fundamental frequency of backfill ranges from initial fundamental frequency to degraded frequency equal to the initial fundamental frequency divided by 1.5 (to account for possible strain-softening effects)
    - fundamental frequency of ground motion, f = 5 Hz
- L = average horizontal distance from back of wall to rock
- Use linear interpolation if 1.5 < L/H < 5.0.

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT: VICTORIA GOLD CORP.

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

BGC ENGINEERING INC.
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TITLE: DYNAMIC EARTH PRESSURE AMPLIFICATION FACTORS, C_1, FOR RESTRAINED WALLS

PROJECT No.	DWG No.	REV.
0792-006	24	0

N:\BGC\Projects\0792 Victoria Gold\006 EG FS Infrastructure\05 Analysis and Design\Retaining Walls

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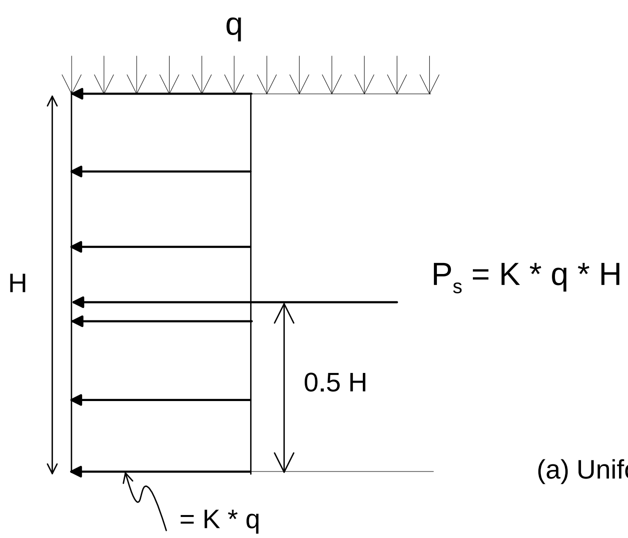
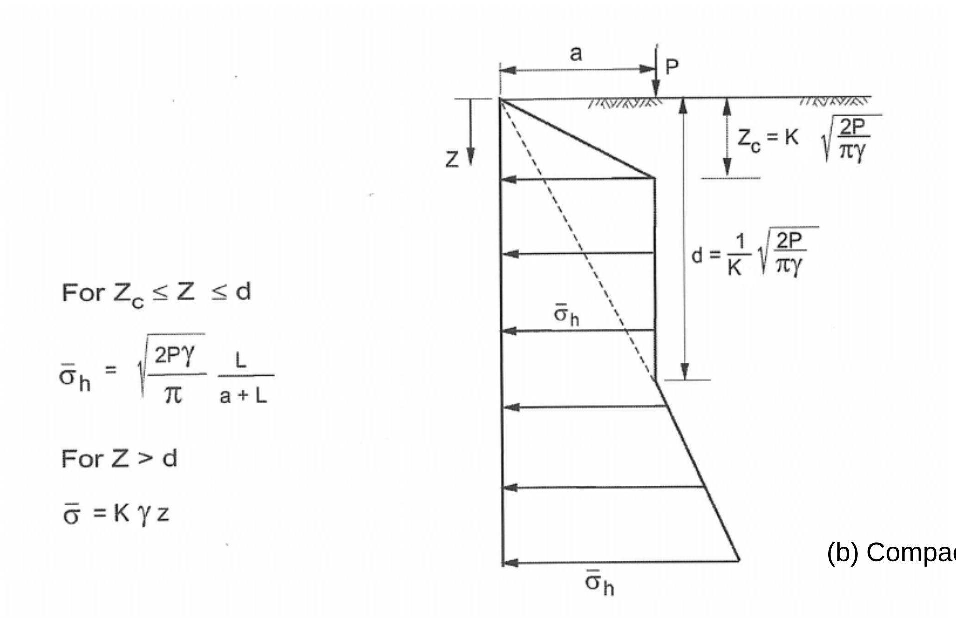


(a) Uniform surcharge



(b) Compaction surcharge

$$P \text{ (roller load)} = \frac{\text{dead weight of roller + centrifugal force}}{\text{width of roller}}$$

a = distance of roller from wall

L = length of roller

Source: From Canadian Foundation Engineering Manual 4th Edition, CGS, 2006

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT: VICTORIA GOLD CORP.

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
BGC
Vancouver, BC
Phone: (604) 684 5900

TITLE: EARTH PRESSURE DISTRIBUTION DUE TO UNIFORM LOAD SURCHARGE AND COMPACTION

PROJECT No.	DWG No.	REV.
0792-006	25	0

N:\BGC\Projects\0792 Victoria Gold\006 EG FS Infrastructure\05 Analysis and Design\Retaining Walls

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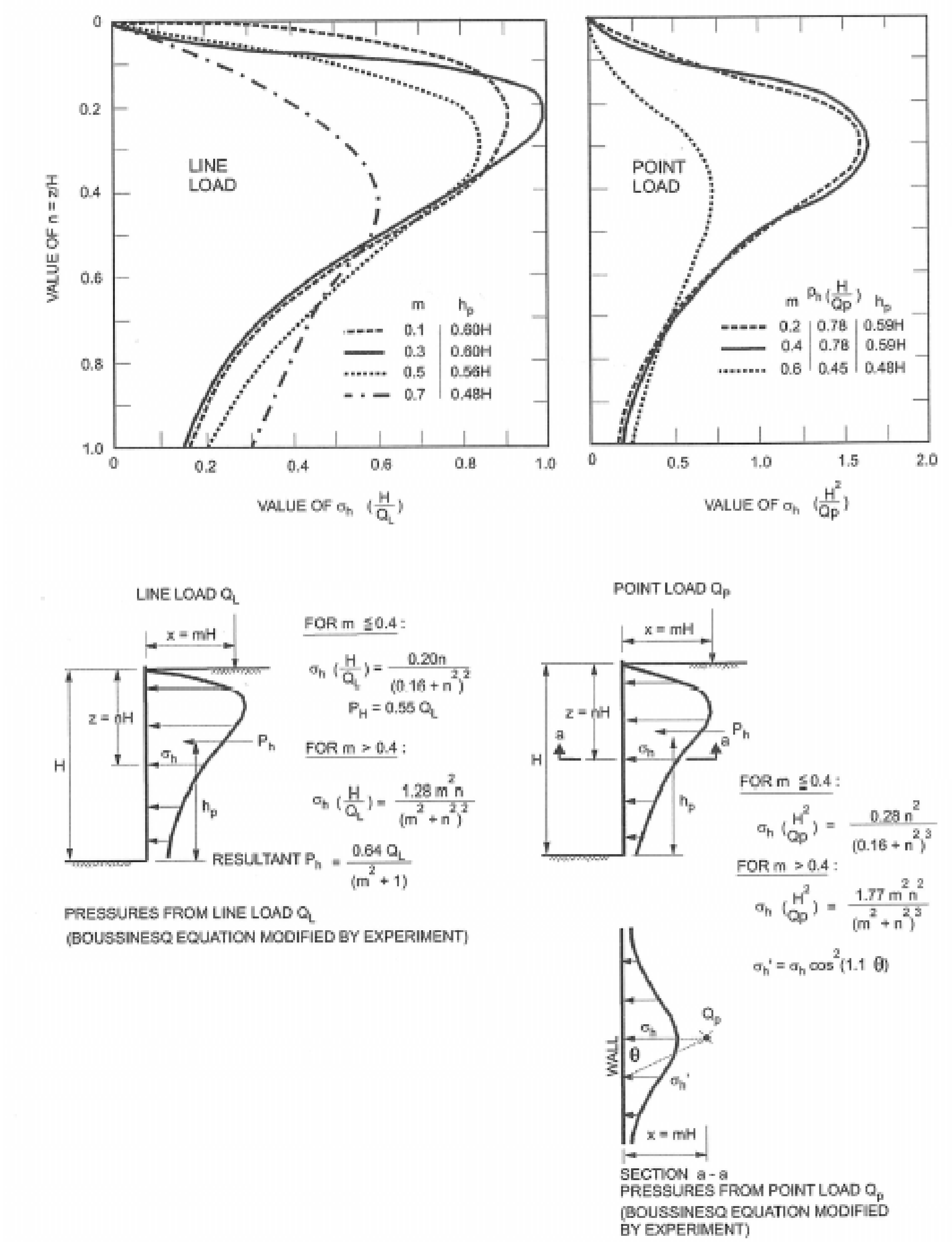

Source: From Canadian Foundation Engineering Manual 4th Edition, CGS, 2006

SCALE:	NTS	DESIGN:	DW
DATE:	JAN 2012	CHECKED:	DW
DRAWN:	DW	APPROVED:	PQ

CLIENT: VICTORIA GOLD CORP.

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Vancouver, BC
Phone: (604) 684 5900

TITLE: EARTH PRESSURE DISTRIBUTION DUE TO POINT AND LINE LOAD SURCHARGES

PROJECT No.	DWG No.	REV.
0792-006	26	0

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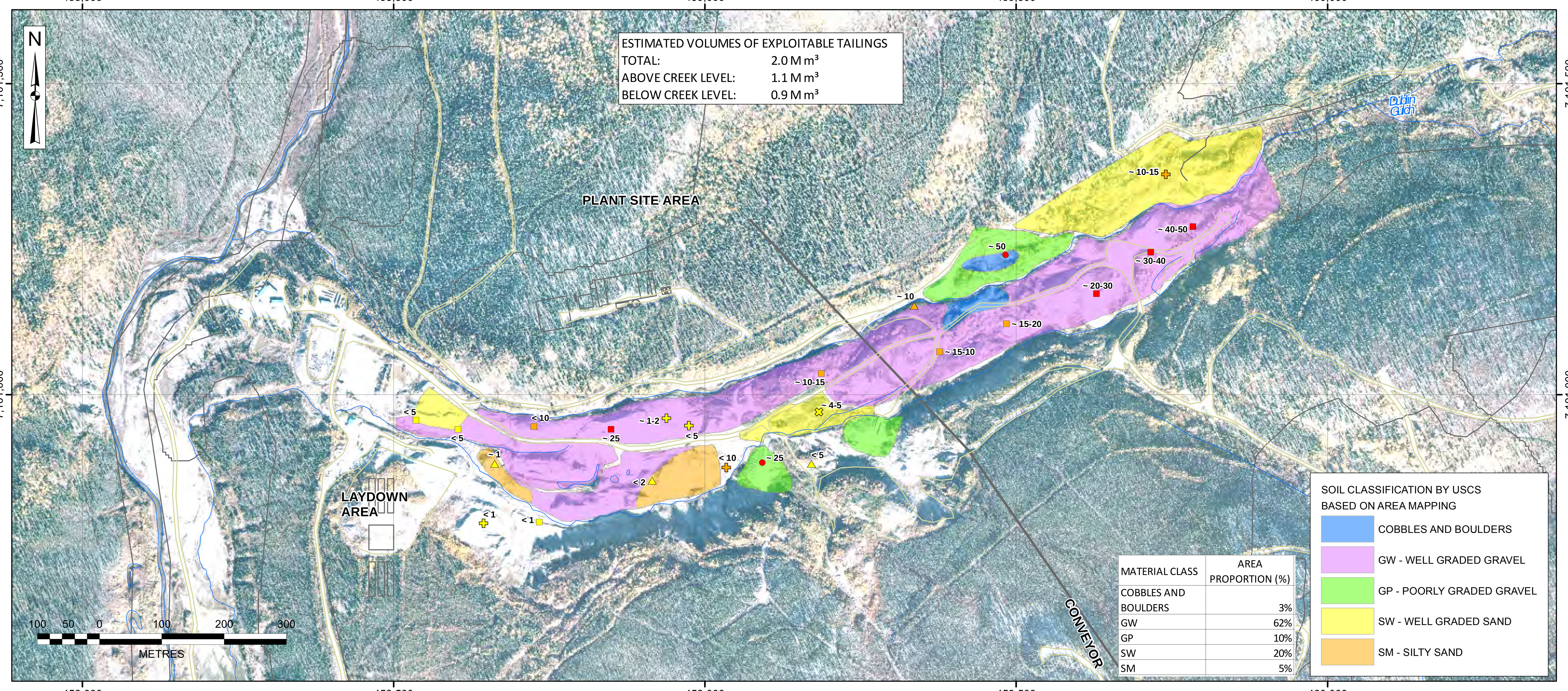
DUBLIN GULCH



ESTIMATED VOLUMES OF EXPLOITABLE TAILINGS	
TOTAL:	2.0 M m³
ABOVE CREEK LEVEL:	1.1 M m³
BELOW CREEK LEVEL:	0.9 M m³

MATERIAL CLASS	AREA PROPORTION (%)
COBBLES AND BOULDERS	3%
GW	62%
GP	10%
SW	20%
SM	5%

SOIL CLASSIFICATION BY USCS BASED ON AREA MAPPING
- COBBLES AND BOULDERS
- GW - WELL GRADED GRAVEL
- GP - POORLY GRADED GRAVEL
- SW - WELL GRADED SAND
- SM - SILTY SAND

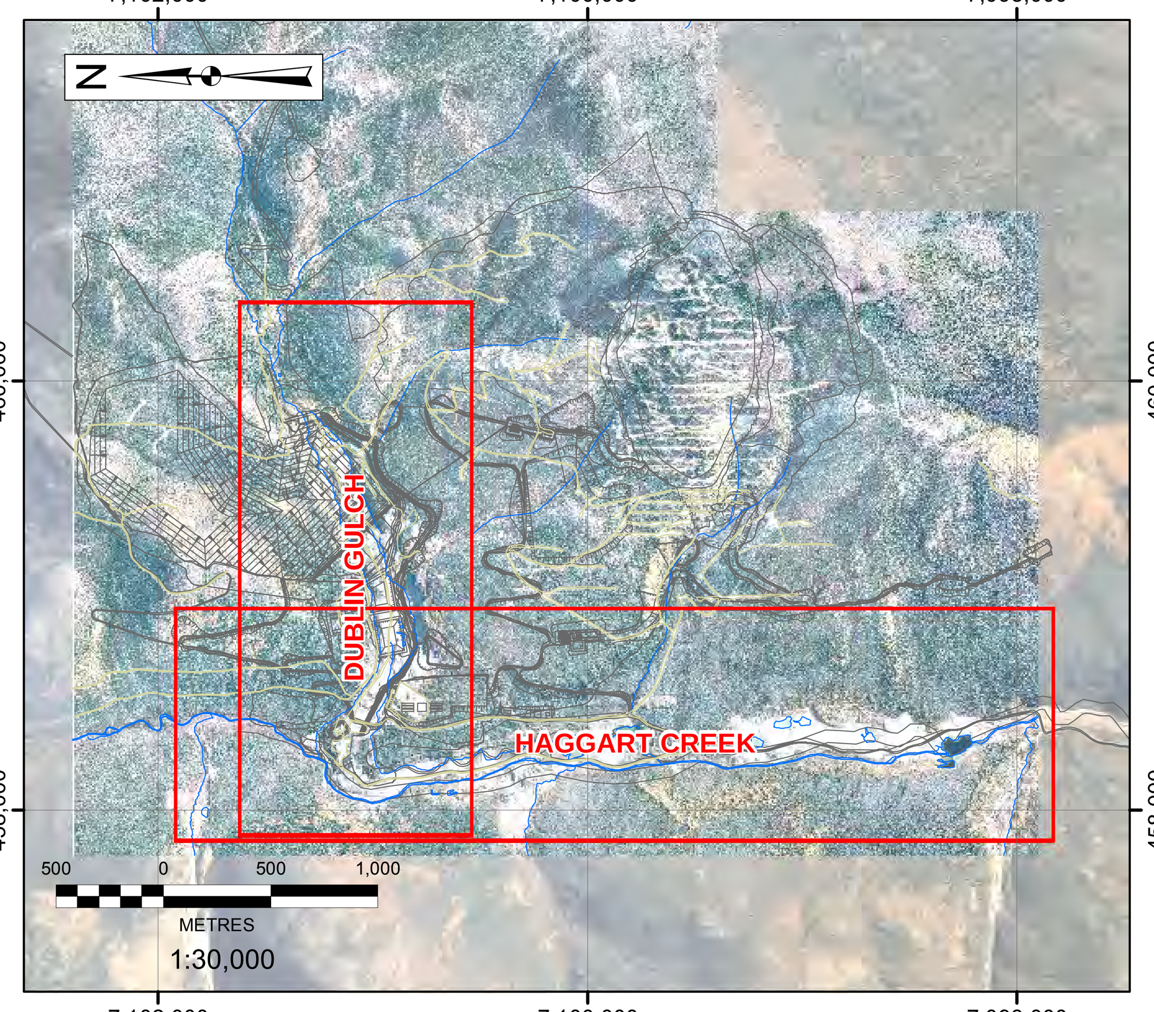
LEGEND
- PROPOSED INFRASTRUCTURE LAYOUT
- EXISTING ROAD

SOIL CLASSIFICATION BY UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) BASED ON ISOLATED SAMPLING
- GP - POORLY GRADED GRAVEL
- GW - WELL GRADED GRAVEL
- SM - SILTY SAND
- SW - WELL GRADED SAND
- NA - NON-SUITABLE MATERIAL

ESTIMATED % OF OVERSIZE PARTICLES (>75 MM)
- N/A
- 0-5 %
- 5-20 %
- >20 %

EXAMPLE:
< 2
WELL GRADED SAND, <2% OF MATERIAL IS GREATER THAN 75MM.

OVERVIEW MAP



HAGGART CREEK

PILE NAME	MATERIAL DESCRIPTION (USCS)	VOLUME (M³)
A	GW-SW - well graded sand and gravel	144,000
B	GW - well graded gravel	91,000
C	GW-GP-SW - sand and gravel,well graded and poorly graded	126,000
D	GW - well graded gravel	44,000
E	GW-SW - well graded sand and gravel	76,000
F	SW - well graded sand	150,000
G	GW - well graded sand	19,000
H	GW - well graded sand	101,000
TOTAL		751,000

NAD 1983 UTM Zone 8N

NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.
2. GRAIN SIZE OBSERVATIONS ARE FROM VISUAL OBSERVATION ONLY.
3. VOLUME ESTIMATES FOR DUBLIN GULCH PLACER TAILINGS BASED ON TEST HOLE AND GEOPHYSICAL OBSERVATIONS.
4. VOLUME ESTIMATES FOR HAGGART CREEK PLACER TAILINGS ARE INFERRED FROM SURFACE OBSERVATIONS AND TOPOGRAPHY. NO SUBSURFACE DATA ARE AVAILABLE IN THIS AREA.
5. IMAGERY PROVIDED BY VICTORIA GOLD CORPORATION.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.
0	JAN 2012	ISSUED WITH FINAL REPORT	LL	DW	PQ
A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE:	1:5,000
DATE:	JAN 2012
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DESIGNED:	LT
CHECKED:	DW
APPROVED:	PQ

PROFESSIONAL SEAL:

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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: BORROW MATERIALS PLACER TAILINGS

PROJECT No.:	DWG No.:	REV.:
0792-006	27	0

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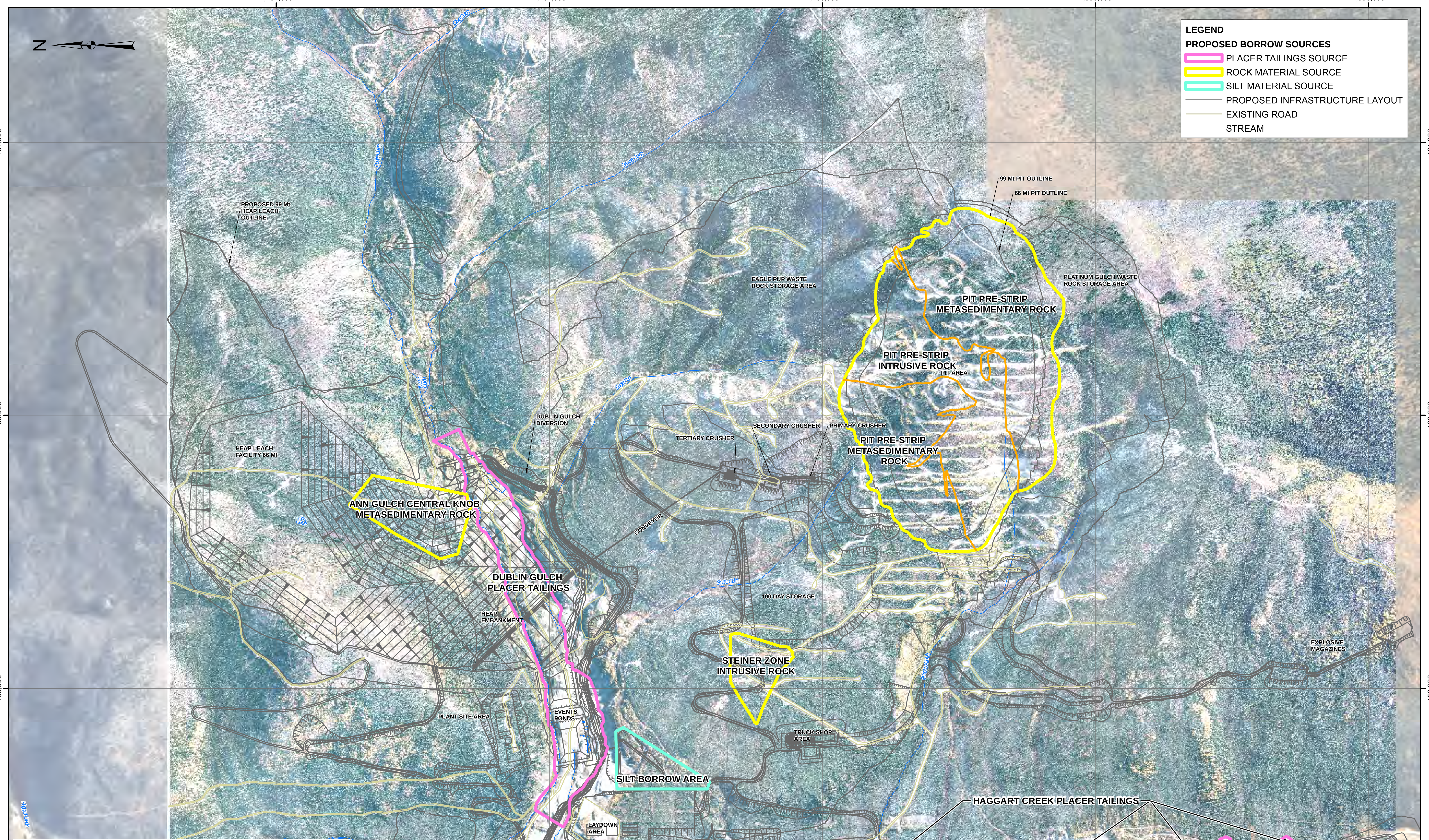

BORROW MATERIAL SUMMARY		
MATERIAL	TYPE	VOLUME
Pit Pre-strip	Surface weathered Granodiorite and Quartzite	Available volumes will be dependant on mining sequencing
Ann Gulch Central Knob	Metasedimentary Rock	Up to approximately 900,000 m^3, pending confirmation of grading plans from Tetra Tech
Steiner Zone	Surface weathered Granodiorite	Assuming a quarry depth of 5 m, approximately 200,000 m^3 of rock is available
Dublin Gulch Placer Tailings	Well graded sand and gravels	2.0 M m^3
Haggart Creek Placer Tailings	Well graded sand and gravels	approx 750,000 m^3
Silt Borrow	Silt (ML), trace sand to sandy, most likely frozen and ice rich.	220,000 m^3

100 50 0 100 200 300
METRES
NAD 1983 UTM Zone 8N

NOTES:
1. PROPOSED INFRASTRUCTURE LAYOUT RECEIVED FROM WARDROP NOV.23, 2011.
2. IMAGERY PROVIDED BY VICTORIA GOLD CORPORATION.

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APPROVED: PQ

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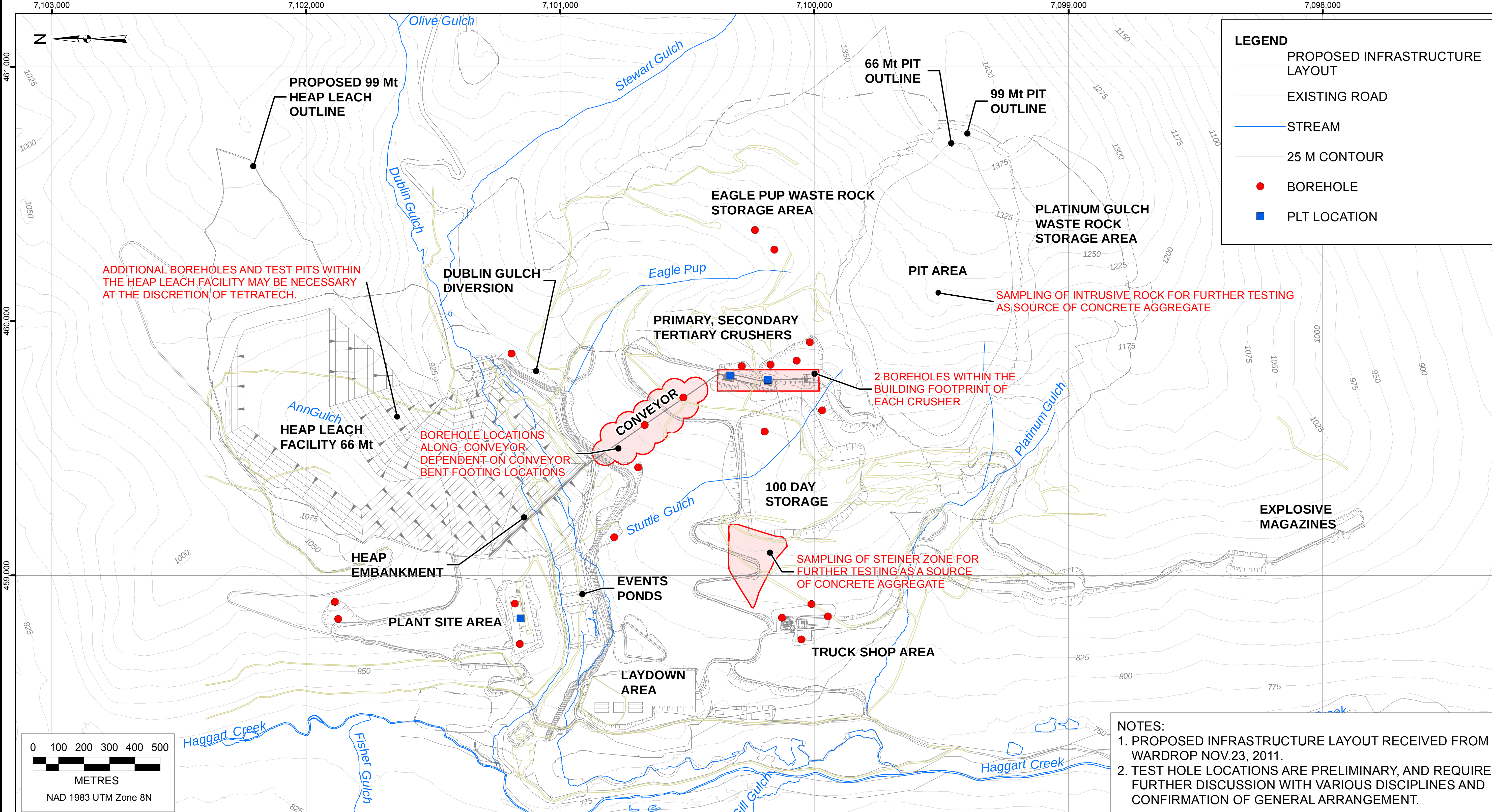
CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: BORROW MATERIAL - OTHER ENGINEERING MATERIALS

PROJECT No.:	DWG No.:	REV.:
0792-006	28	0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012



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A	DEC 2011	DRAFT ISSUED FOR REVIEW	LL	DW	PQ

SCALE: 1:15,000
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DESIGNED: KH
CHECKED: DW
APPROVED: PQ

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
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CLIENT: VICTORIA GOLD CORPORATION

PROJECT: EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT

TITLE: RECOMMENDATIONS FOR FURTHER INVESTIGATION

PROJECT No.: 0792-006
DWG No.: 29
REV.: 0

DWG TO BE READ WITH BGC REPORT TITLED "EAGLE GOLD PROJECT - 2011 GEOTECHNICAL INVESTIGATION FOR MINE SITE INFRASTRUCTURE FOUNDATION REPORT" DATED JANUARY 2012

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APPENDIX A
SUMMARY OF OBSERVED SITE CONDITIONS

1.0 INTRODUCTION

1.1. General

BGC completed site investigations to support design of mine site infrastructure in 2009, 2010 and 2011 (BGC 2010, BGC 2011, BGC 2012). These data, along with data from historic site investigation work completed by Stantec (2010), Knight Piesold (1996a, 1996b), Sitka (1996), GeoViro (1996) and various exploration programs, was used to summarize the site conditions relevant to the development of mine site infrastructure.

This appendix presents a synthesis of all presently available geotechnical data of relevance to the geotechnical design of mine site infrastructure for the Eagle Gold project.

1.2. Generalized Site Conditions in the Mine Site Area

1.2.1. General Site Conditions

The site topography involves moderate to high relief, with ground elevation varying from approximately 800 to 1400 m ASL. Ground conditions are highly variable across the site. Groundwater was observed at varying depths across the site, generally close to the elevation of streams in the valley bottoms, and often below the depth of test pit excavation on the hillsides.

Frozen ground is present in the area, and is relatively warm (typically 0 to -1 degrees Celsius), discontinuous and occasionally contains excess ground ice. Although not specifically controlled by slope aspect, permafrost is found more frequently in the north-facing lower slopes above the south side of Dublin Gulch.

1.2.2. Typical Subsurface Conditions

Subsurface data from BGC geotechnical investigations, and relevant data from prior investigations by others, have been compiled for review in support of this work. The locations of available data are shown on Drawing 11.

Overburden soils encountered on the sloping ground at the mine site typically consist of a veneer of organic soils overlying a blanket of colluvium, which overlies weathered bedrock. The observed thickness of overburden materials is illustrated in Drawing 12.

Glacial till is generally encountered on the lower flanks of the north- and west-facing slopes north and west of the proposed open pit, above Dublin Gulch and Haggart Creek. Placer tailings (fill) cover most of the valley bottom of Dublin Gulch and Haggart Creek. Alluvial soils are occasionally encountered along the undisturbed valley-bottom areas. Surface soils typically consist of organic soil, rootlets, woody debris and plant matter.

Colluvium varies in composition but typically consists of loose to compact subangular to angular gravel and occasional cobbles in a sand and silt matrix, derived from weathered metasedimentary rock. In some zones, the colluvium is gravelly, sandy low plastic silt, also derived from weathered metasedimentary rock. Till is typically firm to stiff sandy silt or compact silty sand with varying proportions of gravel.

Placer tailings (fill) are typically well graded sand and gravel with varying proportions of silt, cobbles and boulders. Particle size distribution and density vary considerably throughout the placer tailings. Drawing 13 shows the distribution of placer tailings thickness, where known. This unit consists of native materials that have been reworked by placer mining activities, and is present in the Dublin Gulch and Haggart Creek valley bottoms.

"Weathered rock" is defined as un-transported bedrock that is completely weathered and weak, with weathering grade of W5 or higher, and intact strength of R0 or less (i.e. UCS less than 1 MPa). "Weathered rock" is expected to behave like a soil, and is therefore included as part of the thickness of overburden illustrated on Drawing 12.

Drawing 14 shows the ground and surface water observations. Drawing 15 shows the distribution of frozen ground, where encountered, which can generally be inferred to be permafrost, but may in some cases be seasonally frozen soils. Frozen ground is more difficult to excavate than unfrozen ground, and can be expected to require ripping. Drawing 15 also shows the distribution of ice rich permafrost, which for the purposes of this report is defined as frozen soils that become very wet and soft when thawed. Ice-rich permafrost soils are unstable as a foundation for an engineering structure when thawed.

The bedrock encountered at the mine site is classified as either intrusive (typically granodiorite, in the uplands) or metamorphosed sedimentary rock (typically schist, phyllite or quartzite), with a variable depth of weathering. Bedrock has been subdivided into three types on the basis of expected engineering characteristics, including, from weakest to strongest: Type 3; Type 2; and, Type 1.

"Type 3" rock is the first "rock-like" material underlying the overburden soil materials, and is defined as being rock that is highly or less weathered (i.e. W4.5 or better), and has intact strength greater than R0 (i.e. minimum UCS strength 1 MPa). It is expected that Type 3 rock can be excavated with normal excavating equipment with some material requiring ripping. Drawing 16 shows the observed depth to "Type 3" rock.

"Type 2" rock is stronger and stiffer than "Type 3" rock. This material is defined as rock with Geological Strength Index (GSI, Hoek and Marinos, 2000) or Rock Mass Rating (RMR, Bieniawski, 1976) of 30 or greater, and core recovery during drilling of 50% or greater. Alternatively, where GSI and RMR data are unavailable, average Rock Quality Designation (RQD) of 10 or greater serves as an approximately equivalent criterion. It is expected that Type 2 rock will require a combination of normal excavation and ripping. Drawing 17 shows the observed depth to "Type 2" rock.

"Type 1" rock is the strongest rock observed during the site investigations. This material is defined as having GSI, RMR or average RQD exceeding 40. It is expected that Type 1 rock will require ripping, and may require local blasting. Drawing 18 shows the observed depth to "Type 1" rock.

2.0 BEDROCK STRUCTURE

Design of selected cut slopes will be controlled by the presence, orientation, persistence and strength properties of discontinuities in bedrock. This section provides a general overview of interpreted structural geology in relation to the design of mine site infrastructure facilities.

2.1. Area overview

Dublin Gulch lies within an area that was deformed and metamorphosed in the late Jurassic to early Cretaceous period by north-directed folding and thrusting. The site sits on the hanging wall to the south of the major thrust faults that accommodated the north-south shortening (Murphy 1997). Intrusive plutons in the area were emplaced by subsequent magmatism associated with this event (Mortensen et al. 2000). Most of the modern rock structure at Dublin Gulch can be attributed to this event, and to a period of north-south extension shortly afterwards – around the time of the gold mineralization at Dublin Gulch – that caused the development of steeply dipping, E- to NE-striking extension veins and NNW-striking strike-slip fault veins (Stephens et al. 2004).

Two main rock types are found within the study area around Dublin Gulch: metasedimentary rocks of the Hyland Group, and a granodiorite intrusive stock belonging to the Tombstone Plutonic Suite (Murphy 1997). The metasedimentary rocks range from quartzite to phyllite, and are contact-metamorphosed to hornfels near the granodiorite intrusion. Their engineering characteristics are primarily determined by their relative content of quartz and mica/phyllite, and by their degree of contact metamorphism (controlled in turn by distance from the granodiorite intrusion). The quartz-rich metasedimentary rocks are strong, blocky, lightly folded, and have a well-jointed structure, whereas the mica-rich phyllite tends to be very weak, friable, intensely folded, and its structure is almost entirely controlled by the closely-spaced foliation planes. The mica-rich phyllite is mainly found north of and within Dublin Gulch, and the rocks south of Dublin Gulch are generally more quartz-rich, with quartz content increasing in proximity to the intrusive body. Where the metasedimentary rocks have been contact metamorphosed, they are much stronger and tend to have rougher, more widely-spaced discontinuities.

2.2. Structural domains

The Dublin Gulch study area was divided into structural domains based mainly on similarity of rock type and structures (see Drawing 19). The intrusive rocks comprise one single domain (B), and the sedimentary rocks were divided into four separate domains (A, C, D, and E). Domains A and C occupy the southern three quarters of the study area and are separated by a major west-plunging anticline that runs from northeast to southwest, with its axis passing just north of the proposed open pit. In domain A on the south limb of the anticline, the average foliation dips shallowly to steeply southwest; in domain C on the north limb of the anticline, the foliation dips moderately northwest. Domain D covers an area around Tin Dome where the bedrock is very phyllitic and intensely folded, resulting in a distribution of somewhat irregular foliation orientations. Domain E, covering the upper eastern side of Ann Gulch, is the southwest corner of an area stretching to the north and east of the study area wherein the foliation dips mostly north.

2.2.1. Domain A – South Limb of Anticline

Much of Domain A is situated around the edges of the granodiorite intrusion, so the metasedimentary rocks observed in the area are almost all contact metamorphosed and are fairly strong. A notable exception is the rocks directly within a few meters of the intrusive contact, metasedimentary and intrusive alike, which tend to be clay-altered and very weak; in places completely disintegrated. The foliation in Domain A is slightly wavy perpendicular to its dip direction at outcrop scale (10s of meters), but noticeably more so at the inter-outcrop scale (100s of meters), with average dips ranging from 25-63 degrees between different outcrops. The poles to foliation planes measured in Domain A fall into a bi-modal distribution on stereonet which suggests the limbs of a series of tilted monoclines (Figure 1). Some two-sided folds occur in this set as well, as indicated by instances of foliation dipping shallowly in the opposite direction of the regional average.

The strongest discontinuity set in Domain A apart from foliation is the set of subvertical, SW-striking extension fractures that host much of the mineralization in the area (JV1). These are highly persistent, planar joints and joint-veins and were observed at most mapped outcrops. A secondary joint set with similar orientation to JV1 but dipping more shallowly and with generally lower persistence was observed sporadically (JS2). A steeply-dipping, NNW-striking set cross-cuts JV1 and was also observed at most mapped outcrops (JS1). Finally, two additional joint sets that dip moderately towards the NE and SE were observed sporadically (JS3 and JS4).The average orientations and properties of the discontinuity sets in Domain A are shown in Table 1.

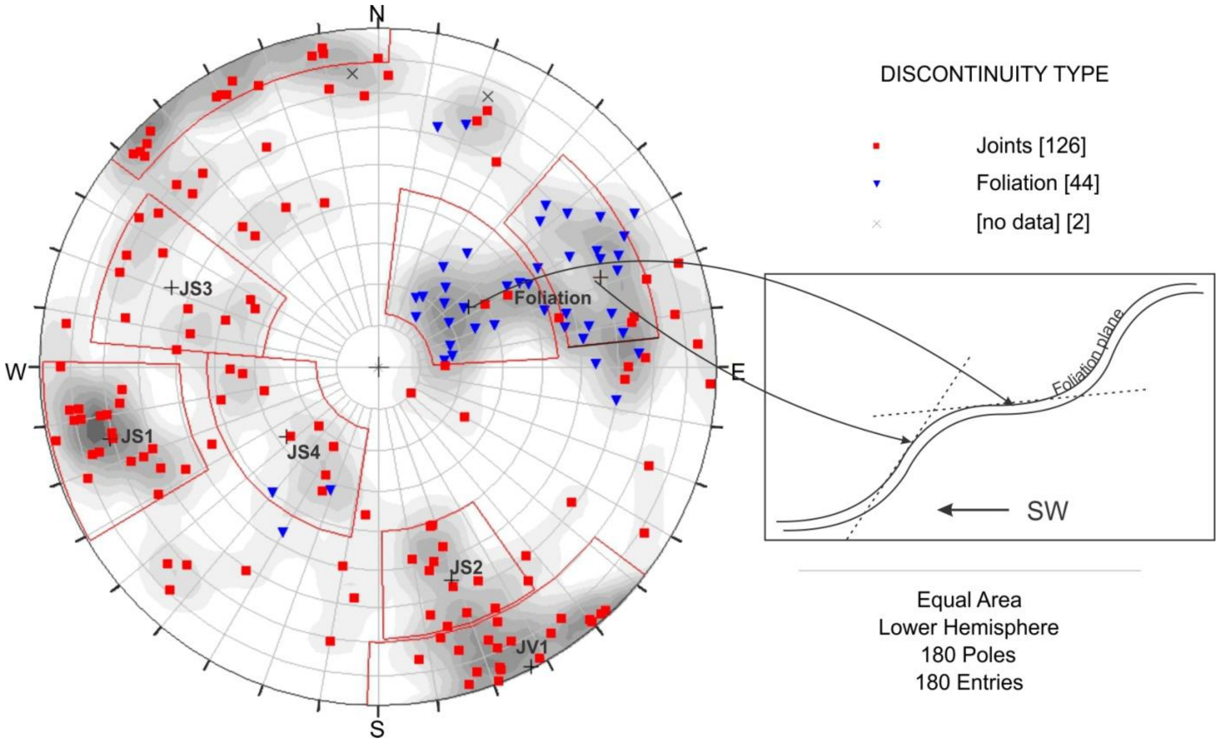

Figure 1. Discontinuities mapped in Domain A, with major sets and conceptual sketch demonstrating potential fold geometry.

Table 1. Discontinuity set average properties in structural domain A.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
Foliation	43	245	11	3.5	2.2	0.08
JV1	87	332	9	4	1.6	0.22
JS1	74	76	10	4	1.8	0.39
JS2	53	343	12	3.5	1.3	0.22
JS3	56	120	9	3.5	1.6	0.47
JS4	29	60	10	3	1.6	0.2

2.2.2. Domain B – Granodiorite Intrusion

Most of the intrusive rocks in Domain B are very strong, with the exception (noted above) of clay-altered rocks often found within a few meters of the contact with the metasedimentary rocks. The sub-vertical ENE-striking, mineral-hosting extension fractures are strongly expressed in Domain B (JV1), as well as an orthogonal sub-vertical set that strikes NNW (JS1; Figure 2). The strike of these two sets varies substantially around Domain B, being rotated clockwise (to the north/east) in the northerly part of the domain. However, their relative orthogonality is consistent everywhere. As in Domain A, these sets tend to be very planar and have high persistence.

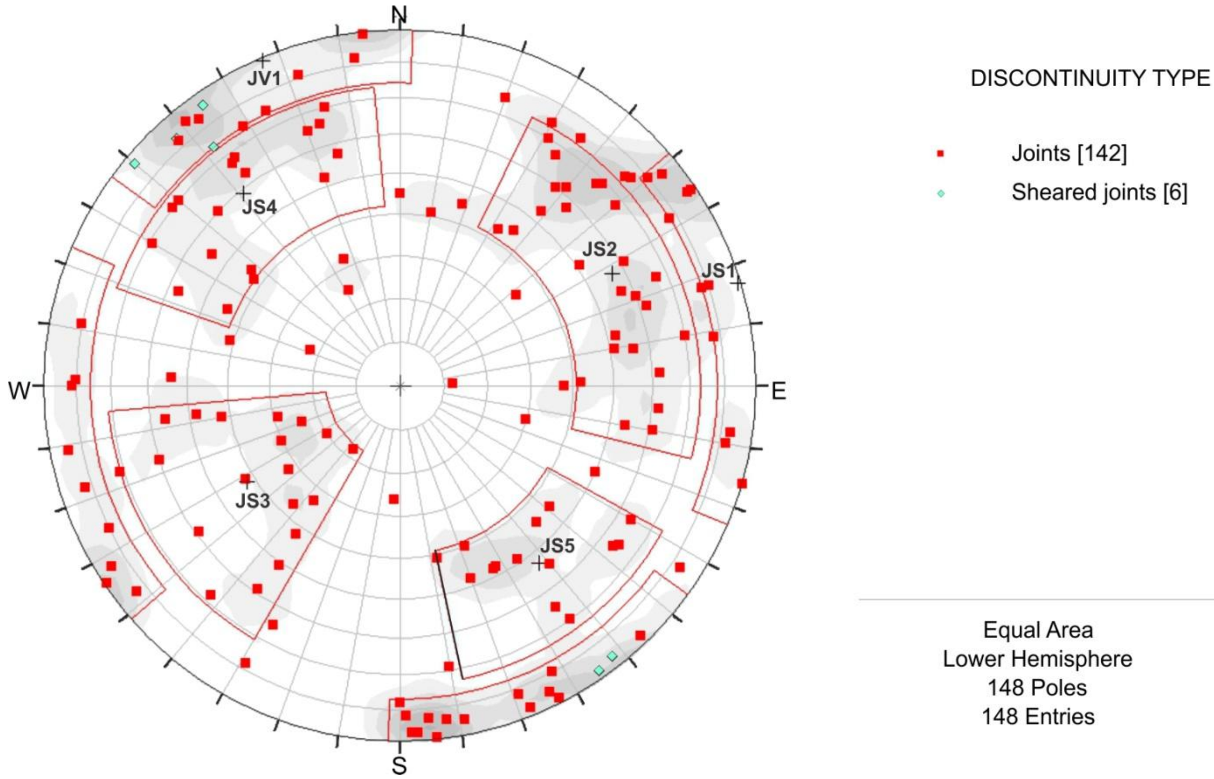

Figure 2. Discontinuities mapped in Domain B, showing major sets.

Four other joint sets were also observed throughout Domain B (JS2-JS5), with moderate average dips from 40-70 degrees. Their average orientations on stereonet suggest two groups of conjugate sets, whose strikes are rotated approximately 15 degrees counter-clockwise from JV1 and JS1. In general, joints in sets JS3 and JS4 tend to be more persistent than those in JS2/JS5, and JS4 joints are smoother than the other three sets. JS2 and JS4 were observed at most outcrops, whereas JS3 and JS5 appear sporadically. Table 2 shows the average orientations and properties of all discontinuity sets in Domain B.

Table 2. Discontinuity set average properties in structural domain B.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
JV1	90	341	10	4.5	4.3	0.41
JS1	89	253	12	4	3.0	0.54
JS2	57	242	14	4.5	2.1	0.56
JS3	58	42	14	4	2.3	0.59
JS4	59	141	10	4	4.0	0.75
JS5	53	322	11	4.5	1.1	0.44
Sheared joints	86	140	5	3.5	2.8	0.5

2.2.3. Domain C – North Limb of Anticline

Domain C represents the northern limb of the major anticline that crosses the study area from east to west. Some hornfels are found in the southeastern corner of the domain, adjacent to the granodiorite intrusion. The rocks in Domain C grade northwards from quartzite into progressively more micaceous phyllitic rock. In general, the part of Domain C south of Dublin Gulch is primarily quartzite, and the part within and north of Dublin Gulch is primarily phyllite. However, these two units are often interbedded in fairly narrow seams (~ 0.1 – 1 m), particularly near Dublin Gulch.

The foliation in Domain C is regularly oriented, dipping shallowly to moderately NW across most of the domain (Figure 3), and relatively planar – the waviness of foliation surfaces perpendicular to the direction of dip is less pronounced than that seen in Domain A. The spacing of foliation planes varies more than an order of magnitude between the quartzite and phyllite rocks.

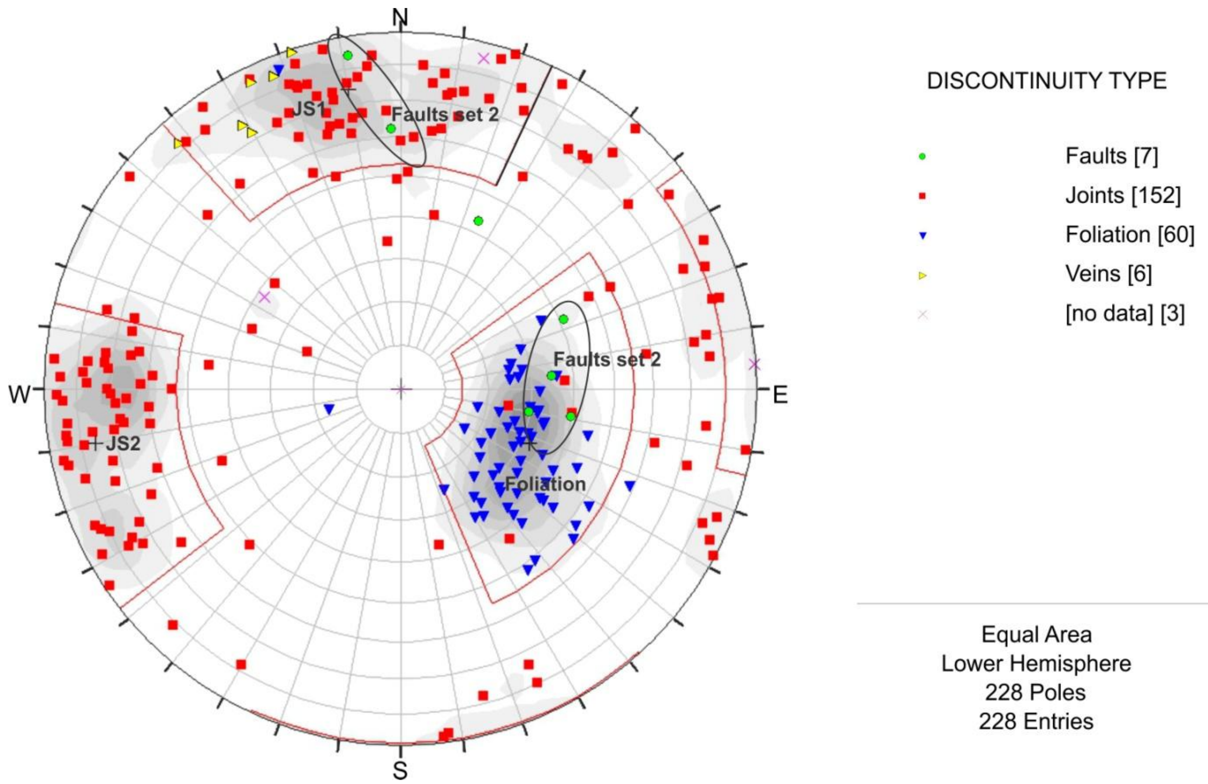

Figure 3. Discontinuities mapped in Domain C, showing major sets.

Two major joint sets cross-cutting foliation were observed at nearly every outcrop in Domain C (JS1 and JS2), one dipping steeply SSE and the other dipping steeply ENE. Their strikes are equivalent to the sub-vertical extension fracture sets JV1 and JS1 in domains A and B, but they dip less steeply and the ENE-striking set only rarely hosts mineralized veins. Two sets of faults were noted in Domain C: one parallel to foliation, and one sub-parallel to JS1. Additional minor joint sets cutting obliquely across foliation exist at most sites, but none stands out as a consistent set on the inter-outcrop scale. Table 3 shows the average orientations and properties of discontinuity sets in Domain C.

Table 3. Discontinuity set average properties in structural domain C.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
Foliation	32	299	13	3	2.1	0.11
JS1	73	173	9	3	2.2	0.29
JS2	76	79	12	3	1.4	0.27
Fault set 1	36	267	19	1	6.0	N/A
Fault set 2	73	174	7	2.5	4.8	N/A

2.2.4. Domain D – Area around Tin Dome

A distribution of irregular foliation orientations was mapped in the well-folded phyllitic rocks around Tin Dome, just north of the bottom of Dublin Gulch. The density of mapping in the area is insufficient to interpret the major geologic structures controlling foliation attitudes, so the rock structure in Domain D cannot be interpolated between mapped sites with good confidence. However, the majority of the bedrock in Domain D is so soft and fractured that its engineering behavior will likely be controlled less by structure than by the intact rock and rock mass properties.

Despite this, some structural interpretation can still be made in Domain D. The two steeply dipping orthogonal extension fracture sets seen elsewhere at Dublin Gulch are present (JS1 and JS2; Figure 4). A widely-spaced minor third joint set dipping moderately southwest is also apparent (JS3). Foliation in the southwest corner of Domain D near the planned plant site dips generally southwest, although at highly variable angles. In general, foliation planes are very wavy due to the high degree of small-scale folding present (1s to 10s of metres-scale).

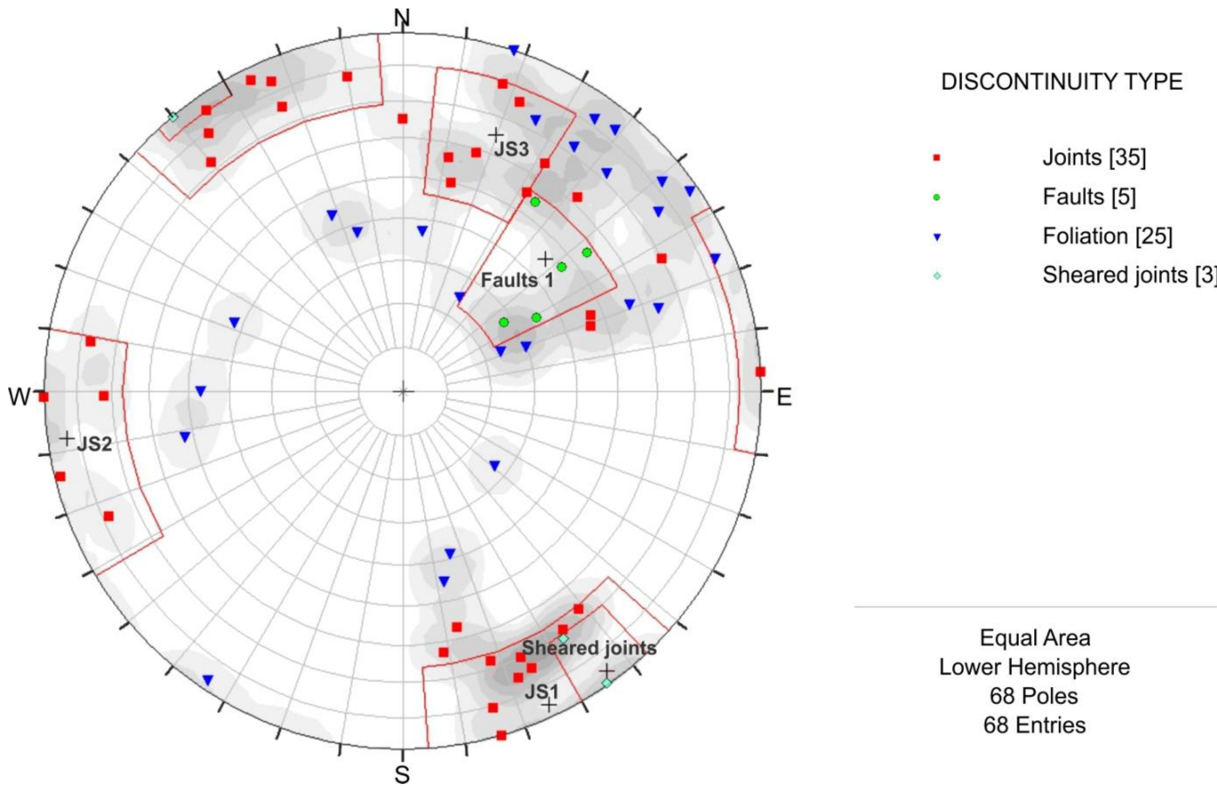

Figure 4. Discontinuities mapped in Domain D, showing major sets.

Table 4. Discontinuity set average properties in structural domain D.

	Avg Dip	Avg Dip Direction	JRC	JWCS	Persistence	Spacing
Foliation	Various	Various	15	2	1.8	0.05
JS1	85	335	8	3	2.2	0.17
JS2	82	84	14	3	1.0	0.13
JS3	62	203	14	3	1.0	0.38
Faults 1	43	230	18	3	9.2	0.5
Sheared joints	83	324	12	3	3	0.1

2.2.5. Domain E – northeast of Ann Gulch

The BGC field program mapped structural data at only one station in Domain E; however, data from others (Stephens, et. al. 2004) in the area north of Dublin Gulch and east of Ann Gulch agrees with the north-dipping foliation observed by BGC (Figure 5).

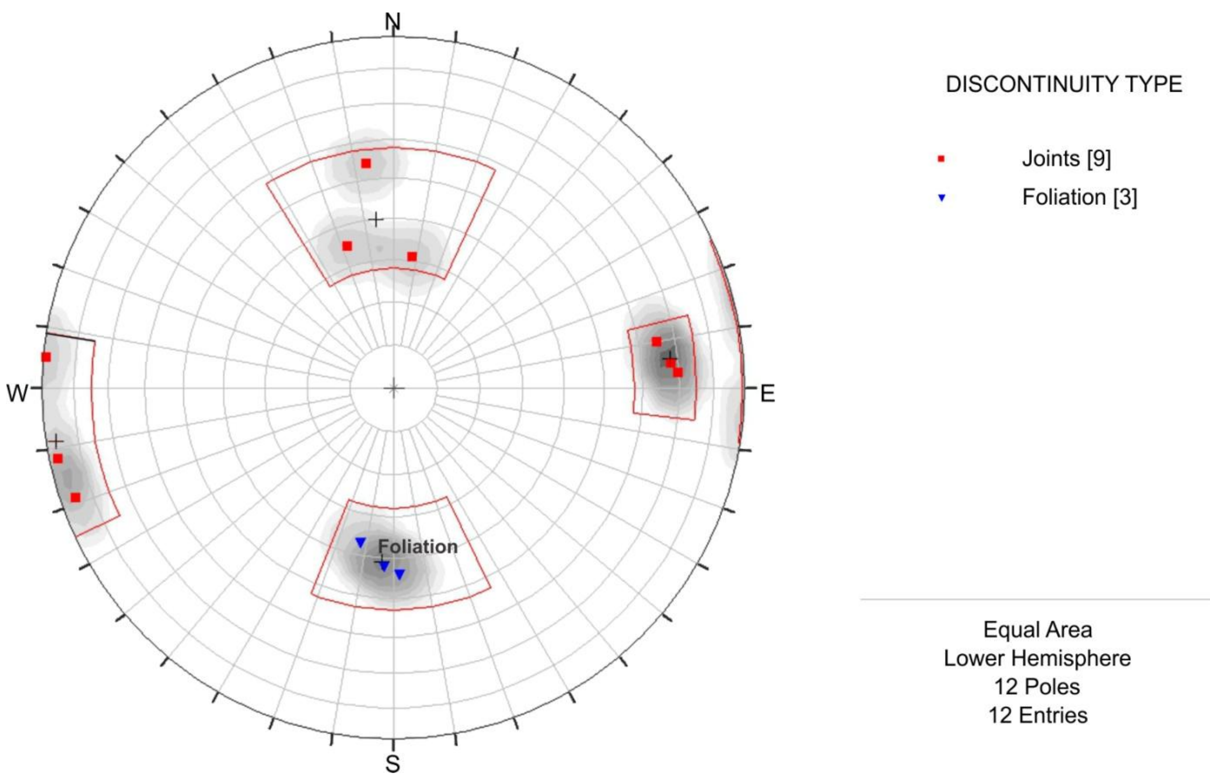

Figure 5. Discontinuities mapped in Domain E.

3.0 AREA-SPECIFIC SUBSURFACE CONDITIONS

3.1. General Overview by Functional Area

The project site has been subdivided into a number of distinct functional areas related to proposed infrastructure elements for the purpose of data synthesis and analysis.

Summary observations for each functional area are presented in Table 5. This table provides an overview of the general conditions within each area, including the observed thickness of overburden, presence or absence of frozen ground and excess ice, and depth, where encountered, to Types 1, 2 or 3 bedrock.

Extensive deposits of placer tailings fill are present in the Dublin Gulch valley bottom in the area of the heap leach pad, heap embankment, a portion of the Dublin Gulch diversion, and ponds or other facilities to be constructed in this area. The observed thickness of placer tailings at 16 test holes had a mean value of about 10 m, with a range between 0.3 m and 19.8 m. Based on shear wave geophysical surveys of the Dublin Gulch valley bottom, the placer tailings likely extend to a depth of up to about 25 m, and possibly deeper toward Haggart Creek.

There is typically a thin cover of organic soils overlying the other overburden units. The observed thickness of this unit varies across the site, ranging between 0 m and 3.7 m, with an average thickness of 0.3 m, and standard deviation of 0.3 m, from 285 observations. All organic materials are unsuitable for re-use as engineering fill materials, but should be suitable for reuse as cover materials for reclamation and should be segregated and separately stockpiled.

The main body of the report includes commentary on earthworks construction in each functional area in relation to these specific observations.

Table 5. Summary Observations of Ground Conditions by Functional Area

Area	Overburden[7] Thickness (m)				Observations of Frozen Ground			Depth to Rock where Encountered (m)					
	Known Thickness[1]		Minimum Thickness[2]		N[3]	Frozen Ground[4], N_f	Excess Ice[5], N_{ei}	Type 3		Type 2		Type 1	
	Median	N[3]	Median	N[3]				Median	N[3]	Median	N[3]	Median	N[3]
100 Day Storage	1.2	7	3	7	14	9	7	1.2	7	1.8	6	N/A	N/A
Conveyors	13.5	2	2.3	9	11	7	6	13.6	2	23.3	1	N/A	N/A
Crushers	3.2	18	4.7	3	21	4	2	2.6	14	4.6	11	17.4	4
Diversion	4.8	10	5.5	12	22	7	4	4.5	9	8.8	7	19.5	1
Dublin Gulch pond	N/A	N/A	16.8	3	3	0	0	N/A	N/A	N/A	N/A	N/A	N/A
Eagle Pup WRSA pond	10.4	3	5.5	3	6	1	1	3.8	3	12.2	3	18.9	2
Eagle Pup WRSA	2.5	39	3.9	38	77	47	29	1.9	36	3	25	21.5	6
Events Ponds	12.2	3	5.5	5	8	0	0	12.2	3	16.2	1	14.9	1
Explosive Storage	2.0	2	N/A	N/A	2	0	0	2.0	2	4.5	1	N/A	N/A
Heap Embankment	8.8	13	5.7	12	25	3	3	9.0	12	14.2	6	31.2	1
Heap Pad	3.5	50	5	21	71	14	6	1.9	42	4.8	29	17.3	4
Laydown Area	N/A	N/A	5.5	11	11	6	6	N/A	N/A	N/A	N/A	N/A	N/A
Main Pond	N/A	N/A	5.8	9	9	7	5	N/A	N/A	N/A	N/A	N/A	N/A
Main truck road	5.1	3	4.9	4	7	1	1	5.1	3	N/A	N/A	N/A	N/A
Plant site	9.1	7	6.4	4	11	2	1	9.1	5	12.3	1	N/A	N/A
Platinum Gulch WRSA pond	N/A	N/A	6.9	4	4	1	1	N/A	N/A	N/A	N/A	N/A	N/A
Platinum Gulch WRSA	2.4	19	3.1	10	29	11	11	2.3	14	3.3	12	10.9	4
Secondary road	N/A	N/A	2.8	10	10	7	4	N/A	N/A	N/A	N/A	N/A	N/A
Truck Shop	7.1	3	2.5	3	6	6	5	7.1	3	N/A	N/A	N/A	N/A

1. "Known thickness" of overburden indicates the full depth is known because bedrock was encountered during drilling or test pitting.
2. "Minimum thickness" of overburden represents observations where the overburden is known to be at least a given thickness, equal to the depth of exploration, but total thickness is not known, since bedrock was not encountered.
3. "N" is the number of observations taken into consideration.
4. N_f is the number of observation locations where frozen ground was noted.
5. N_{ei} is the number of observation locations where excess ice was observed in the frozen ground.
6. "N/A" indicates no data available in that area.
7. Overburden is defined as soil material, including organics, till, colluvium, alluvium, fill (placer tailings) and completely weathered rock.
8. Median values are presented in this table. There is significant variability throughout functional areas. Please see drawings 12 through 18 for detailed illustrations of site conditions.

3.2. Subsurface Conditions at the Proposed Plant Site

3.2.1. General

Drawing 11 shows the distribution of test holes located in the general vicinity of the proposed Plant Site. The subsurface observations from these holes are summarized in Table 6, below.

Table 6. Summary Subsurface Observations in Proposed Plant Site Area

Testhole	Approx. Ground Elevation (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth to Type 3 Rock (m)	Total Depth (m)	Frozen Ground Encountered
TP-BGC09-HL1-1	878	0.3	1.3	4.9	6.5	6.5	Yes
TP-BGC09-HL1-2	845	0.3	5.9	N/A	N/A	6.2	Yes
TP-BGC10-33	868	0.25	4.0	>1.3	N/A	6.5	No
TP-BGC10-34	852	0.2	3.8	>1.5	N/A	6.5	No
TP-BGC11-103[3]	865	0.2	3.3	>3.7	N/A	6.0	No
TP-BGC11-105	878	0.2	2.3	>0.5	N/A	3	No
TP-BGC11-130	865	0.1	>5.9	N/A	N/A	6	No
BH-BGC10-11	857	NR	NR	6.1	13.2	46.6	No
BH-BGC10-12	863	NR	NR	13.2	19.2	28.7	No
BH-BGC11-54	884	NR	NR	15.2	19.8	41.2	No
BH-BGC11-67[4]	867	N/A	N/A	9.1	9.1	9.9	No
BH-BGC11-69[4]	867	N/A	N/A	9.1	9.1	21.34	No

Notes:

1. "NR" = no recovery
2. N/A – not observed or not applicable
3. In TP-BGC11-103, till was present between 3.3 m and 6 m.
4. BH-BGC11-67 and BH-BGC11-69 were drilled in the pit excavated for plate load testing. Base of pit was in completely weathered metasedimentary rock. This pit is logged as TP-BGC11-103.

3.2.2. Overburden

The upper soil unit consists of a horizon of organic soil, rootlets, woody debris and plant matter ranging from 0.1 to 0.3 m thickness and averaging approximately 0.2 m.

The organic cover is immediately underlain by colluvium to depths ranging from 2.0 m to > 5.9 m, with an average depth of approximately 3.4 m. The colluvium consists of loose to compact, subangular to angular gravel and occasional cobbles in a silt and sand matrix, derived from transported weathered metasedimentary bedrock further upslope. Till was present in TP-BGC11-103, and consisted of compact sand and gravel with both granodiorite and metasedimentary clasts.

Weathered rock is present below the colluvium in a number of test pits and boreholes, ranging in thickness from approximately 4 m to 15 m. The material is typically highly to completely weathered, extremely weak to very weak metasedimentary rock.

3.2.3. Bedrock

Bedrock was observed below weathered rock at depths ranging between 6.5 m and 19.8 m below drill collar elevation (average depth to bedrock at 12.8 m) in the test holes shown in Drawing 11. Observed bedrock consisted of slightly to moderately weathered, very weak to weak metasedimentary rock (i.e. Type 3 rock). Type 2 rock was encountered in one borehole (BH-BGC11-69) at 12.3 m depth; it should be noted that BH-BGC11-69 was drilled at the base of an excavation and its collar elevation is approximately 6 m below existing ground surface.

Recovery from drill holes in bedrock was poor in some zones and good in others, ranging from 20% to 100%, with average recoveries of approximately 55%. Rock Mass Rating (RMR, Bieniawski, 1976) values ranging from 15 to 55, with an average of approximately 30, were determined from the observed rock core.

These observations suggest that weathered rock will typically be encountered at foundation grades. The cut for the plant site pad will be primarily in completely weathered rock with a thick blanket of colluvium.

The soft, phyllitic bedrock around the plant site is intensely folded and faulted, resulting in a fairly erratic distribution of foliation attitudes (Figure 6). In large plate load test pits in the footprint of the plant site, two distinct directions of foliation were mapped, dipping 81° towards 213° and 39° towards 169°. On average, foliation in the area dips roughly southwest between 25 and 85 degrees. Sheared fault structures sub-parallel to foliation, some containing clay gouge infill, were observed at outcrops 48A and 51 (BGC 2012). There are also steeply-dipping joint sets that strike NE and NNW; these are consistent with strong regional sets observed at most sites around Dublin Gulch. Although these structures should be considered in engineering analyses of the plant site, the extreme weakness of the bedrock here suggests that its engineering behavior will be controlled primarily by the strength of the intact rock and rock mass.

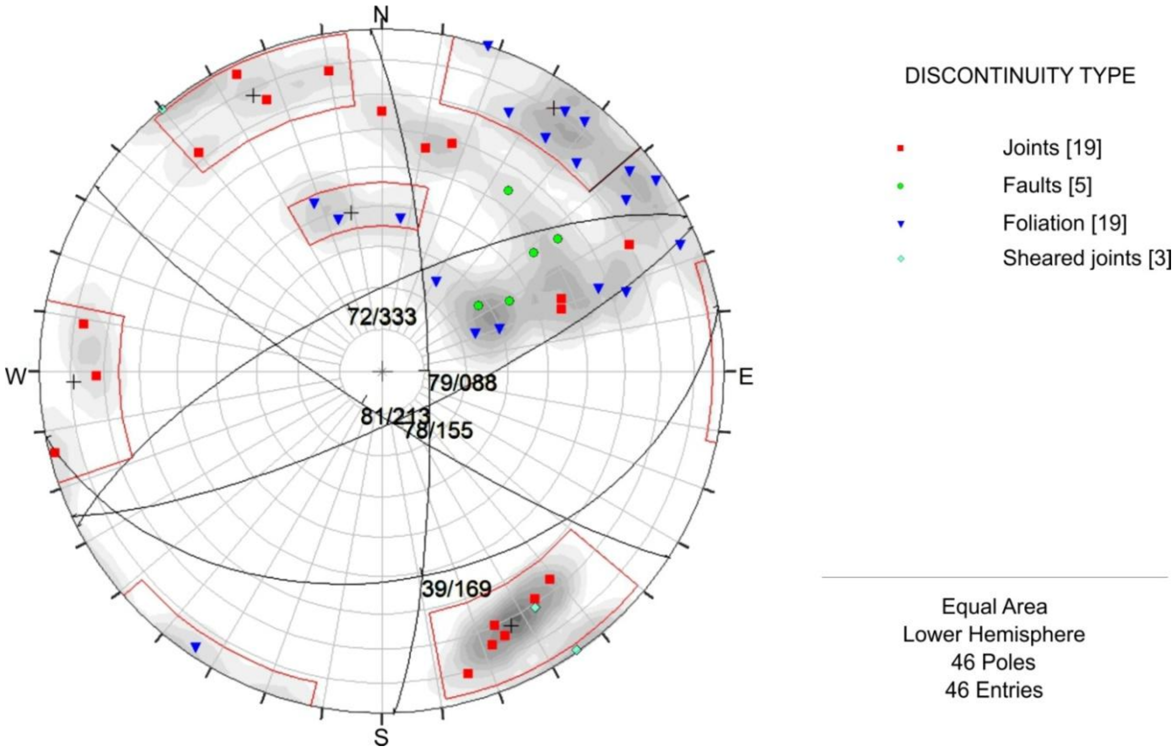

Figure 6. Discontinuities and structural sets mapped around the plant site.

Shear wave velocities were measured in one borehole (BH-BGC11-69) to a depth of 19.5 m. Although this depth is insufficient to calculate a Vs_{30}, a shear wave velocity of 832 m/s was used to approximate the site class as B/Rock (NBCC 2005).

3.2.4. Groundwater

Groundwater seepage was not noted in any of the excavated test pits in proximity to the Plant Site, up to 7.3 m depth below grade. A water level of 11.8 m below ground surface was observed from the vibrating wire piezometer installed in BH-BGC11-54. Groundwater is observed at shallower depths, close to the elevation of Dublin Gulch, in the valley bottom below the proposed plant site location.

3.2.5. Permafrost

Frozen ground was encountered in two (TP-BGC09-HL1-1 and -2) of the seven test pits excavated in this area. This suggests that sporadic permafrost could be encountered during site preparation.

3.2.6. Geological Hazards

No specific geological hazards have been identified by Stantec (2010) in the general area of the proposed plant site. Subsurface investigation (BGC 2010, 2011 and 2012) confirmed the presence of colluvial soils derived from shallow gravitational translational movement of native materials from further upslope, as is typically seen in rugged terrain.

3.3. Subsurface Conditions at the Proposed Crushers and Conveyors

3.3.1. General

The proposed layout of the crushers was changed during BGC's 2011 site investigation program in response to encountering poor rock quality at the locations proposed at the outset of the 2011 site investigation program. Additional holes were drilled at the new proposed location for the secondary and tertiary crushers. The available data from all test holes in the general vicinity of the crushers is summarized below.

Drawing 11 shows the distribution of test holes located in the general vicinity of the proposed crushers and conveyors. The subsurface observations from these holes are summarized in Table 7 and Table 8 below.

Table 7. Summary Subsurface Observations in Proposed Crusher Area

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth to Type 3 Rock (m)	Depth to Type 2 Rock (m)	Depth to Type 1 Rock (m)	Total Depth (m)	Frozen Ground Encountered
BH-BGC10-8	1036	NR[1]	NR	3.4	N/A[2]	3.4	8.5	26.2	No
BH-BGC10-18	1063	NR	4.5	N/A	4.5	7.3	19.5	30.2	No
BH-BGC11-35	986	NR	7.5	7.7	15.2	N/A	N/A	50.3	No
BH-BGC11-36	1002	NR	3.1	N/A	3.1	11.1	N/A	50.3	No
BH-BGC11-37	1034	NR	NR	N/A	N/A	3.8	7.2	43.6	No
BH-BGC11-38	1013	NR	5.2	0.2	5.4	18.3	N/A	50.5	No
BH-BGC11-40A	1050	NR	NR	7.3	7.8	9.1	14.6	33.2	No
BH-BGC11-40B	1050	NR	NR	N/A	8.7	10.7	15.5	45.7	No
BH-BGC11-50	1058	NR	5.2	N/A	5.2	12.2	15.2	41.2	No
BH-BGC11-62	1018	NR	1.5	N/A	1.5	4.6	24.4	35.1	No
TP-BGC09-HL4-5	987	0.2	>6.3	N/A	N/A	N/A	N/A	6.5	Yes
TP-BGC10-05	1064	0.2	0.4	N/A	0.6	4	N/A	4	No
TP-BGC10-06	1038	0.2	1.2	0.7	2.1	4.2	N/A	4.2	No
TP-BGC10-09	1038	0.2	0.5	N/A	0.7	1.0	N/A	1.0	No
TP-BGC10-10	1080	0.2	1.0	N/A	1.2	1.5	N/A	1.5	No
TP-BGC11-50	1011	0.2	3.4	N/A	3.6	N/A	N/A	5.8	No
TP-BGC11-51	972	0.2	>4.5	N/A	N/A	N/A	N/A	4.7	Yes
TP-BGC11-59	1065	0.2	0.9	N/A	1.1	2.6	N/A	2.6	No
TP-BGC11-60	1067	0.2[3]	2	N/A	N/A	N/A	N/A	2.7	Yes

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth to Type 3 Rock (m)	Depth to Type 2 Rock (m)	Depth to Type 1 Rock (m)	Total Depth (m)	Frozen Ground Encountered
TP-BGC11-127	1096	0.1	0.9	N/A	1.1	2.5	N/A	2.5	No
TP-BGC11-138	1050	0.3	1.2	N/A	1.5	N/A	N/A	6.3	No

Notes:
1. “NR” = no recovery
2. N/A – not observed or not applicable

3. A secondary layer of organics was present below the colluvium in TP-BGC11-60.

Table 8. Summary Subsurface Conditions in Proposed Conveyor Area

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Completely Weathered Rock Thickness (m)	Depth[4] to Type 3 Rock (m)	Depth[4] to Type 2 Rock (m)	Depth[4] to Type 1 Rock (m)	Total Depth[4] (m)	Frozen Ground Encountered
BH-BGC10-7	948	NR	18[1]	N/A	18	23.3	N/A	30	No
MW09-STU1	967	NR	9.2	N/A	9.2	N/A	N/A	14.3	Unknown
TP-BGC09-HL4-1	963	0.3	>1.6	N/A	N/A	N/A	N/A	1.9	Yes
TP-BGC09-HL4-2	910	0.3	>2.0	N/A	N/A	N/A	N/A	2.3	Yes
TP-BGC09-HL4-3	913	0.2	N/A	N/A	N/A	N/A	N/A	5	Yes
TP-BGC10-11	945	0.2	>4.1	N/A	N/A	N/A	N/A	4.3	Yes
TP-BGC11-90	981	0.2	>6.3	N/A	N/A	N/A	N/A	6.5	No
TP-BGC11-91	969	0.2	>1.9	N/A	N/A	N/A	N/A	2.1	Yes
TP-BGC11-92	933	0.2	>1.5	N/A	N/A	N/A	N/A	1.7	Yes
TP-BGC11-93	917	0.4	>1.3	N/A	N/A	N/A	N/A	1.9	Yes
TP95-47	N/A	0.2	>5.3	N/A	N/A	N/A	N/A	5.5	No

Notes:

1. The overburden materials may be completely weathered rock below some thickness of colluvium but poor recovery during drilling prevented confident classification.
2. A layer of ice 0.2 m thick was present below the organics in TP-BGC11-93.
3. Recovery was poor in top 18 m of BH-BGC10-7. This zone may be colluvium or weathered rock.
4. Depths indicated are below existing ground surface.

3.3.2. Overburden

The upper soil unit consists of a thin horizon of organic soil, rootlets, woody debris and plant matter ranging from 0.1 m to 0.4 m thickness and averaging approximately 0.2 m.

In the vicinity of the proposed crushers, the organic cover is underlain by colluvium ranging in thickness from 0.4 m to 7.5 m, averaging 2.6 m. The colluvium typically consists of a loose to compact, subangular to angular gravel and occasional cobbles in a silt and sand matrix, derived from transported weathered metasedimentary rock further upslope.

In the vicinity of the proposed conveyor line extending north from the tertiary crusher to the heap, the organic cover is underlain by colluvium (possibly overlying completely weathered rock) to depths up to 18 m (in BH-BGC10-7, where there was poor recovery up to 18 m). The colluvium consists of loose to compact subangular to angular gravel and occasional cobbles in a silt and sand matrix derived from transported weathered metasedimentary bedrock further upslope. This colluvium is expected to be typically frozen and ice-rich; however, limited subsurface information is available at depth.

Completely weathered rock is present below the colluvium and above bedrock in a number of the holes in the vicinity of the crushers. The highly to completely weathered rock ranges in thickness from 0.7 m to 7.7 m, averaging 4.8 m and consists of cobbley or sandy gravel.

3.3.3. Bedrock

In the vicinity of the proposed crushers, bedrock was observed immediately below colluvium in some holes and below a weathered rock horizon in other holes at depths ranging from 0.6 m to 15.2 m, averaging 4.0 m. Observed bedrock when first encountered consisted of moderately to highly weathered metasedimentary rock, typically Type 3 rock. Type 2 and Type 1 rock were encountered below the Type 3 rock. The contact between the metasedimentary and intrusive (granodiorite) rock was encountered in BH-BGC11-50 and both rock types were observed in this hole.

At the location of the proposed primary crusher and primary crusher haul road, the depth to Type 3 rock ranged from 0.6 m to 8.7 m. The depth to Type 2 rock ranged from 2.6 m to 19.5 m and the depth to Type 1 rock ranged from 8.5 m to 15.5 m. Rock mass quality and characteristics were inferred from five bore holes (BH-BGC10-8, BH-BGC10-18, BH-BGC11-40A, BH-BGC11-40B and BH-BGC11-50). Typical Rock Mass Rating (RMR, Bieniawski, 1976) values of about 40 were determined from the observed rock core and ranged from 20 to 75. RMR values generally increased with depth. Given the founding grades at the primary crusher, Type 1 rock is expected at founding elevation and will comprise the majority of the cut at the proposed primary crusher location.

At the current proposed location of the secondary crusher, Type 2 rock was encountered near surface at depths ranging from 1.0 m to 3.8 m. The depth to Type 1 rock was encountered at 7.2 m in BH-BGC11-37. Rock mass quality and characteristics were inferred from one borehole (BH-BGC11-37), located in the footprint of the secondary crusher. Average RMR values of about 40 were determined from the observed rock core and ranged from 20 to 65. RMR values generally increased with depth. The secondary crusher pad is currently proposed to be constructed from a cut fill balance and is planned to be founded on both Type 2 rock and fill.

At the current proposed location of the tertiary crusher, Type 2 rock was encountered at 4.6 m in BH-BGC11-62. Type 1 rock was encountered at 24.4 m. Rock mass quality and characteristics were inferred from one borehole (BH-BGC11-62). Average RMR values of about 35 were determined from the observed rock core and ranged from 20 to 45. RMR values generally increased with depth. The tertiary crusher pad is currently proposed to be constructed from a cut fill balance and is planned to be founded on both Type 2 rock and fill.

Down slope from the currently proposed locations, where the secondary and tertiary crushers were previously planned, rock quality is poorer. Type 3 rock is encountered at greater depths (ranging from 3 m to 15.2 m), with Type 2 rock at encountered at 11.1 m in BH-BGC11-36 and not encountered through the full depth of BH-BGC11-35, to 50.3 m. Type 1 rock was not encountered in the vicinity of the previously proposed secondary and tertiary crusher locations. Average RMR values of approximately 30 were determined from the observed rock core (in two boreholes, BH-BGC11-35 and BH-BGC11-36) and ranged from 19 to 55, typically increasing with depth. Although rock quality was observed to be better at the currently proposed locations for the secondary and tertiary crushers, given the observations of poor rock quality in the general area, it is possible that poor rock quality may be encountered at the currently proposed locations of the secondary and tertiary crushers.

At the location of the proposed conveyor, rock mass quality and characteristics were inferred from two boreholes (BH-BGC10-7 and MW09-STU1). Average RMR values of about 30 were determined from the observed rock core and ranged from 20 to 40. The depth to Type 3 rock was 18 m in BH-BGC10-7 and 9.2 m in MW09-STU1. Type 2 rock was encountered at a depth of 23.3 m and continued through the full depth of the hole to 30 m in BH-BGC10-7 and was not encountered through the full depth of MW09-STU1 to 14.3 m.

Geological mapping of surface structural features was carried out at several locations within 300 to 400 m of the proposed crushers. Mapped sites included natural outcrops, road cuts, test pits, and boreholes. Figure 7 shows joints, faults, and foliation planes mapped by BGC as well as Victoria Gold field geologists.

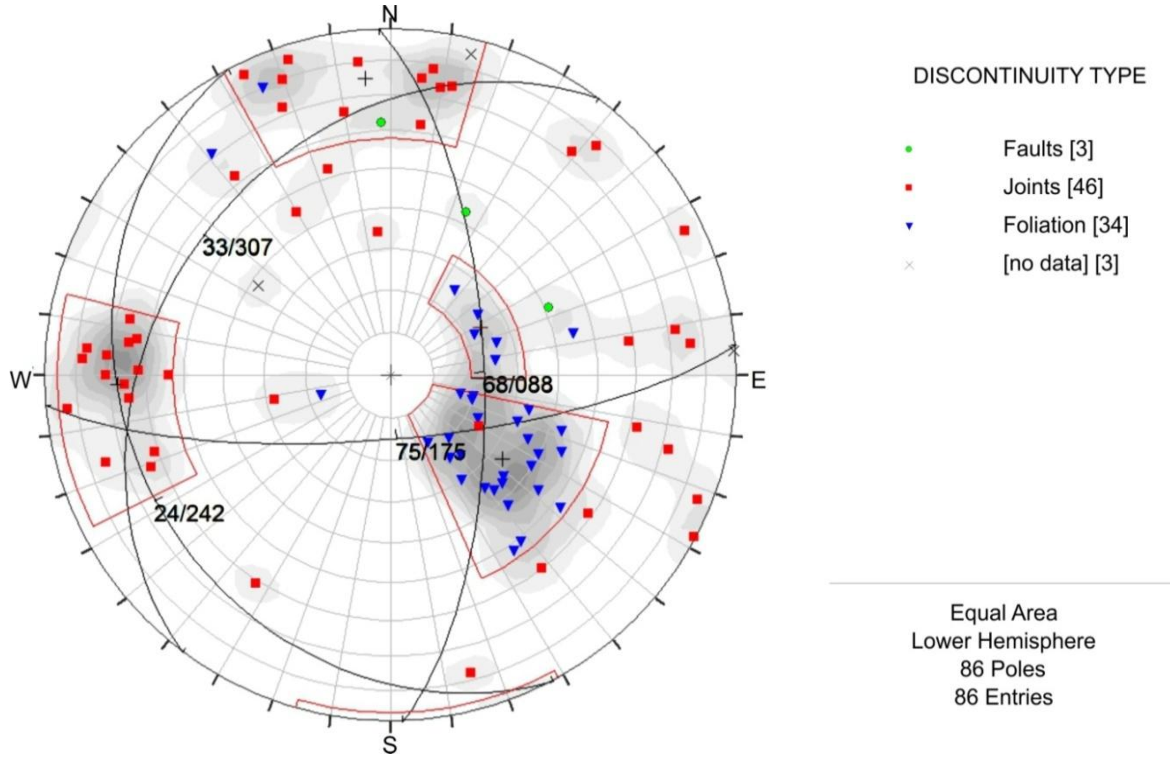

Figure 7. Discontinuities and structural sets mapped around the crushers.

The rock structure near the crushers, particularly the primary crusher, is strongly influenced by the axis of the major east-west anticline shown in Drawing 19 that marks the boundary between structural domains A and C. At the secondary and tertiary crushers on the northern limb of the anticline, the foliation of metasedimentary rocks dips northwest from 20-60 degrees, averaging 33 degrees. Borehole BH-BGC11-50, at the primary crusher, is near the axis of the anticline; here the foliation dips more shallowly west to southwest, averaging 24 degrees. Foliation planes are smooth at a small scale (centimetres to metres), but folded and undulating at the scale of 10s of meters, as shown by the variability of mapped orientations. Foliation in nearby outcrops dips more steeply, with an overall average in the area of approximately 32 degrees.

The angle of foliation planes in relation to the drill core axis in nearby vertical drillholes is generally similar to the field mapping data, with a mean dip of 26 degrees. The core retrieved from these boreholes was not retrieved using oriented core methods, and therefore the dip direction of these discontinuities is unknown. However, based on the relatively tight cluster of surface mapping observations, it can be inferred that many of the discontinuities observed in the core would likely also have similar orientations.

Two steep joint sets that cross-cut the foliation and each other are present near the crushers, dipping an average of 75 degrees towards the south and 68 degrees towards the east. The surfaces of these planes are fairly smooth, with average JRC values between 4 and 12. The bedrock lithology around the crushers is mostly quartzite, with a degree of contact metamorphism that increases towards the intrusive body from north to south.

Shear wave velocities were measured in 3 boreholes (BH-BGC11-36, BH-BGC11-40B and BH-BGC11-62) to a depth of 30 m. The site class (NBCC 2005) varied for this site between B – Rock at BH-BGC11-36 and BH-BGC11-40B and C- very dense soil and soft rock at BH-BGC11-62.

3.3.4. Groundwater

Groundwater seepage, which may represent thawing of seasonally frozen ground, was noted in two test pits in the vicinity of the conveyors (TP-BGC11-92 and TP-BGC11-93) at depths of 0.25 m and 0.4 m respectively. Groundwater seepage was not noted in any of the excavated test pits in the vicinity of the crushers, up to 6.5 m depth below grade. Four standpipe piezometers were installed in the vicinity of the crushers as part of the 2011 site investigation program. Water levels in these piezometers are tabulated in Table 10. Depth to the water table varies from approximately 8.5 m below ground surface near the primary crusher to approximately 20 m to 26 m below ground surface in the vicinity of the secondary and tertiary crushers.

Table 9. Crusher Area Groundwater Observations

Piezometer ID	Groundwater Depth (m below ground surface)	Location of Piezometer
BH-BGC11-35	24.6	Near Tertiary Crusher
BH-BGC11-36	19.9	Near Secondary Crusher
BH-BGC11-38	26.3	Tertiary Crusher
BH-BGC11-40B	8.5	Primary Crusher
MW09-STU1	15.4	Near Conveyor

Given the depth of the proposed cuts for the crushers, groundwater is expected to be encountered in the primary crusher cut. Seepage observations in test pits along the conveyor alignment suggest that groundwater may be encountered during foundation preparation for the conveyor.

3.3.5. Permafrost

Frozen ground was encountered in 11 of the 19 test pits excavated in this area, more commonly near the proposed conveyor line, and less commonly near the crushers. This suggests that sporadic patches of permafrost could be encountered during site preparation.

3.3.6. Geological Hazards

No specific geological hazards have been identified by Stantec (2009) in the general area of the proposed primary crusher (Drawing 20). The secondary crusher and tertiary crusher are located in an area identified as being subject to permafrost processes. Subsurface investigation (BGC 2010, 2011 and 2012) confirmed the presence of colluvial soils, as is typically seen in rugged terrain. The observed frozen ground mentioned in above was observed within the terrain unit identified as subject to permafrost processes, and is therefore consistent with the terrain analysis reported by Stantec (2010).

The conveyor is located in an area identified as being subject to both permafrost processes and surface seepage. Subsurface investigation (BGC 2012) confirmed the presence of colluvial soils, frozen ground and shallow seepage along the proposed conveyor alignment and is therefore consistent with the terrain analysis reported by Stantec (2010).

3.4. Subsurface Conditions at the Proposed Truck Shop

3.4.1. General

Drawing 11 shows the distribution of test holes located in the general vicinity of the proposed truck shop. The subsurface observations from these holes are summarized in Table 10, below.

Table 10. Summary Subsurface Observations in Proposed Truck Shop Area.

Testhole	Approx. Ground Elev. (m)	Organics Thickness (m)	Colluvium Thickness (m)	Depth[2] to Type 3 Rock (m)	Total Depth[2] (m)	Frozen Ground Encountered
BH-BGC11-57	859	0.1	7.0	7.1	12.1	Yes
BH-BGC11-58	859	0.1	8.3	8.4	10.8	Yes
BH-BGC11-60	859	0.1	6.9	7.0	9.2	Yes
TP-BGC11-83	863	0.5	>0.8	N/A[1]	1.3	Yes
TP-BGC11-84	863	0.3	>2.3	N/A	2.6	Yes
TP-BGC11-85	865	0.5	>2.0	N/A	2.5	Yes

Notes:

1. N/A – not observed or not applicable.
2. Depths indicated are below existing ground surface.

3.4.2. Overburden

The upper soil unit consists of a horizon of organic soil, rootlets, woody debris and plant matter ranging in thickness from 0.1 m to 0.5 m and averaging approximately 0.3 m.

The organic cover is underlain by colluvium to depths ranging from 7.0 m to 8.4 m with an average depth of approximately 7.4 m. The colluvium consists of low plastic silt with some sand and some gravel derived from transported weathered metasedimentary bedrock further upslope.

Standard Penetration Testing (SPT) was completed in all three boreholes in the truck shop area. Tests in BH-BGC11-58 and BH-BGC11-60 are considered invalid since recovery in the samples was less than six inches. Three valid tests in colluvium were completed in BH-BGC11-57, all with SPT N_{60} greater than 50.

3.4.3. Bedrock

Bedrock in the proposed truck shop location was observed only by auger drilling and therefore rock mass parameters were not measured. Type 3, moderately weathered metasedimentary rock was present at depths ranging from 7.0 m to 8.4 m. Given the founding grades of the truck shop, Type 3 rock or better is expected at founding grades and will likely comprise the majority of the cut at the proposed truck shop location.

A single SPT test was completed was completed in the moderately weathered bedrock in BH-BGC11-57 with SPT N_{60} greater than 50.

3.4.4. Groundwater

Seepage was observed at 0.5 m in TP-BGC11-84. Two piezometers were installed in BH-BGC11-57 and BH-BGC11-58. No groundwater was observed in either set of measurements taken in late August 2011. The proposed founding grade of the truck shop is approximately 25 m below existing ground surface, so although not observed, groundwater could be present at greater depths.

3.4.5. Permafrost

Frozen ground, including excess ice, was observed in all test pits and boreholes completed in the proposed truck shop location, with excess ice confined to a depth of approximately 2-3 m. Frozen ground conditions are anticipated in the upper portions of the cut at the proposed truck shop location.

3.4.6. Geological Hazards

The truck shop is located in an area identified as being subject to permafrost processes (Stantec 2010, Drawing 20). Frozen ground was observed in both test pits and auger holes in the vicinity of the test pit, which is therefore consistent with the terrain analysis reported by Stantec (2010).

3.5. Subsurface Conditions at the Proposed Heap Leach Pad, Water Diversion and Process Management Ponds

3.5.1. Heap Leach Pad

3.5.2. General

Drawing 11 shows the distribution of test holes located in the vicinity of the proposed heap leach pad. The data suggest that this area can be divided into three zones with distinct overburden conditions: Heap Leach Upland, Heap Leach Valley Bottom, and Heap Leach Southern Edge above Valley Bottom (see Figure 8). The test hole observations from these three zones are summarized in Table 11, Table 12 and Table 13, respectively.

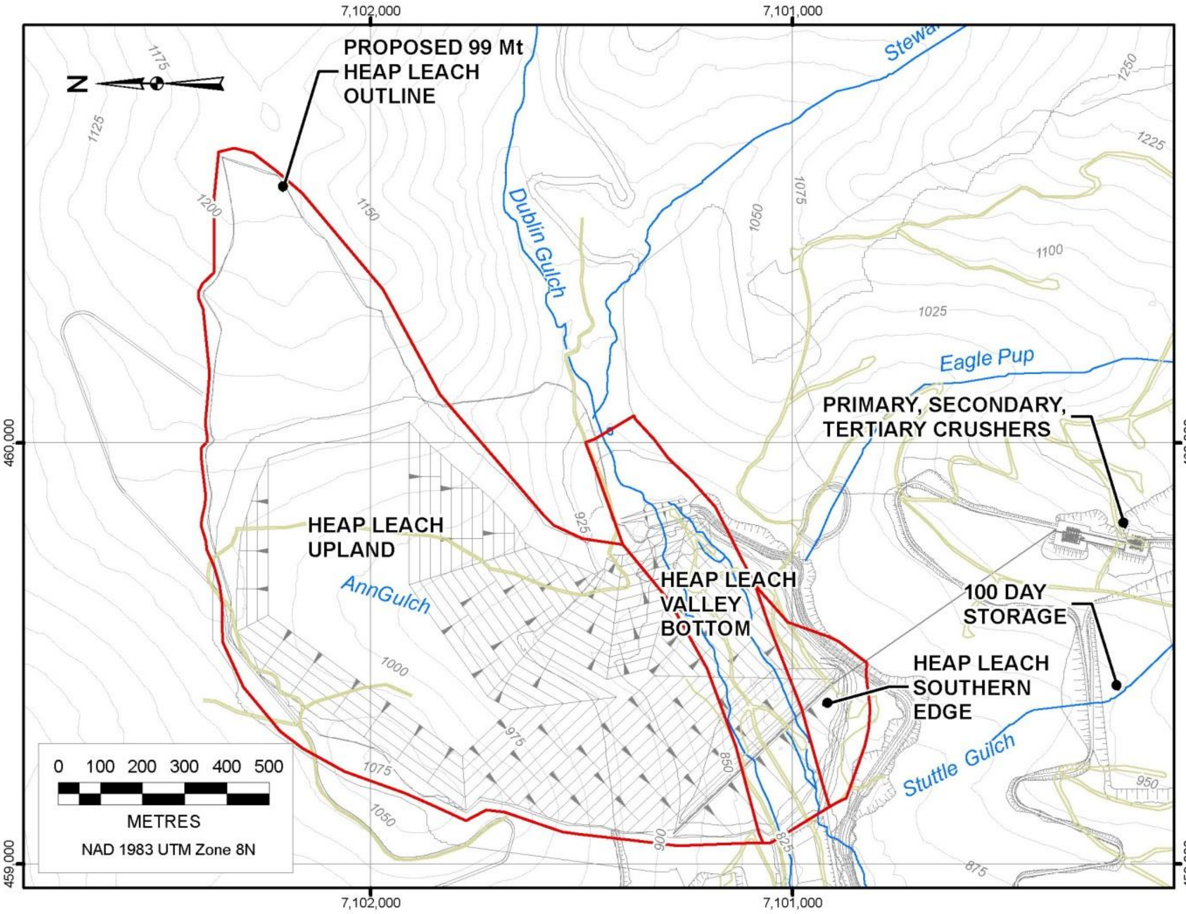

Figure 8. Heap Leach Pad Areas

Table 11. Summary Subsurface Observations in Proposed Heap Leach Pad Area - Upland

Test Hole ID	Approx.Elev[1]. (m)	Overburden thickness (m)					Depth[8] to Type 3 Rock (m)	Depth[8] to Type 2 Rock (m)	Depth[8] to Type 1 Rock	Total Depth[8] (m)	Frozen Ground
		Organics	Colluvium	Till	Fill	Completely Weathered Rock					
TP-BGC09-A1[6]	884	0.2	1.1	>0.9	-	-	-	-	-	2.2	Yes
TP-BGC09-HL6-1	1038	0.1	2.4	-	-	-	2.5	-		6.5	No
TP-BGC09-HL6-2	1024	0.1	0.6	-	-	-	0.7	4.4	-	4.4	No
TP-BGC09-HL6-3	1010	0.2	3.0	-	-	3.0	-	-		6.2	No
TP-BGC09-HL6-4	981	0.2	3.8	-	-	0.8	-	4.8	-	4.8	Yes
TP-BGC09-HL6-5	1022	0.1	0.6	-	-	0.7	1.4	4.0	-	4.0	No
TP-BGC09-HL6-6	1062	0.2	2.8	-	-	2.5	-	-	-	5.5	No
TP-BGC09-HL6-7	1072	0.2	2.3	-	-	2.9	-	-	-	5.4	No
TP-BGC09-HL6-8	920	0.4	> 2.2	-	-	-	-	-	-	2.6	No
TP-BGC09-HL6-9	1042	0.2	0.6	-	-	0.7	1.5	3.8	-	3.8	Yes
TP-BGC09-HL6-10	939	0.2	1.0	-	-	3.6	-	4.8	-	4.8	Yes
TP-BGC09-HL6-11	976	0.2	0.5	-	-	0.2	0.9	2.8	-	2.8	No
TP-BGC09-HL6-12	957	0.1	> 5.9	-	-	-	-	-	-	6.0	No
TP-BGC09-HL6-13	959	0.1	1.5	-	-	0.2	1.8	2.4	-	2.4	No
TP-BGC09-HL6-14[5]	870	0.2	5.6	-	-	-	6.2	-	-	6.2	No
TP-BGC09-HL6-15	979	0.2	4.7	-	-	-	4.9	-	-	5.3	Yes
TP-BGC09-HL6-16	999	0.1	-	-	-	4.4	4.5	-	-	5.3	No
TP-BGC09-HL6-17	984	0.2	1.1	-	-	-	1.3	3.3	-	3.3	No
TP-BGC10-25	1023	0.1	0.7	-	-	-	0.8	-	-	6.5	No

Test Hole ID	Approx.Elev[1]. (m)	Overburden thickness (m)					Depth[8] to Type 3 Rock (m)	Depth[8] to Type 2 Rock (m)	Depth[8] to Type 1 Rock	Total Depth[8] (m)	Frozen Ground
		Organics	Colluvium	Till	Fill	Completely Weathered Rock					
TP-BGC10-26	1023	0.1	1.6	-	-	0.3	2.0	-	-	5.3	No
TP-BGC10-27	1045	0.2	1.5	-	-	2.3	4.0	4.5	-	5.3	No
TP-BGC10-28	1027	0.3	> 0.2	-	-	-	-	-	-	0.5	Yes
TP-BGC10-29	1049	0.2	1.0	-	-	>1.8	-	-	-	3.0	No
TP-BGC10-30	1060	0.2	3.8	-	-	-	4.0	-	-	5.5	No
TP-BGC10-31	1048	0.2	3.0	-	-	>2.1	-	-	-	5.3	No
TP-BGC10-35	880	0.5	2.0	-	-	-	2.5	-	=	5.5	No
TP-BGC10-41	942	0.3	4.0	-	-	1.8	-	-	-	6.1	No
TP-BGC10-42	917	0.3	> 3.2	-	-	-	-	-	-	3.5	Yes
TP-BGC11-52	1051	0.3	1.5	-	-	-	1.8	5.0	-	5.0	No
TP-BGC11-53	1103	0.3	0.7	-	3.0	-	-	4.0	-	4.0	No
TP-BGC11-54	1178	0.2	0.8	-	-	-	1.0	2.0	-	2.0	No
TP-BGC11-55	1209	0.2	0.7	-	-	-	0.9	2.0	-	2.0	No
TP-BGC11-56	1158	0.2	-	-	-	-	0.2	2.0	-	2.0	No
TP-BGC11-57	1144	0.2	2.3	-	-	-	2.5	5.0	-	5	No
TP-BGC11-58	1118	0.2	1.8	-	-	-	2.0	5.0	-	5	No
TP-BGC11-71	885	0.3	1.9	-	-	-	-	-	-	2.2	Yes
TP-BGC11-72	874	0.3	1.1	-	-	-	1.4	-	-	4.3	No
TP-BGC11-86	894	0.3	0.2	-	-	-	0.5	-	-	7.5	No
TP-BGC11-94	930	0.2	>4.8	-	-	-	-	-	-	5.0	Yes

Test Hole ID	Approx.Elev[1]. (m)	Overburden thickness (m)					Depth[8] to Type 3 Rock (m)	Depth[8] to Type 2 Rock (m)	Depth[8] to Type 1 Rock	Total Depth[8] (m)	Frozen Ground
		Organics	Colluvium	Till	Fill	Completely Weathered Rock					
TP-BGC11-132	922	0.1	1.0	-	-	-	1.1	4.0	-	5.3	No
TP-BGC11-133	984	0.2	1.0	-	-	3.2	-	-	-	4.4	Yes
TP-BGC11-145[7]	959	0.2	>4.6	-	0.3	-	-	-	-	5.1	No
DH-BGC09-AG3	884	1.2	6.4	-	-	-	-	7.6	-	13.7	No
BH-BGC10-1	1057	NR				-	1.8	10.4	-	20.4	No
BH-BGC10-2	949	NR				-	7.3	-	=	20.4	No
BH-BGC11-24	1208	NR	1.8	-	-	-	1.8	4.9	10.2	20.9	No
BH-BGC11-25	1183	-	-	-	-	0.9	0.9	2.3	-	20.4	No
BH-BGC11-26	1140	-	-	-	-	-	0	15.4	-	30.2	No
BH-BGC11-27	1100	-	-	-	-	2.1	2.1	13.7	-	26.5	No
BH-BGC11-28	1011	-	-	-	-	-	0	13.7		40.8	No
BH-BGC11-29	1045	-	-	-	-	-	0	18.6	24.1	41.2	No
BH-BGC11-30	952	-	1.5	-	-	-	1.5	-	-	35.1	No
BH-BGC11-31	918	-	15.2	-	-	1.6	16.8	-	-	35.1	No
BH-BGC11-59	884	-	4.6	-	-	1.2	5.8	9.8	-	30.2	Yes
MW09-AG1	1017	-	10.0	-	-	-	10.0	-	-	15.9	No
MW09-AG2	1009	0.3	10.3	-	-	1.9	12.5	-	-	15.9	No
MW10-AG3	997	0.1	7.5	-	-	3.9	11.5	-		16.8	No
MW10-AG5	934	0.2	1.0	-	-	5.1	6.3	-	-	20.8	Yes
MW10-AG6	906	0.2	4.4	-	-	4.6	9.2		-	17.7	No

Test Hole ID	Approx.Elev[1]. (m)	Overburden thickness (m)					Depth[8] to Type 3 Rock (m)	Depth[8] to Type 2 Rock (m)	Depth[8] to Type 1 Rock	Total Depth[8] (m)	Frozen Ground
		Organics	Colluvium	Till	Fill	Completely Weathered Rock					
TP95-51	912	-	> 5.5	-	-	-	-	-	-	5.5	No
TP95-52	899	0.2	-	-	-	2.9	3.1		-	3.1	No
TP95-53	917	0.4	> 1.2	-	-	-	-	-	-	1.6	Yes
TP95-54	911	-	6.4	-	-	0.9	-	-		7.3	No
TP95-55	904	0.2	> 5.2	-	-	-	-	-	-	5.5	No
TP95-56	920	-	> 6.0	-	-	-	-	-	-	6.1	No
TP95-57	902	2.7	-	-	-	2.2	4.9		-	5.5	No
TP95-58	889	1.8	>5.5	-	-	-	-	-	-	7.3	Yes
TP95-59	871	1.2	>4.9	-	-	-	-	-	-	6.1	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. “NR” = no recovery
3. N/A – not observed or not applicable
4. Frozen ground observations from older test pits (TP95-XX or TP96-XX) may not reflect current conditions
5. 0.2 m of Alluvium was present below the colluvium in TP-BGC09-HL6-14.
6. Till was observed in TP-BGC09-A1 to a depth greater than 0.9 m.
7. Drill pad fill was observed in TP-BGC11-145 to a depth of 0.3 m.
8. Depths indicated are below existing ground surface.

Table 12. Summary Subsurface Observations in Proposed Heap Leach Pad Area – Valley Bottom

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[7] toType 3 Rock (m)	Depth[7] toType 2 Rock (m)	Depth[7] toType 1 Rock (m)	Total Depth[7] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP-BGC09-DG1	923	-	-	-	> 2.5	-	-	-	-	2.5	No
TP-BGC09-HL6-14	870	0.2	4.7	-	-	-	4.9	-	-	5.3	No
TP-BGC10-17[4]	873	0.1	-	>1.5	4.4	-	-	-	-	6.0	No
TP-BGC10-18[4]	877	0.2	0.3	>7.0	-	-	-	-	-	7.5	No
TP-BGC10-21[3]	895	0.1	-	-	>6.4	-	-	-	-	6.5	No
TP-BGC10-22[4]	884	0.1	0.8	-	-	0.6	1.5	-	-	5.3	No
TP-BGC10-23[4]	880	-	-	-	> 5.0	-	-	-	-	5.0	No
TP-BGC10-24	858	0.1	-	-	>2.9	-	-	-	-	3.0	No
TP-BGC10-32[4]	902	0.1	-	-	>7.9	-	-	-	-	8.0	No
TP-BGC10-35	880	0.5	2.0	-	-	-	2.5	-	-	5.5	No
TP-BGC10-36	837	-	-	-	>4.5	-	-	-	-	4.5	No
DH-BGC09-DG1	923	-	-	-	6.1	1.5	-	7.6	-	12.8	No
BH-BGC10-3	878	-	-	-	9.3	-	-	9.3	10.5	50.7	No
BH-BGC10-4	858	-	-	-	8.5	0.2	8.7	11.8	25.0	31.0	No
BH-BGC10-5[4]	884	-	-	-	4.3	-	-	4.3	-	21.0	No
BH-BGC10-6[4]	876	-	-	16.4	-	-	-	16.4	-	28.9	No
BH-BGC10-17	836	-	-	-	7.3	-	7.3	17.8	-	37.3	No
BH-BGC10-23	849	-	-	-	>6.0	-	-	-	-	6.0	No
BH-BGC11-33	833	-	-	-	8.5	0.6	9.1	-	-	41.4	No
BH-BGC11-34	848	-	-	-	16.5	-	16.5	28.4	31.2	38.1	No
MW10-DG06	859	-	-	-	2.8	1.5	4.3	-	-	11.9	No

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[7] toType 3 Rock (m)	Depth[7] toType 2 Rock (m)	Depth[7] toType 1 Rock (m)	Total Depth[7] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP95-45	838	-	-	-	>5.5	-	-	-	-	5.5	No
TP95-46	867	-	-	-	2.4	0.7	3.1	-	-	3.1	No
TP95-50	872	-	-	-	2.4	1.3	3.7	-	-	3.7	No
TP96-230	845	-	-	-	>1.5	-	-	-	-	1.5	No
TP96-231	843	-	-	-	>3.5	-	-	-	-	3.5	No
TP96-232	851	-	-	-	>3.6	-	-	-	-	3.6	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. “NR” = no recovery
3. N/A – not observed or not applicable
4. Also considered in the proposed velocity reduction pond and rockfill diversion structures analysis
5. Frozen ground observations from older test pits (TP95-XX or TP96-XX) may not reflect current conditions
6. Stantec monitoring wells MW09-DG1 has been excluded from the table since it did not provide any soil information.
7. Depths indicated are below existing ground surface.

Table 13. Summary Subsurface Observations in Proposed Heap Leach Pad Area – Southern Edge of Proposed Heap above Valley Bottom

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[4] to Type 3 Rock (m)	Depth[4] to Type 2 Rock (m)	Depth[4] to Type 1 Rock (m)	Total Depth[4] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP-BGC10-17[3]	873	0.1	-	>1.5	4.4	-	-	-	-	6.0	No
TP-BGC10-18[3]	877	0.2	0.3	>7.0	-	-	-	-	-	7.5	No
BH-BGC10-6[3]	876	-	-	16.4		-	16.4	22.9		28.9	No
BH-BGC10-16[3]	878	NR				1.5	9.9	10.5	-	28.0	No
BH-BGC11-53	876	-	-	11.4	-	-	11.4	-	-	14.5	No
BH-BGC11-55	881	-	8.8	-	-	-	8.8	-	-	14.5	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. “NR” = no recovery
3. Also considered in the proposed velocity reduction pond and rockfill diversion structures analysis
4. Depths indicated are below existing ground surface.

3.5.3. Overburden

Overburden soil conditions are distinctly different in the Dublin Gulch valley bottom from those encountered above the valley bottom in Ann Gulch and south of Dublin Gulch along the southern edge of the proposed heap.

In the Uplands above the valley bottom, the upper soil unit consists of a thin horizon of organic soil, rootlets, woody debris and plant matter ranging from 0.1 to 2.7 m in thickness and averaging approximately 0.3 m (Table 11). The organic cover in the uplands overlies colluvium ranging in thickness from 0.2 m to 15.2 m, and averaging approximately 2.9 m (Table 11). The colluvium consists of loose to compact angular gravel with occasional cobbles in a silt and sand matrix, derived from transported weathered metasedimentary bedrock. The colluvium may also include variable amounts of organics, which are often observed in distinct layers within the colluvium. Highly to completely weathered metasedimentary rock is present below the colluvium in a number of boreholes and test pits. It ranges in thickness from 0.2 m to 5.1 m, averaging 2.2 m.

The overburden soils in the valley bottom have been reworked by historical placer mining activities. Placer tailings (fill) are observed from the ground surface to bedrock, with thicknesses ranging between 2.4 m and 16.5 m, and an average thickness of approximately 6.6 m (Table 12). The material encountered is generally a well graded, loose to dense, silty sand and gravel, ranging to sand and gravel with some silt and occasional cobbles and boulders. Loose zones were encountered within the placer tailings. There is little to no vegetative cover on most of the placer tailings.

The placer tailings in the valley bottom have highly variable particle size distribution and density, and are generally saturated. Recorded Standard Penetrometer (SPT) blowcounts, N, are summarized in Table 17 for the placer tailings within the footprint of the events ponds. No blowcount data are available in the placer tailings portion of the footprint of the heap leach pad, but the placer tailings materials are expected to have a similar variability in penetration resistance and associated strength and stiffness.

Highly to completely weathered metasedimentary rock is present below the placer tailings in some boreholes and test pits in the valley bottom. It ranges in thickness from 0.2 m to 1.5 m, averaging 0.9 m.

The overburden at the southern edge of the proposed HLF includes 4.4 m of placer tailings at TP-BGC10-17, and a variable thickness of till ranging up to 16.4 m (Table 13). The till is a compact to dense sandy silt to silty sand with some gravel. It must be noted that in borehole BH-BGC10-16 there was no soil recovery, so the contact between fill and undisturbed till has been inferred from observations in the adjacent test pit TP-BGC10-17. SPT blow counts recorded in BH-BGC11-53, within the till, have an average SPT N_{60} value of 44 and range from 23 to 55. Upslope from the valley bottom a debris flow deposit is present to a depth of 8.8 m in BH-BGC11-55. This material consists of fine silty sand with some gravel. SPT blow counts recorded in BH-BGC11-55, within the debris flow/colluvium have an average N_{60} value of 20 and range from 14 to 35.

3.5.4. Bedrock

Drawing 11 shows the plan view of the Heap Leach Pad and includes all the existing test holes in the area. Bedrock was observed in the uplands above Dublin Gulch immediately below colluvium at depths ranging between 0.0 and 16.8 m below existing grade (average depth to bedrock at 3.5 m where observed).

Bedrock was observed in the valley bottom at depths ranging between 1.5 and 16.5 m below existing grade, with an average depth to bedrock at 6.2 m where observed.

Bedrock was observed in four boreholes on the southern edge of the HLF and ranged in depth from 8.8 m to 16.4 m, averaging 11.6 m.

Observed bedrock consists primarily of Type 3 and Type 2 metasedimentary rock. Type 1 metasedimentary rock was encountered in a small number of boreholes, ranging in depth from 10.5 m to 31.2 m. The metasediments in general are observed as strongly foliated yellowish brown to dark grey phyllites interbedded with quartzites. The quartzites are variably gritty, micaceous, and massive. Phyllitic metasediments are composed of muscovite-sericite and chlorite.

The rock mass quality and characteristics have been inferred from observations in the boreholes completed by BGC as tabulated above, which were drilled within the heap leach facility footprint. Average RMR values of approximately 25 were determined from the observed rock core for Type 3 rock, approximately 35 for Type 2 rock and approximately 50 for Type 1 rock. A single SPT test was completed in the moderately weathered rock in BH-BGC11-55 with an N_{60} of 86.

Mapping of structural discontinuities was carried out at road cuts, valley cuts and outcrops within and around the heap leach pad footprint by BGC during summer 2011. The mapped discontinuity features, shown on stereonets below, are divided into two groups by area. Figure 8 shows discontinuities mapped in the upper (northern) portion of the HLF, between Tin Dome and the eastern edge of the heap leach pad. Figure 9 shows discontinuities mapped in Dublin Gulch valley bottom, between the proposed diversion berm and velocity reduction pond to the east, and the proposed process management ponds to the west.

The upper portion of the HLF covers three different structural domains; C, D, and E. Structural data in this area show two major joint sets and three distinct foliation orientations. Foliation dipping southeast, opposite of the regional average, was observed at a small outcrop on top of Tin Dome. This orientation probably represents the eastern limb of a small-scale fold with its axis running perpendicular to the average dip direction of foliation, similar to the folds observed in Domain A. Foliation measured on the upper eastern flank of Ann Gulch dips 27 degrees in the opposite direction (northwest). One mapping station, located at the upper northern end of Ann Gulch, showed foliation dipping north at 41 degrees.

While this is anomalous in the context of the BGC study, north-dipping foliation has been observed in the area north of Ann Gulch by Stephens et al. (2004). The two main joint sets in the upper heap leach facility cross-cut the foliation and each other, dipping 52 degrees towards the south and 84 degrees towards the east. Bedrock lithology in this area is mostly phyllite, with interbedded seams of quartzite 10-30 cm thick.

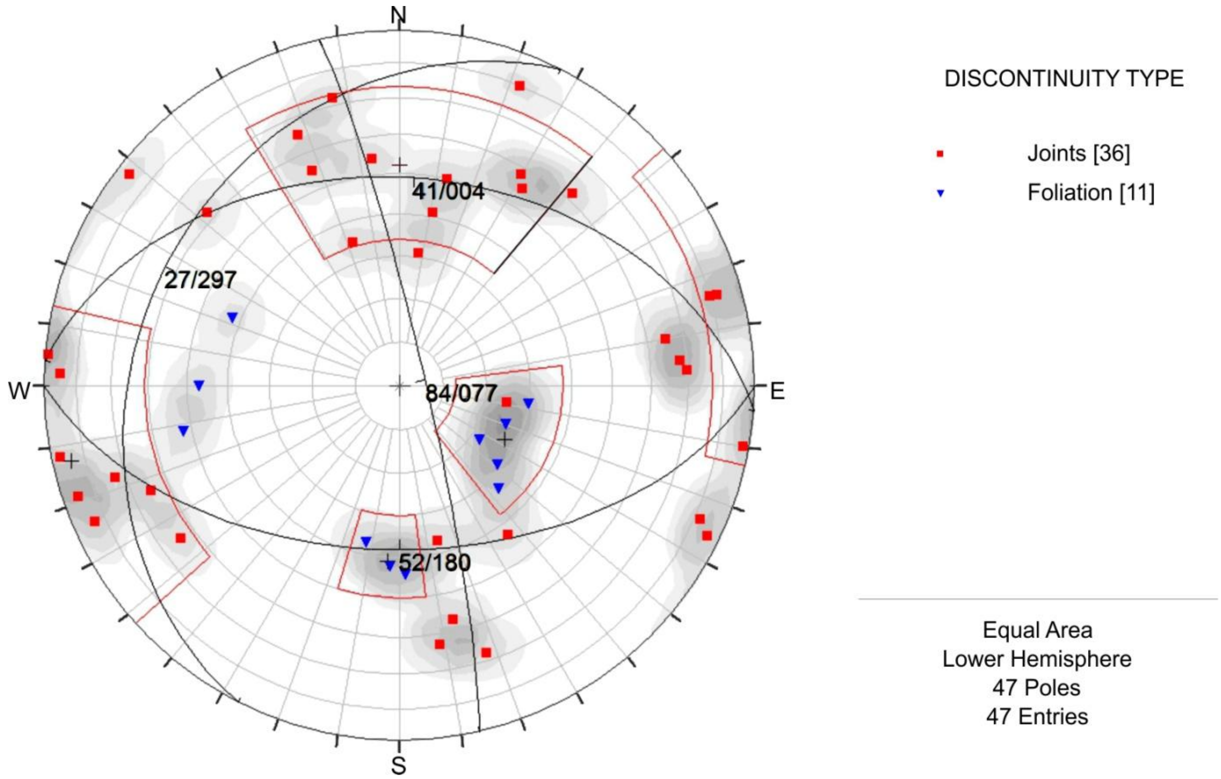

Figure 9. Discontinuities and structural sets mapped in the upper portion of the heap leach facility.

At the lower heap leach facility in Dublin Gulch valley bottom, the foliation dips shallowly (15-30 degrees) at a range of orientations from northwest to south-southwest. The foliation is cross-cut by two major joint sets dipping 81 degrees towards the east-northeast and 84 degrees towards the southeast. A third, minor joint set dips 67 degrees southwest. The surfaces of joints in this area vary from smooth to very rough (JRC 4-20), whereas the wavy foliation surfaces are mostly rough (JRC 16-20). The bedrock lithology in this area is mostly quartzite, with up to 40% phyllite at some outcrops interbedded in seams 10-20 cm thick.

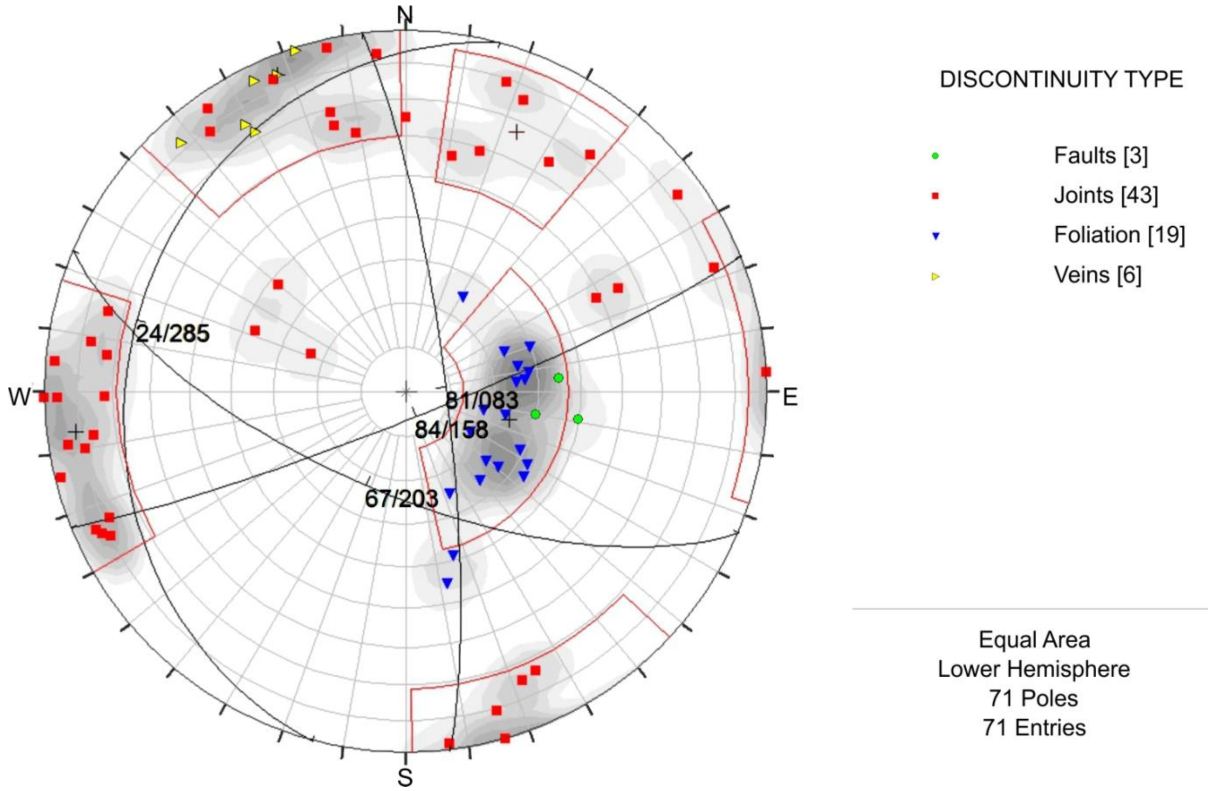

Figure 10. Discontinuities and structural sets mapped in the lower portion of the heap leach facility.

Shear wave velocities were measured in four boreholes within the heap embankment and heap pad (BH-BGC11-28, BH-BGC11-33, BH-BGC11-34 and BH-BGC11-59) to depths of 30 m, 30 m, 30 m and 28 m respectively. Although Vs_{30} values could not be calculated for all boreholes, site class (NBCC 2005) was determined to be C – very dense soil and soft rock.

3.5.5. Groundwater

Groundwater seepage was noted in five test pits, two in the upland areas of the heap leach facility (TP-BGC09-HL6-8 and TP-BGC11-133) and three in the valley bottom (TP-BGC11-131, TP-BGC11-134 and TP-BGC11-135). A number of standpipe piezometers were installed in 2011 in the footprint of the proposed heap leach facility. In addition, a number of standpipe piezometers have been installed in the footprint by Stantec (Stantec 2010). Typical groundwater observations for the area around the HLF are compiled in Table 14.

Table 14. Groundwater Observations in the General Area of the Proposed Heap Leach Pad (data compiled from Stantec 2010, BGC 2012)

Location	Well ID	Typical Groundwater Depth (m below ground surface)
Upland	MW09-AG1[1]	15.4
	MW09-AG2[1]	13.6
	MW10-AG3a[1]	9.9
	MW10-AG5[1]	7.0
	MW10-AG6[1]	12.6
	BH-BGC11-26[2]	16.4
	BH-BGC11-29[2]	7.8
	BH-BGC11-52[2]	4.1
Valley Bottom	MW09-DG1[1]	2.6
	MW09-DG2[1]	2.5
	MW10-DG06[1]	3.4
	BH-BGC11-30[2]	16.3
	BH-BGC11-32[2]	10.8
	BH-BGC11-33[2]	4.2
	BH-BGC11-34[2]	8.3
South Side of Heap Leach	BH-BGC11-55[2]	>12.7

Notes:

1. Stantec water levels are average water level since installation.
2. Water levels in BGC holes were measured in late August 2011.

The observed groundwater depths on the open slopes in the upper Ann Gulch valley range from 4.1 m below grade close to the middle of Ann Gulch to 15.4 m in the headwaters of Ann Gulch. Water levels in the Dublin Gulch valley bottom are variable, but can be expected to be closer to ground surface near streams and deeper below piles of tailings. It is anticipated that these levels will vary seasonally. The groundwater table has not been observed in the south corner of the heap leach facility on the south side of Dublin Gulch.

For preliminary design, it may be assumed that the natural groundwater table will be encountered at approximately 10-15 m depth below grade in the uplands, and at close to the elevation of existing drainage courses in the valley bottom. However, groundwater can be expected to be encountered locally at shallower depths, specifically when approaching the main drainages. This variability should be considered in planning, design and construction.

3.5.6. Permafrost

Frozen ground was encountered in the upper part of the HLF footprint (i.e. Upland area) in test pits TP-BGC09-A1, TP-BGC09-HL6-04, -09, -10, -15, TP-BGC10-28, -42, TP95-53 and -58, and boreholes BH-BGC11-59 and MW10-AG5. When observed in a plan view, many of the test pits are located on the eastern slope of Ann Gulch, and all except for TP-BGC10-28 align in a NE trend, covering the entire HLF footprint, from its most eastern edge to its western end at the heap leach containment dike. The reason for this connection between the frozen ground observations is unknown and might simply correspond to sporadic disconnected patches; nevertheless the continuity of the linear feature may deserve to be studied in more detail and accounted for during site preparation and construction. Frozen ground was typically encountered within gravels and gravels and sands with depths varying between 0.6 m to 2.8 m, and occasionally included excess ice. Test pit TP95-58 encountered visible ice encountered between 6.7 m to 7.3 m depth.

Frozen ground was not encountered in the valley bottom or on the southern edge of the proposed heap leach pad, but localized pockets of frozen ground may be present in these areas, particularly in areas where natural vegetative cover has not been disturbed by prior mining activities.

3.5.7. Geological Hazards

Around the HLF, geological hazards as determined by Stantec (2010) mainly include permafrost processes in the west-facing slopes at the upper part of the valley and surface seepage at the bottom of the valley between the rockfill diversion berm and rockfill embankment Drawing 20. Some of the south-facing lower and steeper slopes above Dublin Gulch are affected by rockfall and rockslide hazards (Drawing 20).

3.6. Water Diversion Structure

3.6.1. General

The water diversion system consists of a rockfill diversion berm and velocity reduction pond which will divert water coming from Dublin Gulch into a diversion channel. The channel carries water on the south side of Dublin Gulch, adjacent to the events ponds prior to discharging into Haggart Creek.

Overburden conditions encountered along the proposed diversion channel alignment, east of Stuttle Gulch, are generally different than those encountered further west in the valley bottom. The first segment is located at a higher elevation containing primarily colluvium and till; whereas, the second segment is underlain by placer tailings (fill, see Drawing 11 and Drawing 13). The ground conditions for the second (lower) segment of the diversion channel and sediment ponds are discussed in Section 3.7.

Ground conditions at the proposed Dublin Gulch diversion berm and velocity reduction pond are similar to those encountered at the valley bottom component of the heap leach pad.

Subsurface conditions in the area of the proposed Dublin Gulch diversion to the Stuttle Gulch energy dissipation structure are summarized below in Table 15.

Table 15. Summary Subsurface Observations in Proposed Dublin Gulch Diversion Area.

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[6] to Type 3 Rock (m)	Depth[6] to Type 2 Rock (m)	Depth[6] to Type 1 Rock	Total Depth[6] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
TP-BGC09-HL4-2[4]	910	0.3	>2.0	-	-	-	-	-	-	2.3	Yes
TP-BGC10-17[4]	873	0.1	-	>1.5	4.4	-	-	-	-	6.0	No
TP-BGC10-18[4]	877	0.2	0.3	>7.0	-	-	-	-	-	7.5	No
TP-BGC10-19[4]	899	0.2	>7.3	-	-	-	-	-	-	7.5	Yes
TP-BGC10-20[4]	905	0.2	0.4	-	-	-	0.6	-	-	3.2	No
TP-BGC10-21[3]	895	0.1	-	-	>6.4	-	-	-	-	6.5	No
TP-BGC10-22[3]	884	0.1	0.8	-	-	0.6	1.5	-	-	5.3	No
TP-BGC10-32[3]	902	0.1	-	-	>7.9	-	-	-	-	8.0	No
TP-BGC10-40[4]	816	-	-	-	>5.5	-	-	-	-	5.5	No
TP-BGC11-88[4]	922	0.2	>5.8	-	-	-	-	-	-	6.0	No
TP-BGC11-92[4]	933	0.2	>1.5			N/A	N/A	N/A	N/A	1.7	Yes
TP-BGC11-93[4]	917	0.4	>1.3			N/A	N/A	N/A	N/A	1.9	Yes
TP-BGC11-104[4]	832	0.1	0.6	2.8	0.3	3.5	-	-	-	5.6	No
TP-BGC11-110[4]	942	0.1	4.9	-	-	-	5.0	-	-	5.0	No
TP-BGC11-131[3]	921	0.1	>3.4	-	-	-	-	-	-	3.5	No
TP-BGC11-136[3]	910	0.1	>4.7	-	-	-	-	-	-	4.8	No
TP-BGC11-137[4]	943	0.1	1.0	-	-	1.4	2.5	5.0	-	5.0	No
DH-BGC09-DG-2[4]	828	-	-	-	14.6	-	-	14.6	-	16.3	No
BH-BGC10-5[3]	884	-	-	-	4.3	-	-	4.3	-	21.0	No
BH-BGC10-6[4]	876	-	-	16.4	-	-	16.4	22.9	-	28.9	No

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[6] to Type 3 Rock (m)	Depth[6] to Type 2 Rock (m)	Depth[6] to Type 1 Rock	Total Depth[6] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
BH-BGC10-15[3]	893	NR				-	-	8.8	-	21.0	No
BH-BGC10-16[4]	878	NR				1.5	9.9	10.5		28.0	No
BH-BGC11-41[4]	914	0.3	2.5	-	-	-	2.8	4.3	-	5.0	No
BH-BGC11-52[3]	909	-	11.3	-	-	0.6	11.9	14.3	19.5	22.6	No
BH-BGC11-53[4]	876	-	-	11.4	-	-	11.4	-	-	14.5	No
BH-BGC11-55[4]	881	-	8.8	-	-	-	8.8	-	-	14.5	No
TP96-127[4]	909	0.4	>5.1	-	-	-	-	-	-	5.5	Yes
TP96-129[4]	904	0.2	-	-	>4.9	-	-	-	-	5.1	Yes
TP96-130[4]	893	-	-	-	1.5	-	1.5	1.8	-	1.8	No
TP96-131[4]	901	-	2.3	-	1.5	-	3.8	4.2	-	4.2	Yes
TP95-49[4]	886	-	>4.9	-	-	-	-	-	-	4.9	Yes
DH95-152[4]	865	-				-	12.2	-	-	30.2	No
GT96-13[4]	904	-				-	-	12.2	18.3	36.3	No
MW96-15a[4]	943	-	4.5	-	-	-	4.5	-	-	9.2	No
MW09-STU2[4]	857	-	4.0	>6.1	-	-	-	-	-	10.1	No

Notes:

1. "NR" = no recovery
2. not observed or not applicable
3. Test holes relevant to the proposed rockfill diversion structure
4. Test holes relevant to the proposed diversion channel
5. Frozen ground observations from older test pits (TP95-XX or TP96-XX) may not reflect current conditions.
6. Depths indicated are below existing ground surface.

3.6.2. Overburden

The current diversion berm arrangement, as shown in Drawing 11, is located entirely within the approximate extent of placer tailings. There is a thin organic layer approximately 0.1 m thick underlain by placer tailings with thickness varying between 4.3 m to greater than 7.9 m. The tailings are generally loose to compact silty sands and gravels and soft to firm sandy silts. Recorded Standard Penetration Test (SPT) blowcounts, N, are summarized in Table 17 for the placer tailings within the footprint of the proposed process management ponds.

The first segment of the diversion channel runs along the north facing slope, south of Dublin Gulch, at an elevation of approximately 900 m, and is generally outside the extent of placer tailings. The overburden consists of a thin horizon of organic soil ranging from 0.1 to 0.4 m thick and averaging approximately 0.2 m. The organic cover is underlain by colluvium ranging in thickness from 0.3 m to 8.8 m, with an average thickness of approximately 3.5 m. Colluvium is described as a loose to compact gravelly sand with some silt to gravelly silt with some sand with occasional cobbles and boulders. Glacial till is observed locally west of Eagle Pup and east of Stuttle Gulch along the proposed alignment of the diversion channel. The observed thickness of the till unit varied between 2.8 m to 16.4 m. In this area till is described as being a firm to stiff (or compact to dense) silt and sand with some gravel.

3.6.3. Bedrock

The bedrock near the diversion berm was observed at a maximum depth of 11.9 m in borehole BH-BGC11-52 and a minimum depth of 1.5 m in TP-BGC10-22. The rock is described as slightly to moderately weathered metasedimentary rock (W2 – W3), weak to medium strong (R2 – R3), and with very closely spaced discontinuities and is Type 3 and Type 2 rock. The rock mass rating (RMR ’76) ranges from 20 to 50 with an average rating of about 40. For the mapped geological structures in this area refer to Drawing 19.

In the proposed diversion channel footprint, metasedimentary bedrock was encountered at depths ranging from 0.6 m to 16.4 m, averaging 13.1 m. Type 3 rock is present up to depths ranging from 0.6 m to 22.9 m with Type 2 rock below. Type 1 rock was encountered in two boreholes at depths of approximately 19 m. An average RMR value of approximately 35, ranging from 19 to 54, was determined from the observed rock core (BH-BGC10-6 and BH-BGC10-16).

Shear wave velocities were measured in one borehole (BH-BGC11-52) to a depth of 21 m. Although this depth is insufficient to calculate a Vs30 value, a shear wave velocity of 439 m/s was used to approximate a site class (NBCC 2005) of C – very dense soil/soft rock.

3.6.4. Groundwater

Within the diversion berm footprint, seepage was observed in two test pits at a depth of 3.0 m. Groundwater is expected to be close to the existing grade in the valley bottom near existing drainages and deeper further upslope on either side of the valley.

Within the proposed diversion channel footprint, seepage was observed in six test pits at depths ranging from 0.1 m to 5.5 m. A standpipe piezometer installed in BH-BGC11-55 downslope of the diversion channel alignment was dry to a depth of 12.7 m in August 2011.

3.6.5. Permafrost

Frozen ground was not encountered in test pits and boreholes within the footprint of the proposed diversion berm. Frozen ground was encountered in nine test pits along the proposed diversion channel alignment.

3.6.6. Geological Hazards

As shown in Drawing 20, the geological hazards identified by Stantec (2010) that might affect the construction of the diversion berm include surface seepage within the footprint of the placer tailings.

For the upper segment of the diversion channel, the presence of permafrost may affect construction and operation, while surface seepage in creek crossings will need to be considered also.

3.7. Events Ponds

3.7.1. General

The proposed event ponds are located immediately downstream (west) of the heap leach pad and below (south of) the process plant (Drawing 11), and are to be constructed in the Dublin Gulch valley bottom, between Stuttle Gulch in the east and Haggart Creek to the west.

The overburden soil encountered in the vicinity of the proposed process management ponds area mainly comprises placer tailings and occasional colluvium or till. Subsurface conditions in the area are summarized below in Table 16.

Table 16. Summary Subsurface Observations in Proposed Process Management Ponds Area

Test Hole ID	Approx. Elev.[1] (m)	Overburden thickness (m)					Depth[3] to Type 3 Rock (m)	Depth[3] to Type 2 Rock (m)	Depth[3] to Type 1 Rock (m)	Total Depth[3] (m)	Frozen Ground
		Organics	Colluvium	Till	Placer Tailings	Completely Weathered Rock					
BH-BGC11-32	820	-	-	-	19.8	-	19.8	-	-	24.4	No
BH-BGC11-65	820	-	-	-	>6.9	-	-	-	-	6.9	No
BH-BGC10-13	824	-	1.1	-	11.1	1.1	12.2	-	14.9	19.5	No
DH-BGC09-DG3	844	-	-	-	12.1	-	12.1	16.2	-	20.7	No
TP-BGC09-DG3	837	-	-	-	>5.0	-	-	-	-	5.0	No
TP-BGC10-38	830	-	-	-	>4.8	-	-	-	-	4.8	No
TP-BGC10-39	825	-	-	-	>5.5	-	-	-	-	5.5	No
TP95-43	822	-	-	-	>5.5	-	-	-	-	5.5	No
TP95-44	828	-	-	-	>5.5	-	-	-	-	5.5	No

Notes:

1. Approximate ground elevation is inferred from available digital elevation model based on assumed approximate horizontal position.
2. N/A – not observed or not applicable.
3. Depths indicated are below existing ground surface.

3.7.2. Overburden

The placer tailings within the footprint of the events ponds and lower segment of the proposed diversion channel, above Dublin Gulch, have a variable thickness up to 19.8 m.

The placer tailings encountered within the footprint of the events ponds are generally a well graded, loose to compact, sand and gravel with some fines and some cobbles. Table 17 below summarizes the available SPT N-value for the boreholes within the area of the proposed events ponds. Detailed records of recorded N values can be found on the borehole logs in BGC's site investigation data reports (BGC 2011, 2012).

Table 17. Summary of Standard Penetration Test N-values for the placer tailings within the Process Management Ponds Footprint

Borehole ID	Depth Interval tested (m)	USCS	Number of Tests Meeting Refusal	N-value (raw blowcount, blows / 300 mm)	
				Average	Standard Deviation
BH-BGC10-13	0.8 – 5.0	GW, trace SW	1	30	8
BH-BGC11-65	0.8 – 6.9	SW/GW	2	21	6

3.7.3. Bedrock

Bedrock was encountered underlying the placer tailings within the footprint of the proposed events ponds in boreholes BH-BGC09-DG3, BH-BGC10-13 and BH-BGC11-32 (Drawing 11). Depth to bedrock ranged between 12.1 m and 19.8 m below existing grade. The placer tailings surface is highly variable and the majority of holes were completed on top of piles of placer tailings. Based on shear wave geophysical surveys, the typical thickness of placer tailings within the events ponds is approximately 10 m.

Observed bedrock consisted of moderately to highly weathered metasedimentary rock (i.e. Type 3 rock as described above (Section 1.2). Type 2 rock was encountered at 16.2 m in DH-BGC09-DG-3. Type 1 rock was encountered in a depth of 14.9 m in BH-BGC10-13. The metasediments are moderately to strongly foliated highly fractured.

Shear wave velocities were measured in one borehole (BH-BGC11-32) to a depth of 21 m. Although this depth is insufficient to calculate a Vs30 value, a shear wave velocity of 367 m/s was used to approximate a site class (NBCC 2005) of C – very dense soil and soft rock.

3.7.4. Groundwater

Groundwater was observed at approximately 3 m depth in two test pits within the valley bottom (TP-BGC09-DG3 and TP95-44).

A standpipe piezometer was installed in BH-BGC11-32; the groundwater level in this hole observed at 10.8 m below existing grade. BH-BGC11-32 is located near the crest of a placer tailings pile. The groundwater table is expected to be at or near the elevation of the Dublin Gulch surface water course in the vicinity of the proposed events ponds.

3.7.5. Permafrost

While frozen ground was not observed within the placer tailings in the valley bottom, isolated patches of permafrost may be encountered.

3.7.6. Geological Hazards

The geological hazards identified by Stantec (2010) that might affect the construction of the process management ponds are limited to surface seepage within the footprint of the placer tailings (Drawing 20).

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