FMC237

Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex Vangorda Creek Diversion

Design Report- Draft

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and

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Vangorda Creek Diversion
Design Report - Draft

Government of Yukon

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1 Introduction

1.1 General

The Yukon Government (Energy, Mines and Resources Assessment and Abandoned Mines Branch) has assumed responsibility for the Faro Mine Complex, and Denison Environmental Services (DES) has been awarded a contract to provide care and maintenance services to Yukon Government at this site. As part of the ongoing work, a number of projects are planned as part of the early remediation of the project to reduce the risk of impacts to the environment. One of these projects is the construction of a permanent diversion of Vangorda Creek at the Vangorda /Grum mine site as shown on Figures 1 and 2.

The existing flume diverts Vangorda Creek around the pit. Failure of the flume during a major flood event, could result in excess water the flooding of the pit with a significant increase in the volume of contaminated water. SRK has examined a number of alternative long-term options for this diversion. The current preferred design (as of February 2010) for the new diversion follows a near-surface route varying in grade from 1.5 percent in the upper reaches to a relatively steep grade of approximately 11 percent in the lower reach of the Phase 1 construction. A plan view of this alignment is shown on Figures 3 and 4 and the profile is shown on Figure 6.

The proposed channel (February 2010 design) is located to the north of the Pit and would be constructed in two phases. The first phase would extend from a new intake structure located about 500 m north of the existing diversion headworks on Blind Creek road to the existing plunge pool and dropbox structure at the haul road. The total length of the initial phase of the diversion would be about 1325 m. The future second phase of the work would extend the diversion by about 180m to intersect the original Vangorda Creek. YG contracted SRK Consulting (Canada) Inc. (SRK) to carry out a geotechnical field investigation and to provide preliminary engineering designs for the construction of the new diversion. The geotechnical field investigation component was carried out between May 28 to June 19, 2009 and involved the excavation of a number of test pits, as well as, a drilling program along the centerline of the originally proposed diversion alignment. During the field investigation, Yukon Engineering Services (YES) completed a land survey along the alignment of the proposed diversion. This report presents a preliminary design of the Vangorda Creek Diversion, based on the YES survey and the results of the recent field investigation, and focuses on moving ahead with the concepts behind the currently adopted design.

1.2 Background

The development of the Vangorda Pit in the early 1990s required the diversion of Vangorda Creek around the perimeter of the pit. Between 1991 and 1992, the diversion was realigned due to the changing footprint of the pit. Since the cessation of mining activities at Faro Mine in January 1998, the diversion channel has been maintained in order to ensure a slow rate of pit filling while a long-term management plan for the site is developed. In 1999, there was a rock fall from a near vertical slope, which overlooks one section of the flume. This rock fall necessitated emergency replacement of approximately 39 meters of the flume.

The flume is also subjected to continuous pressure annually from ice build-up. The cross braces have buckled and many of the seals between each of the flume sections were damaged. Seepage loss from the flume is partially collected in an underdrain beneath the lower reaches of the flume but most of the seepage actually flows beneath the flume discharging into the plunge pool at the outlet of the diversion. The quantity of leakage is considered minor and is believed to not significantly impact the groundwater nor the stability of the pit walls.

The existing Vangorda Creek Diversion system consists of the following components:

- Headworks comprising an 8 m high earth dam and a 1.5 m diameter culvert within the dam which directs the water into an 800 m long, 2400 mm dia. half round CSP culvert section.
- A plunge pool or stilling basin located at the end of the culvert sections.
- A 3000 mm diameter drop box structure, a 2000 mm CSP culvert, and a 1600 mm outfall culvert convey the discharge from the plunge pool beneath the haul road and back into Vangorda Creek.
- Two 1000 mm diameter culverts at the headworks provide an emergency spillway to the Vangorda Pit for events that exceed the 100 year event.

The following list summarizes the key parameters of the original design for the flume:

- The diversion channel was designed to accommodate the 1:100 year event, with a peak instantaneous flow of 10.0 m^3/s.
- The diversion channel was designed to be reasonably watertight. Hence, the use of a half-round corrugated steel pipe (CSP).
- The upstream headworks was designed to retain water to the 1:100 year level of 1168 m allowing for one metre of freeboard. The dam crest was built to Elevation 1169 m.
- A 1.5 m diameter CSP was designed to convey the water through the upstream collection dam.
- The main diversion channel was designed with the CSP flume in a riprap-lined trapezoidal section. A longitudinal slope of 0.5% was selected to ensure subcritical flow within the section, which was considered preferable for this application. However, the as-built grade is steeper than 0.5% in the lower reaches of the flume, which could cause overtopping of the channel during supercritical flow conditions. Icing conditions and uplift of the CSP flume in the flatter sections of the diversion are chronic maintenance issues.
- During the 100-year flood event the depth of water flow in the channel is expected to rise to a maximum of 0.73 m.

In general, since construction of the flume, the system has successfully conveyed Vangorda Creek during normal runoff events. However, in June 2004, a rainfall-on-snow flood, estimated to have a return period of 100 years, damaged the piping system and nearly overtopped the headworks dam. The following year, an overflow spillway was installed in the headworks dam to prevent overtopping and redirect the flow into Vangorda Pit.

This event also necessitated reconstruction of the flume and upgrade to the culvert that empties into the drop box. The original design and alignment has not changed.

1.3 Project Schedule

TBD

2 Site Description and Current Conditions

2.1 Location

The centerline of the proposed 'long-term' Vangorda Creek Diversion is located on the hillside approximately 50 to 100 m upgradient of the existing Vangorda Creek flume. The location and existing surface conditions are shown in Figures 3 and 4.

2.2 Site Conditions

2.2.1 Field Investigation

To determine the site conditions of the proposed Vangorda Creek Diversion alignment, a geotechnical investigation was completed in June, 2009 by SRK. The field work involved field reconnaissance, higher resolution survey coverage, the excavation of 12 test pits, and the completion of seven boreholes and additional laboratory testing on a glacial till and two silty sand samples.

Additional survey coverage was gained along the length of the proposed Vangorda Creek Diversion with emphasis placed on the areas near the most north-eastern end of the alignment, (expected entrance of Vangorda Creek to the diversion), as well as near the most southwest extents, (where the diversion will again reconnect with the natural alignment of Vangorda Creek).

The test pits were excavated using a thumbed CAT 345 C Excavator, operated by Denison Environmental Services (DES) personnel. The depths of the test pits varied from about 1.25 to 6.4 m below the existing ground surface.

Boreholes were drilled through the use of a 3-person crew using Geotech Drilling Services Ltd's track mounted double rotary Fraste Multi Drill – XL, with 'overburden drilling with eccentric bit' ODEX and diamond core drilling capabilities. The depths of the boreholes ranged from about 11.0 m to 21.3 m below the existing ground surface. During ODEX drilling cutting samples were retrieved and Standard Penetration Tests (SPT), as well as split spoon sampling was performed. During diamond core drilling, core was recovered and logged. Standpipe piezometers/monitoring wells were installed at four locations along or near the VCD alignment at borehole locations SRK09-DHVD01, 04B, 05 and 06.

Test pit and borehole locations are presented in Figure 3 and detailed findings from the geotechnical investigation can be found in Appendix I.

2.2.2 Surface and Topography

Upstream and downstream of the proposed diversion, the channel of Vangorda Creek has a gradient of about 5%, a base width of approximately 4 to 5 m and sideslopes of 2.5 horizontal (H): 1 vertical (V). On the sides of the creek, alluvial sand gravel, cobbles and boulders up to 1.2 m diameter have been historically observed (SRK, 1990). Near the intake location of the proposed Vangorda Diversion (see Figures 3 and 4), the elevation of the natural creek is about 1199 m. The sidehills slope down from north to south varying in grade from 15 to 25%.

The majority of the ground along the most northern half of the proposed Vangorda Creek Diversion alignment appears to have been largely undisturbed by mining activities. The more southern portion of the proposed diversion has been moderately disturbed by mining activities such as roads and buildings. Undisturbed ground along the formally glaciated hillslopes neighbouring Vangorda Pit are well vegetated with native trees, scrub and low profile vegetation. The vegetation along the southern portion of the hillsides in closer proximity to the Vangorda Pit, is less dense than the northern slopes.

2.2.3 Subsurface

Soil

The soil encountered in the test pits and drillholes generally followed a sequence of organic rich topsoil overlying glacial till above metamorphic bedrock consisting predominantly of phyllite. The phyllite bedrock generally was subjected to moderate fluid flow and occasional igneous granodiorite intrusions were encountered at depth. In the lower 200 m of the proposed diversion, the overburden primarily consists of predominantly run-of-mine waste rock. It should also be noted that near the north-central portion of the alignment there is a deep section of silty sand, as illustrated in the profile along the alignment in Figure 6. Generalized soil descriptions are presented below:

- **Topsoil**: dark brown to black, fibrous, consisting of an organic matt with soft silts, clays, some sand and some to trace gravel. Frozen sections were often observed near the base of this unit. These frozen sections are believed to be remnants of the winter frost. Permafrost is not expected in the new alignment.
- **Glacial Till**: greyish brown to medium brown moderately to well graded, silty sand to sandy silt with gravel, sub-rounded cobbles, *boulders*, some clay and exhibiting generally medium plasticity. Glacial till is the main unit comprising the overburden, however near the southern

extents of the diversion, near the existing plunge pool, there is also a road/fill material component to the overburden.

- **Silty Sand**: this unit appears to be glaciofluvial or glacialoutwash brown-yellow permeable silty sand with some to trace gravel and clay. The plasticity of this unit varies from low to extremely low.

Bedrock

The bedrock-overburden contact is at greatest depths in the north-central and southern extents of the proposed Vangorda Diversion alignment, the estimated bedrock contact is illustrated in the Figure 6 profile. Generally the bedrock in this area can be characterized as phyllite. The till becomes shallower near the central portion of the alignment and the bedrock outcrops near the proposed intake structure.

In general, RQD (Rock Quality Designation) is poor, ranging from 25 to 50% with some thicknesses of very poor RQD characterized by higher intensity fractures and planar foliation breaks.

A slightly higher fair quality RQD, with an expected range from 50 to 75% and slightly lower hydraulic conductivities, are expected to be encountered near the proposed intake structure. However, the latter is based primarily on visual observations as drilling access was restricted to this area. More foundation and abutment investigation is expected to be required prior to construction.

In order to determine the acid rock drainage (ARD) potential of the rock and soil removed during the excavation of the channel, samples were tested from five holes drilled during SRK's Vangorda Diversion Field Investigation in June 2009. A total of 17 samples were collected representing three types of material: granodiorite, phyllite and till. Samples of till were collected from bagged material from the upper portions of the drill holes. All samples were submitted to CEMI in Vancouver, BC. Results of the testwork are provided in Appendix II.

The results indicate that most of the potentially acid generating (PAG) samples have an acid potential (AP) of less than 10 kg/tonne which is probably representative of all of the rock in the region. It is concluded that the exposed rock in the excavation for the diversion is unlikely to produce appreciable amounts of acidity or metal leaching – especially from the walls. However, it is recommended that the PAG rock removed during the diversion excavation be disposed of in the Grum dump and incorporated under the cover. It is also recommended that extraction tests be performed on the weathered phyllite to assess metal content and to confirm that a mitigation plan would not be required for the bedrock sideslopes of the new channel.

The depth to the water table was not rigorously assessed during the field investigation. However, a number of piezometers were installed in some of the drillholes to allow the water table to be monitored prior to construction. Water inflows into the excavations should be expected, especially around the central portion of the proposed diversion alignment (0+850 towards 1+050).

3 Design Considerations and Objectives

3.1 Review of Project Alternatives

The original closure plan for the Vangorda Pit presented in the 1990 Water Licence involved the redirection of Vangorda Creek back into the pit to form a lake. Flow was to then discharge via a spillway into Vangorda Creek on the south side of the pit. Portions of the pit walls and pit floor were to be covered with till to control metal leaching and acid generation. Based on the results of an ARD evaluation completed by SRK in 2003, it was determined that flow through the pit has the potential of meeting water quality objectives in Vangorda Creek but would require many ancillary measures including maximum input dilution, clean up of the waste rock dumps and pit walls and insitu water treatment of the initial pit lake. As it was recognized that a more rigorous geochemical assessment of this option would be required before it could be considered a serious contender, the Pit lake (Flow Through) option was not included in a review of diversion alternatives prepared for Deloitte and Touche dated April 2003.

In the April 2003 study the following alternative schemes were evaluated:

- Option 1: Upgrading the existing flume diversion in an open channel around the north perimeter of the Vangorda Pit, removing the drop box structure, excavating through the existing haul road and relocating the plunge pool;
- Option 2: Rediverting the creek back along the alignment of the original creek bed and constructing an open channel within the partially backfilled Vangorda Pit; and
- Option 3: Realigning Vangorda Creek into an open channel located above the Vangorda Pit over to Dixon Creek to the south.

As the recommended closure plan is based on a stabilize-in-place approach, Option 2 was excluded from further evaluation. Option 3 was also excluded based on a study completed by EBA Engineering in August 2005, which concluded that the stream flow of Dixon Creek is so small that there is no continuous surface channel of Dixon Creek for a considerable distance downstream from the proposed point of entry of the diversion channel. Significant potential environmental impacts to the upper watershed of Dixon Creek by the diversion of the full flow of Vangorda Creek would require extensive and major engineering works to mitigate.

The approach adopted for the proposed diversion of Vangorda Creek presented in this report is a variation of Option 1, the main difference being the final alignment. Option 1 had the Vangorda Creek routed north of the pit, closely following the present route of the Vangorda Creek Diversion. The channel would have been slightly re-aligned and also widened and deepened. A plunge pool was to be constructed at the bottom of the diversion to dissipate energy before discharging into Vangorda Creek. The risk of eventual pit failure remained a concern and the decision was made to push the alignment further north outside any possible impact of a pit wall failure. Several secondary benefits would also be realized with this alignment. Firstly, the new diversion could be constructed while maintaining the existing diversion channel. Secondly, the original diversion channel, although no longer protected by the half-round CSP flume, would remain to provide overflow capacity in the event of a flood event occurring that exceeds the selected design capacity of the new diversion. Thirdly, the original channel would minimize inflows to the Vangorda Pit by intercepting leakage from the upper diversion and also runoff generated by the small intervening catchment between the intake structures of the new and old diversions.

SRK concluded that while the proposed alignment presents issues related to energy dissipation and erosion protection because of the steep grade in the approximately 500 m long lower reach, measures to mitigate the associated risks can be incorporated into the channel, and it remains one of the two preferred options. Compared to the current diversion, the proposed system will accommodate higher flows, will minimize the impact of glaciation, provide a more natural stream appearance and require less maintenance, over the long term. Currently, Vangorda Creek flume requires annual upgrades and maintenance.

This report primarily focuses on the design of a diversion channel for a proposed alignment completely beyond the perimeter of the Vangorda Pit. The single most challenging element of this new alignment is the steep reach at the lower end of the diversion. Given uncertainties associated with the design of erosion protection for steep channels, SRK was requested to investigate alternatives to the current design to identify layouts with milder maximum slopes. The memorandum prepared to summarize the alternatives analysis is presented in Appendix VIII. The key finding of the memorandum was that the so-called pushback option (Option 1 described above in Section 3.1) should be retained as a candidate option for providing a closure diversion. The pushback option would run along benches of the open pit, but would have a maximum longitudinal gradient of about 7%, or much less than required for a channel completely outside the perimeter of the pit. The risk of pit wall failure could potentially be mitigated by provision of a buttress at the toe of the north pit wall.

3.2 Flood Hydrology

3.2.1 Introduction

The overall diversion system has been designed to accommodate the 500-year flood (estimated peak instantaneous flow of 30 m^3/s). However, given the challenges of designing a channel to safely pass such a large event down a slope of 11% (Phase 1 construction), it was decided to design the new channel to accommodate a lesser event such as the 200-year flood (peak instantaneous discharge of 13 m^3/s). In the event that floods occur with peak flows in excess of 13 m^3/s, the design incorporates a feature in the headworks of the diversion that will direct the excess flow down the old Vangorda Creek channel. This flow would then pass along the route of the existing flume diversion, which will be retained after closure. If the bypass flows exceed the capacity of the existing diversion, flow will be directed into the Vangorda Pit.

The following hydrological studies were completed:

- A technique for estimating the instantaneous peaks of floods on Vangorda Creek for a range of return periods from 100 to 500 years.
- Summary of a recent comprehensive flood study prepared by the Alberta government. This study is relevant to the design of the Vangorda Diversion because it can be used to help identify potential conservatism in the flood estimates made for the Vangorda Diversion.
- A technique for estimating the volume characteristics of floods on Vangorda Creek for a range of return periods. The need for this second flood-estimation technique is related to the design of the proposed intake structure for the diversion, as explained later in this section.

Figures developed in support of the hydrological analyses are presented in Appendix V. The remainder of this section presents the estimates of flood hydrology that have been adopted for the design of the new diversion channel.

3.2.2 Peak Instantaneous Flood Estimates

The intake of the proposed diversion channel, as shown on Figure 3, will control a drainage area of 17.7 km^2 . The incremental area draining laterally along the full length of the diversion (at end of Phase 2) will be approximately 1.8 km^2. Thus, the total catchment area for the proposed diversion is estimated to be 19.5 km^2 (see Figure 5).

The flood hydrology of Vangorda Creek was estimated using a technique known as Regional Analysis. This entailed performing three broad tasks. Firstly, a search was made of government streamflow gauging networks to locate stations that measure flows from relatively small drainages and have long periods of record. To find an adequate sampling of such stations, the networks of Environment Yukon, Water Survey of Canada and the US Geological Survey were searched. The second task involved fitting the annual series of peak flows at each regional station to theoretical frequency distributions to estimate peak flood rates for a range of return periods from 2 to 500 years. The final step involved identifying trends that could be used to transpose the flood estimates at the regional streamflow gauging stations to Vangorda Creek. Appendix V presents the working plots used to identify the key trend upon which the flood hydrology was transposed, namely plots of unit flood discharge versus contributing catchment area. Using the figures presented in Appendix V, the peak instantaneous discharges for the 200-year and 500-year floods were estimated to be 13 m^3/s and 30 m^3/s, respectively. SRK has adopted a nominal design life for the new diversion of 100 years. The chances of floods greater than the 200-year and 500-year floods occurring in a 100-year period are 40% and 18%, respectively.

The existing Vangorda Diversion has been in operation for 19 years. The largest flood over this period occurred on June 8, 2004. Using high water marks at three locations along the diversion, the instantaneous peak of this flood was estimated to fall in the range of 10 m^3/s to 12 m^3/s.

3.2.3 Flood Volume Estimates

As stated above, the new diversion has been designed to accommodate the 200-year flood event. However, the overall diversion system will be designed to survive the 500-year flood event. During this latter event, flow down the new diversion will be limited to 13 m^3/s (i.e., magnitude of 200-year peak instantaneous discharge) with the remaining flow being directed into the existing channel of Vangorda Creek. This excess flow will then pass into the existing flume diversion and rejoin the new diversion at the Phase I plunge pool. If the flow exceeds the capacity of the retained channel, the excess flow would enter the Vangorda Pit.

Increases in the amount of water allowed to enter the pit will result in a decrease in the maximum discharge rate that the new diversion would be required to handle. A preliminary trade-off analysis was carried out to determine an optimum split between these two components of flow.

The handling of extreme floods with both the diversion and the pit storage allows for a smaller design discharge to be specified for the diversion channel than would otherwise be possible. As a first approximation, the capital costs associated with the construction of the diversion can be considered to be proportional to the design discharge. The decreased capital costs associated with the resulting smaller diversion would, however, be offset by increased water treatment costs because the water entering the pit would have to be treated prior to being released to the lower Vangorda Creek catchment. A limit exists as to how far the tradeoff between the diversion size and pit storage can go. The amount of water allowed to flow to the pit cannot exceed the buffer storage that would be dedicated in the open pit for dealing with such water. Otherwise, the risk exists that the open pit would fill and spill contaminated water to the receiving environment.

For the first iteration of the new diversion design, the assumption was made that the diversion would be sized to handle all flows up to the 200-year event (or 13 m^3/s). If the flow event exceeds 13 m^3/s, then the headworks has been designed to divert the excess flow down the original Vangorda Creek Channel towards the existing diversion and the Vangorda Pit. The diversion of this water would be controlled by an erodible fuse plug built into the headworks dam. The plug has been designed to readily erode during the rising limb of an extreme flood at the point where the 200-year flood magnitude has been exceeded. The final design of the headworks dam and the erodible plug will require further analysis of the structure's hydraulics using a backwater model. However for the purpose of this design report, the hydraulics were approximated using the weir formula for a broad-crested trapezoidal weir. A potential geometry for the spillway would be:

- base width of permanent spillway = 12 m;
- sideslopes of permanent spillway = 2H:1V;
- elevation of base of fuse plug, or crest elevation of permanent spillway = 0.55 m above local invert of diversion channel; and
- elevation of crest of fuse plug = 1.40 m above local invert of diversion channel.

Figures 11 and 12 illustrate the design concept.

With this configuration, the full flood hydrograph of all events with an instantaneous peak of 13 m^3/s or less would be directed to the diversion. No flow would pass to the open pit.

When the flow in Vangorda Creek exceeds 13 m^3/s at the new diversion intake (an event expected to occur about once every 200 years), water would begin to overflow the fuse plug and would attack the highly erodible downstream slope of the plug. The material associated with the fuse plug would be removed, leaving a wide spillway available to divert flow to the existing flume and possibly the open pit. The removal of the fuse plug would reduce the driving head, thus reducing the inflow to the new diversion below the design discharge.

If the 500-year flood hit the Vangorda Creek, the fuse plug would erode during the rising limb of the flood hydrograph. When the flood reached its peak (30 m^3/s), a flow of 13 m^3/s would be directed to the diversion, while the remainder of the flow would pass over the spillway and eventually reach the existing diversion and the open pit.

Overflow to the open pit would continue until the flow in Vangorda Creek dropped to about 3.5 m^3/s. At that point, the water level in the stream would drop below the crest of the spillway and all of the Vangorda Creek flows would be directed into the new diversion. Within a few weeks of an event that activated the overflow spillway, a new fuse plug should be constructed within the headworks.

An analysis was undertaken to estimate the volume of water that could potentially spill to the open pit if the diversion system was hit by the 500-year flood. The basis of the prediction was a flood frequency analysis of eight streamflow records at gauging stations within the Yukon and east central Alaska. An emphasis was placed on utilizing the records of stations with small catchments and long records. The flood frequency analysis is presented in Appendix V.

The flood frequency analysis provided the basis for estimating the shape of the 500-year flood hydrograph at a daily interval. For the purpose of the present analysis, it was only necessary to identify the portion of the hydrograph with daily average flows greater than 3.5 m^3/s (i.e., greater than the flow required to activate the spillway of the intake dam once the fuse plug has been removed). An examination of the constructed hydrograph revealed that a total of four days during the 500-year flood would have daily average flows greater than the threshold required to activate the spillway (see Appendix V for more details). Thus, the excess flow to the pit would amount to about 970,000 m^3.

To put this quantity into perspective, the average annual inflow volume to the Vangorda Pit is about 420,000 m^3. The average annual flow of Vangorda Creek at the inlet of the new diversion is approximately 7 million m^3. Thus, the estimated spill to the open pit during a 500-year flood would be roughly equal (on a volume basis) to roughly double the average annual inflow to the pit, and about 14% of the average annual flow of Vangorda Creek at the proposed intake location. Assuming the pit is at or near to the recommended maximum level, this would result in a water level rise of about 12 meters. The adopted estimate of the 500-year flood hydrograph was based on envelope curves and, therefore, is probably a conservative estimation of the true flood regime of Vangorda Creek (i.e., probably high estimates of actual flow during period when flows would be great enough to cause spillage to the open pit).

It should be noted that with minor changes to the erodible plug dimensions, expected after further modeling has been completed, slight changes to the predicted excesses flow volume may result. The final estimate of the excess flow into the pit would be presented/confirmed once the embankment arrangement has been finalized.

To consider this information in an economic analysis, it is necessary to estimate the long-term average spill to the pit that would occur as a result of flood events that caused erosion of the fuse plug. If the erosion is to be initiated during events with instantaneous peaks of 13 m^3/s (200-year return period), then one would expect the fuse plug to fail 2 or 3 times during a 500-year period. The average inflow to the pit during the failures would probably be less than the 970,000 m^3 computed above (i.e., some of the exceedances would likely be caused by events between the 200-year and 500-year floods). If, as a first approximation, it is assumed that all spill events result in a flow volume of 970,000 m^3 to the pit, then the total inflow over a 500-year period would average about 2.4 million m^3. If averaged over 500 years, the average annual inflow to the pit from spillage is calculated to be 5,000 m^3, or 1.2% of the current average annual inflow rate to the pit.

Accordingly, the penalty for not designing the diversion ditch to handle the instantaneous peak of the 500-year flood is that the mine will have to treat an extra 5,000 m^3 of water per year, on average. Given these favourable results, the adoption of the 200-year flood event for the new diversion over of a 500 year event is an acceptable risk.

3.3 Headworks

3.3.1 Dam and Erodible Plug Functions

The principal function of the new headworks at the intake of the new Vangorda Creek Diversion is to direct flow into the diversion and to limit the flow to a maximum of 13 m^3/s using an erodible plug. The primary design objectives for the headworks structure are:

- Long-term stability;
- Seepage control;
- Accommodation of excessive flow from primary Vangorda Diversion channel; and
- An erodible plug.

These objectives are further developed in the following sections.

3.3.2 Consequence of Failure and Long-Term Stability

Failure of the entire headworks dam embankment is very unlikely to result in any fatalities, so the "environmental, socioeconomic and financial" consequences are most relevant. Failure of the headworks dam would release 'geochemically clean' water and sediment into the downstream environment. The environmental impact would be localized and of short duration. It is expected that if a full failure of the headworks resulted that most of the sediment would settle out as it progresses downstream through the existing diversion channel (retained after the steel flume sections have been removed), or will settle in Vangorda Pit. By applying the stability criteria provided by the Canadian Dam Association (CDA) and applying the CDA guidelines shown in Table 1, a "very low" consequence of failure classification was adopted for the Headworks structure.

Table 1: CDA (1999) Dam Classification in Terms of Consequences of Failure

Consequence Category	Potential Incremental Consequences of Failure [a]	
	Life Safety[b]	Socioeconomic, Financial & Environmental[c]
Very High	Large number of fatalities	Extreme damages
High	Some fatalities	Large damages
Low	No fatalities anticipated	Moderate damages
Very Low	No fatalities	Minor damages beyond owner's property

Notes to Table 3.1

a) Incremental to the impacts which would occur under the same natural conditions (flood, earthquake or other event) but without the failure of the dam. The consequence (i.e. loss of life or economic losses) with the higher rating determines which category is assigned to the structure. In the case of tailings dams, consequence categories should be assigned for each stage in the life cycle of the dam.

b) The criteria which define the Consequence Categories should be established between the Owner and the regulatory authorities, consistent with societal expectations. Where regulatory authorities do not exist, or do not provide guidance, the criteria should be set by the owner to be consistent with societal expectations. The criteria may be based on levels of risk which are acceptable or tolerable to society.

c) The owner may wish to establish separate corporate financial criteria which reflect their ability to absorb or otherwise manage the direct financial loss to their business and their ability to pay for damages to others.

As is discussed more fully in the design section of this report, the headworks dam is expected to be constructed primarily of general fill consisting of a granular material that is obtained from either borrow area(s) or from works excavation that is free of organics, well graded and heterogeneous with a grain size distribution that meets the fill specification (see Technical Specifications document). Foundation settlements are expected to be negligible in the phyllitic bedrock observed around the headworks location. However, some foundation modification to ensure stability and to control settlement should be expected.

The slope stability requirements for earth and rock fill dams specified by the CDA guidelines and by the International Committee on Large Dams (ICOLD) are summarized in Table 2 below.

Table 2: Minimum Factors of Safety from CDA (1999) and ICOLD

Loading Condition	Minimum Factor of Safety	Slope
Steady state seepage with maximum storage pool	1.5	Downstream
Full or partial rapid drawdown	1.3	Upstream
End of construction before reservoir filling	1.3	Downstream and Upstream
Earthquake (pseudo-static)	1.1	Downstream

The maximum height of the headworks structure is about 3m with sideslopes of 2:1 (H;V). As the dam is lined and as seepage through the dam would be minimal, conformance of the structure to the above stability criteria has been established.

3.3.3 Seepage Criteria

Seepage control through the foundation and through the embankment is provided by using an upstream cutoff and a synthetic liner. Seepage through the embankment would be limited primarily by the upstream cut-off trench and low hydraulic gradient due to the length of the flow path by which water must travel under the dam.

3.3.4 Erodible Plug and Accommodation of High Flow Events

Extreme floods will be handled by a combination of diversion and temporary storage of water in the Vangorda Pit. To implement this strategy, the headworks will have to incorporate a feature that will split the flow into two components, one directed to the diversion and the other directed to the open pit. Two methods were evaluated for splitting the flow: a fixed-invert spillway and a spillway partially filled with an erodible plug (also known as a fuse plug).

Early in the design process, the fixed-invert spillway was rejected because it would lead to a higher frequency and greater volume of spill to the open pit than would be experienced with the spillway and fuse plug combination. To limit the flow down the diversion to 13 m^3/s for all events up to and including the 500-year flood, the invert of the spillway would have to be set at a fairly low elevation. Without a fuse plug, the water level in the stream would rise above this invert on a nearly annual basis, thus resulting in frequent spills to the pit.

An erodible plug in the headworks dam embankment would minimize the number of times the overflow spillway would be activated. The crest of the erodible plug would be set at an elevation that would only allow overtopping during events greater than the peak of the 200-year flood, or a discharge of 13 m^3/s. The erodible section should be designed to collapse and fail gradually as it is overtopped. As the structure fails, the excess flood discharge can then be released without compromising the main diversion structure. Limiting the maximum flow in the new diversion to 13m^3/s provides a safety factor that would ensure the functionality and stability of the stepped chute structures proposed for the steep grades in the lower reaches of the of new diversion. Specific design criteria adopted herein for the erodible plug design are:

- Hydraulic capacity of overflow spillway equal to difference between 200-year and 500-year peak instantaneous discharges, or 17 m^3/s.
- Geometry and material placement required to ensure failure of the erodible section.
- Foundation requirements to withstand the erosive action of flow over the washed out erodible plug section during the extreme flood event.
- The requirements for an exit chute/apron designed to lead flow from the erodible dam section into the previous or original alignment of Vangorda Creek.
- Minimum requirements for long-term maintenance.

3.4 Energy Dissipation

3.4.1 Alternative Solutions

To achieve an optimized alignment for the proposed Vangorda Creek Diversion, SRK evaluated a number of channel configurations, alignments and grades. Parameters including the volume of excavation, hydraulics, constructability, static and geotechnical stability were assessed. Consistently the most favourable diversion arrangements exhibited a low grade section near the diversion intake, transitioning into a long steeper grade of 11% in the lower Phase 1 reaches.

Conveying the diversion design discharge of 13 m^3/s down an approximately 460 m long Phase 1 channel segment with a 11% grade requires some form of energy dissipation. A number of alternative solutions for energy dissipation were evaluated. These included a uniform-sloped chute, a stepped chute cut into the bedrock and the glacial till sections (the latter being concreted) and a series of step-pools.

The uniform chute option was discarded early in the assessment due to the high velocities and large sized riprap that would be required. The option to construct concrete steps was considered but was dropped due to the high volume of concrete required. It also proved to be very costly and impractical. The study then focused on the two remaining options: the stepped chute and step-pools. Both of these options would require cutting into the bedrock and the overburden (till with some smaller sections of road fill material expected). Appendix VIII presents some additional information on the alternatives analyzed.

As shown in longitudinal profile on Figure 6, the steep slope section is expected to be partly in bedrock from about STA 0+870 to STA 1+200 and partly in overburden from STA 1+200 to STA 1+300.

Review of diamond drill holes completed in the bedrock shows phyllite rock materials that are highly foliated and of variable strength between R3 and R4 (25 – 100MPa), with frequent foliation parallel jointing, and occasional sub-vertical jointing. Based on core alpha angles, the exposed rock mass will have a foliation orientation that is sub-horizontal to horizontal. Experience at other cold temperature sites with similar bedrock shows the phyllite to be susceptible to damage related to freeze-thaw cycles with water entering the open foliation/jointing, and splitting the rock along the sub-horizontal foliation planes. This process will be aggravated by the cutting of 'steps' into the phyllite. The phyllite would typically not erode much due to water flow, but it is felt that the freeze-thaw cycles will initiate this erosion especially in the areas of the cut steps. Although the erodibility potential of the phyllitic bedrock can be assessed, as presented in Appendix III, it is more difficult to determine the rate of erosion and hence the requirement for the placement of riprap layer over the bedrock in the steps to slow the erosion process.

A discussion on the two options is provided below.

Step-Pools

Step-pools exist in nature and in theory would appear to be an ideal solution. Until recently, little was known about the forms and processes of step-pool channels. However, over the past two decades significant advances in the theory of step pool sequences have been made (Chin et al., 2008). In evaluating this option SRK conducted an extensive literature review for a step-pool design. Appendix VI presents a memorandum summarizing the preliminary step-pool design. A comprehensive list of the papers utilized in assisting to establish the concepts, theory and criteria for the step pool design is presented in the reference section of this report.

As commented by NHC in their review of the Step-pool concept (See Appendix VII), most of the studies are qualitative and lack the essential detail to provide an acceptable comfort level for a final design. There is a general lack of engineering based criteria for step-pools. By observing what happens in nature, step-pools are very effective in dissipating energy. However, the stability of any riprap lining that would be used as erosion protection in the pools, against the turbulent flow conditions is difficult to determine. The other concern is the stability of the large boulders that would be used to form the steps against rotation or toppling. Furthermore if there is a failure of one step-pool, a progressive failure of the subsequent pools (56 in Phase 1) could occur.

Stepped Chute

For the long steep slope section we have in the lower reaches of this diversion, a stepped chute cut into the bedrock and overburden is the preferred solution. The primary concern with this approach is the erodibility of the bedrock and the glacial till overburden. As discussed above, the phyllite bedrock would typically not scour significantly as a result of water flow, but there is a high potential for erosion caused by freeze-thaw cycles, due primarily to the near-horizontal orientation of the bedrock fractures. We are unaware of any method for reliably estimating the rate of bedrock erosion caused by freeze-thaw cycles. Given the concerns of the stability of the riprap lining in the step-pool approach plus the potential for glaciation in the pools that will be left undrained, the stepped chute approach remains the preferred option despite the erodibility potential of the bedrock foundation. Design details of the stepped chute approach are provided in Section 4.

3.4.2 Plunge Pools

The primary function of the plunge pool in Phase 1 of the Vangorda Creek Diversion is to dissipate energy in the flow prior to entering the existing drop box system at the base of the diversion. The existing plunge pool at the base of the flume diversion will be expanded to accommodate flow from the new diversion and any overflow from the existing diversion. The plunge pool locations are shown in Figures 3 and 4.

The base and sideslopes of the plunge will be lined with riprap to provide erosion protection and resistance to excessive scouring.

3.5 Erosion Control

Erosion protection using riprap will be required in most sections of the new diversion to ensure the long-term stability of the structures. Following the guidelines set by the United States Department of Agriculture (USDA, 2007) and the British Columbia Ministry of Environment, Lands and Parks (BC MELP, 2000) consideration of riprap size, material strength, density, angularity, durability, geometric ratios, gradation, bedding, piping potential, and channel curvature were included in the design and construction considerations. Riprap sizing is presented in the design section of this report.

3.6 Surface Water Management

It is expected that in a number of locations along the proposed alignment, hillside runoff above the diversion will need to be directed in the new channel. This will be achieved by the construction of riprap lined channels or half round flumes running down the sideslopes of the diversion. In addition, where applicable, the side slopes of the diversion should be hydroseeded. Hydroseeding will not only increase the aesthetics of the diversion channel but will give the benefits of increased slope protection, surface water management and slope stability.

4 Design

4.1 Overview

The proposed horizontal alignment for the new diversion is shown in Figures 3 and 4. Several vertical and horizontal alignments were examined to optimize the cut and fill volumes. As shown on Figure 6, the chosen vertical alignment follows a near-surface route varying in grade from 1.5 percent in the upper reaches to a relatively steep grade of 11% in the lower reach of Phase 1 before entering the plunge pool and the dropbox system.

As discussed in Section 3, the selected design flood for the new channel is the 200-year event which is estimated to have peak flow of 13 m^3/s.

As discussed in Section 3, a stepped chute approach is presented in this report as the preferred option for energy dissipation in the steep sections of the diversion. As explained earlier, a number of mitigative measures have been included in the design to provide structural stability of the channel particularly in the steeper sections and to provide reasonable erosion control. Details are provided in Section 4.5.

Riprap is the primary means of erosion control where applicable. The riprap size and depth vary depending on the channel grade and configuration. Nonwoven geotextile filter fabric has been specified where riprap is placed on sand or cohesive soils such as the glacial till. No filter fabric has been specified where riprap is placed directly on the weathered or more competent phyllite bedrock. The design will also include placement of a synthetic LLDPE liner (approximately 2,400 m^2) which would be placed over the section of the channel that is to be excavated in the silty sand (see Figure 8). The channel is primarily cut with the exception of two short sections along the channel where fill is required on both sides. In general, the channel has sideslopes of 2H:1V and base widths varying from 3 to 6 m. Those sections in competent rock would have sideslopes of 1.5H:1V. In those sections of the channel where the total length of the sideslope exceeds the practical reach of an excavator, 5 m wide benches have been included in the design. These benches would double as road access on either side of the channel. Phase 1 of the projects ends at the existing plunge pool which will be upgraded to accommodate flow from the new diversion. The pool will be widened and deepened where required. Riprap erosion protection will be placed on the base and sideslopes of the upgraded pool. The existing outlet from the plunge pool is a 2000 mm diameter CSP culvert which will be retained until Phase 2 of the project is constructed. Also retained will be the 3000 mm drop box structure and the 1600 mm culvert beneath the haul road which takes the flow to the original Vangorda Creek channel. In Phase 2 of the project, the haul road will be removed and the new channel will be extended through the breached road to a new plunge pool before discharging into Vangorda Creek.

It is estimated that Phase 1 of the project will require about 160,000 cubic meters of excavation and the placement of about 10,000 cubic meters of riprap. The Stepped Chute approach for the steep section of the diversion will require the placement of boulders in Phase 1. It is estimated that about 300-475 boulders that have an average diameter in excess of 1 m will be required. Most of these boulders would be sourced from within the mine site. The total excavation for both phases is estimated to be about 206,000 m^3.

Subsurface conditions along the alignment vary from glacial till to both weathered and competent graphitic phyllite bedrock. The test pits indicated that the weathered phyllite would be easily excavated by machine. Although the more competent phyllite encountered proved difficult for the excavator in the test holes, it may be possible to rip the material when completely exposed. For the purposes of this design report, it is assumed that there is about 26,000 m^3 of phyllite rock (weathered and competent) that will need to be removed in Phase 1. It is estimated that about half of this will require blasting.

4.2 Channel Entrance and Headworks

Figures 11 and 12 (Drawings V-11, V-12), shows a plan and detailed section views of the channel entrance and headwork components.

4.2.1 Channel Entrance

The Vangorda Creek Diversion channel will be field fit to maximize the diversion intake contact with Vangorda Creek. The channel invert elevation of 1199.6 m will be tied into the natural Vangorda Creek. Based on field observation of competent bedrock outcrops near the creek edge, some degree of blasting effort is expected to be required. The diversion inlet will have a downward gradient of about 1.5% and will require riprap protection on the side slopes as well as on the base of the channel. Similar to station 0+000 the base of the channel will be approximately 3 m wide with 1.5H: 1V side slopes along the northwest extents.

4.2.2 Primary Earth Dam Embankment

Prior to construction of the primary earth dam embankment "knobs" of bedrock currently upstream of the future dam site will likely have to be removed/blasted to the 1199 m elevation.

The design for establishing the crest elevation was determined from the site topography as well as from the peak discharge for the 500 m flood event. The east access road traverses adjacent to the Vangorda Diversion until it transition onto the top of the dam surface.

A temporary cofferdam will be set up to divert Vangorda Creek into the constructed Vangorda Diversion Channel. The cofferdam construction will allow for dry working conditions required for the placement of the dam fill material. Any water retained behind the coffer dam would be pumped into the constructed diversion.

Figure 11 (Drawing V-11, section H) details the embankment design. The existence of the headworks will force water to form a pool at the diversion intake. This pool will act as a bedload trap so that sediments may fall out of suspension and decrease the bedload transport into the diversion. Having a lower bedload transported in the diversion is favourable for the long-term stability and functionality of the stepped chute structures. To allow for a minimum freeboard of 1 m the crest of the dam embankment has been established at 1202.1 m.

Some foundation work may be required, such as stripping or grouting, to ensure that the dam has adequate foundation conditions. To reduce seepage loss through the foundation, an upstream cutoff and installation of a synthetic liner would be constructed. This cutoff would extend across the whole width of the creek and be somewhat similar to an impervious upstream apron. The synthetic liner would be installed at a 2H:1V slope down 3 m vertically from where the toe of the dam embankment touched the existing ground. The expected trench created to construct the cutoff and install the liner would be backfilled with compacted suitable general fill. Basic liner tie-in details are presented on Figure 13, and are also outlined in the Preliminary Technical Specifications Draft document. The proposed dam embankment would be constructed with 2H:1V upstream and downstream side slopes. The crest width would be approximately 10 m. The main section of the embankment would be constructed of general fill. The upstream and downstream slopes would be protected by sandwiching two layers of geotextile filter fabric between a 60 mil LLDPE liner (approximate quantity 960 m^2). Riprap would then be placed on the upstream and downsteam slope to protect the liner and add additional protection for the expected overtopping associate with the failure of the erodible plug. A minimum riprap thickness of 0.6 m should be placed on the upstream and downstream faces.

Further finite element stability modeling will be performed to check the preliminary dam design. This modelling is scheduled to be completed before the final dam design drawings are issued for construction.

4.2.3 Erodible Dam Plug

For this preliminary stage of design, the hydraulics of the headworks were approximated using broad-crested weir formulae. The hydraulic calculations were set up to represent the conditions at the arrival of the instantaneous peak of the 500-year discharge, 30 m^3/s. At this point, the fuse plug would have already failed and been washed away. One of the key findings of the calculations was the base width of the overflow section of the dam would have to be about 17 m (further details presented in section 3.2.3).

The function of the erodible plug is to divert flows in excess of 13 m^3/s first into the existing Vangorda flume alignment then secondly into the Vangorda Pit. In addition to the diversion of excessive flow, the erodible plug acts as a safety measure to accommodate the possibility of glaciation processes blocking the upper diversion reaches.

The preliminary modeled hydraulics for the erodible plug section are an approximation of the true hydraulics. Before release of the final design drawings, a more accurate representation of the headworks hydraulics will be made using a backwater model (e.g. the US Army Corps of Engineers HEC-RAS program).

The base of the erodible plug has been established at el. 1200.15 m, or 0.55 m vertically above the diversion channel invert at its inlet. The top elevation of the erodible plug would be at 1201.00 m, or the water level at the headworks when Vangorda Creek is flowing at a rate of 13 m^3/s. Therefore, the overall height of the erodible plug design section is 0.85 m.

The fuse plug consists of a thin 2m downstream inclined till core with a highly erodible sand and gravel material behind it to ensure that effective erosion and washout results. The key element when constructing the erodible plug is the relatively impervious core. The till core prevents washouts from occurring for discharges less than the 200-year design flood. As the impervious core is expected to be generally above the normal water level against the upstream dam face, the core has the potential to dry and crack. Filters consisting of sand and gravel are suggested to cover the core to prevent piping and premature washout (Khatsuria, 2005), thus sand and gravel is specified on either side of the inclined core. The predefined erodible plug extents are designed to fail in a controlled manner. Slope protection consisting of riprap and the geomembrane liner is placed on both sides at 2H:1V slope. In addition to the slope protection, these coarse materials are used to assist in segregating the components of the non-overtopping portion from the highly erodible body of the plug.

Utilizing the 13 m^3/s flood discharge through the erodible plug opening, treating it again as a broad crested weir and by employing the model studies conducted by USBR, the lateral erosion rate (after the initial breach) for a given erodible plug embankment design and flow depth can roughly be estimated by ER = 14.6*Hf + 48 (where ER = lateral erosion rate in m/hr, and Hf = height of the fuse plug in meters) (Khatsuria, 2005). By utilizing the latter empirical relationship we can roughly estimate that for our fuse plug the rate of failure might be expected around the magnitude of (14.6*0.7 + 48 =) 58 m/h. These estimations have evolved from model structures (Khatsuria, 2005). It is understood that over time the fuse plug material will slightly compact due to traffic, vegetal growth and from the weight of the armoring material. This would again reduce or lower this lateral failure rate (experienced after the initial breach). The time for the initial breach is harder to estimate and therefore slight allowances have been made to ensure that the erodible plug section overtops and washes out at the time the design discharge hits.

In the event that the erodible plug is overtopped and washes away, expected to happen approximately every 200 years, all flows in Vangorda Creek greater than about 3.5 m^3/s will result in spillage at the headworks. The fuse plug should be reconstructed within a few months of its failure.

The performance of the erodible plug in the headworks dam is critical to the overall design of the diversion. It is therefore recommended that a live test be carried out on the headworks dam in the first year to provide comfort that the plug will erode when it should. This test would involve damming off the entrance to the new diversion to force overtopping of the fuse plug portion of the headworks dam and failure of the erodible section. Assuming the plug erodes, the headworks dam should be repaired and the erodible section of the dam rebuilt.

4.2.4 Exit Chute/ Apron

Figure 11 shows the area where an exit chute would be constructed from the erodible plug section of the earth dam embankment into the original Vangorda Creek Channel. This channel requires riprap protection. To ensure that the channel does not clog by the eroded dam plug material and to provide a more favourable geometry, a portion of the neighbouring slope is expected to be excavated or blasted. As well, the riprap can be utilized to develop additional buttressing force against the downstream toe portion of the diversion dam which will remain after the erodible plug has washed-out. Due to the presence of the downstream geomembrane liner the function of the riprap becomes primarily to protect the liner and not the main dam embankment itself. A portion of the neighbouring slope is expected to be excavated or blasted to provide a more favorable chute geometry to promote flow into the original Vangorda Creek alignment.

4.3 Channel Configuration and Layout

A schematic of the Vangorda Creek Diversion channel layout is presented in Figure 4 (Drawing V-4). Figures 7 to 10 (Drawings V-6 to V-9) show "typical" design cross-sections at various stations along the alignment. The long section or profile is presented in Figure 5 (Drawing V-5). These designs represent the general design approach based on the anticipated geotechnical site conditions. Slight modifications or field-fits are expected to be made at the discretion of the on site engineer. The Vangorda Creek Diversion has an approximate length of 1506 m and drops from el. 1199 m at the northern headworks intake to el. 1102.5 m at the southern re-entrance into Vangorda Creek.

Site conditions and the Manning's equation were used to determine the ditch geometry based on a design flow rate of 13 m^3/s. Given the site-specific topography, the diversion was broken into three segments, based on the optimized and most practical final design grade. In general, the channel has sideslopes of 2H:1V and base widths varying from 3 to 6 m. However, in those sections which are in competent rock, sideslopes of 1.5H:1V can be constructed. The channel grades and geometries utilized in this design report are presented in Table 3 below.

All sections of the channel were designed to contain the maximum flow depth plus 1.0 m of freeboard. The 1.0m freeboard was adopted to increase the design factor of safety and to allow for possible variance in the sideslopes from 2H:1V to 1.5H:1V. Additionally this 1.0 m of freeboard will allow for increased flow capacity while the erodible section of the dam is progressing to failure, likely to be briefly required when a greater than 200 year storm event occurs. In the event that some basal freezing results in the channel, then again this freeboard should limit/avoid channel overflow.

Table 3: General Channel Configuration Details

Channel Reach	Approximate Length	Slope Grade	Base Width	Side Slopes**	Freeboard
Upper	0+000 to 0+820	1.50%	3 m	2H:1V	1 m
Transition	0+820 to 0+875	5.26%	3 to 6 m	2H:1V	1 m
Lower (Phase 1)	0+875 to 1+325	11%	6 m	2H:1V	1 m

*The general channel configuration for the lower Phase 2 section is currently designed at~13% however the alignment and final grade of this section is expected to be slightly redesigned based on Phase 1 construction.
** Side slopes expected to be reduced to 1.5H:1V in bedrock sections.

Side slopes will be steeper in a portion of the upper 1.5% grade reach where bedrock is expected to be encountered, approximately from station 0+000 to 0+075. For the lower 11% Phase 1 reach a 6 m channel base has been specified to accommodate stepped chute construction and to reduce the expected depth of flow on this steep section. Areas where the channel passes through soil or weathered phyllite bedrock will be covered with non-woven geotextile in addition to the required riprap armoring. Over the sections of the channel excavated in the silty sand (See Figure 8) the placement of a synthetic LLDPE liner (approximately 2,400 m^2) is specified.

Based on piezometric data and field investigation observations, some degree of excavation dewatering is expected to be needed to facilitate construction of the Vangorda Diversion. Secure dewatered conditions would likely assist in speeding construction and assist in ensuring proper engineered as-built conditions. A dewatering plan should be further developed with contractors when construction commences.

Due to the associated diversion construction activities there is a heightened potential for the total suspended solids to be temporarily increased as the diversion first experience the flows diverted from Vangorda Creek.

4.4 Riprap Requirements

The requirements for riprap can generally be divided into five categories as follows:

- Headworks riprap;
- Riprap required for the 1.5% grade, 3m wide base channel typically with 2H: 1V side slopes;
- Riprap required for the 11% grade, 6m wide base channel with 2H: 1V side slopes to be constructed in Phase 1. (This is fully detailed in section **4.5 'Stepped Chute'**);
- Riprap required for the 5.26% grade, 3 to 6m wide base, 2H: 1V side sloped transition zone channel; and
- Riprap required for the plunge pool structures.

As suggested by the USDA and BC MELP riprap guidelines, a riprap blanket thickness of 2 x D50 has generally been adopted for all areas, excluding section of the stepped chute and for protection of the headwork dam liner. It should be noted that D_x is defined as the particle size such that x% of the particles in the soil or rock matrix are smaller than. The USDA sizing method was adopted at this stage of the design for determining the main riprap size requirements. The USDA riprap sizing method assumes angular rock is used; larger sizes are required for round rock. The required riprap size is a function of the roughness (or Manning's n), depth of flow and flow velocity. Estimates of channel roughness were made using the empirical relationship developed by Rice et al. 1998, resulting in estimates of Mannings's n falling in the range of 0.04 to 0.065. Table 4 summarizes the basic riprap requirements for components of the Vangorda Creek Diversion.

Table 4: Basic Riprap Requirements for the Vangorda Creek Diversion

Rip-rap area	Description	Requirement	Comment
Headworks	Upstream face of dam	D_{50} = 0.54 m	Angular, FOS=1.2
	Downstream face of dam	D_{50} = 0.60 m	Rounded, FOS≈1.2
Diversion Channel	1.5% upper reach	D_{50} = 0.30 m	Rounded, FOS>1.2
Diversion Channel	11% lower Phase 1 reach	D_{50} = 0.63 m	USACE , FOS≈1.2
Diversion Channel	5.26% transition reach	D_{50} = 0.50 m	Rounded, FOS>1.2
Plung Pool	Phase 1	D_{50} = 0.50 m	Angular, FOS≈1.2

Piping of the soil underlying the riprap could lead to failure of the riprap protection. The segments of the diversion excavated in the phyllite bedrock are typically expected to not require a riprap filter layer, with the possible exceptions of areas where the bedrock is observed to be highly fractured. Where the alignment overlies overburden, a non-woven geotextile is required as a filter layer for the riprap. Based on field observations, the glacial till appears to be poorly graded and, as a result, a long-term filter may self-form as the geosynthetic deteriorates.

4.5 Stepped Chute

The approach would involve constructing a stepped chute in the bedrock and overburden foundation in the steep sections of the diversion (STA 0+870 to STA 1+325). Each step would be a nominal 1m in height and the bench length between steps would be 9.0 m for Phase 1 construction (spacing will likely be reduced to around 7.0 m for Phase 2 construction). The base of the channel would be 6 m in width.

Each step in the bedrock sections will be cut out of the phyllite. At each step, large boulders (mean diameter of 1.3 m) would be concreted into the foundation. These boulders are mandatory in the overburden sections; however, in the bedrock sections these have been included as a recommended contingency item at this stage. The boulders in the bedrock section would act as hardpoints to mitigate erosive effects on the bedrock. These boulders will be placed in an arch formation to provide additional stability through the transfer of a portion of the fluid forces to the banks of the channel. To stabilize these boulders against sliding or rotation, it is proposed to install 100 mm diameter concrete filled, galvanized steel pipes up against the boulders. The aforementioned pipes are referred to in this document as 'retaining piles'. The retaining piles would be buried to depths of about 3m into the bedrock. The bench sections of the stepped chute in bedrock would be excavated with a relatively flat grade. No measures to prevent erosion of the phyllite bedrock exposed in the flat bench sections of the chute would be applied. However, it is proposed that an adaptive management approach be taken for the bedrock erosion potential. The rate of erosion would be carefully monitored during the first few years of operation and if it was found that the rate of bedrock erosion was impacting the structural integrity of the diversion, mitigative measures such as placing a concrete surface over the flat section of stepped chute would be implemented.

The portion of the stepped chute founded on overburden would require erosion protection. It is proposed to provide this protection with a minimum 0.5 m thick layer of concrete grouted or mortared riprap placed on the flat bench section of the chute. Concrete filled galvanized pipes would also be required to stabilize the anchor boulders, however, the embedment depth would increase to about 5 m. Voids in boulders forming the steps would be grouted. During construction of the stepped chute, a high degree of site management and supervision is required to ensure that the energy dissipation measures are constructed in accordance with the specifications.

Figures 13 shows a plan and sections through a typical section of the stepped chute structure. As well, Figure 14 shows a typical profile through a section of the stepped chute. The stepped chute introduced will cause the dissipation of energy and create grade control during high flows. When developing the stepped chute design, the risks and benefits associated with changes to the step arrangements, geometry, and material sizing were taking into account.

4.5.1 Step Chute Boulder Sizing

The boulders were sized using two force diagrams, one assuming failure by sliding and the other assuming failure by rotation. The algorithms for applying the force diagrams are documented in papers by Lenzi, 2001 and Fischenich and Seal, 2000. The rock sizes estimated by these techniques are probably conservative because they look at the rock in isolation. They do not account for additional resistance forces realized by the interlocking contacts with neighbouring rocks, which can be substantial. The published techniques indicated that boulders forming the steps would require diameters ranging from 1.0 to 1.3 m.

4.5.2 Boulder Placement

Four to six boulders are required per step. The boulders should be arranged in a broad upstream facing U or V or crescent shape with pointed, curved or apex pointing upstream. This crescent shape takes advantage of the additional strength gained from the "jammed state" (Church , 2007) while promoting the alignment of the flow towards the centre of the channel, thereby helping to maintain a constant downstream scour position and limiting excessive bank erosion.

The boulders should be placed along the side of the channel and between these boulders on the outside channel flanks. Transverse boulder work is required in the overburden foundation sections to limit lateral erosion around the steps. The step boulders should be ideally be 'seated', dug in the overburden section or cut into the phyllitic bedrock and concreted in place. Based on site conditions in the overburden sections additional riprap and boulder material may need to be placed against the downstream face of the step to provide additional buttressing support and to reduce the likelihood of scour undermining the steps. The latter will have to be reassessed during construction and will be determined if required at the discretion of the onsite engineer. Some preferable boulder arrangements are detailed in Figure 13 and 14. Note that the long or A-axis of the rock should ideally be orientated parallel to the direction of flow to obtain the most stable configuration.

4.5.3 Stepped Chute Geometry

The weir height, or the difference in elevation of crests of neighbouring steps should be 1 m. Steps should be created consecutively with a crest to crest spacing or step length (L) of approximately 9 m (1m weir height/ 0.11 slope grade = 9 m). The base of the channel should be 6 m. Figure 13 illustrates the stepped chute in plan and typical section. The profile detailed in Figure 14 illustrates the arrangement of the dimensions required to derive the desired 11% slope grade. On average the channel slopes between steps are expected to be constructed with slight slope to limit/avoid the formation of a pool/ponding.

4.5.4 Stepped Chute Lining

It is proposed that a minimum 0.5 m thick layer of riprap be placed on the flat bench overburden section of the chute. This 0.5 m does not account for some material being slightly embedded into the overburden foundation. The riprap would be concrete grouted or mortared in place. The riprap specifications (D_{50} and gradation) should be consistent throughout the full length of the chute. Owing to the importance of interlocking forces between rocks, a weak zone in the chute could lead to a catastrophic failure of the chute. Again, concrete grouting or mortared in place has been specified to increase the factor of safety for the rip-rap lining specified in the overburden sections, and to reduce the potential chance of castrophic chute failure. Based on the USACE method a chute with a 11% slope, 6 m base width and a design discharge of 13 m^3/s would require a D_{50} of 0.63 m. Riprap protection with a D_{50} of 0.63 m has been determined to remain stable on a chute.

4.6 Plunge Pool

Figure 15 shows details and the typical arrangement of the plunge pools. The end of Phase 1 construction will tie into the existing plunge pool. The water level will be controlled by the inlet of the pipe leading to the drop structure, which in turn feeds the culvert running under the haul road. At the design discharge of 13 m^3/s, the water level in the pool would rise to approximately elevation 1127.8 m. The base of the plunge pool will be set a minimum of 0.30 m below the invert of the existing 2000 mm CSP pipe. The primary function of the plunge pool in Phase 1 of the Vangorda Creek Diversion is to dissipate energy of flow existing the stepped chute and to generate adequate head to pass flow into the pipeline leading to the drop structure. The riprap lining of the existing pool is expected to be upgraded to a minimum thickness of 0.5 m and the sideslopes regarded to a maximum slope of 2H:1V. Overall, the final base geometry of the Phase 1 plunge pool is expected to be about 25 m in width by approximately 25 to 30 m in length.

In Phase 2, a plunge pool will be constructed at the outlet of the diversion in order to dissipate the energy of the water before it re-enters Vangorda Creek south west of the pit. Generally the base of this pool is expected to be 25 m in length by approximately 15 m in width with 2H:1V sideslopes and will be lined with riprap ($D50 \approx 0.5$ m).

4.7 Access Road

In order to construct the channel in those sections with long side slopes, 5 m wide benches have been incorporated into the design. These benches will also provide permanent access roads to the channel.

4.8 Quantity Estimates

The quantities and volumes used for the Vangorda Creek Diversion project were based primarily on the results of the 2009 SRK field investigation findings, through the use of Gemcom GEMS© 3D modelling and AutoCAD Civil 3D© drafting software, as well as through cross-section checks along the alignment. All volumes in Table 5 and 6 below are reported as in place volumes; no bulking factors have been applied.

The Vangorda Creek Diversion (VCD) was divided into three regions and then further subdivided into the two construction phases. The estimated excavation volumes are summarized in Table 5 for the final four options assessed.

Table 5: Comparison of Estimated Excavation Volumes for Various VCD Geometries

Option	Geometry				Volumes (m^3)		
	~0+000 to 0+825 at a 1.5% grade		~0+875 to 1+506 at a ~11% grade				
	Base Width (m)	Side Slopes	Base Width (m)	Side Slopes	Phase 1	Phase 2	Total
1	6	2H:1V	3	2H:1V	159,902	42,066	201,968
2	3	2H:1V	3	2H:1V	150,171	38,632	188,803
3	3	2H:1V	3	1.5H:1V	123,484	38,632	162,116
4	6	2H:1V	3	1.5H:1V	133,210	42,066	175,276

Notes:

1. The transition zone geometry from station 0+825 to 0+875 was kept constant
2. Table does not account for any overexcavation.

To more fully satisfy the design criteria, Option 1 was selected out of the options detailed above in Table 5. Table 6 below presents a detailed breakdown of the estimated geological materials expected to be encountered during construction, as well as estimates of the required cut/excavation, fill and riprap volumes for Option 1.

Table 6: Preliminary Summary of Estimated Vangorda Creek Diversion Volumes

Item		Volume (m^3)	
		Unit	Total
Excavation/Cut	Topsoil	30,592	201,968
	Overburden/Till	130,942	
	Silty Sand	11,588	
	Phyllite/Bedrock	28,846	
Fill	Phase 1	13,353	13,353
	Phase 2	0	
Riprap	Phase 1	9,180	11,456
	Phase 2	2,276	
Step Boulders	Phase 1	474	697
	Phase 2	222	
Stepped Chute Over-Excavation	Phase 1	2,837	3,960
	Phase 2	1,123	

Notes:

1. The Volumes presented are solely for the Vangorda Creek Diversion channel and do not include the headworks components.
2. Step boulders are assumed to be halfway between a sphere and a cube.

Table 7 below presents a preliminary breakdown of estimated material quantities expected to be encountered and utilized during the headworks construction.

Table 7: Preliminary Summary of Estimated Headwork Volumes

Item			Volume (Bm^3)	
			Unit	**Total**
Foundation and Key Trench	Excavation/Blast	Topsoil	0 - minor	250
		Overburden/Till	150	
		Weathered Bedrock	50	
		Drill/Blast Bedrock	50	
Embankment	Main Embankment	General Fill (not bulked)	1,272	1,272
	Protection	Riprap	360	360
	Erodible Plug	Sand and Gravel	92	110
		Compact Core Material	18	
Other	Exit Chut/Apron	Riprap	918	918

4.9 Cost Estimate

The equipment rates in Table 8, were used to develop the unit rates for handling soil and rock material for the various material types. Equipment rates are based on the rates published by the BC Government in the 2008-2009 Blue Book. Unit rates are based on in-house experience, the CAT performance handbook and the 2008-2009 Blue Book. In addition the aforementioned, unit rates were developed to include the equipment type and amount of trucks required, travel distance, average grade, load, haul, dump and place for the simplified material types.

Table 8: Preliminary Equipment Unit Rates

Model	Equipment Rates ($/hr)	Operators
Compactor		
CAT CP74	105.97	1
Dozer		
CAT D7	201.02	1
Drill		
Air Rotary, 200 cfm compressor	345.49	2
Excavator		
CAT 336CL	187.67	1
CAT 345	287.67	1
Lifting		
TL642	81.97	1
Loader		
CAT 972	193.12	1
Truck		
CAT 740	219.22	1

Note: Fuel rates are not included in the above table.

The productivity of the rock and soil moving operations was controlled primarily be the number of trucks selected to haul material. Unit rates would vary depending on the actual number of trucks used. A preliminary estimate for the construction costs associated with the Vangorda Creek Diversion (based on the older November 2009 design, see additional comments in section 4.10) is presented in Appendix IX.

4.10 Technical Specifications

SRK has prepared a preliminary draft of the technical specifications for the Phase 1 Vangorda Creek Diversion Channel Construction. The first revision of the document (Revisions A) was issued for review in November 2009. This document was based on the November 2009 design. The February 2010 revision (Revision B) of this document has been update to reflect the step chute concept at the current time (See Appendix IV).

5 Post Construction Monitoring

A post construction monitoring program should be implemented upon completion of each phase of the Vangorda Creek Diversion construction. This program should be designed to monitor the physical and environmental stability during and after the closure period. Routine inspections should be done to ensure conformance of the diversions implementation/construction with the closure plan. Regular inspections should be performed by both the owner, or owner's representative, and by the design engineer (SRK), or another qualified geotechnical engineer. Inspections should be more frequent over the first year, after the initial phase of construction has been completed. Subsequent year's inspections can be reduced to once per year. The initial increased frequency of inspection will allow for the functions and characteristics exhibited by the diversion to be observed. With the information and observations gathered, design revisions, upgrades and contingency measures can be implemented in the following year. This initial phase of modifications will allow for a more optimal, robust, and long term functioning diversion, as well as assisting with finalizing the alignment and design of the Phase 2 component.

Some of the key items which should be inspected include:

- Embankment seepage, settlement, erosion, piping, cracking;
- Erodible plug, settlement, erosion, piping (it is recommended that in the first year a live test be carried out on the headworks dam to provide comfort that the plug will erode);
- Diversion blockages, channel settlement, lateral movement, erosion, cracking, icing, concentration of high flows in corners;
- Performance of riprap (e.g. around headworks, plunge pool, stepped chute, general diversion);
- Performance of geosynthethics and geomembrane sections;
- Water quality in the diversion; ensure geochemical standards are met/upheld;

- Performance of stepped chute, erosion (step erosion, scour between steps, lateral erosion around steps), sediment load, settlement, deformation, steel pipes and step boulder function; and
- Side slopes of diversion channel, slope stability, surface water management measures.

Survey markers should be established on the embankment as well as at various locations along the diversion channel to quantify settlement and lateral movements. These should be surveyed monthly initially and then less frequently if movements are negligible.

Finally, the contingency plan should be further defined and developed to better address any possible disruptions to the facility.

6 Final Remarks

This report primarily focuses on presenting the work done to date on the preliminary Vangorda Creek Diversion design. The preferred approach presented in this report to address the steeper sections of the proposed diversion alignment is the stepped chute approach. This approach would involve cutting a series of steps into the channel base that have a rise of 1m and a tread of 9 m. The primary concern with this approach is in regards to the erodibility of the bedrock and the glacial till overburden. To address this concern, the following strategy is recommended. Build the stepped chute section of the diversion in the rock as designed with erosion protection on the till only. Monitor the erodibility of the bedrock foundation in the stepped chute over the first few years of operation and make any modifications necessary to protect the integrity of the channel.

The alternative approach to the above the North Wall pushback option discussed in Appendix VIII. This option involves upgrading the existing flume diversion to a riprap lined open channel around the north perimeter of Vangorda Pit. The prime advantage of this option is the maximum channel gradient for Phase 1 of the project would be limited to no steeper than 7% over a distance of about 220 m, which compares to the 11% slope over a distance of 460 m for the main option presented in this report. In Phase 2 however, the gradient of the channel from the plunge pool to Vangorda Creek would still be on the order of 10%, but over a much shorter distance and in a less critical section of the diversion. The main disadvantage of this option is the need to construct this diversion in the winter as Vangorda Creek would need to be diverted around the construction zone using pumps and pipes. It may also be necessary to buttress the pit wall below the diversion to provide an increased factor of safety against failure in the long term. This option also removes the flexibility of retaining the existing flume as a back up in the event flows exceed the design event.

Along the currently proposed diversion alignment, approximately from stations 1+150 to 1+500, it is unknown if the base of the diversion channel will be in overburden or bedrock. It is recommended that an additional three drill holes be drilled in the lower reach to determine if the overburden stepped chute design will be required to be constructed in this section. Drill hole locations would be chosen approximately at locations around station 1+200, around the existing plunge pool (approximately around station 1+300) and through the existing haul road (around station 1+375).

Alternatively if the latter drilling is not done it is recommended that this area be highlighted to the bidding contractors. Further, if drilling in this section of great uncertainty is not undertaken, then additional time for field fitting and design revision will likely be required and should be planned for accordingly.

This report, **"Faro Mine Complex, Vangorda Creek Diversion, Design Report – DRAFT"**, has been prepared by SRK Consulting (Canada) Inc.

Prepared by

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Reviewed by

Cameron Scott
Principal Engineer

All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

7 References

Aberle, J. and Smart, G.M. (2003).*The Influence of Roughness Structure on Flow Resistance on Steep Slopes*. Journal of Hydraulic Research Vol. 41, No. 3 (2003), pp. 259-269.

BC MELP (2000), *Riprap Design and Construction Guide*. Public Safety Section, Water Management Branch, Province of British Columbia, Ministry of Environment, Lands and Parks (BC MELP). March 2000.

BCRBHCA, 2009. The Blue Book: 2008 – 2009 Equipment Rental Rate Guide. B.C. Road. *Builders and Heavy Construction Association*. Authorized by the Government of British Columbia. July, 2009.

Biedenharn, D.S., and Smith, J.B., 1997. Design considerations for grade control siting. *Proc. Conf. on Management of Landscapes Disturbed by Channel Incision*, Wang, S.S.Y, Langendoen, E.J., Shields F.D. Jr (eds.), 229-234.

Carling, P., Tinkler, K. (1998). Conditions for the Entrainment of Cuboid Boulders in Bedrock Streams: An Historical Review of Literature with Respect to Recent Investigations. Rivers Over Rock: Fluvial Processed in Bedrock Channels, Geophysical Monograph 107, Copyright 1998 by the American Geophysical Union.

Chin, A (2002). *The Periodic Nature of Step-Pool Mountain Streams*. Americal Journal of Science, Vol 302 February 2002, pp. 114-167.

Chin, A. and Wohl, E. (2005). *Toward a Theory for Step Pools in Stream Channels*. Progress in Physical Geography 29, 3 (2005), pp. 275-296.

Chin,A, Anderson, S., Collison, A., Ellis-Sugai, B., Haltiner, J., Hogervorst, J., Kondolf, G.M., O'Hirok,L., Purcell, A., Riley, A., Wohl, E. (2008). *Linking Theory and Practice for Restoration of Step-Pool Streams*. Environmental Management (2009) 43:645-661, doi 10.1007/S00267-008-9171-x. February 2008.

Church, M. and Zimmermann, A. (2007). *Form and Stability of Step-Pool Channels: Research Progress*. Water Resources Research Vol. 43, WO3415, doi:10.1029/2006WR005037. March 2007.

Comiti, F., Cadol, F.D. and Wohl, E. (2009). *Flow Regimes, Bed Morphology, and Flow Resistance in Self-formed Step-pool Channels*. Water Resources Research, Vol. 45, WO4424, doi:10.1029/2008WR007259. January 2009.

Fang, Hsai-Tang, editor (1991). *Foundation Engineering Handbook – Second Edition.* Kluwer Academic Publishers, sixth printing 2002. pp 22-25.

Fieschenich, C., Seal, R. (2000). *Boulder Clusters.* USAE Research ad Development Center, Environmental Laboratory, 3909 Halls Ferry Rd., Vicksburg, MS 39180, ERDC TN-EMRRP-SR-11.

Gallegos H.A. and Abt, S.R. (2001). *Design Criterial for Rounded/Angular Rock Riprap in Overtopping Flow.* Graduate Research Assistant, Department of Civil Engineering, Colorado State University, Fort Collins, CO 80523.

Khatsuria, R.M. (2005). Hydraulics of Spillways and Energy Dissipaters: Ch 12. Fuse Plugs and Fuse Gate Spillways. Marcel Dekker, New York, 2005 p.261-282.

Lenzi, M.A. (2001). *Step-Pool Evolution in the Rio Cordon, Northeastern Italy.* Earth Surface Process and Landforms 26, 991-1008 (2001), DOI: 10.1002/esp.239.

Lenzi,M.A. (2001). Stream Bed Stabilization using boulder check dams that mimic step-pool morphology features in Northern Italy. Geomorphology 45 (2002) 243-260. From www.sciencedirect.com.

Lenzi, M.A., Marion, A., Comiti, F., Gaudio, R. (2002). *Local Scouring in Low and High Gradient Streams at Bed Sills.* Journal of Hydraulic Research, Vol 40, 2002, No.6.

Lenzi, M.A., and Comiti, F. (2002). *Local Scouring and Morphological Adjustments in Steep Channels with Check-dam Sequences.* Geomorphology 55 (2003) 97-109. From www.sciencedirect.com.

Lenzi, M.A. (2004). *Displacement and Transport of Marked Pebbles, Cobbles and Boulders during Floods in a Steep Mountain Stream.* Published online 12 May 2004 in Wiley InterScience (www.interscience.wiley.com). DOI: 10.1002/hyp.1456.

Lorenz, E.A., P.E. and M.N. Lobrecht, P.E., Robinson, K.M., USDA (2000). *An Excel Program to Design Rock Chutes for Grade Stabilization.* 2000 ASAE Annual Internation Meeting. Midwest Express Center, Milwaukee, Winsconsin, July 2000.Transaction of the ASAE Vol. 41 (3): 621-626, 1998.

Mendrop, K.B., and Little, C.D., 1997. Grade stabilization requirements for incised channels. *Proc. Conf. on Management of Landscapes Disturbed by Channel Incision*, Wang, S.S.Y, Langendoen, E.J., Shields F.D. Jr (eds.).

Montgomery, D., Buffington, J., Smith, R., M. Schmidt, K. and Pess, G. (1995), *Pool Spacing in Forest Channels.* Water Resources Research, Vol. 31., No. 4, Pages 1097-1105, April 1995.

Peirson, W. and Cameron, S. (2006). *Design of Rock Protection to Prevent Erosion by Water Flows Down Steep Slopes.* Journal of Hydraulic Engineering, ASCE. October 2006.

SRK (1990). Geotechnical/Hydraulic Design, Vangorda Creek Diversion Facility, Vangorda Plateau Development. Report prepared for Curragh Resources Inc. Report 60638. November 1990.

SRK (1991). *Construction Report, Vangorda Creek Diversion, Vangorda Plateau Development.* Report prepared for Curragh Resources Inc. Report 60639. August 1991.

SRK (2002). *Engineering Analysis of Vangorda Pit Wall Stability.* Report prepared for Deloitte and Touche Inc. Project No. 1CD003.15. November 2002.

SRK (2003). *Alternatives Assessment for the Vangorda Creek Diversion.* Report prepared for Deloitte and Touche Inc. Project No. 1CD003.15. April 2003.

SRK (2004). *Vangorda Creek Diversion Inspection Report.* Report prepared for Deloitte and Touche Inc. Project No. 1CD003.62. June 2004.

SRK (2004). *Vangorda Creek Diversion Design of Emergency Spillway and Recommendations for Flume Upgrade.* Report prepared for Deloitte and Touche Inc. Project No. 1CD003.62. October 2004.

SRK (2004). *Colomac TCA Diversion Ditch Project Design and As-Built Report.* Report prepared for Indian and Northern Affairs Canada and Public Works and Government Services Canada. Project No. 1CP001.24. June 2004.

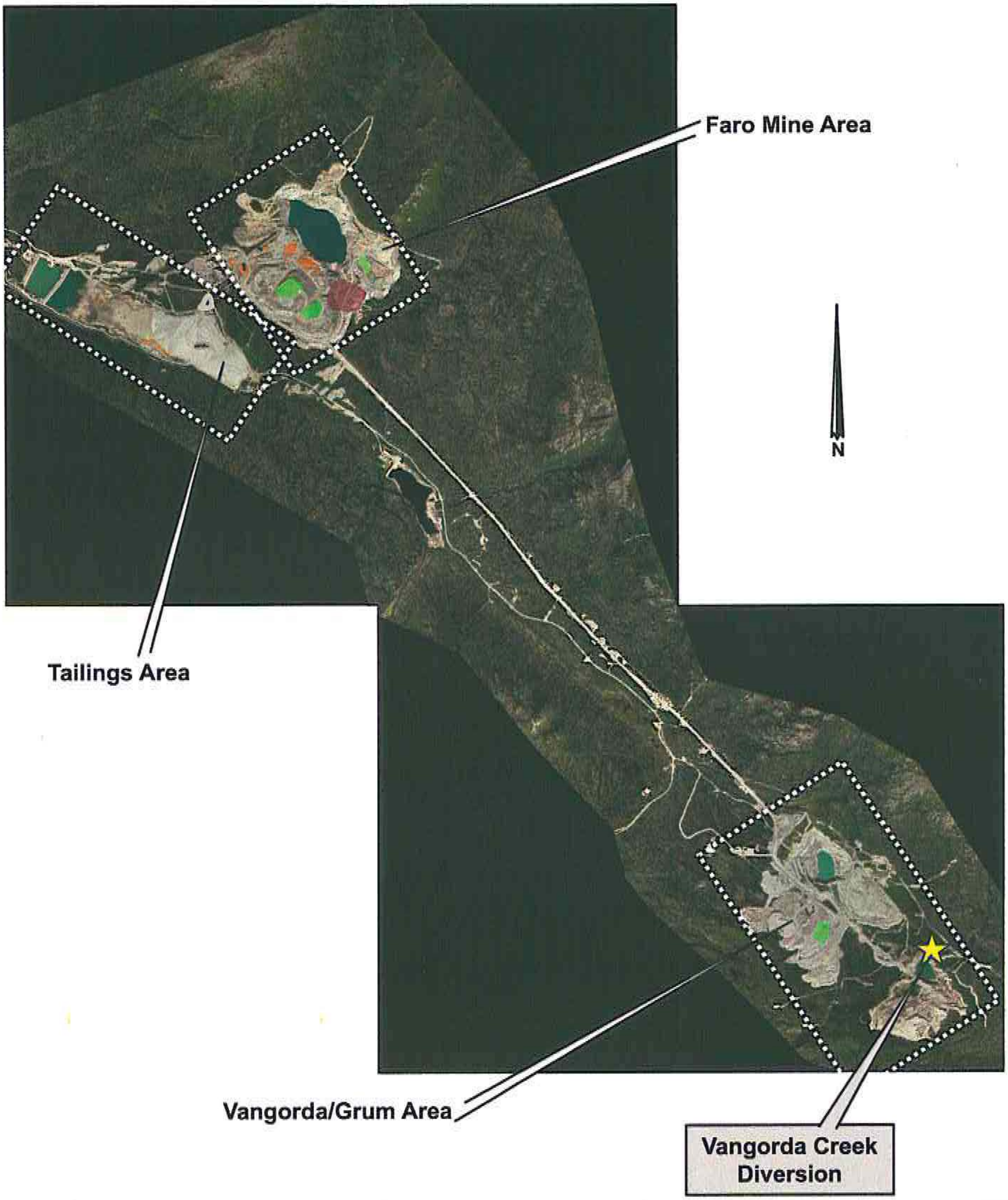
SRK (2005). *Final Design Report for Dam 1B, Colomac site, NT.* Report prepared for Indian and Northern Affairs Canada and Public Works and Government Services Canada. Project No. 1CP001.038. October 2005.

Thomas, D.B., Mussetter, R.A., Abt, S.R., Harvey, M.D. (2000). *A Design Procedure for Sizing Step-Pool Structures*. Mussetter Engineering Inc. 1730 S. College Avenue, Suite 100, Fort Collins, Colorado 80525. ASCE Conf. Proc. 104, 340 (2000), DOI:10.1061/40517(2000)340.

United States Department of Agriculture (USDA) (2007). *Grade Stabilization Techniques.* Technical Supplement 14G. Part 645 of National Engineering Handbook. 210-VI-NEH, August 2007.

United States Department of Agriculture (USDA) (2007), *Stone Sizing Criteria.* Technical Supplement 14C. Part 654 of National Engineering Handbook. 210-VI-NEH, August 2007.

Figures



SRK Consulting Engineers and Scientists VANCOUVER	Faro Mine Closure Fermeture de la mine Faro	Vangorda Creek Diversion **Vicinity Map**
Job No: 1CY001.031 Filename: Figure 1_Vicinity_20090825.ppt	Faro Mine Complex Final Closure and Remediation Plan	Date: February 2010 Approved: Figure: 1

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Job No: 1CY001. 031
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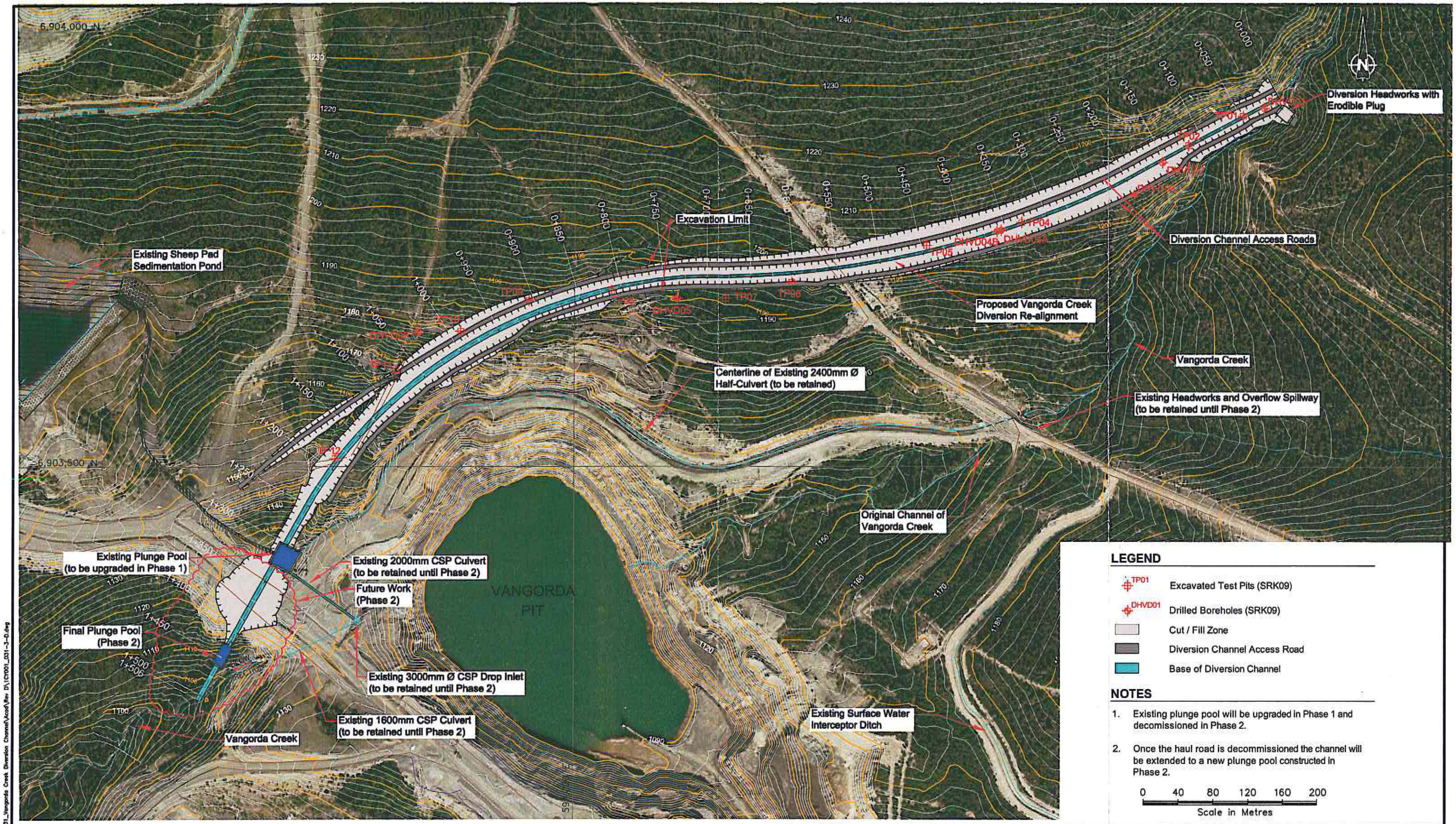
Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex
Final Closure and Remediation Plan

Vangorda Creek Diversion

Location Map
(3D Image)

Date: February 2010
Approved:
Figure: 2



LEGEND

TP01	Excavated Test Pits (SRK09)
DHVD01	Drilled Boreholes (SRK09)
▭	Cut / Fill Zone
▬	Diversion Channel Access Road
▬	Base of Diversion Channel

NOTES

1. Existing plunge pool will be upgraded in Phase 1 and decommissioned in Phase 2.
2. Once the haul road is decommissioned the channel will be extended to a new plunge pool constructed in Phase 2.

0 40 80 120 160 200
Scale in Metres

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Faro Mine Closure
Fermeture de la mine Faro

SRK JOB NO.: 1CY001.031
FILE NAME: 1CY001_031-3-D.dwg

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

General Arrangement
(with Orthophoto)

DATE: Feb. 2010
APPROVED: PMH
FIGURE: 3

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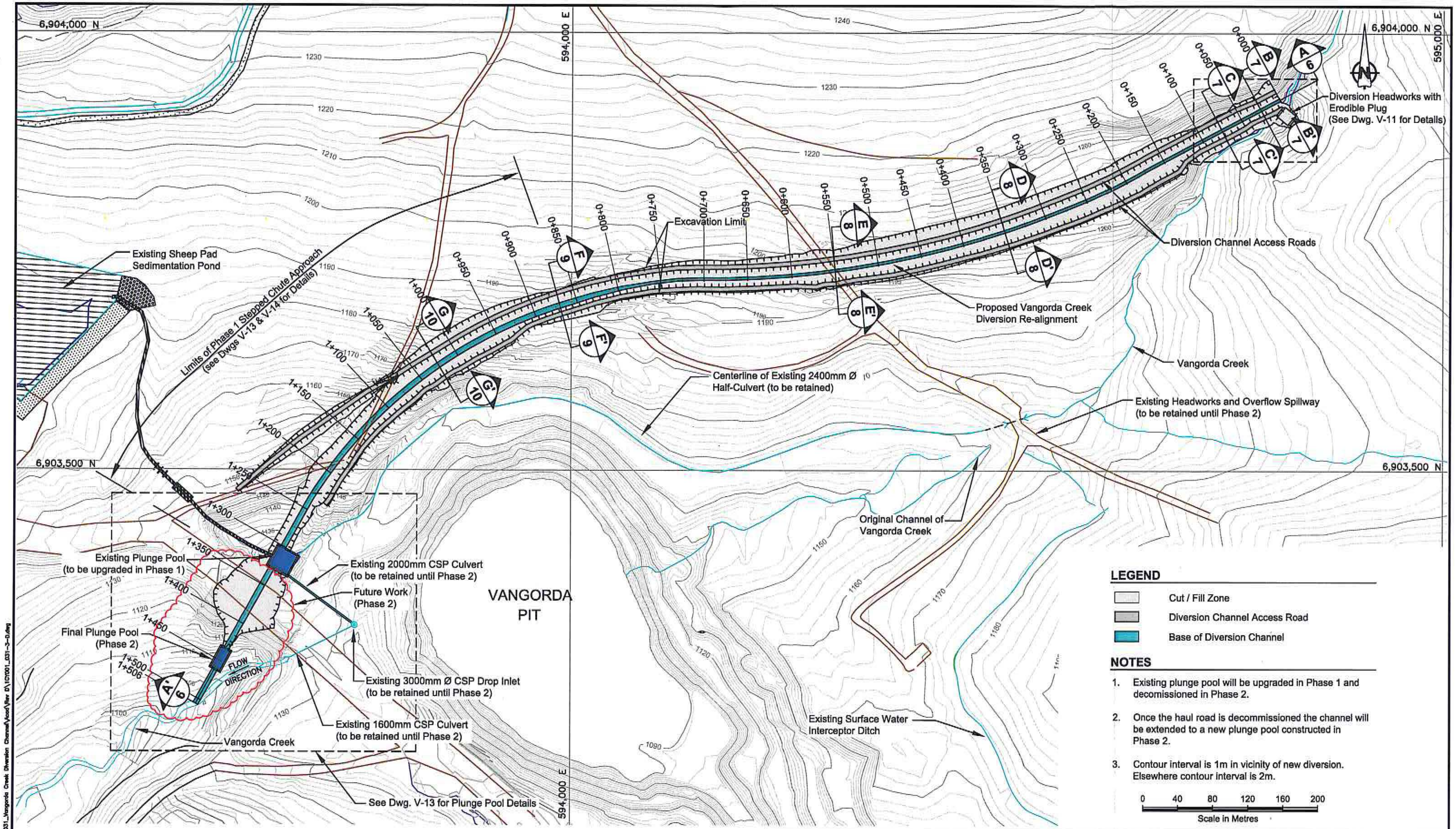

LEGEND

▢	Cut / Fill Zone
▩	Diversion Channel Access Road
🟦	Base of Diversion Channel

NOTES

1. Existing plunge pool will be upgraded in Phase 1 and decomissioned in Phase 2.
2. Once the haul road is decommissioned the channel will be extended to a new plunge pool constructed in Phase 2.
3. Contour interval is 1m in vicinity of new diversion. Elsewhere contour interval is 2m.

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Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

General Arrangement

DATE:	APPROVED:	FIGURE:
Feb. 2010	PMH	4

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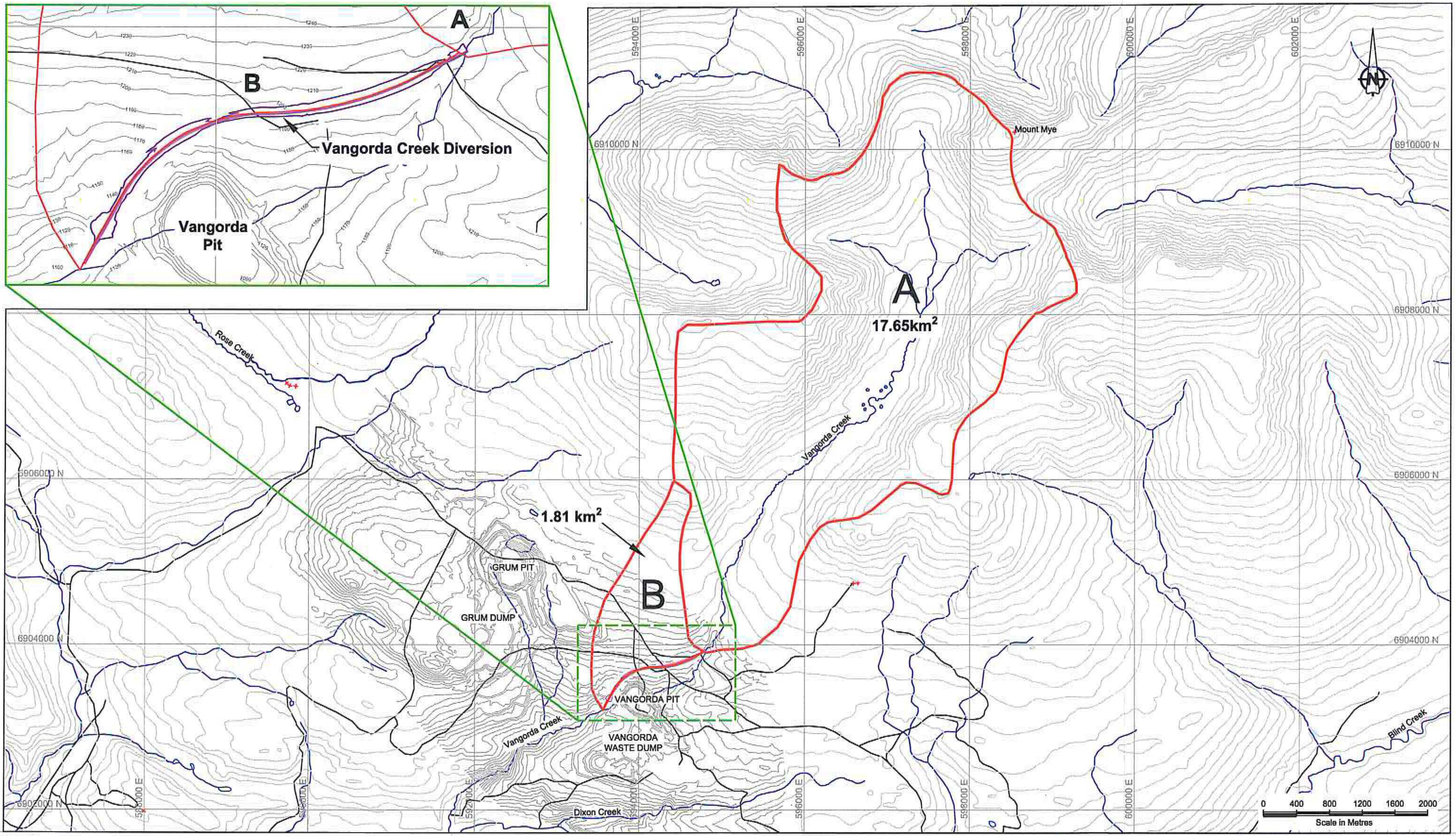

LEGEND

Catchment Area

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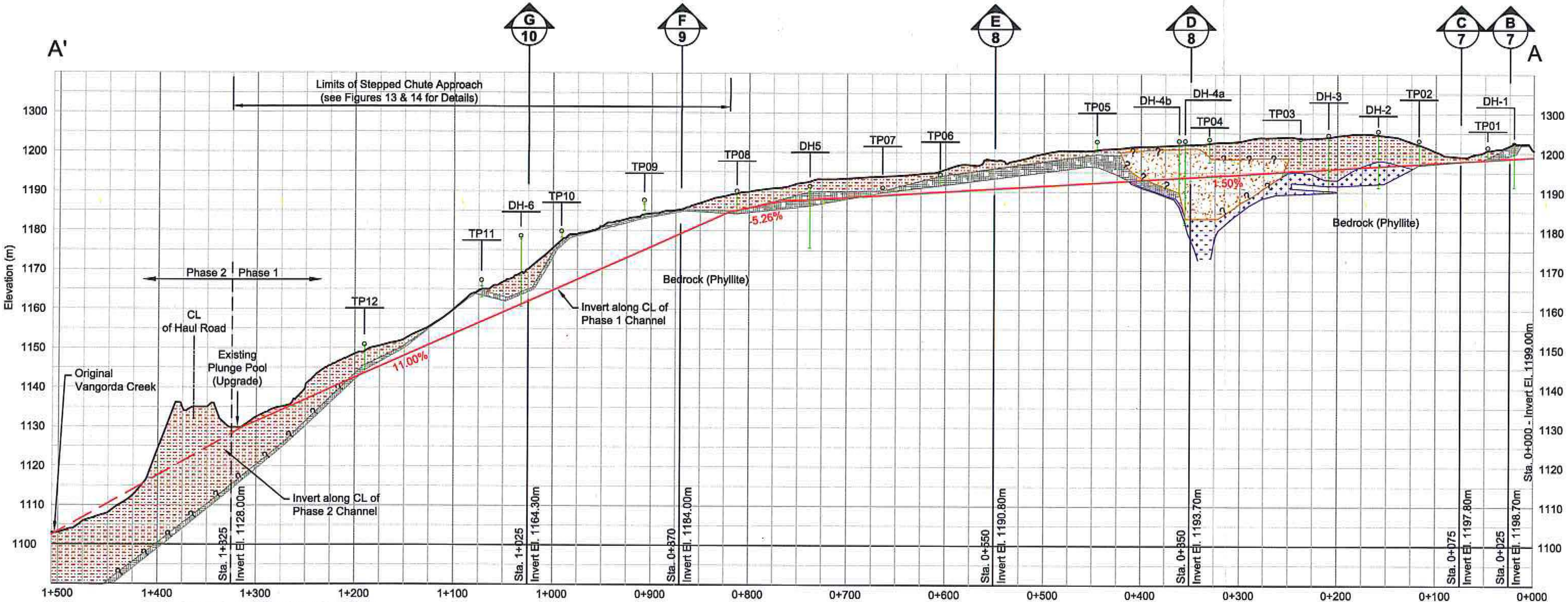
Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Creek Diversion

Catchment Areas

DATE: Feb. 2010
APPROVED: PB
FIGURE: 5



Current Ground Elevation at CL of Station

	1+500	1+400	1+300	1+200	1+100	1+000	0+900	0+800	0+700	0+600	0+500	0+400	0+300	0+200	0+100	0+000
Current Ground Elevation at CL of Station	1103.00	1124.45	1132.50	1148.50	1159.86	1175.50	1184.32	1189.92	1193.33	1195.65	1199.71	1201.53	1202.95	1204.33	1200.40	1200.69
Design CL Elevation of Vangorda Diversion Channel Invert (before Riprap Placement)	1102.47	1115.47	1128.47	1141.47	1154.47	1167.47	1180.47	1186.98	1188.48	1189.98	1191.49	1192.99	1194.49	1195.99	1197.50	1199.00

Design CL Elevation of Vangorda Diversion Channel Invert (before Riprap Placement)

LEGEND

— Existing Ground at Centerline

— Proposed Channel Invert

—?— Inferred Strata Contact

DH-2 — Drillhole / Test Pit ID

Collar Elevation Projected to Alignment Centerline

Strata Contact

Drillhole / Test Pit Trace

▨ Overburden

▨ Silty Sand

▨ Granodiorite

▨ Weathered Phyllite

Top soil layer varies from 0-0.9m and is not shown for clarity.

Overburden includes primarily glacial till with some road fill material incorporated near the south extents of the proposed diversion.

4x Vertical Exaggeration

0 10 20 30 40 50

Vertical Scale in Metres

0 40 80 120 160 200

Horizontal Scale in Metres

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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Diversion Channel

Profile Along Centerline of Channel
(Section A-A')

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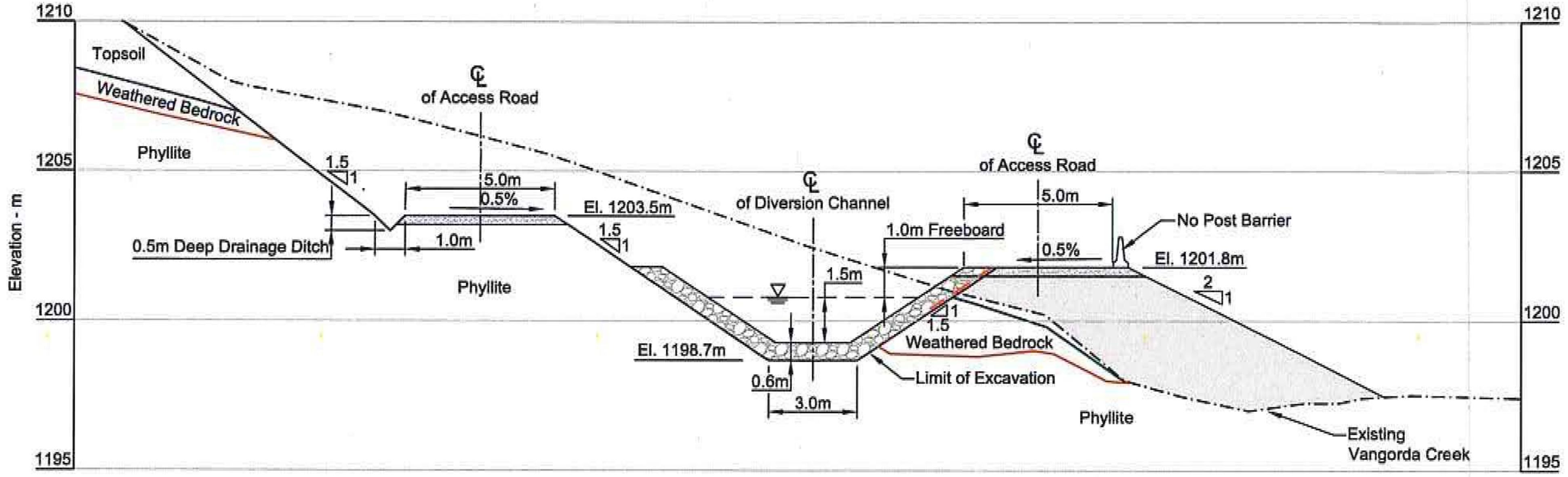
FILE NAME:	1CY001_031-5-D.dwg

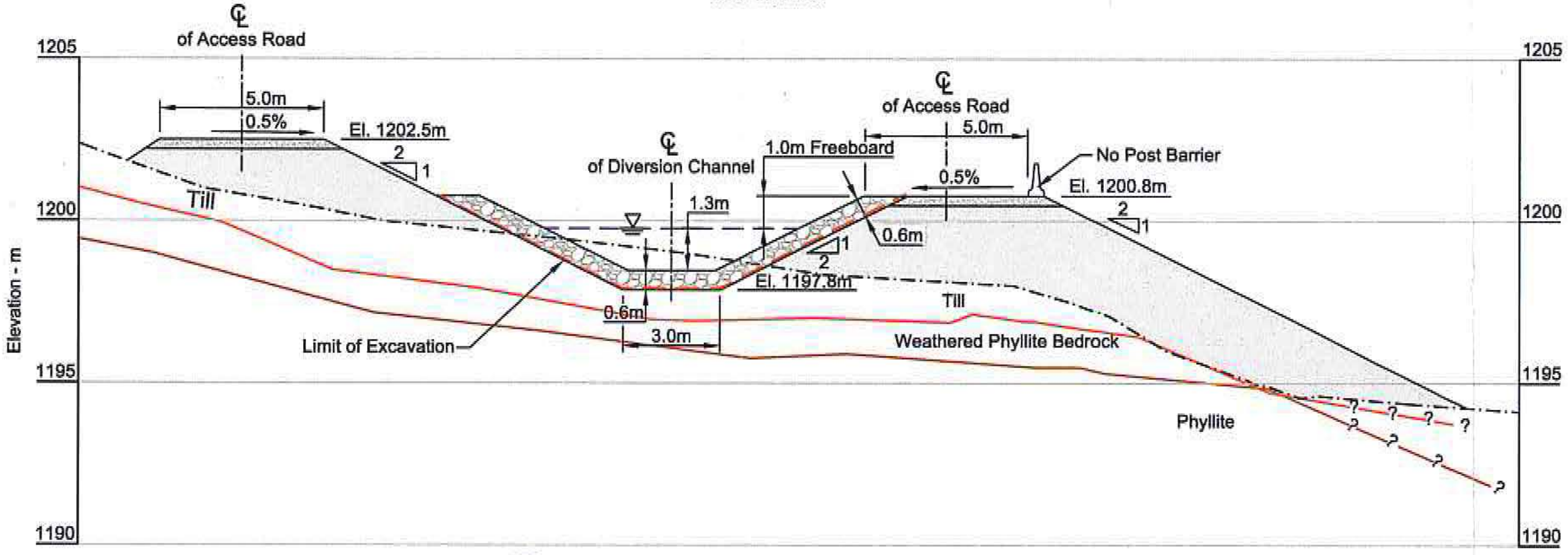
FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

DATE:	APPROVED:	FIGURE:
Feb. 2010	PMH	6



B 7 SECTION AT STATION 0+025

0 2 4 6 8 10
Scale in Metres



C 7 SECTION AT STATION 0+075

0 2 4 6 8 10
Scale in Metres

LEGEND

—·—·—	Original Ground Surface	▨	Sand and Gravel	—	Topsoil Base
—·—·—	LLDPE Liner	▭	General Fill	—	Glacial Till Base
— — —	Geotextile Filter Fabric	▨	Rip Rap	—	Weathered Phyllite Base

NOTE

1. Channel designed for maximum flow of $13m^3/sec$ (200 year event). Flow for events that exceed $13m^3/sec$ will be directed to existing flume and/or Vangorda pit.

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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Diversion Channel

Section B-B' (0+025) and
Section C-C' (0+075)

SRK JOB NO.: 1CY001.031
FILE NAME: 1CY001_031-6-D.dwg

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

DATE: Feb. 2010
APPROVED: PMH
FIGURE: 7

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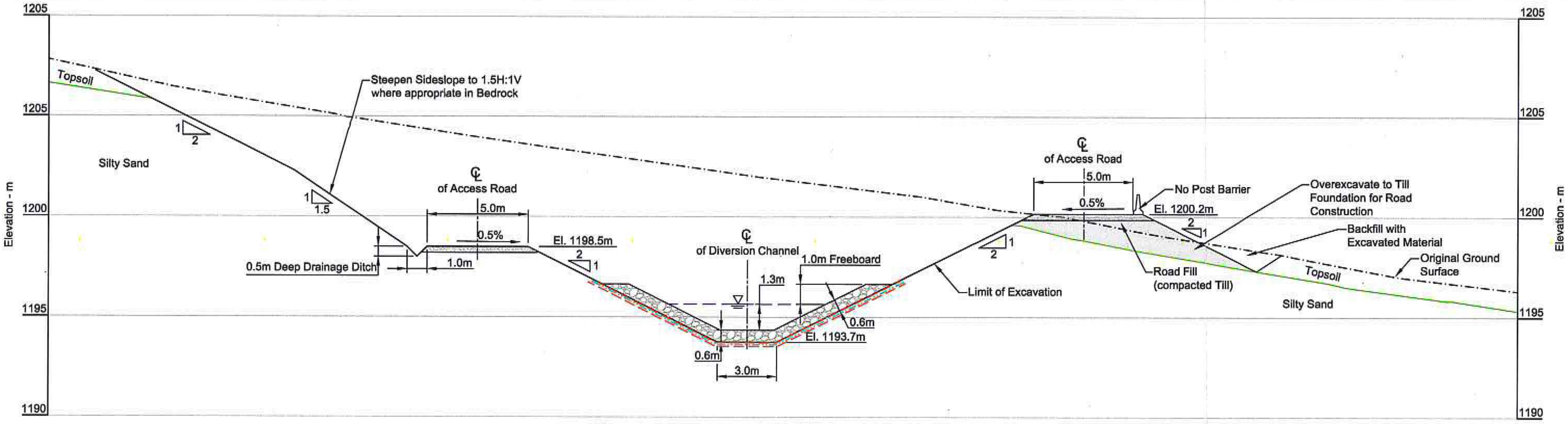
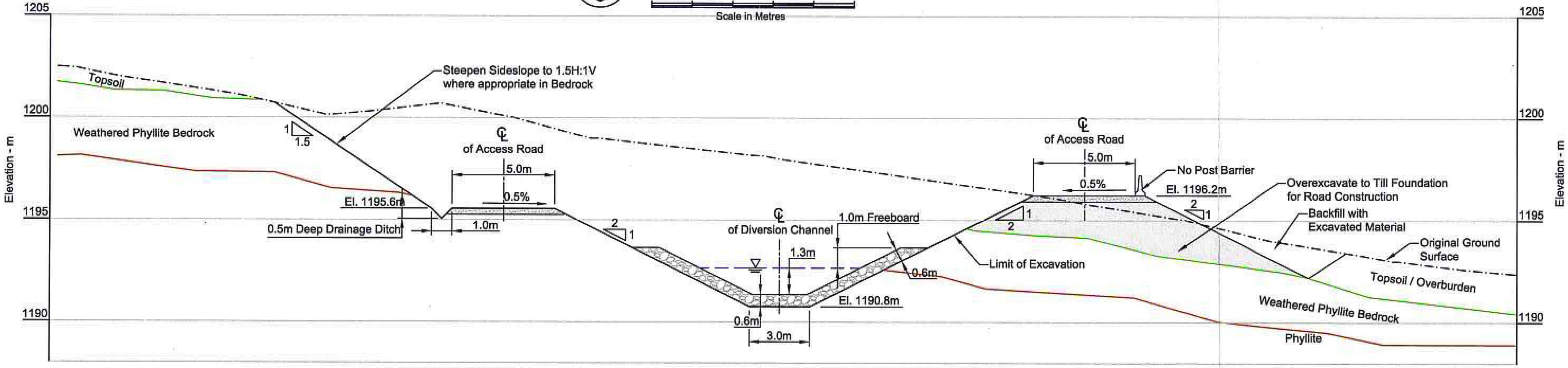


D/8 SECTION AT STATION 0+350

0 2 4 6 8 10

Scale in Metres



E/8 SECTION AT STATION 0+550

0 2 4 6 8 10

Scale in Metres

LEGEND

- — · — Original Ground Surface
- — · · — LLDPE Liner
- — — Geotextile Filter Fabric
- ▨ Sand and Gravel
- ▒ General Fill
- ▩ Rip Rap
- — Topsoil Base
- — Glacial Till Base
- — Weathered Phyllite Base

NOTE

1. Channel designed for maximum flow of 13m^3/sec (200 year event). Flow for events that exceed 13m^3/sec will be directed to existing flume and/or Vangorda pit.

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Engineers and Scientists
Vancouver B.C.

SRK JOB NO.: 1CY001.031
FILE NAME: 1CY001_031-6-D.dwg

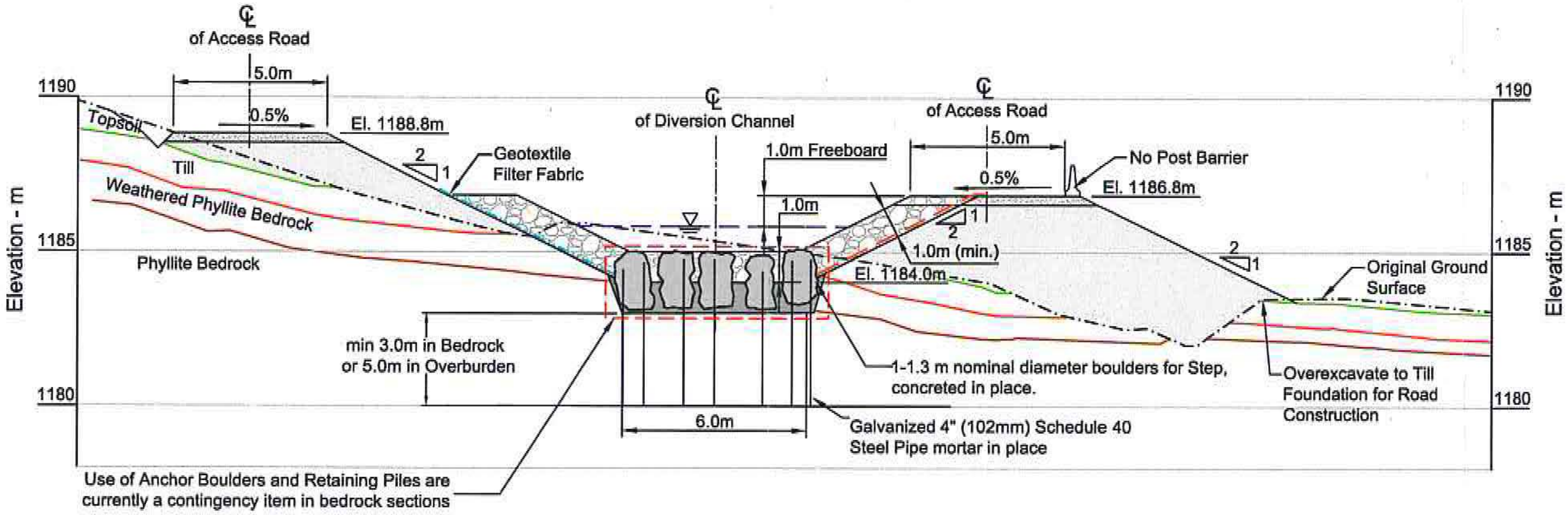
Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

Section D-D' (0+350) and
Section E-E' (0+550)

DATE:	APPROVED:	FIGURE:
Feb. 2010	PMH	8



F
9

SECTION AT STATION 0+870

LEGEND

–·–·–	Original Ground Surface	▨	Sand and Gravel	—	Topsoil Base
–··–··–	LLDPE Liner	☐	General Fill	—	Glacial Till Base
– – –	Geotextile Filter Fabric	▨	Rip Rap	—	Weathered Phyllite Base

NOTES

1. Channel designed for maximum flow of 13m^3/sec (200 year event). Flow for events that exceed 13m^3/sec will be directed to existing flume and/or Vangorda pit.
2. Riprap placed in overburden and highly weathered phyllite sections.

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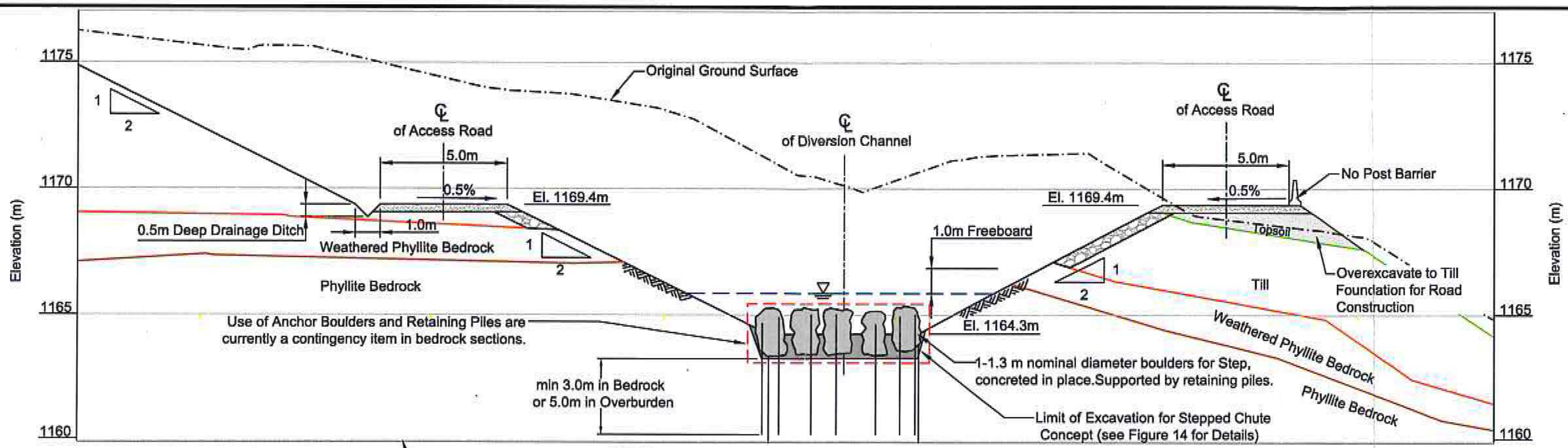
Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

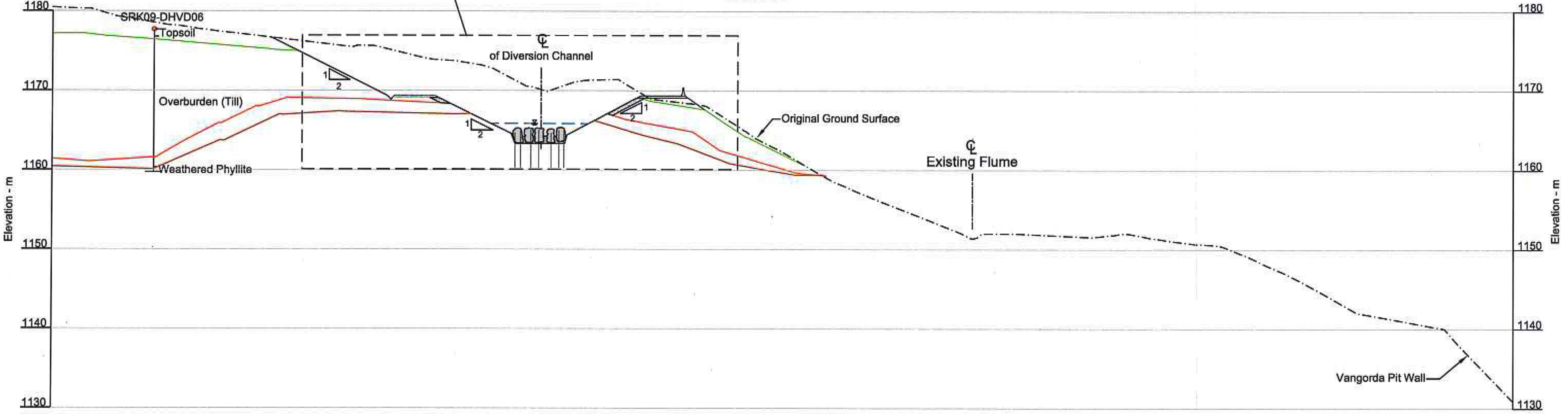
Section F-F' (0+870)

DATE: Feb. 2010
APPROVED: PMH
FIGURE: 9



G 10 SECTION AT STATION 1+025

0 2 4 6 8 10
Scale in Metres



G 10 SECTION AT STATION 1+025

0 5 10 15 20 25
Scale in Metres

LEGEND

— Original Ground Surface
-··-··- LLDPE Liner
- - - Geotextile Filter Fabric
▨ Sand and Gravel
▨ General Fill
▨ Rip Rap
— Topsoil Base
— Glacial Till Base
— Weathered Phyllite Base

NOTE

1. Channel designed for maximum flow of 13m^3/sec (200 year event). Flow for events that exceed 13m^3/sec will be directed to existing flume and/or Vangorda pit.

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Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

Section G-G' (1+025)

DATE:	APPROVED:	FIGURE:
Feb. 2010	PMH	10

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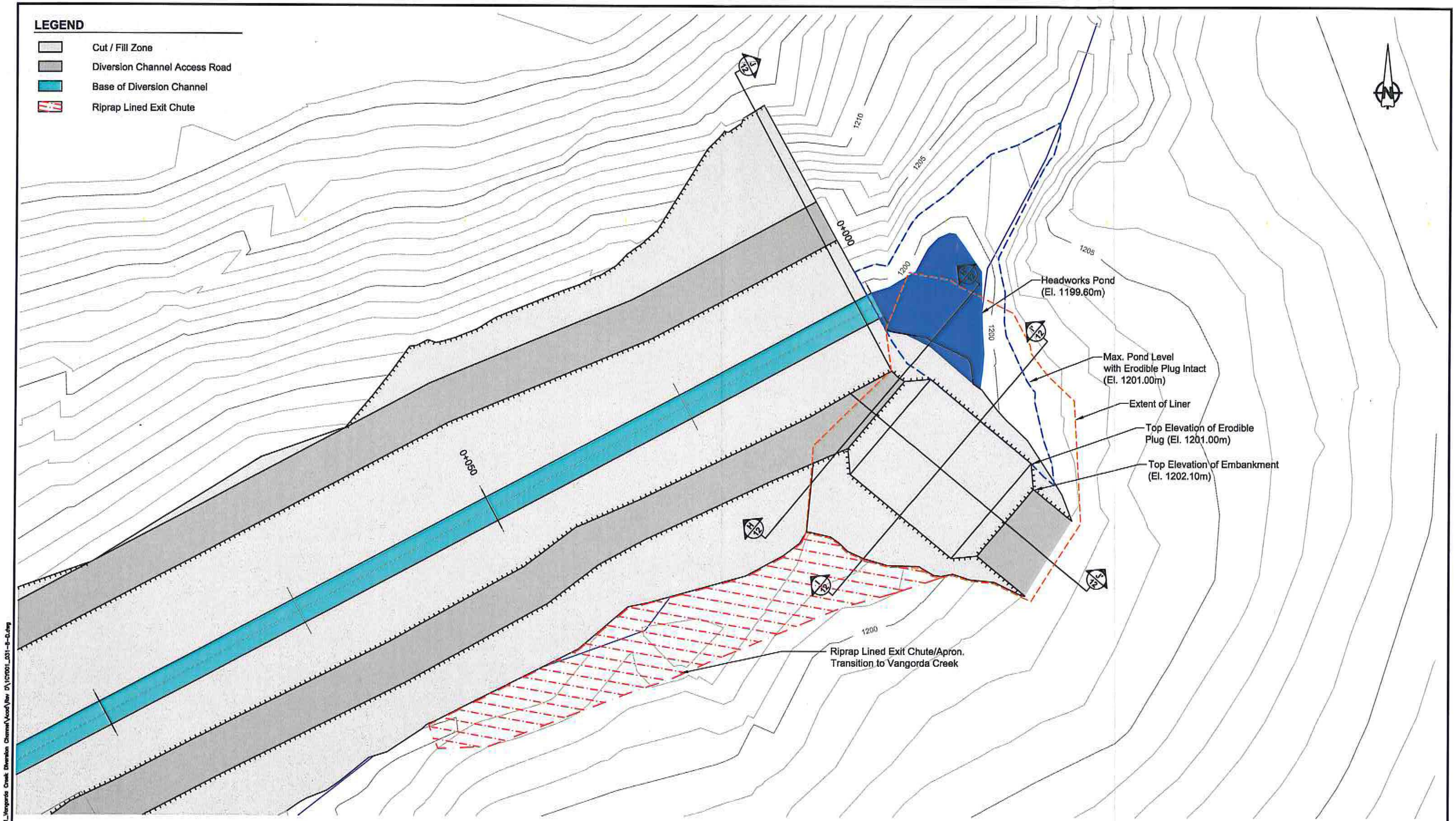

0 2 4 6 8 10
Scale in Metres

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Vancouver B.C.

SRK JOB NO.: 1CY001.031
FILE NAME: 1CY001_031-8-D.dwg

Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

Headworks Site Plan

DATE: Feb. 2010
APPROVED: PMH
FIGURE: 11

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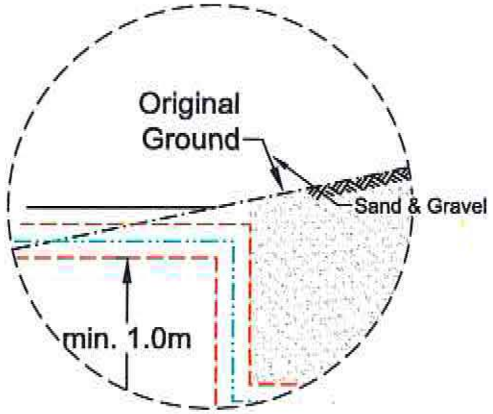
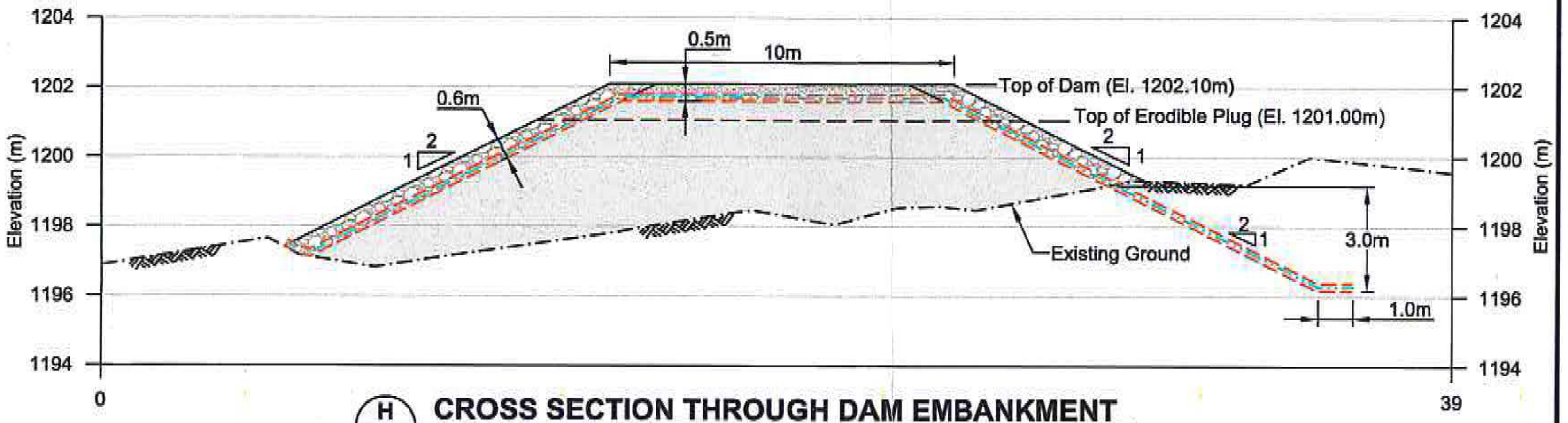
LEGEND

- -·-·- Original Ground Surface
- -··-·· LLDPE Liner
- - - - Geotextile Filter Fabric
- ▨ Sand and Gravel
- ☐ General Fill
- ▨ Rip Rap
- ■ Core Material



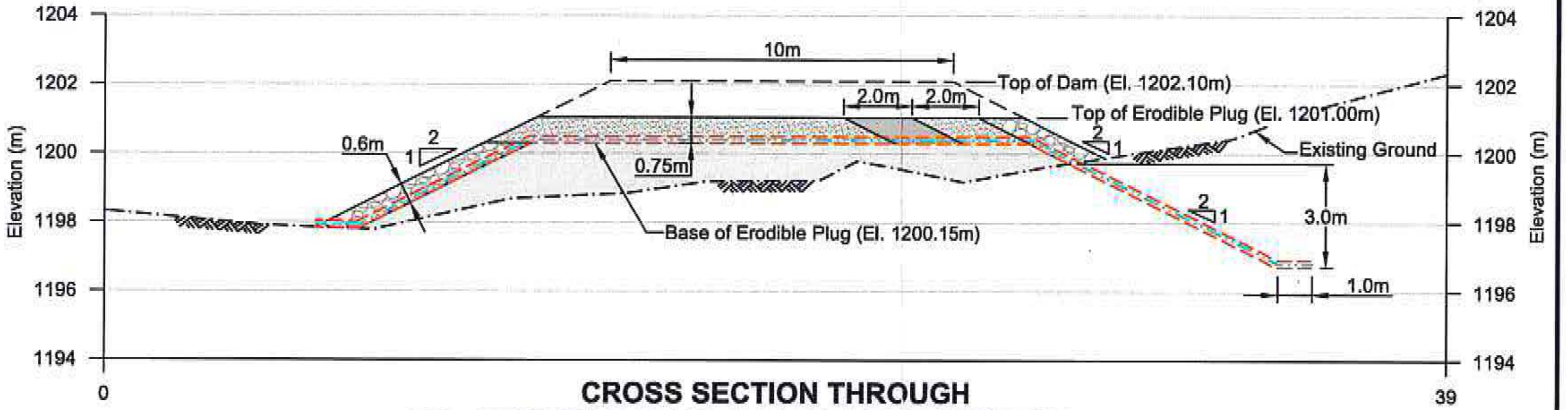
① DETAIL 1

0 0.4 0.8 1.2 1.6 2
Scale in Metres



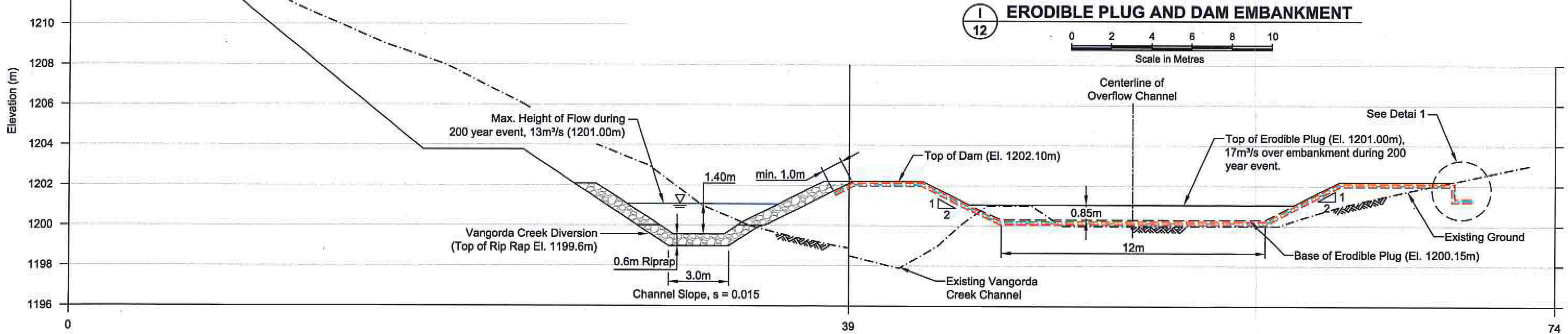
H/12 CROSS SECTION THROUGH DAM EMBANKMENT

0 2 4 6 8 10
Scale in Metres



I/12 CROSS SECTION THROUGH ERODIBLE PLUG AND DAM EMBANKMENT

0 2 4 6 8 10
Scale in Metres



J/12 LONGITUDINAL SECTION THROUGH HEADWORKS

0 2 4 6 8 10
Scale in Metres

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FILE NAME: 1CY001_031-8-D.dwg

Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

Headworks Sections

DATE:	APPROVED:	FIGURE:
Feb. 2010	PMH	12

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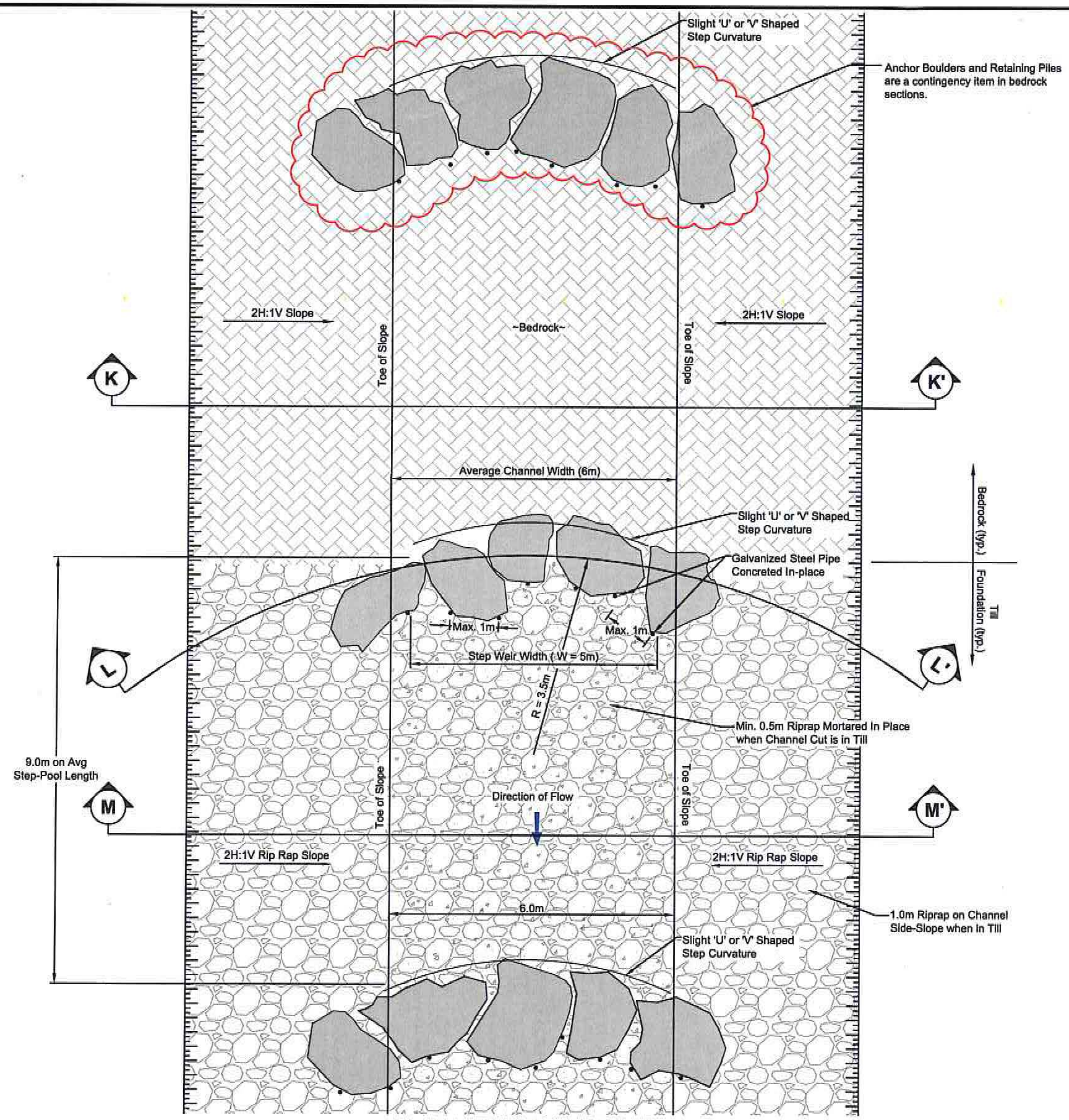


PLAN VIEW OF STEP POOL

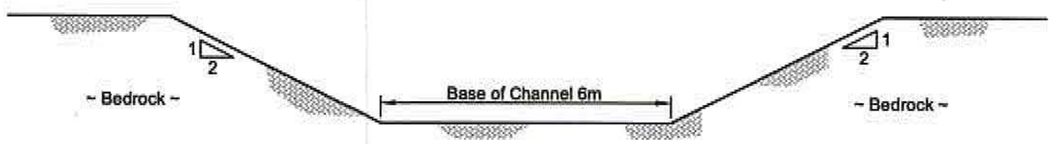
0 1 2 3 4 5
Scale in Metres

LEGEND

▨	Sand and Gravel
○	Rip Rap
○	Mortared Rip Rap
⬢	Boulder
▬	Concrete

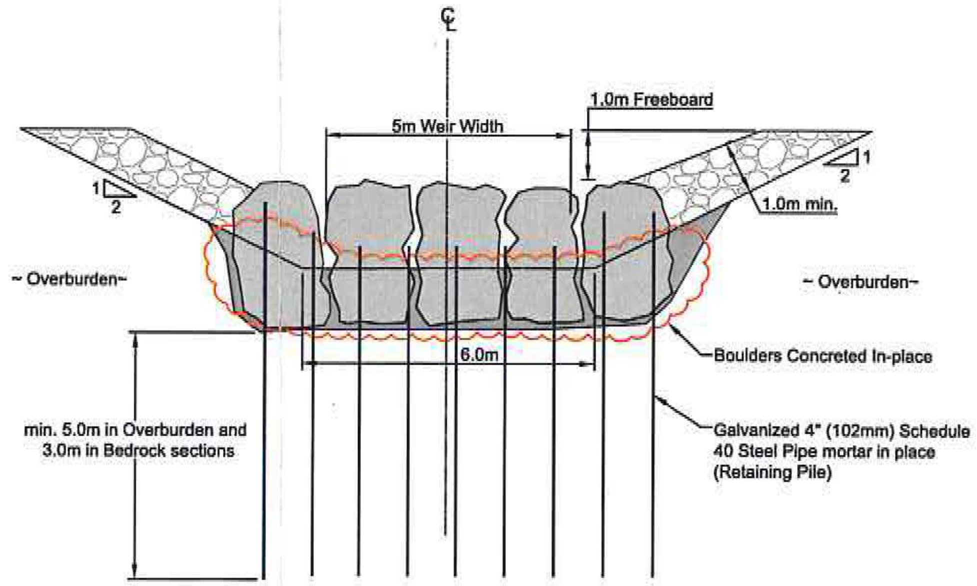
NOTES

1. Phase 1 design shown.
2. Use of anchor bouldering and retaining piles in overburden would be subject to confirmation by future drilling program below STA 1+025.



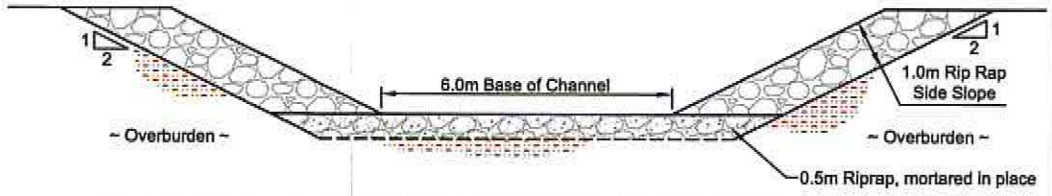
K TYPICAL SECTION IN BEDROCK

0 1 2 3 4 5
Scale in Metres



L TYPICAL SECTION AT STEP IN OVERBURDEN

0 1 2 3 4 5
Scale in Metres (Boulders NTS)



M TYPICAL SECTION IN OVERBURDEN FOUNDATION

0 1 2 3 4 5
Scale in Metres

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FILE NAME: 1CY001_031-7-D.dwg

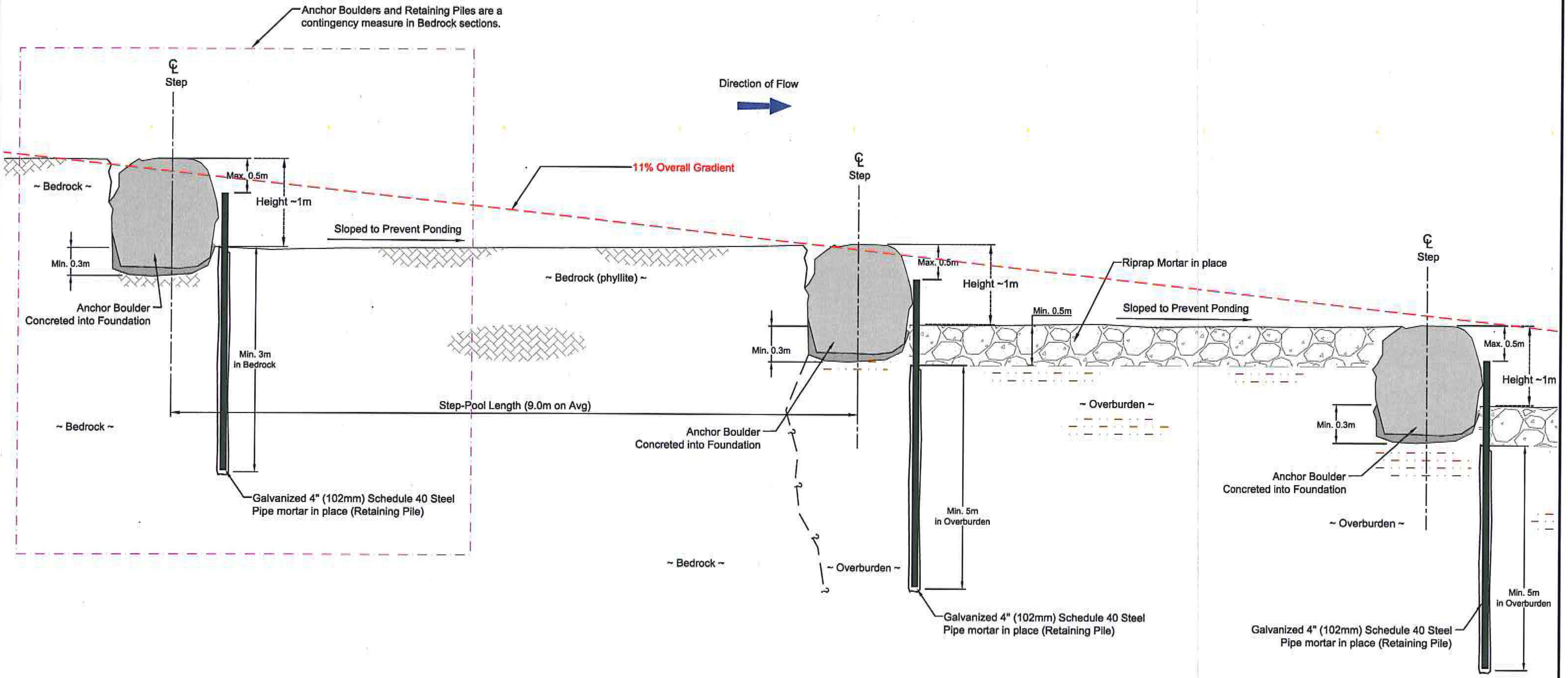
Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

Stepped Chute
Plan and Section

DATE: Feb. 2010
APPROVED: PMH
FIGURE: 13



SCHEMATIC PROFILE OF STEPPED CHUTE

0 0.5 1 1.5 2 2.5
Scale in Metres

LEGEND

- ┈ Geotextile Filter Fabric
- ▨ Sand and Gravel
- Rip Rap
- Mortared Rip Rap
- ● Boulder
- Bedrock
- Till
- ■ Concrete

NOTE

1. Phase 1 design shown.

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FILE NAME: 1CY001_031-7-D.dwg

Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

Stepped Chute
Typical Profile

DATE: Feb. 2010
APPROVED: PMH
FIGURE: 14

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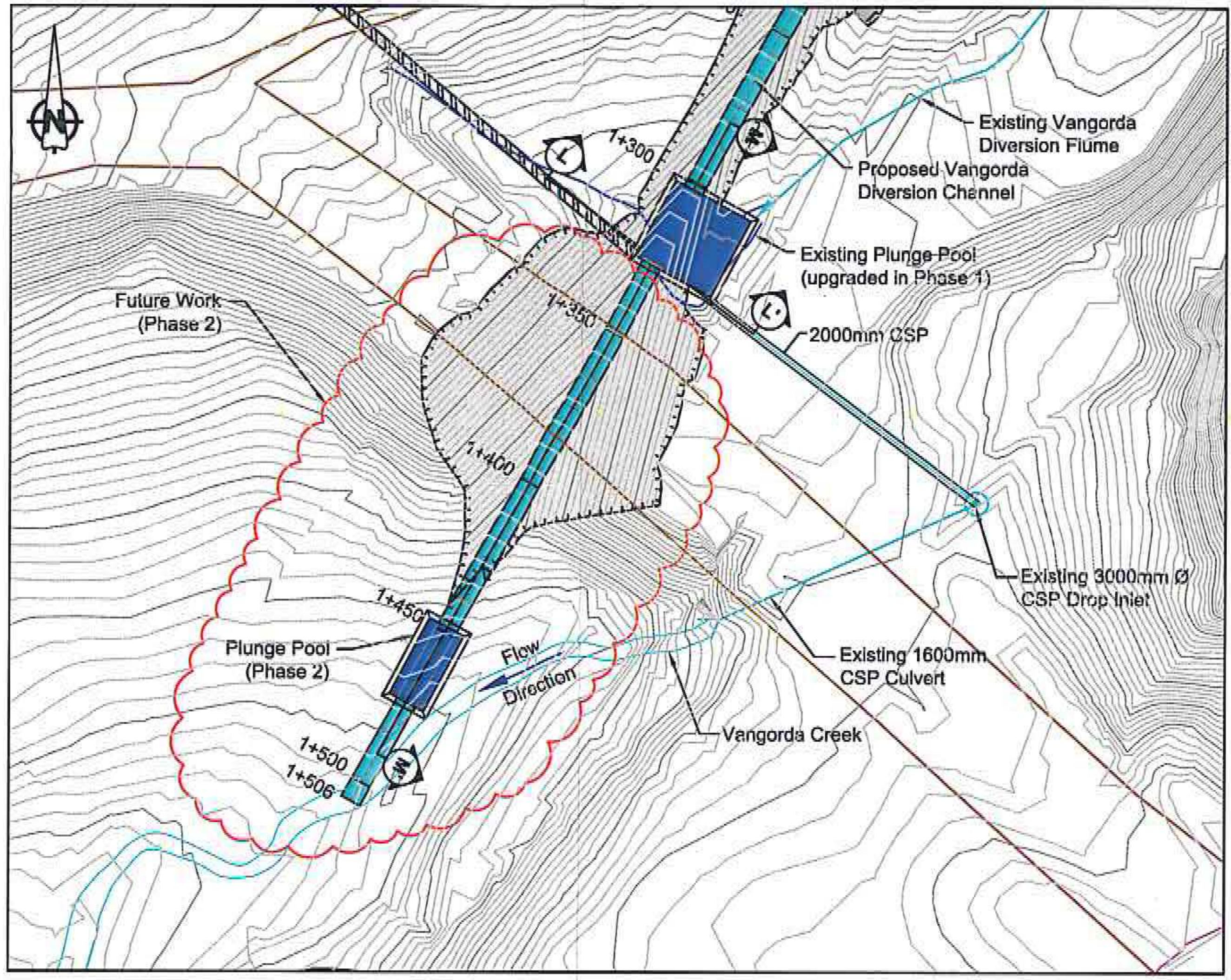
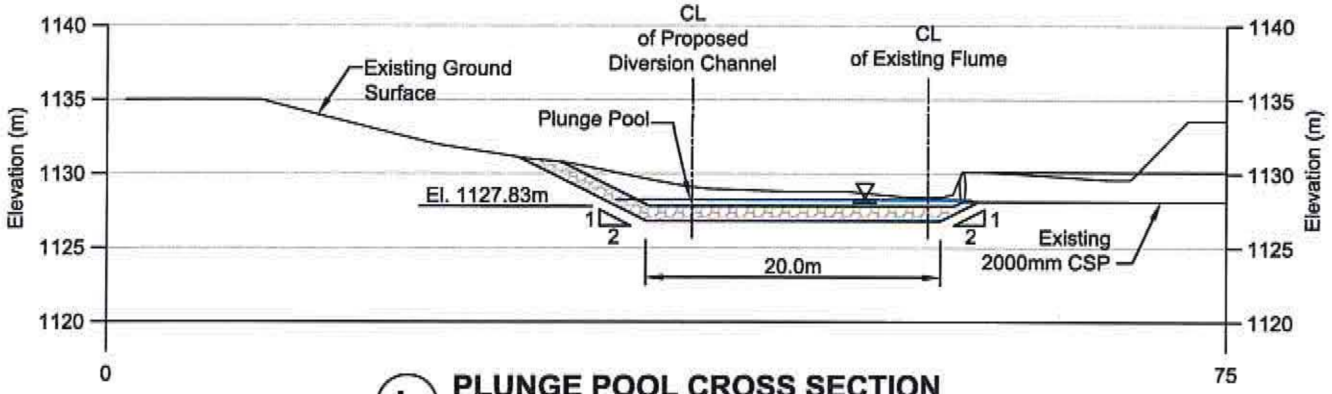
NOTES

1. Existing plunge pool will not be removed until the haul road is decommissioned in Phase 2.
2. Transition into existing plunge pool will be field fit to ensure a minimum pool base elevation 0.30m lower than the bottom elevation of the existing 2000mm CSP.
3. Once the haul road is decommissioned the channel will be extended to a new plunge pool constructed in Phase 2.



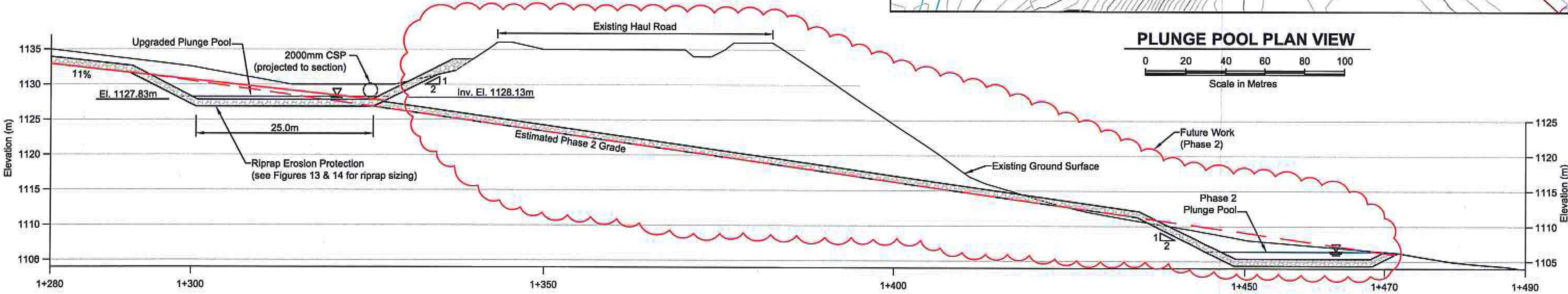
PLUNGE POOL PLAN VIEW

0 20 40 60 80 100
Scale in Metres



L PLUNGE POOL CROSS SECTION

0 5 10 15 20 25
Scale in Metres



M LONGITUDINAL SECTION THROUGH PLUNGE POOLS

0 5 10 15 20 25
Scale in Metres

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Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Diversion Channel

Plunge Pool
Plan and Sections

DATE:	APPROVED:	FIGURE:
Feb. 2010	PMH	15

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Appendix I
Geotechnical Field Investigation

Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex
Vangorda Creek Diversion

Geotechnical Field Investigation
- Draft -

Prepared for:

Indian and Northern Affairs Canada
Affaires indiennes et du Nord Canada

and

Yukon Government

Prepared by:

SRK Consulting
Engineers and Scientists

Project Reference Number
SRK 1CY001.031

August 2009

Vangorda Creek Diversion
Geotechnical Investigation

Government of Yukon

Assessment and Abandoned Mines
Energy, Mines and Resources
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SRK Project Number 1CY001.031.0002

August 2009

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1 Introduction

1.1 General

The Yukon Government (Energy, Mines and Resources Assessment and Abandoned Mines Branch) has assumed responsibility for the Faro Mine Complex, and Denison Environmental Services (DES) has been awarded a contract to provide care and maintenance services to Yukon Government at this site. As part of the ongoing work, a number of projects are planned for 2010 to reduce the risk of impacts to the environment. One of these projects is the construction of a permanent diversion of Vangorda Creek at the Vangorda /Grum mine site.

The existing flume which carries Vangorda Creek around the pit is in an advanced state of disrepair. Failure of the channel would likely result in flooding of the pit and consequently, an unmanageable increase in the volume of contaminated water. A new channel is proposed to the North of the Pit and would be constructed in two phases. The first phase would extend from a new intake structure located about 500m north of the existing diversion headworks on Blind Creek road to the existing plunge pool and dropbox structure at the haul road. The total length of the initial phase of the diversion would be about 1325m. The future second phase of the work would extend the diversion by about 125 to intersect the original Vangorda Creek.

YG contracted SRK Consulting (Canada) Inc. (SRK) to carry out a geotechnical field investigation and engineering design for the construction of the new diversion. The geotechnical field investigation component was carried out between May 28 to June 19, 2009. The investigation involved the excavation of a number of test pits and a drilling program along the centreline of the proposed diversion alignment. The report presents the results of that investigation and provides the in interpretation of the insitu surface and subsurface site conditions.

1.2 Background

To allow for the mining activities in the early 1990's at the Vangorda Pit, a diversion of the Vangorda Creek, upstream of the pit, was required. A general overview of the Vangorda and Grum mine developments including the development of the conceptual design for diversion are presented in the December 1989 "Vangorda Plateau Development, Water Licence Application". In 1990, as part of the preliminary geotechnical and hydraulic design of the Vangorda Creek Diversion, an initial field investigation, consisting of mainly test pitting, was done downslope of the currently proposed diversion.

Since its construction in 1991, SRK has completed yearly inspections of the existing Vangorda Creek Diversion flume and the surrounding Vangorda areas. A detailed inspection of the original Vangorda Creek Diversion, performed after the flood in 2004, which necessitated the upgrade and reconstructions to the current diversion, can be found in the "Vangorda Creek Diversion Inspection Report, June 2004". In addition to the latter, a field inspection of the overall pit wall conditions was carried out during a site visit in 2002 and the finding presented in the, "Engineering Analysis of Vangorda Pit Wall Stability" report.

2 Field Activities

The Vangorda Creek Diversion (VCD) geotechnical field investigation was carried out between May 28, 2009 and June 19, 2009 by SRK site staff. The SRK site investigation team consisted of John Kurylo (onsite from May 28 to June 19) and Peter Healey (on site from June 2 to 4).

SRK carried out the following tasks as part of this program:

- Drill rig selection/organization
- General Site Reconnaissance
- Surficial geology visual interpretation
- Road clearing/ development of drill access
- Excavating of twelve test pits
- Test pit stratigraphy logging/classification
- Drill hole layout
- Diversion alignment ground survey, 1m survey resolution
- Drilling, seven boreholes (deepest one being 21.34m-deep)
- Borehole cuttings logging
- Simplified geotechnical rock core logging
- Standard Penetration/in-situ density testing
- Disturbed and undisturbed (split spoon) sample collection
- Well installation, four standpipe piezometers
- Groundwater flow/piezometer measurements
- Soil samples, core sample and gain size analysis testing

Throughout the field program the winds were generally moderate to slight with daytime temperatures reaching highs of 22°C (average) and lows down to 10°C (average). Conditions ranged from sunny clear skies to overcast with periods of intense rainfall. All field work was completed under the supervision of SRK site staff.

3 Investigation Methods

3.1 Test Holes

All test pits were excavated using a thumbed CAT 345 C Excavator, operated by Denison Environmental Services (DES) personnel. A field traverse of an access route along a preliminary alignment of the VCD was first flagged by utilizing a handheld Garmin GPS unit. Trees and vegetation were cleared by the excavator along the flagged alignment creating a pathway for drill access. After sections had been cleared for access, test pits were excavated. Test pitting progressed from the northeast extent to the southwest of the proposed diversion alignment. Test pit locations were selected to provide an even spatial distribution over the extent of the diversion alignment; these locations were slightly field fit to encountered site conditions.

Test pits were advanced until the excavator reached refusal on bedrock, frozen ground, dense cobble rich till or on boulders. SRK field staff logged and photographed the excavator progress as well as soil and rock conditions encountered. For safety test pits greater than 2m depth or with steep unstable side slopes were not entered. When unable to enter a test pit, changing soil stratigraphy and overburden samples were gathered from the excavator's bucket for closer classification and visual examination. Upon completion of the field test pit logging the test pits were backfilled with the excavated material and compacted through tamping with the back of the excavator bucket.

Grab samples were taken of overburden glacial till when encountered during the test pit excavations. One test pit sample from SRK09-TP02 (second test pit excavated) was delivered to EBA Engineering's soil testing laboratory in Whitehorse (see Section 3.5 for results). Notes and photographs were taken on the remaining soil samples. After the test pits had been backfilled the locations of the test pits were flagged and later picked up as waypoints in the ground survey.

The surveyed coordinates and total depth of the completed test pits are provided in Table 1. Figures 1 and 2 show the test pit locations excavated in the Vangorda Creek Diversion field investigation area.

Table 1: Test Pit Locations

Test Pit ID	Northing[1]	Easting[1]	Ground Surface Elevation (m)	Total Depth (m)
SRK09 - TP01	6903901.9	594771.2	1200.5	3
SRK09 - TP02	6903868.3	594708.7	1202.2	6.2
SRK09 - TP03	6903799.6	594629.4	1202.6	6
SRK09 - TP04	6903779.8	594516.7	1202.5	5.8
SRK09 - TP05	6903753.3	594406.1	1201.9	2.5
SRK09 - TP06	6903710.5	594250.5	1193.4	2.8
SRK09 - TP07	6903692.5	594175.2	1190.5	1.25
SRK09 - TP08	6903701	594043.8	1189.2	5.8
SRK09 - TP09	6903689.7	593948.4	1186.8	2.1
SRK09 - TP10	6903654.5	5938710	1178.9	1.6
SRK09 - TP11	6903607.4	593797.2	1166.4	4.3
SRK09 - TP12	6903507.8	593728.5	1150	6.4

1. UTM Projection NAD 27, Zone 08 V.

3.2 Drilling

Boreholes were drilled by a 3-person crew using Geotech Drilling Services Ltd's track mounted double rotary Fraste Multi Drill – XL. The Fraste Multi Drill was supported by two Morookas, MST-1500 and MST-1600 series, transporting the drill rods, bits, accessories, general equipment, as well as the air compressor. This drill was selected as it has the capabilities to drill through cobble and boulder rich overburden through the use of the 'Overburden drilling with eccentric bit' or ODEX drilling method as well as diamond core capabilities for bedrock drilling. All boreholes were vertical and were completed using drill steel with either compressed air during ODEX, or water during diamond core drilling.

During ODEX drilling, air is injected down the hole to return the air, soil and rock chips (cuttings) generated from the percussion action of the ODEX bit, to a cuttings sampler spout at the ground surface via the interior of the drill steel. Cutting samples were generally collected by hand or in a 20 litre pail placed under the cutting spouts at defined intervals and stratigraphic changes. In the case of the diamond core drilling, water is injected down the hole to assist with the diamond drilling and the core is lifted to the surface in the core sampling assembly/rods. All holes were commenced with 10.16 cm (4-inch) ODEX bit drilling until bedrock was reached. HQ or 63.5 cm diameter diamond core drilling resulted in four of the seven boreholes. Core runs were collected to gain a better understanding of the geotechnical bedrock conditions. Boreholes were advanced to refusal in either weathered phyllite or more competent fractured phyllite bedrock and were drilled to final depths below the expected diversion invert, as determined using AutoCAD.

In addition to rock core sampling Standard Penetration Tests (SPT) as well as split spoon sampling was performed periodically to gather in-situ samples and overburden density measurements. These tests were performed by removing the ODEX bit and entering the casing with soil sampling and hammer assembly or coring equipment.

SRK field engineering staff John Kurylo supervised the drill, logged the recovered core and soil materials, and collected representative soil samples for geotechnical testing. Two in-situ split spoon soil samples from SRK09-DHVD04A and SRK09-DHVD04B, drilled adjacent to and within 5m of each other, were collected and delivered to EBA Engineering's soil testing laboratory in Whitehorse.

Borehole locations were determined and marked after the test pitting activities and staked during the ground survey. The surveyed coordinates, depth and orientation of the completed boreholes are presented in Table 2.

4 Investigation Results

4.1 Test Pits – Surficial Geology

Complete test pit logs are provided in Appendix A and photographs are provided in Appendix B. Test pits are named in the order they were excavated, starting at the northeast extent of the diversion and working down alignment to the southwest. Locations of the test pits are shown on Figures 1 and 2 as well as in Table 1 and Appendix A.

The depth of the test pits varied from 1.25 m to 6.40 m. Based on field test pit classifications of the soils and weathered bedrock, the stratigraphic profile consists of the following:

- 0.1 -0.9 m (0.5m average) dark brown to black, fibrous topsoil, consisting of an organic matt with soft silts, clays, some sand and some to trace gravel. Typically sections near the base of the topsoil stratigraphy were observed to be frozen. There appears to be a deeper pocket ranging in thickness from 0.9-1.5 m closer to the central extents of the proposed VCD alignment around SRK09-DHVD05 and DHVD06. This topsoil layer was covered by overlying shrub, grass and tree vegetation and was observed to be wet to saturated with some surficial ponding observed.
- In about one third of the test pits a layer of medium brown, soft, wet, silty sand to sandy silt with some clay, gravel and cobbles was encountered beneath the topsoil. This layer typically ranges from 0.1 to 0.3 m and contains a high organic content with some root mass. This stratigraphy appears to be an extension of the upper topsoil layer and would likely be stripped as part of the upper organic rich topsoil layer. Near the top of this stratigraphy occasionally a creamy thin 0.1 m thick volcanic ash or tephra layer was observed to be transitioning into this unit.
- The next stratigraphic unit encountered was a glacial till. Typically this material was a greyish brown to medium brown, silty sand to sandy silt with gravel, sub-rounded cobbles, mainly igneous boulders and some clay. This layer is variable in thickness but typically is greater than 1 m. This material was observed to generally be dense, moist, exhibiting medium plasticity and was the predominant overburden material observed.
- Near the south-western portion of the VCD alignment, closer to the area where the diversion will again reconnect with the natural alignment of Vangorda Creek, there was approximately 2 m or greater thickness of rock road fill material over the glacial till overburden unit. This is depicted in the SRK09-TP12 log.
- The underlying phyllitic bedrock is undulating and fractured to heavily fractured and weathered. Refusal of tests pits on bedrock occurred at depth typically around 1.3 m to 6.4 m below the original ground surface, however test pits SRK09-TP02, 03 and 04 never reached bedrock. The undulating bedrock shows deep pockets and shallow knolls and excavation at any location can vary significantly. The weathered extent of the phyllitic bedrock, which was rippable by the CAT 345 C excavator, showed signs of oxidization and heavy fracturing and ranged from 0.1 to 3.5m in thickness (average thickness being closer to 1.5m).

Figures 3 to 6 present typical cross sections through the site investigation area near the areas where test pits were excavated, illustrating the interpreted subsurface profile as described above.

4.2 Boreholes – Subsurface Investigation

For unconsolidated and loosely consolidated silts and clay rich overburden section, the drill rods fell almost entirely under their own weight as the ODEX bit was cleared with compressed air. Soft ground conditions did not provide sufficient resistance and the rods frequently become 'caked' with soil fines resulting in a slower rate of progression. Poor cuttings sample recovery was achieved in unconsolidated and loosely consolidated soils. Good cutting sample recovery was achieved in the dense, often cobble and boulder rich overburden glacial till sections. Recovery from the cuttings spout was constant so that the layering of the stratigraphy was typically preserved. However, precision in depth determination was estimated to be approximately ±0.5 m. During diamond core drilling poor recovery was observed in heavily quartz intruded or fractured bedrock while good to excellent recovery was achieved in more competent bedrock. Although general geotechnical rock core alpha angle trends were recorded, as no drill orientation to correlate runs together was know, the angles presented may vary by ±180° intervals.

The Geotech drilling team commented that some hydrofracturing resulted when diamond drilling with down borehole injected water in the heavily fractured phyllitic bedrock. The resulting hydrofracturing could be observed as the casing became lodged down borehole as it was removed. In one instance as result of the weak fractured phyllite bedrock, as well as from the additional induced drilling hydrofracturing, a HQ core barrel was lost down borehole SRK09-DHVD05 at approximately 10.7 m depth (the inner tube and head assembly was retrieved). This hydrofracturing phenomenon was observed on site in the weak fractured phyllite bedrock typically close to regions of historic metamorphic fluid flow, observed as competent quartz veins, and not encountered when ODEX drilling with compressed air.

Complete stratigraphic logs of the completed geotechnical boreholes, the SPT blow count data and the installation details for the monitoring wells are provided in Appendix C of this Investigation report. Photographs of the core runs drilled are presented in Appendix D. Boreholes are labelled in a similar manner as the test pits, starting at the northeast extent of the diversion and working down alignment to the southwest. Locations are shown on Figures 1 and 2. The following sections summarize the results of the drilling program.

SRK09 - DHVD01

Borehole DHVD01 was drilled to a depth of 11.02 m. Sample recovery from DHVD01 was generally excellent. Approximately 0.1 m of topsoil then 2.2 m of weathered phyllite was found to overlay phyllitic bedrock exhibiting near horizontal planar cleavage. The phyllite bedrock was generally moderately to heavily fractured and jointed displaying heavy micro defects and soft red brown staining. Typically this rock was of poor quality, Rock Quality Designation (RQD) in the range of 29 to 46% and the jointing and foliation breaks were found to be orientated in the range of 60° to 90° and 110° to 120°. Upon completion of the borehole a monitoring well was installed.

SRK09 - DHVD02

Borehole DHVD02 was drilled to a depth of 14.33 m. Sample recovery from DHVD02 was generally good to moderate, moderate in poor quality rock. Approximately 6.7 m of glacial till overburden was found above 6.6 m of granodiorite rock overlying weathered, highly planar fractured phyllitic bedrock. Generally the granodiorite lithology was extremely weathered for the first 0.6 m then exhibited heavy hematite alteration, hard red staining, and moderate micro defects. Variable but generally weak poor rock quality, RQD 20 to 31% was encountered. Typically the jointing in the granodiorite was observed to be orientated in the range of 50° to 65°.

SRK09 - DHVD03

Borehole DHVD03 was drilled to a depth of 14.17 m. Sample recovery from DHVD03 was generally moderate to poor. Approximately 8.7 m of glacial till overburden was found above 3.4 m of granodiorite rock overlying interbedded phyllitic and granodiorite bedrock. From the cross cutting relationship it appears as if the granodiorite intruded the phyllitic bedrock unit around this area. Generally the sections of the granodiorite runs recovered were of strong but very broken and poor quality rock, RQD in the range of 21%, and the phyllite core recovered showed moderate strength and fair rock quality with lots of micro defects and foliation breaks, RQD in the range of 46%. Typically the jointing in the granodiorite was observed to be orientated in the range of 60° to 75°, while the phyllite joint and foliation orientated was in the range of 65° to 80°.

SRK09 - DHVD04A

Borehole DHVD04A was drilled to a depth of 21.34 m. Sample recovery from DHVD04A was solely from ODEX cuttings and was generally excellent. Approximately 0.9 m of topsoil was found over a massive silty sand layer 18.7 m in thickness. The cuttings from the silty sand layer were characterized as brown-yellow silty sand with some to trace gravel and clay. Upon drilling through the silty sand layer the granodiorite lithology was first encountered for approximately 0.6 m then the subsurface profile transitioned into the phyllitic bedrock. The bottom of the borehole was observed to be dry at the time of drilling.

SRK09 - DHVD04B

Borehole DHVD04B was drilled to a depth of 21.34 m. Sample recovery from DHVD04B was solely from ODEX cuttings and was generally excellent. Borehole DHVD04B was drilled near to DHVD04B as an additional borehole to gain more geotechnical data about the massive silty sand unit encountered in DHVD04A. Approximately 0.9 m of topsoil was found over massive silty sand 12.2 m in thickness. The massive silty sand stratigraphy was observed to have reduced by 6.5 m from the thickness observed in the neighbouring DHVD04B indicating that the maximum thickness of this stratigraphy is likely on the order of 19 m as observed in DHVD04A. This lithology is more typical of water deposited type sediments, perhaps glacial fluvial or more likely from a glacial outwash channel origin. The cuttings from the silty sand layer were characterized as brown-yellow silt and sand with some to trace gravel and clay. Upon drilling through the silty sand layer weathered phyllitic bedrock was encountered. The bottom of the borehole was observed to be dry at the time of drilling; however, a monitoring well was installed at the base of the silty sand layer to confirm expected local hydrogeological site conditions.

SRK09 - DHVD05

Borehole DHVD05 was drilled to a depth of 15.65 m. Sample recovery from DHVD05 was generally moderate to excellent. Approximately 1.5m of topsoil then 4.5m of weathered phyllite was found to overlay phyllitic bedrock exhibiting near horizontal planar cleavage. The phyllite bedrock was generally very heavily fractured and jointed and displaying heavy micro-defects, soft red brown staining and frequent foliation breaks. Typically the rock mass was poor to very poor quality, RQD 0 to 51%. Jointing and foliation breaks were found to be orientated in the range of 70° to 85° and 110° to 125°. A monitoring well was installed at this location.

SRK09 - DHVD06

Borehole DHVD06 was drilled to a depth of 17.98 m. Sample recovery from DHVD06 was solely from ODEX cuttings and was generally excellent. Approximately 0.9m of topsoil was found over 16.5m of glacial till which overlaid weathered phyllite. Water could be heard entering the borehole at depth during the monitoring well installation at this location.

The interpreted subsurface profiles are presented in typical cross sections through the site investigation area near the areas where boreholes were drilled, Figures 3 to 6. The results from the field borehole SPT density testing and in-situ sampling are presented in Table 4.

Table 4: Field Standard Penetration Testing Results

Location[1]	Depth (Type)[2]	Blows				N Value[3]
		1st 6"	2nd 6"	3rd 6"	4th 6"	
SRK09-DHVD01	1.47 - 1.93 m (SM/BR)	37 (rock)	19	22		41
SRK09-DHVD02	2.44 - 3.00 m (SM/SC)	4	4	7	9	11
SRK09-DHVD03	1.22 (SM)	3	5	5		10
	2.74 (SM/GM)	10	23	17		40
	4.27 (ML/SM)	11	36 (cobbles)	23 (stopped)		~
	7.32 (SM/GM)	21	36	Rock (stopped)		~
SRK09-DHVD04A	0 (PT/SM)	4	7	8		15
	2.74 (SW/SM)	5	12	20		32
	7.32 (SW w. GM)	12	23	27		50
SRK09-DHVD04B	4.27 (SM)	12	16	15		31
	8.85 (SM)	17	21	39		60
SRK09-DHVD05	1.22 (PT/SM)	3	3	15		18

1. UTM Projection NAD 27, Zone 08 V.
2. Soil type is designated soil symbol according to the Unified Soil Classification System (USCS); See Table 3.
3. Field N values calculated = sum of blows from 2nd and 3rd 6" intervals.

4.3 Standpipe Piezometers

Standpipe piezometers were installed to provide data on groundwater flow and pore water pressure that should be expected during the VCD construction. Water level readings were taken from the piezometers in detail on July 19, 2009 prior to leaving site. These measurements were taken after completion of piezometer installation and then left to allow time to pass so that piezometric pressures could stabilize and more accurate head measurements could be obtained. The well logs/piezometer as-builts are presented in Appendix C. Table 5 below presents the piezometric measurements gathered on July 19, 2009.

Table 5: Piezometric Measurements (Taken on July 19, 2009)

Location/Well ID[1]	Top of Screen[2,3] (m)	Screened Units[4]	Depth[2] (m)		Height to top of Casing (m)	Water Level Elevation (m)
			To Water	Total Wells[5]		
SRK09-DHVD01	7.92	GM/BR	4.505	11.852	0.794	1197.79
SRK09-DHVD04B	8.35	SM	DRY	12.065	0.687	< 1188.44
SRK09-DHVD05	4.57	Fractured BR	2.667	8.400	0.731	1188.56
SRK09-DHVD06	9.14	SM to GM	8.707	13.043	0.715	1169.71

1. Please refer to 'Table 2: Geotechnical Borehole Locations' for GPS coordinates and original ground surface elevation.
2. Measured vertically down from original ground.
3. All monitoring wells/standpipe piezometers installed had a 3.05m length screen.
4. Soil type is designated soil symbol according to the Unified Soil Classification System (USCS); see Table 3 notes for further details.
5. Includes length of pvc in casing above original ground level.

4.4 Ground Survey - Topography

The results of the 1m resolution ground surface GPS site survey conducted by YES is shown in Figure 1. Additional survey coverage was gained in the areas near the most north-eastern end of the alignment, (expected entrance of Vangorda Creek to the diversion), as well as near the most southwest extents, (where the diversion will again reconnect with the natural alignment of Vangorda Creek). The opposite side of Vangorda Creek was surveyed in order to provide assistance with final design of the VCD earthern intake structure. Near the south-western portion of the VCD alignment the road fill material was observed to be thinner in thickness to the west. Based on the survey contours and field observations it is estimated that this fill material trends from approximately 18 to 22 m in thickness, moving from west to east (near the southwest extents of the proposed VCD alignment). Photographs of approximate road fill thickness and ground conditions near the southwest diversion alignment extents are provided in Appendix F.

4.5 Laboratory Testing

Three overburden samples were subject to basic geotechnical classification testing, with the primary results summarized in Table 6. Complete laboratory sheets are included as Appendix E.

Table 6: Particle Size Analysis Test Results (ASTM D422 & C136)

Soil Type (Field Classification)	Sample Location [Lab ID]	Clay Size (%)	Silt Size (%)	Sand Size (%)	Gravel Size (%)	Moisture Content (%)
SAND TILL - gravelly, some silt to silty, trace clay and cobbles	SRK09-TP02 (4.00m) [TP02][1]	6	21	39	35	9.5
SAND - trace gravel, silt, clay	SRK09-DHVD04B (7.32m) [DH04][1]	1	7	85	8	6.9
SAND and SILT - trace to some gravel, trace clay and cobbles	SRK09-DHVD04B (8.85m) [DHVD04B][1]	5	35	51	10	8.0

Note 1: Values have been extrapolated from the laboratory results presented in Appendix E and grouped by material proportion and grain size as classified by the Modified Unified Soil Classification System.

The samples tested were classified as dense well-graded silty sand with some gravel, typically referred to as a glacial till, and as silty sand. Water content of the samples is estimated to vary around 9.5% in the glacial till to 6.9% in the silty sand stratigraphy leading to an average moisture content within the overburden of 8.1%. During the excavation some boulders were encountered, primarily in the glacial till overburden stratigraphy, these are not reflected in the grain size distribution curves. The larger boulders are expected to be encountered occasionally within the material and would be separated manually from the soil matrix or worked around then fractured into smaller portions to be removed.

5 Final Remarks - General Site Conditions

Based on the completed geologic field investigation outlined in this report, the development of the Vangorda Creek Diversion will encounter the following subsurface geology:

- **Topsoil**; dark brown to black, fibrous, consisting of an organic matt with soft silts, clays, some sand and some to trace gravel.
- **Glacial Till**; dense, greyish brown to medium brown, silty sand to sandy silt with gravel, sub-rounded cobbles, boulders, some clay and exhibiting medium to low plasticity.
- **Silty Sand**; brown-yellow permeable silty sand with some to trace gravel and clay.
- **Granodiorite**; salt and pepper grey to pink grey in colour, generally fair to poor rock quality.
- **Weathered Phyllite**; grey, highly fractured, planar cleavage, oxidized, very poor rock quality.
- **Phyllite**; grey, fractured, quartz veined often altered poor quality erosive rock.

The development of the Vangorda Creek Diversion area will include clearing trees and thick vegetation, as well as grubbing the remaining scrub and low profile vegetation. Typically the overburden glacial till is expected to be removed with a excavator, as was demonstrated during test pitting. It should be noted however, that with increasing depth harder denser till was observed. In areas where the resulting overburden excavation will be greater than six meters or in areas where the denser till is observed to be cobble and boulder rich, progress can be expected to be slow and additional effort beyond excavator digging and dozer stripping should be expected. The weathered phyllite layer generally appears to be favourable for mechanical ripping activities. This weathered phyllitic bedrock unit is expected to be able to be ripped on average for one meter of vertical excavation before refusal on more competent rock results.

Based on piezometric data, water levels were observed to vary from approximately 2.7 to 8.7 m below the original ground surface; frequently above the expected elevation of the proposed Vangorda Creek Diversion invert. In addition to the aforementioned saturated and frozen soil horizons were observed near surface during the field investigation activities. Water inflow on the order of magnitude from 2 L/min to greater than 3 to 5L/s should be expected throughout most of the Vangorda Creek Diversion construction.

At the time of this report only one set of piezometer water level measurements had been collected. A a piezometer monitoring program will be implemented so that additional hydrogeologic data can be collected for further analysis. The piezometer monitoring program for the Vangorda Creek Diversion will comprised bimonthly to monthly piezometer measurements taken until the commencement of the Vangorda Creek Diversion construction activities.

This report, **"1CY001.031.002 – Vangorda Creek Diversion, Geotechnical Field Investigation,"** was prepared by SRK Consulting (Canada) Inc.

Prepared by

John Kurylo

Reviewed by

Peter Healey
Principal Engineer

6 References

Fang, Hsai-Tang, editor (1991). Foundation Engineering Handbook – Second Edition. Kluwer Academic Publishers, sixth printing 2002. pp 22-25.

SRK (1990). Geotechnical/Hydraulic Design, Vangorda Creek Diversion Facility, Vangorda Plateau Development. Report prepared for Curragh Resources Inc. Report 60638. November 1990.

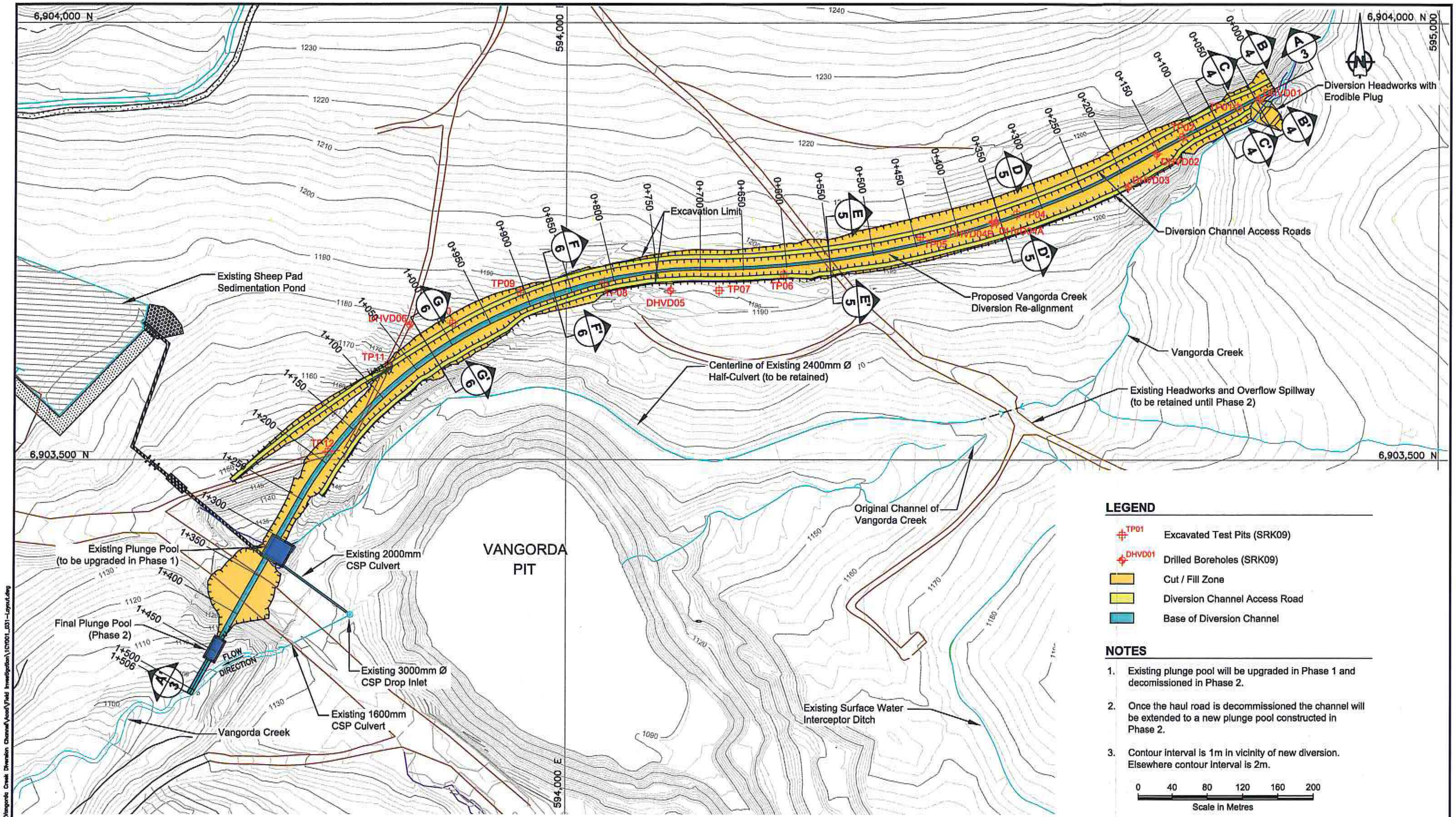
SRK (1991). Construction Report, Vangorda Creek Diversion, Vangorda Plateau Development. Report prepared for Curragh Resources Inc. Report 60639. August 1991.

SRK (2002). Engineering Analysis of Vangorda Pit Wall Stability. Report prepared for Deloitte and Touche Inc. Project No. 1CD003.15. November 2002.

SRK (2004). Vangorda Creek Diversion Inspection Report. Report prepared for Deloitte and Touche Inc. Project No. 1CD003.62. June 2004.

SRK (2004). Vangorda Creek Diversion Design of Emergency Spillway and Recommendations for Flume Upgrade. Report prepared for Deloitte and Touche Inc. Project No. 1CD003.62. October 2004.

Figures



LEGEND

TP01	Excavated Test Pits (SRK09)
DHVD01	Drilled Boreholes (SRK09)
🟧	Cut / Fill Zone
🟨	Diversion Channel Access Road
🟦	Base of Diversion Channel

NOTES

1. Existing plunge pool will be upgraded in Phase 1 and decomissioned in Phase 2.
2. Once the haul road is decommissioned the channel will be extended to a new plunge pool constructed in Phase 2.
3. Contour interval is 1m in vicinity of new diversion. Elsewhere contour interval is 2m.

0 40 80 120 160 200
Scale in Metres

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FILE NAME: 1CY001_031-Layout.dwg

Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Creek Diversion

General Arrangement

DATE:	APPROVED:	FIGURE:
Aug 2009	PMH	1

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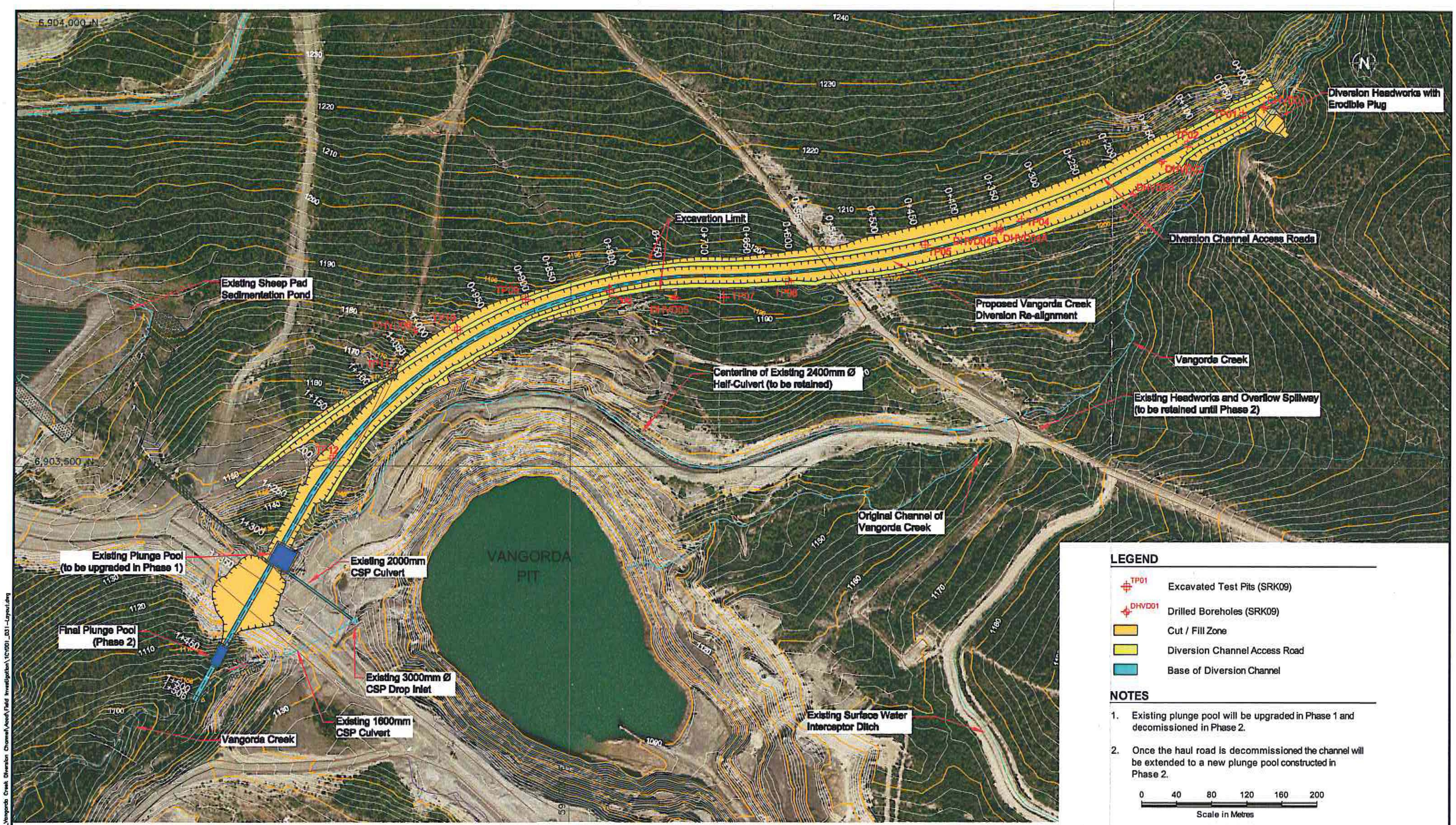

LEGEND

TP01	Excavated Test Pits (SRK09)
DHVD01	Drilled Boreholes (SRK09)
🟧	Cut / Fill Zone
🟨	Diversion Channel Access Road
🟦	Base of Diversion Channel

NOTES

1. Existing plunge pool will be upgraded in Phase 1 and decomissioned in Phase 2.
2. Once the haul road is decommissioned the channel will be extended to a new plunge pool constructed in Phase 2.

0 40 80 120 160 200
Scale in Metres

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Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Creek Diversion

General Arrangement
(With Orthophoto)

DATE: Aug 2009
APPROVED: PMH
FIGURE: 2

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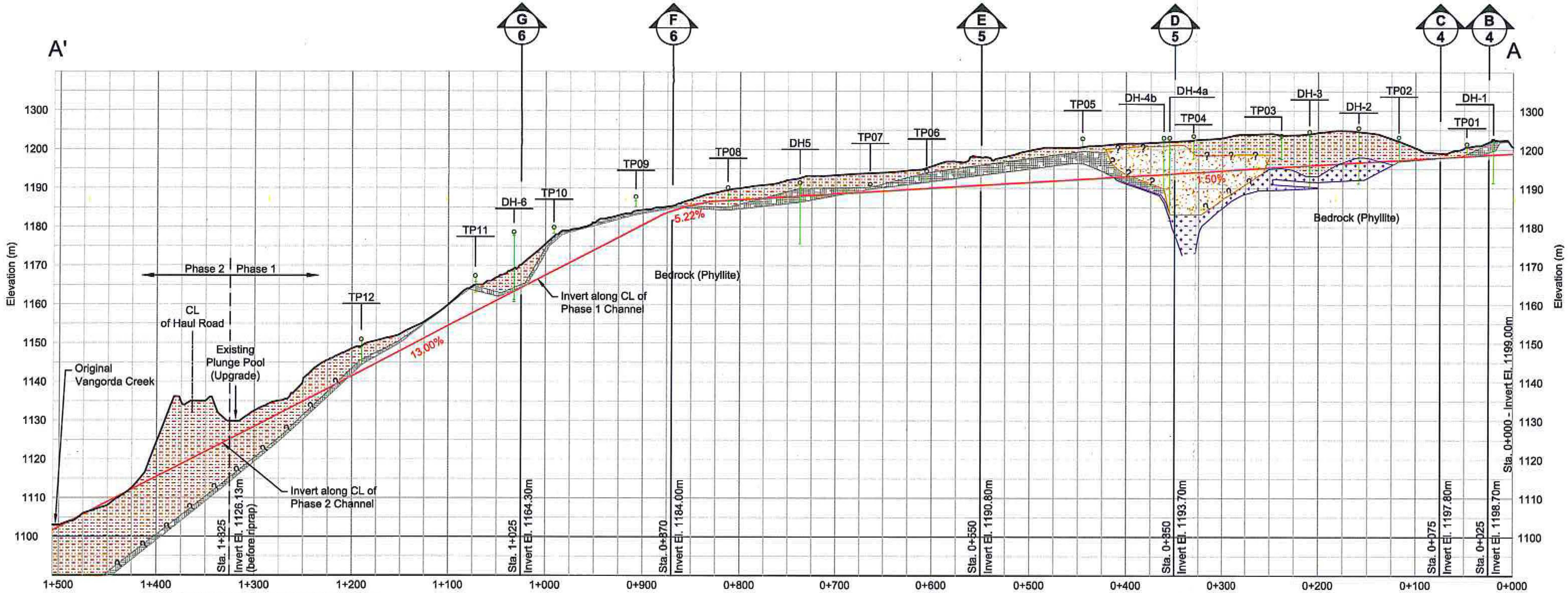

Station	Current Ground Elevation at CL of Station	Design CL Elevation of Vangorda Diversion Channel Invert (before Riprap Placement)
1+500	1103.00	1102.47
1+400	1124.45	1115.47
1+300	1132.50	1128.47
1+200	1148.50	1141.47
1+100	1159.86	1154.47
1+000	1175.50	1167.47
0+900	1184.32	1180.47
0+800	1189.92	1186.98
0+700	1193.33	1188.48
0+600	1195.65	1189.98
0+500	1199.71	1191.49
0+400	1201.53	1192.99
0+300	1202.95	1194.49
0+200	1204.33	1195.99
0+100	1200.40	1197.50
0+000	1200.69	1199.00

LEGEND

- Existing Ground at Centerline
- Proposed Channel Invert
- ? Inferred Strata Contact
- DH-2 Drillhole / Test Pit ID
- Collar Elevation Projected to Alignment Centerline
- Strata Contact
- Drillhole / Test Pit Trace
- ▨ Overburden (Primarily Glacial Till)
- ▨ Silty Sand
- ▨ Granodiorite
- ▨ Weathered Phyllite

Top soil layer varies from 0-0.9m and is not shown for clarity.

4x Vertical Exaggeration

0 10 20 30 40 50

Vertical Scale in Metres

0 40 80 120 160 200

Horizontal Scale in Metres

Overburden describes on grouped unit of primary glacial till with road/fill material near the areas around and south of the existing plunge pool.

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FILE NAME: 1CY001_031-4-Profile.dwg

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Creek Diversion

Profile Along Centerline of Diversion (A-A')

DATE: Aug 2009
APPROVED: PMH
FIGURE: 3

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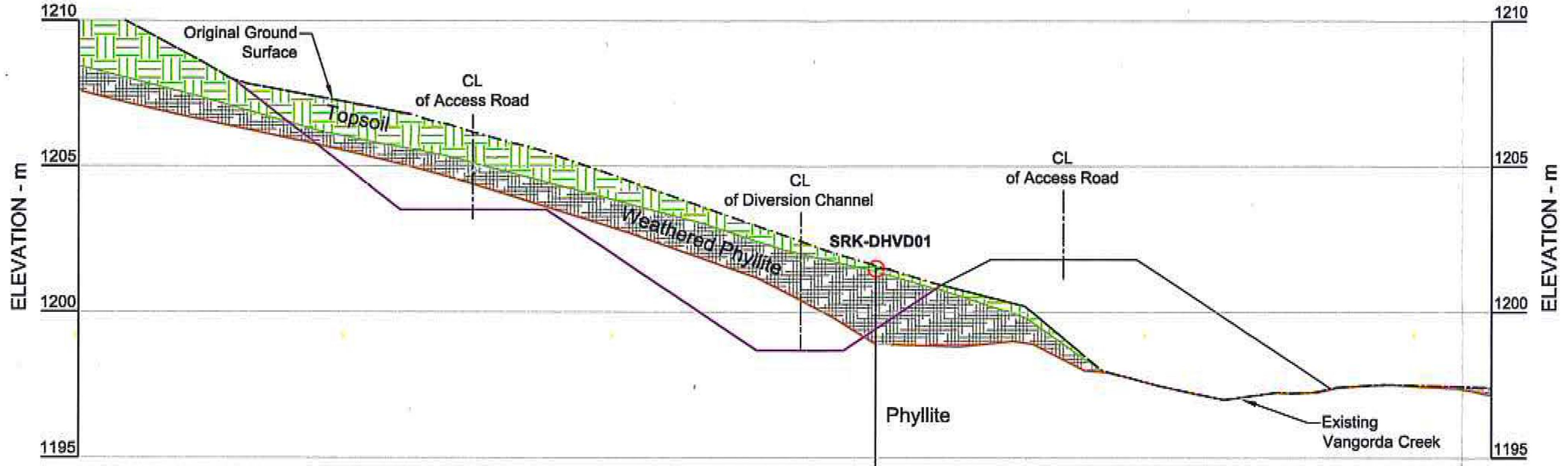
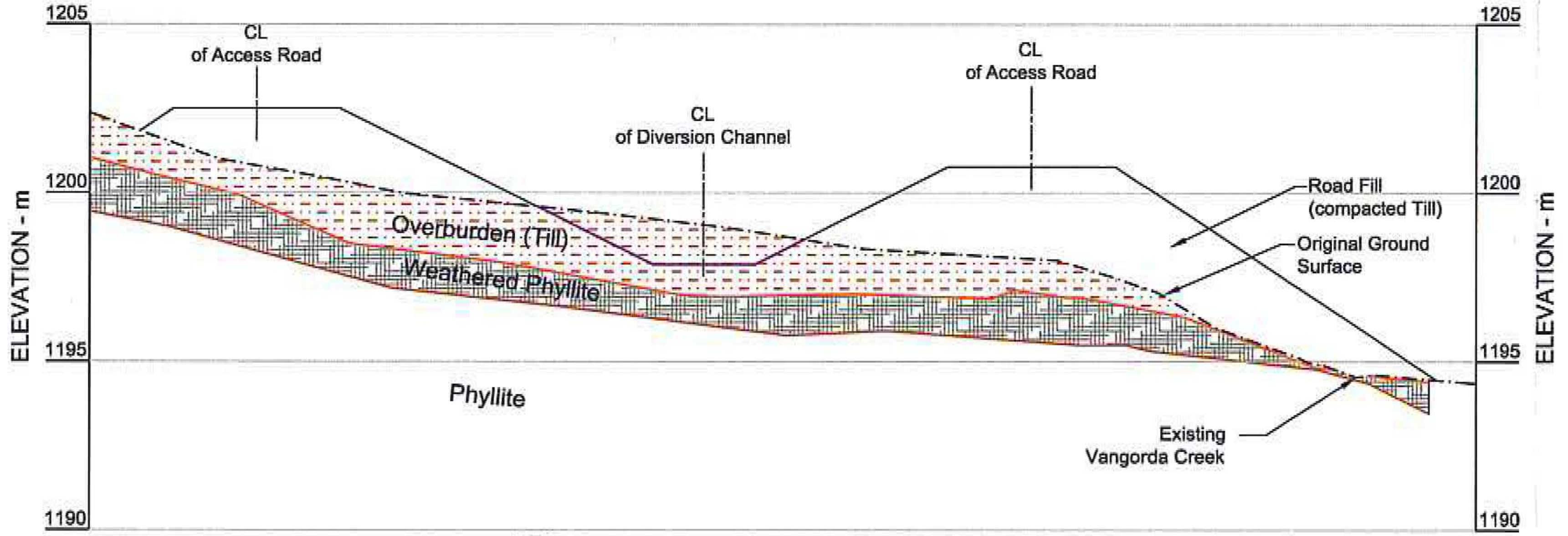


B/4 SECTION AT STATION 0+025

0 5 10 15 20 25
Scale in Metres



C/4 SECTION AT STATION 0+075

0 5 10 15 20 25
Scale in Metres

LEGEND

-·-·- Existing Ground at Section
— Proposed Channel Base
— Topsoil Base
— Till Base
— Weathered Phyllite Base

DH-2 Drillhole / Test Pit ID
Collar Elevation Projected to Alignment Centerline
Strata Contact
Drillhole / Test Pit Trace

▨ Topsoil
▨ Overburden (Primarly Glacial Till)
▨ Silty Sand
▨ Granodiorite
▨ Weathered Phyllite

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FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Creek Diversion

Geologic Cross-Sections at 0+025 and 0+075

DATE: Aug 2009
APPROVED: PMH
FIGURE: 4

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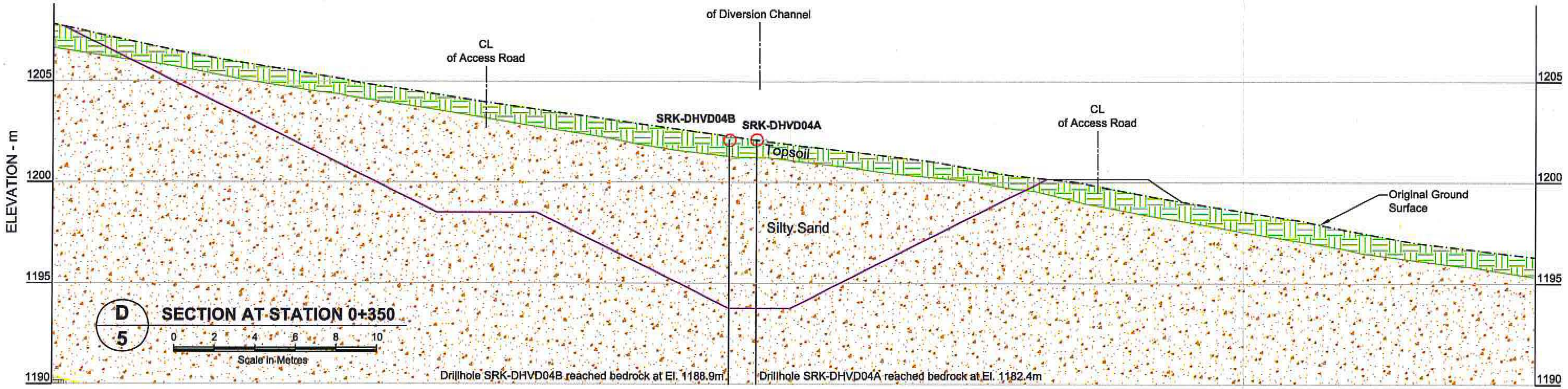
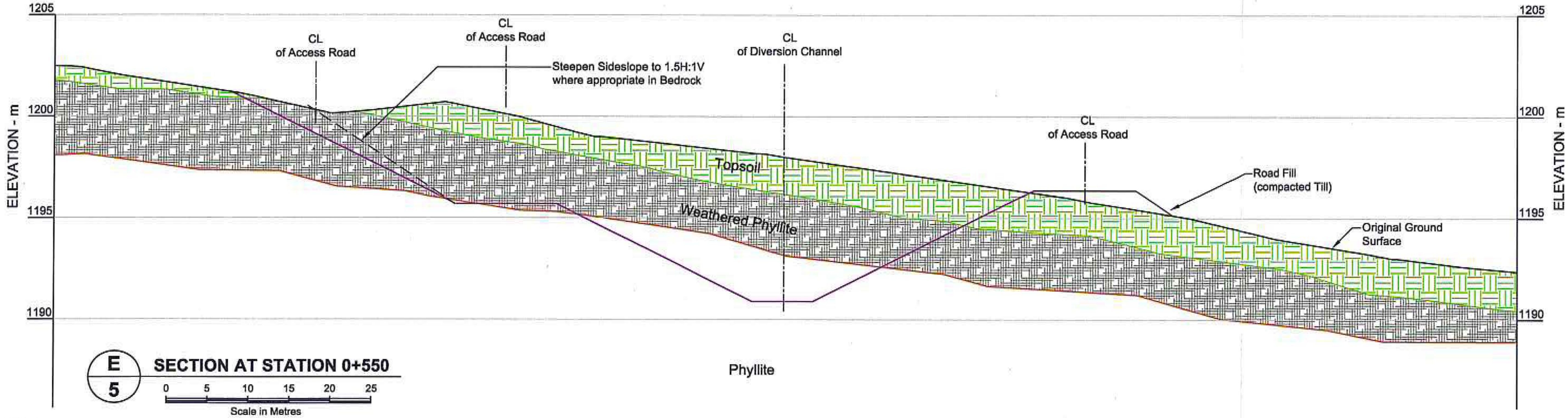


D
5
SECTION AT STATION 0+350



E
5
SECTION AT STATION 0+550

LEGEND

–·–·–	Existing Ground at Section
──	Proposed Channel Base
──	Topsoil Base
──	Till Base
──	Weathered Phyllite Base

DH-2 — Drillhole / Test Pit ID
○ — Collar Elevation Projected to Alignment Centerline
Strata Contact
Drillhole / Test Pit Trace

▦	Topsoil
▤	Overburden (Primarly Glacial Till)
░	Silty Sand
⁘	Granodiorite
▩	Weathered Phyllite

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FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Creek Diversion

Geologic Cross-Sections at 0+350 and 0+550

DATE:	APPROVED:	FIGURE:
Aug 2009	PMH	5

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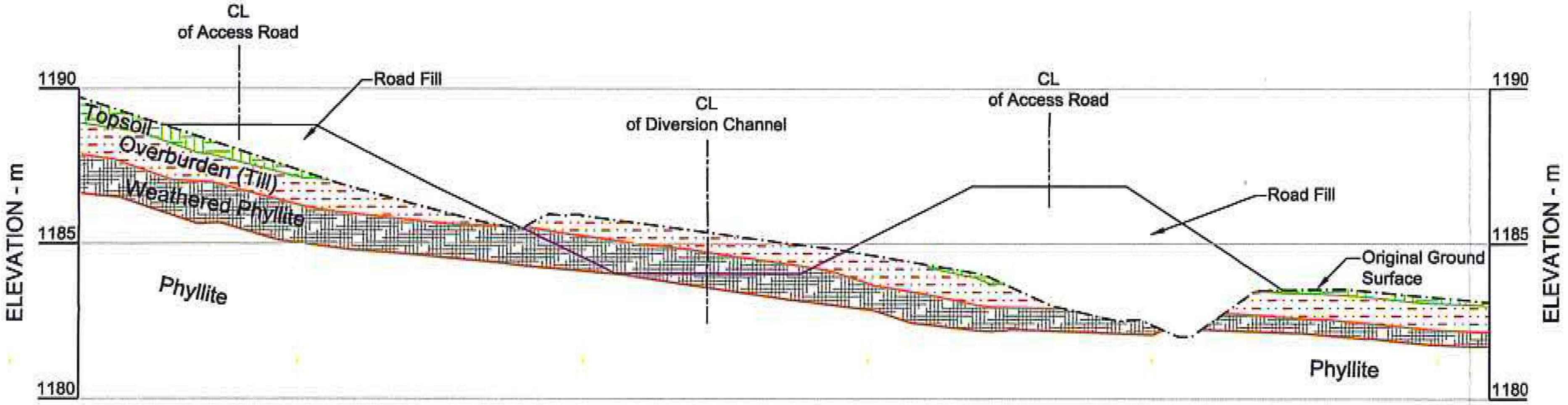
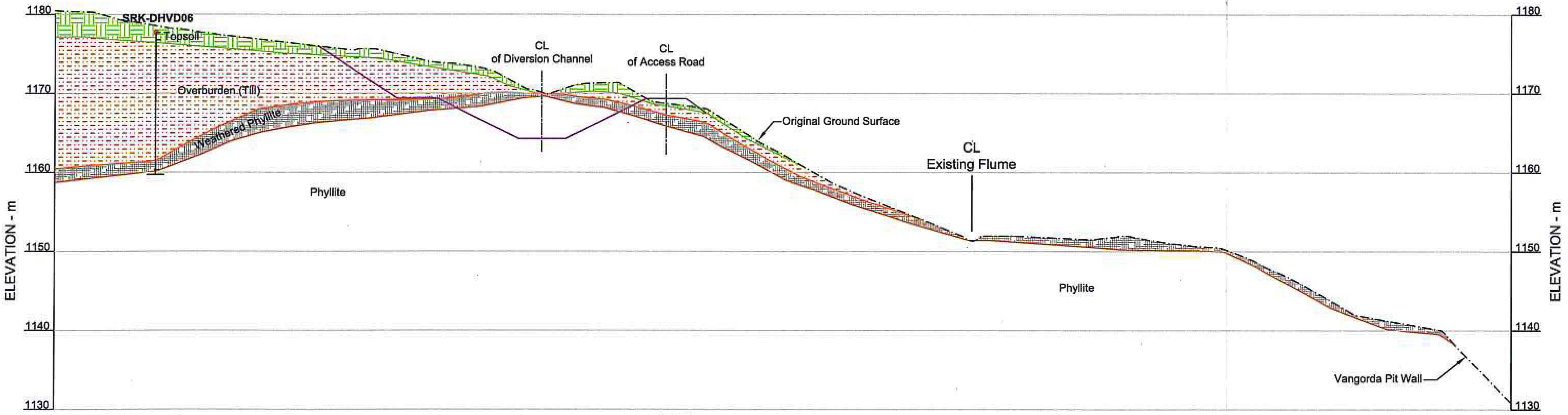


F
6
SECTION AT STATION 0+870

0 2 4 6 8 10
Scale in Metres



G
6
SECTION AT STATION 1+025

0 5 10 15 20 25
Scale in Metres

LEGEND

-·-·- Existing Ground at Section
Proposed Channel Base
Topsoil Base
Till Base
Weathered Phyllite Base

DH-2 Drillhole / Test Pit ID
Collar Elevation Projected to Alignment Centerline
Strata Contact
Drillhole / Test Pit Trace

▨ Topsoil
▨ Overburden (Primarly Glacial Till)
▨ Silty Sand
▨ Granodiorite
▨ Weathered Phyllite

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FILE NAME: 1CY001_031–Sections.dwg

Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

Vangorda Creek Diversion

Geologic Cross-Sections at
0+870 and 1+025

DATE:	APPROVED:	FIGURE:
Aug 2009	PMH	6

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Appendices

Appendix A

Test Pit Logs

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
PAGE: 1 **OF** 1
FILE No: FARO (1CY001.031)
DRILL TYPE: Excavator
BORING DATE: 2009-06-01 **TO** 2009-06-01
DRILL: CAT 345C Excavator
DIP: 90.00 **AZIMUTH:**
CASING: NA
COORDINATES: 6903901.90 N 594771.20 E **DATUM:** UTM NAD 27, 08 V

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND

■ Bentonite Grout	░ Sand
▧ Bentonite	
☐ Cuttings	
▨ Cement	

SOIL LEGEND

〰 Topsoil	● Gravel
● Boulder	░ Sand
▨ Clay	▥ Silt
● Cobble	

SAMPLE CONDITIONS

- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◨ Core

SAMPLE TYPE

- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	STRATIGRAPHY DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1200.50							
			0.00	ORGANIC/ VEGETATION MATT	〰					
			1200.30 0.20	Dark TOPSOIL	〰					
			1200.00 0.50	Medium brown subangular to rounded clast. COBBLES, SAND, some gravel and silt. TILL.	●					
5	1 2		1199.50 1.00	Medium brown subangular to rounded clast. COBBLES, SILT, SAND, some gravel. TILL. Grab sample taken at 2.75m.	●	BS-Grab 1	☒	0	0	
			1197.60 2.90	Refusal on BEDROCK	▤					
10	3		1197.50 3.00	END OF HOLE						

J106_REFERENCE_MATERIAL\Sgeotec\lod\templates\boring_SRK_FARO_JK_bh#3.sty PLOTTED: 2009-07-27 17:05hrs

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-01 **TO** 2009-06-01
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903868.30 N 594708.70 E **DATUM:** UTM NAD 27, 08V

PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▨ Bentonite
- Cuttings
- Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W P W W L 20 40 60 80
			1202.20 0.00	VEGETATION MATT/moss						
			1201.90 0.30	Dark brown silty CLAY (frozen) with some sand and gravel Seepage observed at ~0.5m						
	1		1201.20 1.00	Dark to medium greyish brown GRAVEL, COBBLE, SILT some clay. This OVERBURDEN was frozen and became slightly more frozen with depth.						
5	2			Considerable more boulders observed after 2m depth.						
10	3									
	4		1198.20 4.00	SILT and GRAVEL some cobbles, boulders and some to trace clay; medium plasticity. Grab sample taken at 4m. GRAIN SIZE LAB RESULTS: 6% Clay 21% Silt 39% Sand 39% Gravel		BS-Grab 1	☒	0	0	▲
15	5									
20	6		1196.00 6.20	END OF HOLE - Refusal on frozen ground						

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-01 **TO** 2009-06-01
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903799.60 N 594629.40 E **DATUM:** UTM NAD 27, 08 V

PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND		**SOIL LEGEND**		**SAMPLE CONDITIONS**	**SAMPLE TYPE**
Bentonite Grout	Sand	Topsoil	Gravel	☒ Remoulded	DC Diamond Core
Bentonite		Boulder	Sand	▨ Undisturbed	GS Grab Sample
Cuttings		Clay	Silt	■ Lost	SS Split Spoon
Cement		Cobble		Core	

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY ELEVATION - m DEPTH - m	DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1202.60 0.00	VEGATATION MATT (moss, bush)						
			1202.45 0.15	Grey SILT						
			1202.30 0.30	Light brown TILL lots of cobbles, boulders and sand some silt and trace clay. No water observed Roots to 0.5m						
5	1		1201.10 1.50	Medium to dark brown TILL lots of cobble and boulders, sand some silt and trace clay. Grab sample taken from gravel and cobble rich section at ~2m depth.		BS-Grab 1	☒	0	0	
10	2 3		1199.60 3.00	Dark brown-grey SAND some cobbles, gravel and some clay						
15 20	4 5 6		1196.60 6.00	END OF HOLE - Refusal on boulders and frozen ground. No seepage observed in test pit.						

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-01 **TO** 2009-06-01
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903779.80 N 594516.70 E **DATUM:** UTM NAD 27, 08 V

PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- 🟩 Cuttings
- ▨ Cement

SOIL LEGEND
- ≈ Topsoil
- 🟩 Gravel
- ▨ Boulder
- ▨ Sand
- ▧ Clay
- ▥ Silt
- 🟥 Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY ELEVATION - m / DEPTH - m	STRATIGRAPHY DESCRIPTION	STRATIGRAPHY SYMBOL	SAMPLES TYPE AND NUMBER	SAMPLES CONDITION	SAMPLES RECOVERY %	SAMPLES N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1202.50 0.00	VEGATATION MATT/moss	≈					
			1202.20 0.30	Frozen dark/black organic rich TOPSOIL	▨					
			1202.00 0.50	Greyish brown, COBBLES, GRAVEL and SILT, some boulders, sand and clay	▥					
	1									
5										
	2									
10	3		1199.50 3.00	Medium greyish brown, tightly packed greyish brown, COBBLES, GRAVEL and SILT, some boulders, sand and clay. More silt then above; low plasticity.	▥					
	4									
15										
	5			Grab sample taken from ~5-5.5m depth		BS-Grab 1	☒	0	0	
			1196.70 5.80	Excavator progressed at a slower rate/harder digging in this test pit due to the high frequency of boulders observed. END OF HOLE - Refusal on tightly packed grey boulders						
20	6									

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-01 **TO** 2009-06-01
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903753.30 N 594406.10 E **DATUM:** UTM NAD 27, 08 V

PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ⛆ Sand
- ▧ Bentonite
- ● Cuttings
- ▨ Cement

SOIL LEGEND
- 〰 Topsoil
- ● Gravel
- ● Boulder
- ⛆ Sand
- ▨ Clay
- ▥ Silt
- ● Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1201.90 0.00	ORGANIC/ VEGETATION MATT, moss shrubs etc..	〰					
			1201.80 0.10	TOPSOIL, lots of organics	〰					
			1201.60 0.30	Grey SILT, some sand and clay	▥					
	1		1201.50 0.40	Highly WEATHERED and oxidized brown PHYLLITE with some quartz veins						
5	2		1200.40 1.50	Hard digging, less weathered grey PHYLLITE. ~0.3cm think planar cleavage sheets	▤					
			1199.40 2.50	END OF HOLE - Refusal on bedrock						

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
PAGE: 1 **OF** 1
FILE No: FARO (1CY001.031)
DRILL TYPE: Excavator
BORING DATE: 2009-06-02 **TO** 2009-06-02
DRILL: CAT 345C Excavator
DIP: 90.00 **AZIMUTH:**
CASING: NA
COORDINATES: 6903710.50 N 594250.50 E **DATUM:** UTM NAD 27, 08 V

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- ● Cuttings
- ▽ Cement

SOIL LEGEND
- ~ Topsoil
- ● Gravel
- ● Boulder
- ▨ Sand
- ▧ Clay
- ▥ Silt
- ● Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1193.40 0.00	TOPSOIL, lots of organics SW of SRK09-DHVD05						
			1193.20 0.20	Siltier TOPSOIL to really weathered phyllite						
			1193.10 0.30	Highly WEATHERED grey PHYLLITE; easy for excavator to rip.						
	1									
5										
			1191.60 1.80	WEATHERED PHYLLITE; still able to be ripped but slightly more competent than above. seepage/ water inflow at ~1.81m depth. On the order of 2 L/s.						
	2									
			1190.60 1.81 1191.40 2.80	END OF HOLE - Refusal on bedrock						

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-02 **TO** 2009-06-02
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903692.50 N 594175.20 E **DATUM:** UTM NAD 27, 08 V

PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

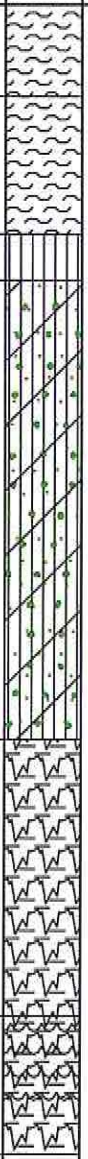
WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- Cuttings
- Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	STRATIGRAPHY DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1190.50 0.00	ORGANIC/ VEGETATION MATT (moss, roots, etc..)						
			1190.40 0.10	Black frozen organic TOPSOIL						
			1190.25 0.25	Cream/white TEPHRA/ volcanic ash						
			1190.20 0.30	Compact/dense organic rich SILT, some clay, gravel and sand						
			1189.70 0.80	Rippable WEATHERED PHYLLITE. Bedding dipping at ~20 to 25 degrees from the horizontal.						
	1									
				Seepage: ~2L/min at ~1.1m depth.						
			1189.25 1.10	END OF HOLE - Refusal on bedrock						
			1189.30 1.25							



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TEST PIT LOG

PROJECT: Vangorda Diversion Field Investigation
BOREHOLE: [illegible]
LOCATION: Vangorda Diversion Preliminary Alignment
PAGE: 1 **OF** 1
FILE No: FARO (1CY001.031)
DRILL TYPE: Excavator
BORING DATE: 2009-06-02 **TO** 2009-06-02
DRILL: CAT 345C Excavator
DIP: 90.00 **AZIMUTH:**
CASING: NA
COORDINATES: 6903701.00 N 594043.80 E **DATUM:** UTM NAD 27, 08 V

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- Bentonite Grout
- Sand
- Bentonite
- Cuttings
- Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY ELEVATION - m DEPTH - m	DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1189.20 0.00	Dark brown to black TOPSOIL, organic rich Soft ground near test pit location. Ground deflection observed as the excavator progressed through this area and as well the excavator experienced minor settlement into the topsoil.						
	1		1188.70 0.50	TILL. Lots of COBBLES, GRAVEL and silty SAND, occassional boulders						
5 10	2 3		1187.70 1.50	Saturated soil. Operator commented that this soil was very dense and heavy to move. Medium brown SILT, CLAY and GRAVEL, some cobbles occassional boulders. Heavy/quick seepage: ~ 5L/s seepage observed at 1.5m depth. Excavation experience sluffing as water flowed intot he open test pit.						
15	4		1185.70 3.50	Very dense CLAY and SILTY TILL.Medium brown SILT, CLAY and GRAVEL, some cobbles occassional boulders.						
	5		1184.20 5.00	Slightly WEATHERED grey PHYLLITE		BS-Grab 1	☒	0	0	
			1183.40 5.80	END OF HOLE - Refusal on bedrock						

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TEST PIT LOG

LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-02 **TO** 2009-06-02
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903654.50 N 593948.40 E **DATUM:** UTM NAD 27, 08 V

PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Bentonite
- Cuttings
- Cement
- Sand

SOIL LEGEND
- Topsoil
- Boulder
- Clay
- Cobble
- Gravel
- Sand
- Silt

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY: ELEVATION - m / DEPTH - m	STRATIGRAPHY: DESCRIPTION	STRATIGRAPHY: SYMBOL	SAMPLES: TYPE AND NUMBER	SAMPLES: CONDITION	SAMPLES: RECOVERY %	SAMPLES: N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1186.80 0.00	VEGETATION MATT (moss roots, etc..)□□VEGETATION MATT (moss, roots, grass, shrubs)						
			1186.70 0.10	Frozen black TOPSOIL, lots of organics, some sand, silt and gravel						
			1186.40 0.40	Dark grey brown to black GRAVEL, SILT, SAND, trace clay, some organics and cobbles. TILL						
5	1 2		1185.80 1.00	WEATHERED PHYLLITE, rippable. Some redish oxidation staining. Planar near horizontal bedding. Evidence of small scale folding can be seen. No seepage observed in test pit.						
			1184.70 2.10	END OF HOLE - Refusal on bedrock						

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TEST PIT LOG

PROJECT: Vangorda Diversion Field Investigation
BOREHOLE: [illegible]
LOCATION: Vangorda Diversion Preliminary Alignment
PAGE: 1 **OF** 1
FILE No: FARO (1CY001.031)
DRILL TYPE: Excavator
BORING DATE: 2009-06-01 **TO** 2009-06-01
DRILL: CAT 345C Excavator
DIP: 90.00 **AZIMUTH:**
CASING: NA
COORDINATES: 6903654.50 N 593871.00 E **DATUM:** UTM NAD 27, 08 V

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- ● Cuttings
- ▨ Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1178.90 0.00	VEGETATION MATT (moss, roots, grass etc..) Water observed ponded near area						
			1178.85 0.05	Dark TOPSOIL high silt and sand content some gravel.						
			1178.65 0.25	highly WEATHERED/ fractured and slightyl oxidized grey brown PHYLLITE. Sub horizontal dip (~20deg). Can see evidence of folding.						
	1		1177.90 1.00	Slightly dense/ more competent WEATHERED PHYLLITE Seepage ~ 3-5L/s at 1m depth.						
5										
			1177.30 1.00	END OF HOLE - Refusal on bedrock						
			1177.70 1.60							

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TEST PIT LOG

PROJECT: Vangorda Diversion Field Investigation
LOCATION: Vangorda Diversion Preliminary Assignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-02 **TO** 2009-06-02
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903607.40 N 593797.20 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-TP11
PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- ● Cuttings
- ▨ Cement

SOIL LEGEND
- ▨ Topsoil
- ▨ Gravel
- ● Boulder
- ▨ Sand
- ▧ Clay
- ▥ Silt
- ● Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m DEPTH - m	STRATIGRAPHY DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1166.40 0.00	VEGETATION MATT (moss, grass, roots, organics)						
			1166.35 0.05	Medium brown sandy TILL. Rounded COBBLES, SILT, SAND, some gravel and trace clay and boulders						
	1		1165.90 0.50	Medium brown sandy TILL. SILT, SAND, some gravel, cobbles and trace clay and boulders						
5 10	2 3		1164.90 1.50	WEATHERED gey PHYLLITE. Planar near horizontal cleavage. No oxidization observed						
	4		1162.90 3.50	Grey PHYLLITE. Much more competent than section above.						
			1162.10 4.30	END OF HOLE - Refusal on bedrock						

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TEST PIT LOG

PROJECT: Vangorda Diversion Field Investigation
LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-02 **TO** 2009-06-02
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903507.80 N 593728.50 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-TP12
PAGE: 1 **OF** 1
DRILL TYPE: Excavator
DRILL: CAT 345C Excavator
CASING: NA

GENERAL COMMENTS:
Cleared route/road for drill access first then excavated TEST PIT.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- ● Cuttings
- ▨ Cement

SOIL LEGEND
- ~ Topsoil
- ● Gravel
- ● Boulder
- ▨ Sand
- ▧ Clay
- ▥ Silt
- ● Cobble

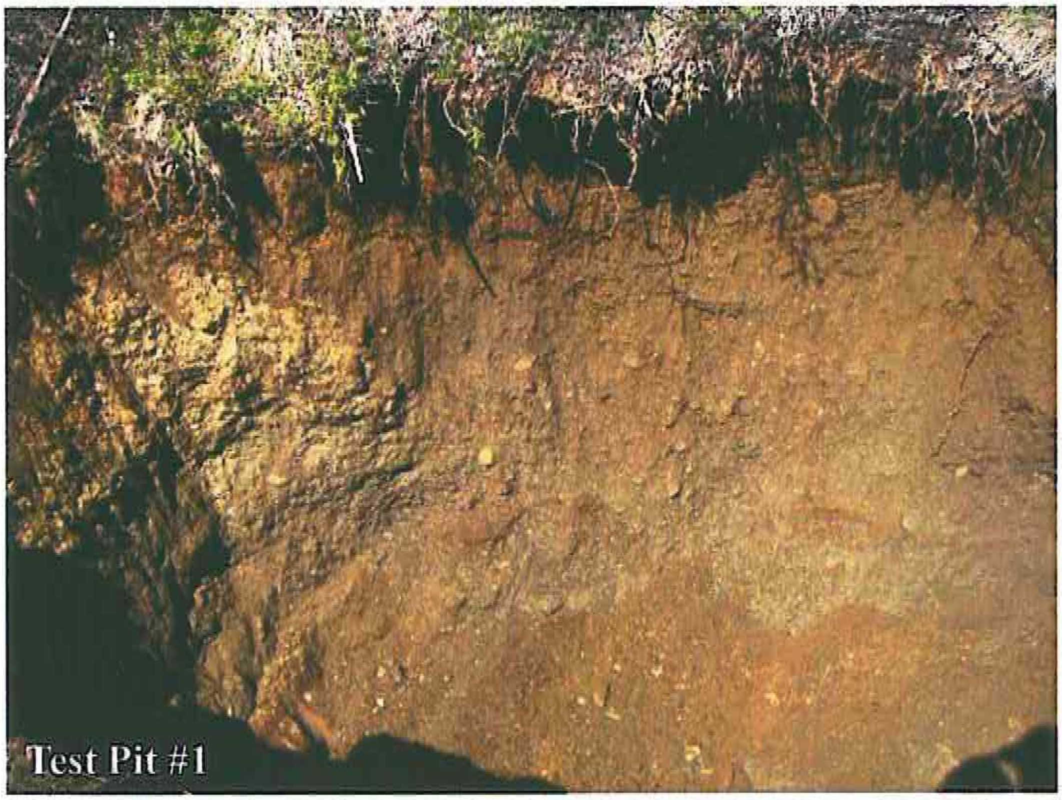
SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1150.00 0.00	FILL, brough to area to build road surface. Comprised of: overburden/till with some weathered phyllite. Similar to the soil observed at the W wall of the Grum 'Slot Cut' area. Dark grey SILT, some sand and gravel, trace clay.						
5	1									
	2		1148.00 2.00	Medium brown sandy silty TILL. SAND and SILT some cobbles and gravel.						
			1147.60 2.40	ORGANICS. Wood, roots, lots of tree remains.						
10	3		1147.30 2.70	Dense brown TILL. Clayey SILT and GRAVEL some to trace cobbles						
	4		1146.20 3.80	Dense cobble rich TILL. SAND, COBBLES, some silt. Seepage near base of layer.						
15			1145.40 4.50	Seepage: Observed at ~ 4.5m depth, ~5L/min flow.						
20	5 6		1145.40 4.60	WEATHERED PHYLLITE. Rippable, wet, neat horizontal planar cleavage.						
			1143.60 6.40	END OF HOLE - Refusal on bedrock						

Appendix B

Test Pit Photographs



Test Pit #1

Test Pit #2

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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion Field Investigation

Test Pit Photographs

Job No: 1CY001.031 Filename: APPENDIX B – Test Pit Photographs	**Faro Mine Complex**	Date: July 2009	Approved: JBK	Photo Page: 1

Test Pit #3

Test Pit #4

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VANCOUVER

Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversi
Investigation

Test Pit Photogra

Job No: 1CY001.031
Filename: APPENDIX B – Test Pit Photographs

Faro Mine Complex

Date: July 2009

Approved: JBK

Test Pit #5

Test Pit #6

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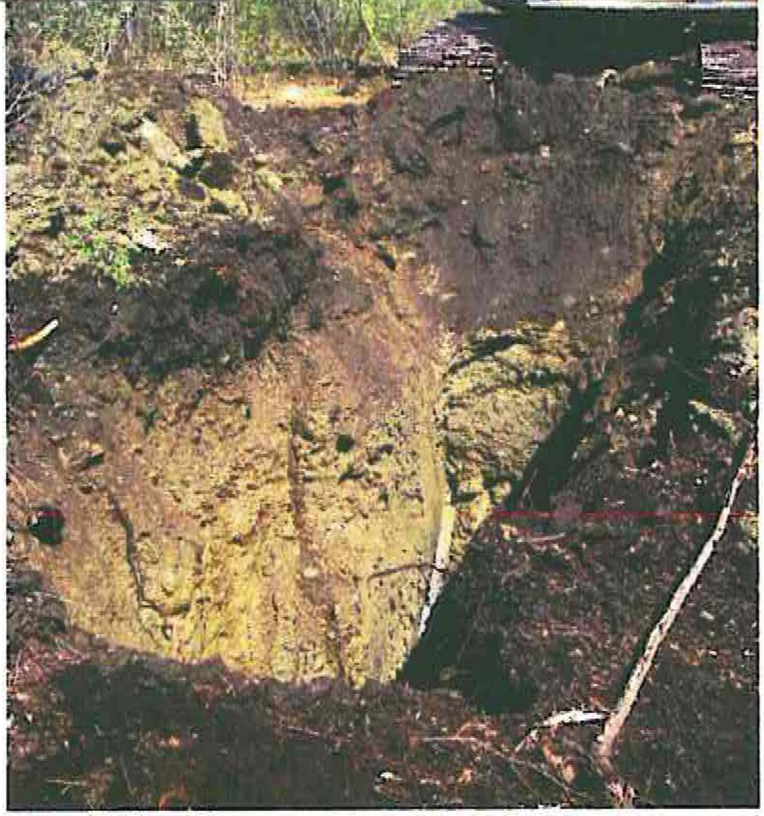
Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion Field Investigation

Test Pit Photographs

Job No: 1CY001.031 Filename: APPENDIX B – Test Pit Photographs	Faro Mine Complex	Date: July 2009	Approved: JBK	Photo Page: 3

Test Pit #7



Test Pit #8

SRK Consulting
Engineers and Scientists
VANCOUVER

Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion Field Investigation

Test Pit Photographs

Job No: 1CY001.031
Filename: APPENDIX B – Test Pit Photographs

Faro Mine Complex

Date: July 2009
Approved: JBK
Photo Page: 4

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Engineers and Scientists

BOREHOLE LOG

PROJECT: Vangorda Diversion Field Investigation
LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-07-07 **TO** 2009-07-07
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903770.70 N 594493.30 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-DHVD04A
PAGE: 1 **OF** 3
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

GENERAL COMMENTS:
Massive silty sand unit encountered at this location. Hole has been backfilled with cuttings and then capped with bentonite chips to surface.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- Cuttings
- Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- ▨ Clay
- ▥ Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W
			1202.10 0.00	Organic rich TOPSOIL	Topsoil					
			1202.00 0.10	STP done from surface, 0m. SPT Sample: 1st 10cm are a organic rich topsil to vegatation mat. Black to dark brown organic rich SILT, some clay and sand, with trace to no gravel. A small amoun tof intermixed white tephra layer observed near 0.4m depth. High moisture content to saturated; medium plasticity Blows 4 1st 6" 7 2nd 6" 8 3rd 6" Black to dark brown organic rich SILT, some clay and sand, with trace to no gravel.	Silt, Clay	SS-SPT 1	▨	0	15 0	
5	1		1201.19 0.91	Medium yellowish brown silty SAND, some gravel	Sand, Gravel					
10	2 3		1199.36 2.74	Light yellow-brown coarse SAND, some silt and gravel, trace to no clay. SPT done at 2.74m depth. SPT Sample: Light brown coarse sand, some silt and gravel, trace to no clay. Blows 5 1st 6" 12 2nd 6" 20 3rd 6"	Sand	SS-SPT 2	▨	0	32 0	
15	4 5				Sand					
20	6		1196.61 5.49	Light yellow-brown coarse SAND then fine sand section, some silt, trace to no clay, trace to no gravel	Sand					

WATER CONTENT and LIMITS (%): W_P W W_L; 20 40 60 80

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CASING: Steel

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Massive silty sand unit encountered at this location. Hole has been backfilled with cuttings and then capped with bentonite chips to surface.

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- ▧ Bentonite
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- ▵ Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- ▧ Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
25	8		1194.78 7.32	Light brown dense SAND with gravel and some silt SPT done at 7.32m depth. SPT Sample: Light yellow brown, dense sand with gravel, some silt and trace to no clay; fairly well sorted but trending towards the finer envelope. Moisture from the drill rods observed at the top of the sample.	SS-SPT 3	▨	0	50 0	▲
30	9		1194.18 7.92	GRAIN SIZE LAB RESULTS: 1% Clay 7% Silt 85% Sand 8% Gravel Blows 12 1st 6" 23 2nd 6" 27 3rd 6" Light yellow brown SAND with some fines, silt and clay					
35	10 11 12								
40	13		1189.91 12.19	Light yellow brown SAND with some fines (mainly silt), and some gravel					
45	14		1188.38 13.72	Light yellow brown SAND with some fines (maily silt), and some gravel; maily fine sand and fines, low plasticity					

J:\06 REFERENCE MATERIAL\Sigeotec log\templates\loglog SRK FARO JK bh#3.sty PLOTTED: 2009-07-09 08:57hrs

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BOREHOLE LOG

PROJECT: Vangorda Diversion Field Investigation
LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-07-07 **TO** 2009-07-07
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903770.70 N 594493.30 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-DHVD04A
PAGE: 3 **OF** 3
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

GENERAL COMMENTS:
Massive silty sand unit encountered at this location. Hole has been backfilled with cuttings and then capped with bentonite chips to surface.

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- ● Cuttings
- ▨ Cement

SOIL LEGEND
- ▨ Topsoil
- ● Gravel
- ● Boulder
- ▨ Sand
- ▨ Clay
- ▥ Silt
- ● Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◨ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
50										
			1186.56 15.53	Light yellow brown SAND with some gravel some silt and some trace clay Grab sample of the light yellow brown SAND with some fines (silt and clay), and some gravel taken at 15.54m depth.		GS-Grab 1	☒	0	0	
	16									
55	17									
60	18									
	19									
			1182.90 19.20	Light yellow brown SAND with some gravel, silt and some to trace clay. PHYLLITE FRAGMENTS starting to be observed in cuttings	SMGK					
			1182.44 19.66	Cuttings: IGNEOUS BEDROCK. Appears to be similar to granodiorite unite previously observed around this area.	+ +					
65	20									
			1181.98 20.12	Cuttings:fine powder with light to pinkish brown GRANODIORITE rock fragments	+ + +					
			1181.37 20.73	Cuttings: Apppear to be transitioning back into the PHYLLITE unit. Phyllite chips exhibit planar cleavage as expected.	■					
	21									
70			1180.76 21.34	END OF HOLE						
	22									

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BOREHOLE LOG

PROJECT: Vangorda Diversion Field Investigation
LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-09 **TO** 2009-06-09
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903769.40 N 594488.10 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-DHVD04B
PAGE: 1 **OF** 2
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

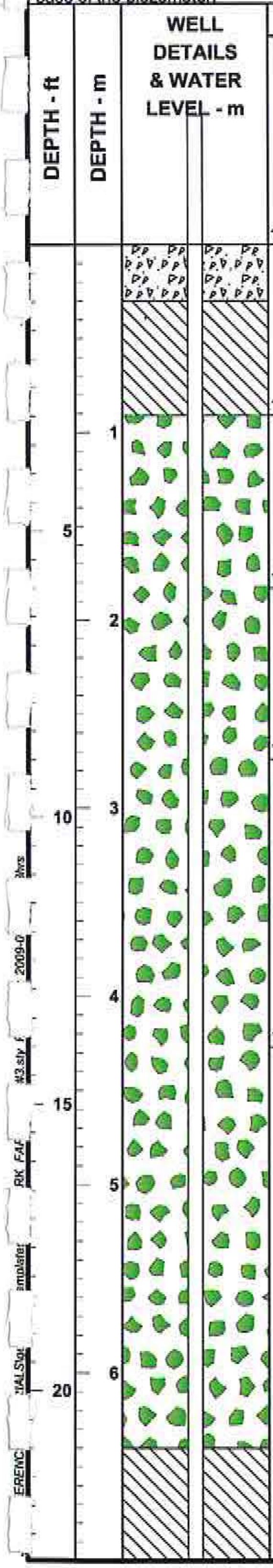
GENERAL COMMENTS:
WELL DETAILS: (June 19th, 2009)
-Depth to water = DRY
-Total depth of well= 12.065m
-Height to top of casing/pvc= 0.687
-A small amount of moisture appeared to be at the base of the piezometer.

WELL PLUG MATERIAL LEGEND
- Bentonite Grout
- Sand
- Bentonite
- Cuttings
- Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon



STRATIGRAPHY			SAMPLES				WATER CONTENT and LIMITS (%)
ELEVATION - m DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	W_P W W_L 20 40 60 80
1202.00 0.00	Dark brown/black TOPSOIL						
1201.09 0.91	Light to med yellow brown coarse SAND and GRAVEL with swilt						
1200.17 1.83	In medium yellow SAND some silt						
1199.26 2.74	Light brown SAND, some silt						
1197.73 4.27	Finer light yellow-brown SAND some silt, some to trace gravel SPT done at 4.27m depth. SPT Sample: light brown sand and gravel, some silt, fairly well graded, moist. Blows 12 1st 6" 16 2nd 6" 15 3rd 6"		SS-SPT 1	▨	0	31 0	

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DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903769.40 N 594488.10 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-DHVD04B
PAGE: 2 **OF** 2
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

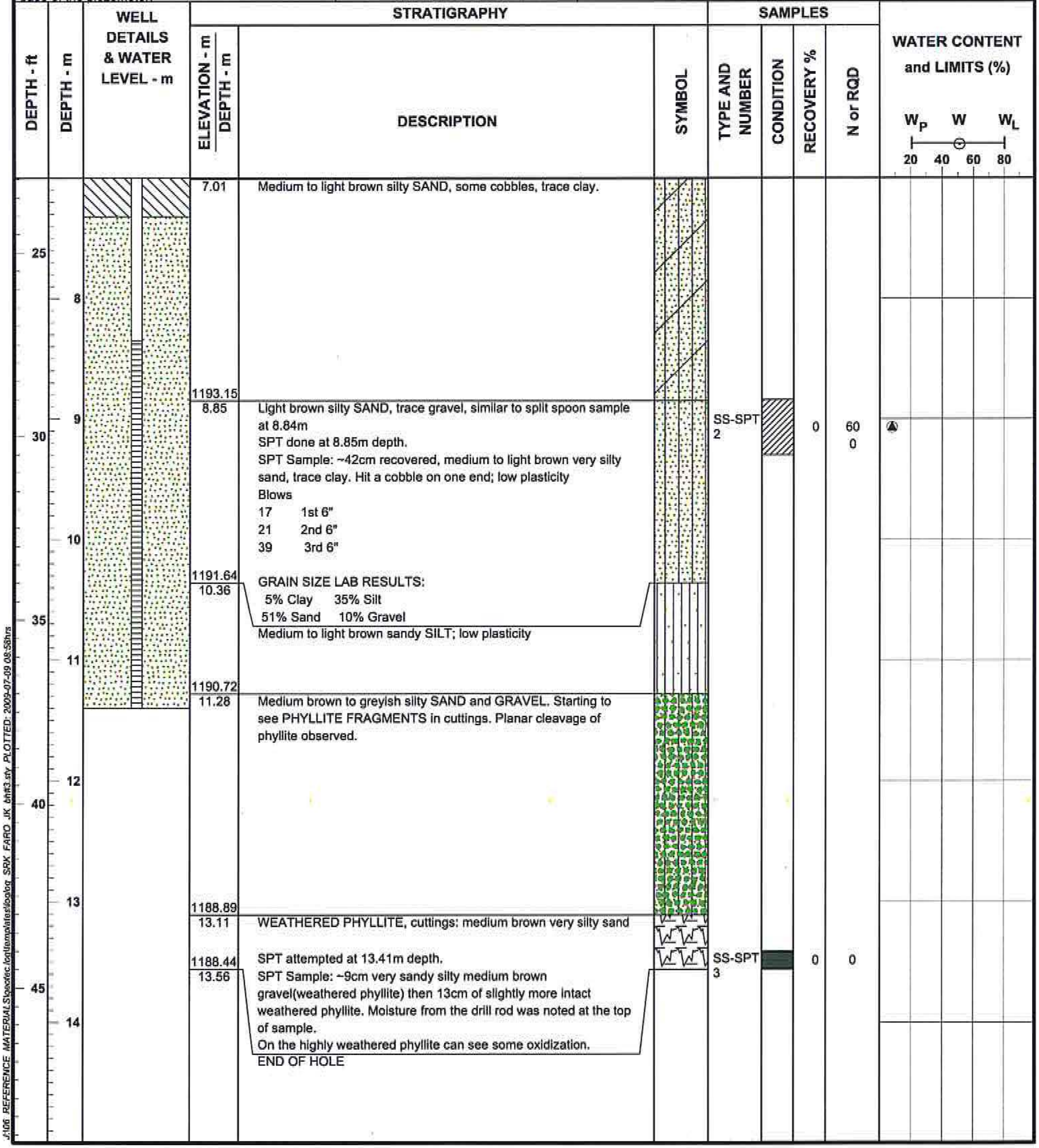
GENERAL COMMENTS:
WELL DETAILS: (June 19th, 2009)
-Depth to water = DRY
-Total depth of well= 12.065m
-Height to top of casing/pvc= 0.687
-A small amount of moisture appeared to be at the base of the piezometer.

WELL PLUG MATERIAL LEGEND
■ Bentonite Grout
⁘ Sand
▨ Bentonite
Cuttings
Cement

SOIL LEGEND
Topsoil
Gravel
Boulder
Sand
Clay
Silt
Cobble

SAMPLE CONDITIONS
☒ Remoulded
▨ Undisturbed
■ Lost
◧ Core

SAMPLE TYPE
DC Diamond Core
GS Grab Sample
SS Split Spoon



ELEVATION - m / DEPTH - m	DESCRIPTION	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD
7.01	Medium to light brown silty SAND, some cobbles, trace clay.				
1193.15 8.85	Light brown silty SAND, trace gravel, similar to split spoon sample at 8.84m SPT done at 8.85m depth. SPT Sample: ~42cm recovered, medium to light brown very silty sand, trace clay. Hit a cobble on one end; low plasticity Blows 17 1st 6" 21 2nd 6" 39 3rd 6" GRAIN SIZE LAB RESULTS: 5% Clay 35% Silt 51% Sand 10% Gravel	SS-SPT 2	▨	0	60 0
1191.64 10.36	Medium to light brown sandy SILT; low plasticity				
1190.72 11.28	Medium brown to greyish silty SAND and GRAVEL. Starting to see PHYLLITE FRAGMENTS in cuttings. Planar cleavage of phyllite observed.				
1188.89 13.11	WEATHERED PHYLLITE, cuttings: medium brown very silty sand				
1188.44 13.56	SPT attempted at 13.41m depth. SPT Sample: ~9cm very sandy silty medium brown gravel(weathered phyllite) then 13cm of slightly more intact weathered phyllite. Moisture from the drill rod was noted at the top of sample. On the highly weathered phyllite can see some oxidization. END OF HOLE	SS-SPT 3	■	0	0

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BOREHOLE LOG

PROJECT: Vangorda Diversion Field Investigation
LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-07 **TO** 2009-06-08
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903692.20 N 594119.60 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-DHVD05
PAGE: 1 **OF** 3
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

GENERAL COMMENTS:
WELL DETAILS: (June 19th, 2009)
-Depth to water = 2.667m
-Total depth of well= 8.400m
-Height to top of casing/pvc= 0.731m
Rods became stuck at 35', left in ground then revisited on June 9th to retrieve (lost only the bit).

WELL PLUG MATERIAL LEGEND

■	Bentonite Grout
⛆	Sand
▧	Bentonite
●	Cuttings
▵	Cement

SOIL LEGEND

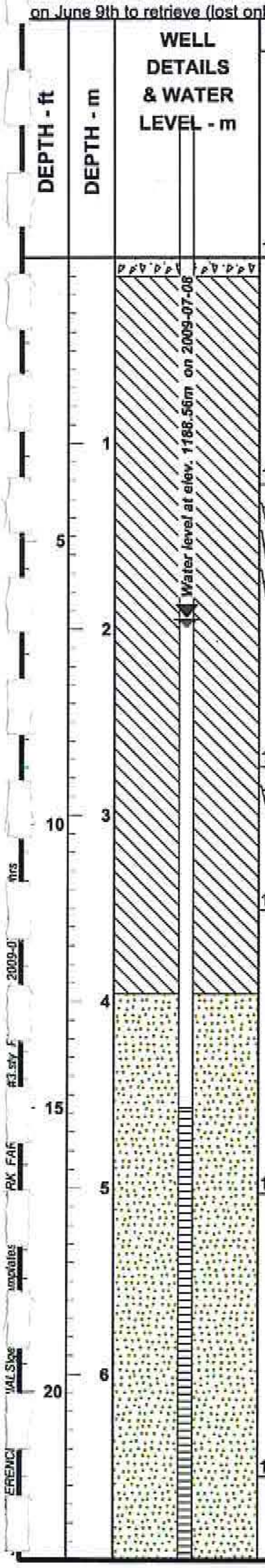
~	Topsoil
●	Gravel
●	Boulder
⛆	Sand
▧	Clay
▥	Silt
●	Cobble

SAMPLE CONDITIONS

☒	Remoulded
▨	Undisturbed
■	Lost
◧	Core

SAMPLE TYPE

DC Diamond Core
GS Grab Sample
SS Split Spoon



STRATIGRAPHY

ELEVATION - m / DEPTH - m	DESCRIPTION
1190.50 0.00	Dark brown black organic rich TOPSOIL
1189.28 1.22 1189.18 1.32 1189.03 1.47 1188.82 1.68	Black organic rich silty sand and gravel some clay, organic rich TOPSOIL SPT done at 1.22m depth. SPT Sample: Topsoil for first 10cm then black organic rich silt, sand, gravel some clay for 10cm then for the next 15cm is a dark brown silt, clay, gravel(phyllite chips) some sand and trace organics exhibiting medium plasticity. Blows 3 1st 6" 3 2nd 6" 15 3rd 6"
1187.76 2.74	Dark brown gravely SILT and CLAY, with some phyllite fragments, some sand and trace organics; medium plasticity
1187.66 2.84	Highly weathered PHYLLITE,exhibiting near horizontal planar cleavage
	Weathered PHYLLITE BEDROCK. Cutting: black grey silty sand with lots of phyllite chips, some gravel
	Weathered PHYLLITE BEDROCK
1186.99 3.51	RUN: Dark grey soft highly weathered PHYLLITE. Appears break quite easily on folliation. Just past 2.84m casing started to wash down due to nature of weathered phyllite. Grey water obsered to be coming out of cuttings spout (on the order of ~1/3 L/s) OF=10 J=9(Rz) F=1: Alpha angles ~70deg IRS= R2 Microdefects: heavy, weak; lLarge scale straight planar joints with medium soft infill Rubble zone (RZ) at 2.89m to 3.12m
1185.47 5.03	RUN: Grey brown highly weathered PHYLLITE OF=41 J=7; Alpha angles~110deg CJ=2; Alpha angles open~70, closed~30deg F=26; Alpha angle ~105-110deg. IRS= R2/R1 Microdefects heavy, weak; Large scale joints: straight; Staining: soft red brown
1183.95 6.55	RUN: Grey weathered PHYLLITE. Green grey weathered chlorite altered phyllite to 5.63m then more compenent OF=34 J=11; Alpha angles, weathered~120, more competent ~100 CJ=2; Alpha angles ~120-125deg F=17; Alpha angle ~95-105deg. IRS= R3/R2 Microdefects heavy to mod, weak; Large scale joints: straight

SAMPLES

TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD
SS-SPT 1	▨	0	18 0
DC-Run 1	◧	45	0
DC-Run 2	◧	94	7
DC-Run 3	◧	99	33

WATER CONTENT and LIMITS (%)

W_P W W_L

20 40 60 80

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BOREHOLE: SRK09-DHVD05
PAGE: 2 OF 3
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

GENERAL COMMENTS:
WELL DETAILS: (June 19th, 2009)
-Depth to water = 2.667m
-Total depth of well= 8.400m
-Height to top of casing/pvc= 0.731m
Rods became stuck at 35', left in ground then revisited on June 9th to retrieve (lost only the bit).

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ⁘ Sand
- ▧ Bentonite
- ☐ Cuttings
- ⁘ Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	STRATIGRAPHY DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
25	8		1182.42	RUN: Grey PHYLLITE, some quartz veins OF=33 J=6; Alpha angles ~75deg CJ=3; 2 open 1 closed Alpha angles ~85deg F=21; Alpha angle ~95-105deg. IRS= R3/R2 Microdefects heavy, weak; Large scale joints: straight, linear. Infill: soft for close joint		DC-Run 4		58	0	
30	9		8.08 1180.90	RUN: Grey chlorite altered PHYLLITE, can see lots of evidence of fluid flow/quartz vein intrusions OF=53 J=6+14(Rz); Alpha angles ~115-120deg CJ=6; Alpha angles ~150deg F=26; Alpha angle ~85-100deg most at 100deg. RZ from 8.08 to 8.18m IRS= R3/R0, mostly in the R1 range Microdefects heavy, weak; Large scale joints: straight undulating to stepped. RZ at 8.32m RZ at 8.50m RZ at 8.67m		DC-Run 5		94	0	
35	10 11		9.60 1179.43	RUN: Grey PHYLLITE. More competent, can see minor sulphides, minor quartz veining. OF=23+3(MRz) J=14; Alpha angles ~75-85, 105 & 55deg CJ=1; Alpha angles ~70deg, twisted/folded F=8; Alpha angle ~70-80deg. IRS= R3 Microdefects heavy, sometimes break; Large scale joints: straight undulating to samll scale straight stepped. Staining: minor, hard white		DC-Run 6		102	51	
40	12		11.07 1177.90	RUN: Grey-green PHYLLITE, highly quartz intruded. Large quartz veins, chlorite alteration and sulphides observed OF=20 J=12; Alpha angles ~120-125 & 110deg CJ=1; Alpha angles ~85deg F=6; Alpha angle ~85 to/most at 105deg. IRS= R4/R3 Microdefects heavy, sometimes break		DC-Run 7		95	48	
45	13		12.60	RUN: Grey-green PHYLLITE, quartz intruded. Chlorite alteration observed. Most joints associated with quartz veins OF=50 J=11; Alpha angles ~70deg CJ=2 F=34; Alpha angle ~105deg then twist at 12.98m to ~75-80deg. IRS= R3/almost R2 in areas Microdefects heavy		DC-Run 8		102	15	

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DIP: 90.00 **AZIMUTH:**
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BOREHOLE: SRK09-DHVD05
PAGE: 3 **OF** 3
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

GENERAL COMMENTS:
WELL DETAILS: (June 19th, 2009)
-Depth to water = 2.667m
-Total depth of well= 8.400m
-Height to top of casing/pvc= 0.731m
Rods became stuck at 35', left in ground then revisited on June 9th to retrieve (lost only the bit).

WELL PLUG MATERIAL LEGEND
- ■ Bentonite Grout
- ▨ Sand
- ▧ Bentonite
- ☐ Cuttings
- ☐ Cement

SOIL LEGEND
- ☐ Topsoil
- 🟩 Gravel
- ☐ Boulder
- ▨ Sand
- ▧ Clay
- ▥ Silt
- ☐ Cobble

SAMPLE CONDITIONS
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
			1176.38 14.12	RUN: Grey-green white PHYLLITE, quartz intruded (thin quartz veins). Chlorite alteration observed. Highly fractured, some sulphides observed OF=54+4(MRz) J=10+8(Rz); Alpha angles ~70 then a twist after 14.48cm then 115deg IRS= R3/almost R2 in areas Microdefects heavy RZ at 14.96m RZ at 15.04m	▤	DC-Run 9	◧	93	7	
	15									
50										
			1174.85 15.65	END OF HOLE						
	16									
55										
	17									
	18									
60										
	19									
65										
	20									

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BOREHOLE LOG

PROJECT: Vangorda Diversion Field Investigation
LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-08 **TO** 2009-06-09
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903652.80 N 593822.10 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-DHVD06
PAGE: 1 **OF** 3
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

GENERAL COMMENTS:
WELL DETAILS: (June 19th, 2009)
-Depth to water = 8.707m
-Total depth of well= 13.043m
-Height to top of casing/pvc= 0.715m
Mud on electrode of water tape (appeared to likely be more of a silty clay).

WELL PLUG MATERIAL LEGEND
- Bentonite Grout
- Sand
- Bentonite
- Cuttings
- Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- Remoulded
- Undisturbed
- Lost
- Core

SAMPLE TYPE
- DC Diamond Core
- GS Grab Sample
- SS Split Spoon

ELEVATION - m DEPTH - m	STRATIGRAPHY DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80
1177.70 0.00	Dark/rich TOPSOIL						
1176.79 0.91	Medium brown sandy GRAVEL and COBBLE rich TILL						
1176.48 1.22	Light to medium brown sandy SILT, TILL, trace to no clay						
1175.57 2.13	More fines present. Medium to almost rusty brown sandy SILT with some gravel and trace clay.						
1175.11 2.59	Igneous BOULDER, appeared granite like.						
1174.96 2.74	Brown sandy SILT with some gravel and trace clay.						
1174.65 3.05	Igneous BOULDER, granite to granodiorite like.						
1174.35 3.35	Medium red brown sandy SILT/ TILL. Odex would slow in section when it hit large cobbles/ boulders.						
1173.74 3.96	Brown sandy SILT/ TILL, some gravel trace clay						
1173.43 4.27	Igneous BOULDER, granite like						
1173.20 4.50	Back into silty sandy TILL. Medium brown, silty SAND some gravel						
1173.13 4.57	BOULDER, granite like						
1172.37 5.33	Brown, sandy SILT						
1172.21 5.49	Brown, sandy SILT, some gravel and trace clay. Starting to see phyllite gravel in TILL as well as granite like fragments.						
1171.30 6.40	Medium brown silty SAND/TILL some gravel. Seeing more flaky shiny, mica like, fragmetns in odex cuttings.						
1169.78							

DEPTH - ft: 5, 10, 15, 20, 25
DEPTH - m: 1, 2, 3, 4, 5, 6, 7

WELL DETAILS & WATER LEVEL - m
Water level at elev. 1169.71m on 2009-06-19

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BOREHOLE LOG

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LOCATION: Vangorda Diversion Preliminary Alignment
FILE No: FARO (1CY001.031)
BORING DATE: 2009-06-08 **TO** 2009-06-09
DIP: 90.00 **AZIMUTH:**
COORDINATES: 6903652.80 N 593822.10 E **DATUM:** UTM NAD 27, 08 V

BOREHOLE: SRK09-DHVD06
PAGE: 2 **OF** 3
DRILL TYPE: Odex 4"/Diamond
DRILL: Froste, Track Mounted
CASING: Steel

GENERAL COMMENTS:
WELL DETAILS: (June 19th, 2009)
-Depth to water = 8.707m
-Total depth of well= 13.043m
-Height to top of casing/pvc= 0.715m
Mud on electrode of water tape (appeared to likely be more of a silty clay).

WELL PLUG MATERIAL LEGEND
- Bentonite Grout
- Sand
- Bentonite
- Cuttings
- Cement

SOIL LEGEND
- Topsoil
- Gravel
- Boulder
- Sand
- Clay
- Silt
- Cobble

SAMPLE CONDITIONS
- Remoulded
- Undisturbed
- Lost
- Core

SAMPLE TYPE

DC	Diamond Core
GS	Grab Sample
SS	Split Spoon

ELEVATION - m / DEPTH - m	STRATIGRAPHY DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%)
	Medium brown silty SAND/TILL some gravel.						
1169.17 8.53	Into a very rocky TILL. GRAVEL, SAND some silt, and some cobbles						
1168.86 8.84	Another boulder, granodiorite like.						
1168.56 9.14	Rocky TILL. Starting to see more fines in cuttings again. Medium brown sandy SILT some gravel.						
1167.64 10.06	Brown GRAVEL and SAND, some silt, cobble and boulder rich, TILL.						
1167.03 10.67	Medium to rich brown sandy SILT TILL with cobbles. Encountered a bit of water in till while drilling. Observed through cuttings spout.						
1166.42 11.28	Brown sandy SILT TILL with cobbles. Not seeing as much water in this region.						
1165.51 12.19	Wet SAND and GRAVEL some silt and some to trace clay. Water observed in cuttings/ out cuttings spout again at 12.19m.						
1164.90 12.80	Wet SAND and GRAVEL some silt and some to trace clay. Higher sand and gravel content.						
1164.29 13.41	Wet SAND and GRAVEL some silt and some to trace clay. Still quite moist.						
1163.68 14.02	Medium brown, sandy TILL with some silt, gravel and some to trace clay						

DEPTH - ft: 30, 35, 40, 45
DEPTH - m: 9, 10, 11, 12, 13, 14
WELL DETAILS & WATER LEVEL - m
W_P W W_L: 20 40 60 80

J:\06 REFERENCE MATERIAL\Sgeotec.log\templates\loglog SRK FARO JK bh#3.sty PLOTTED: 2009-07-09 09:02hrs

DHVD01
05/06/09 -06/06/09
BOX 1
259 to 694.5cm

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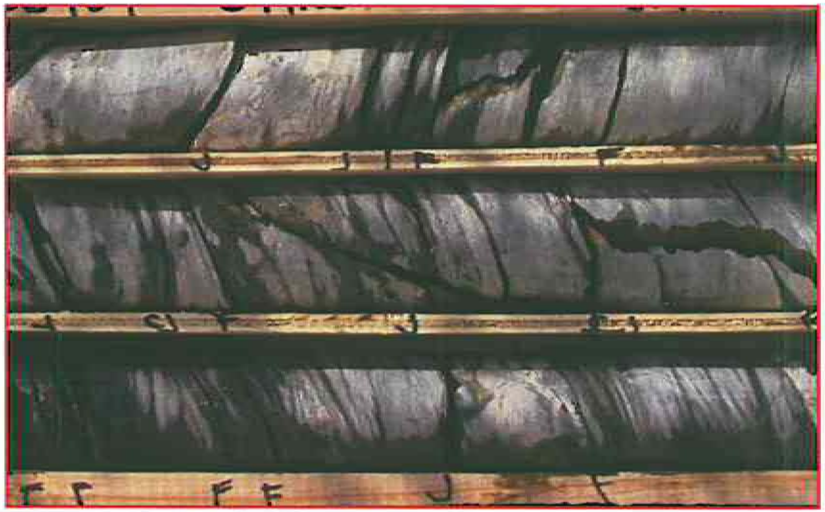
Engineers and Scientists
SRK JOB NO.: 1CY001.031
FILE NAME: APPENDIX D

Faro Mine Closure
Fermeture de la mine Faro
Faro Mine Complex

Vangorda Creek Diversion
Core Box/Split Spoon Photographs
DATE: July 2009
APPROVED: JBK
FIGURE: 1



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Faro Mine Closure
Fermeture de la mine Faro

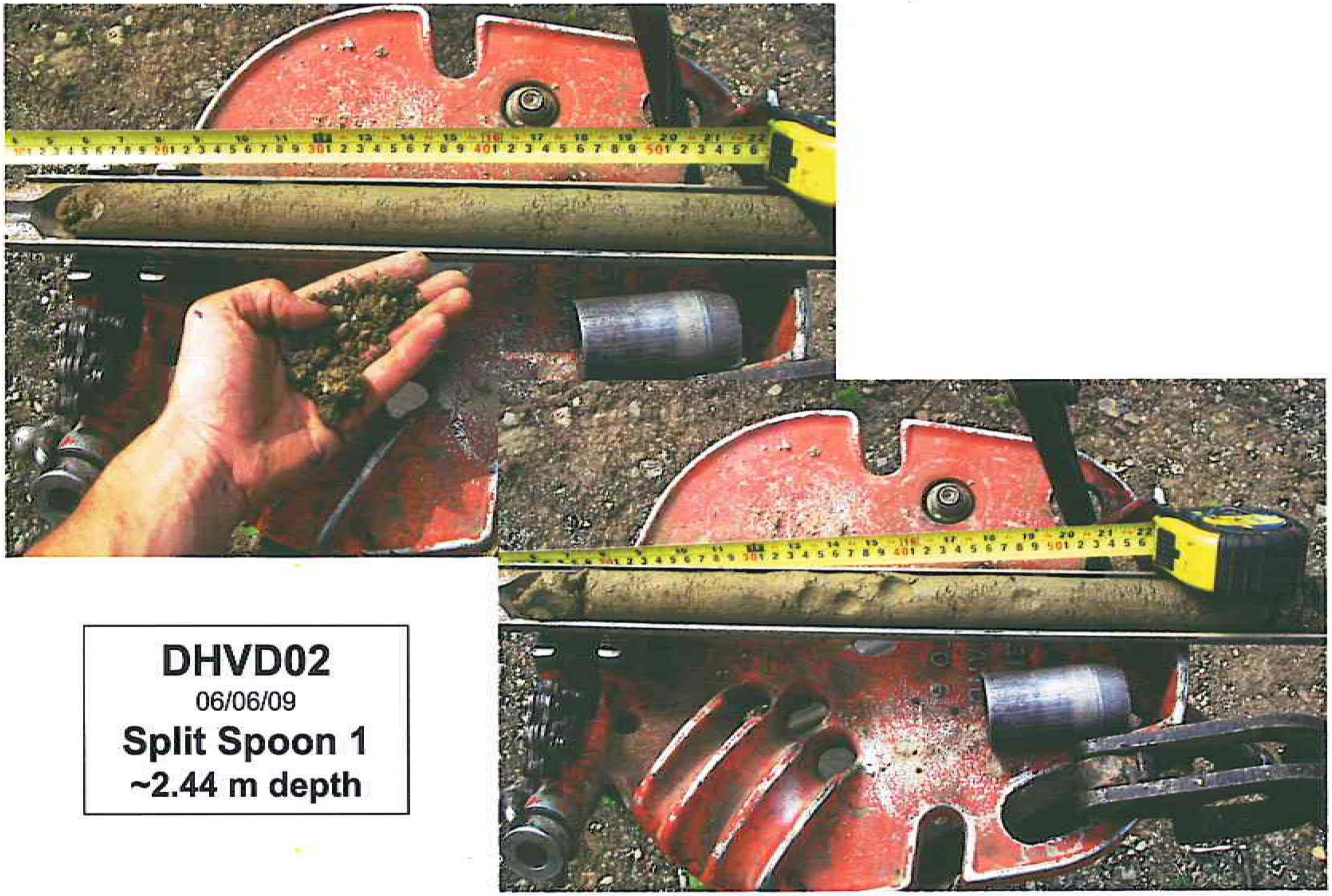
Vangorda Creek Diversion

Core Box/Split Spoon Photographs

SRK JOB NO.: 1CY001.031
FILE NAME: APPENDIX D

Faro Mine Complex

DATE: July 2009
APPROVED: JBK
FIGURE: 2



DHVD02
06/06/09
Split Spoon 1
~2.44 m depth

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Faro Mine Closure
Fermeture de la mine Faro

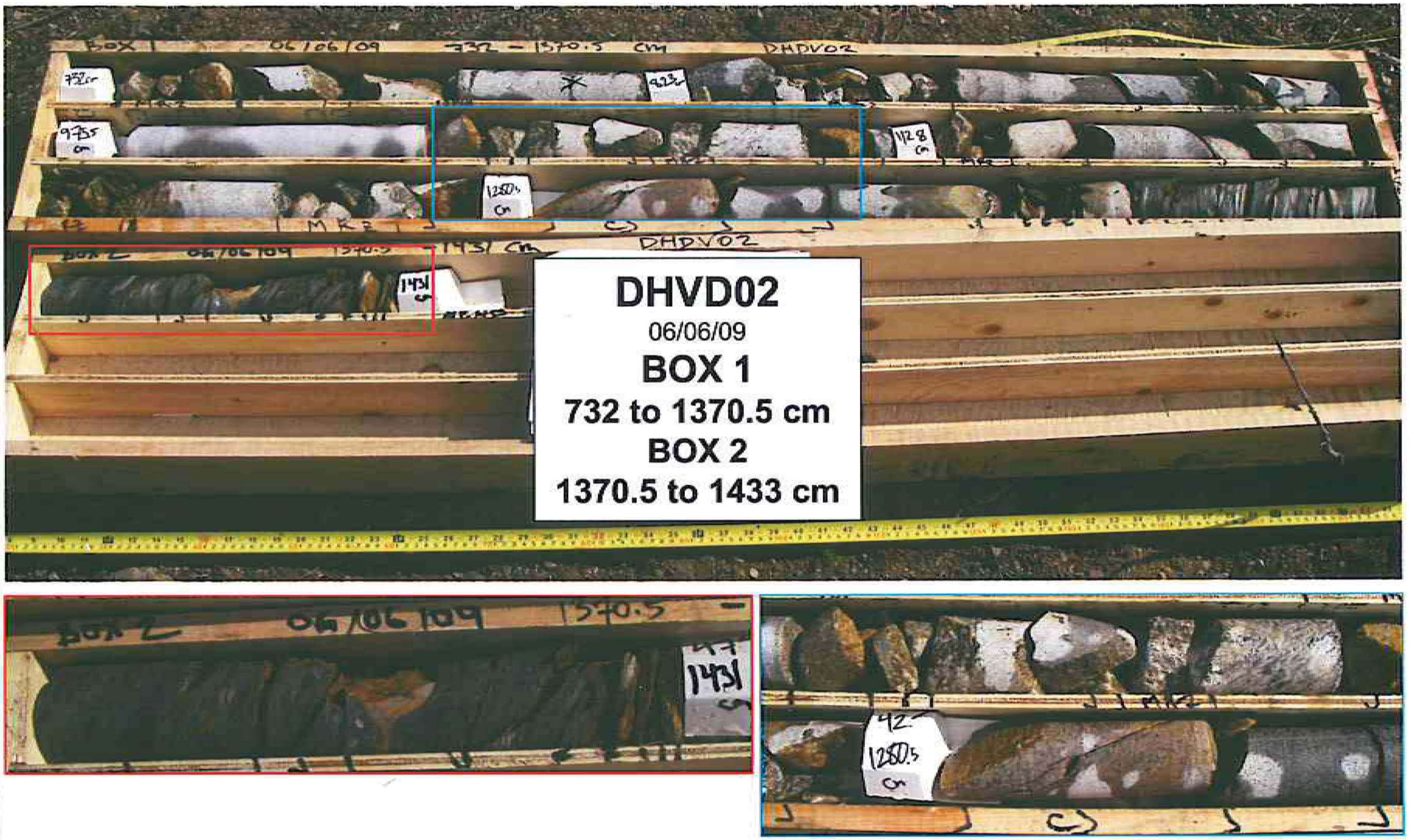
Vangorda Creek Diversion

Core Box/Split Spoon Photographs

SRK JOB NO.: 1CY001.031
FILE NAME: APPENDIX D

Faro Mine Complex

DATE: July 2009
APPROVED: JBK
FIGURE: 3



DHVD02
06/06/09
BOX 1
732 to 1370.5 cm
BOX 2
1370.5 to 1433 cm

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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

SRK JOB NO.: 1CY001.031
FILE NAME: APPENDIX D

Faro Mine Complex

DATE:	APPROVED:	FIGURE:
July 2009	JBK	4

DHVD03
07/06/09
Split Spoon 1
~1.22 m depth

DHVD03
07/06/09
Split Spoon 2
~2.74 m depth

SRK Consulting Engineers and Scientists	Faro Mine Closure Fermeture de la mine Faro	Vangorda Creek Diversion
		Core Box/Split Spoon Photographs
SRK JOB NO.: 1CY001.031 FILE NAME: APPENDIX D	Faro Mine Complex	DATE: July 2009 APPROVED: JBK FIGURE: 5

DHVD03
07/06/09
Split Spoon 3
~4.27 m depth

DHVD03
07/06/09
Split Spoon 4
~7.32 m depth

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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

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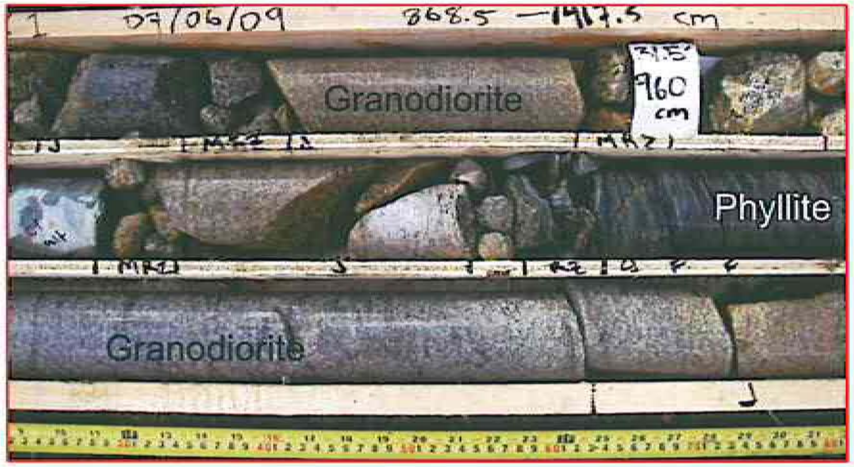
FILE NAME: APPENDIX D

Faro Mine Complex

DATE: July 2009
APPROVED: JBK
FIGURE: 6



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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

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Faro Mine Complex

DATE: July 2009
APPROVED: JBK
FIGURE: 7

DHVD04A

07/06/09

Split Spoon 1

Ground Surface

DHVD04A

07/06/09

Split Spoon 2

~2.74 m depth

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SRK JOB NO.: 1CY001.031 FILE NAME: APPENDIX D	Faro Mine Complex	DATE: July 2009 APPROVED: JBK FIGURE: 8

DHVD04A
07/06/09
Cutting Samples

Sand cutting observed at ~18.29 m depth.

Igneous cutting observed at ~20 m depth.

Phyllite cutting observed at ~21m depth.

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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

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FILE NAME: APPENDIX D

Faro Mine Complex

DATE: July 2009
APPROVED: JBK
FIGURE: 9

DHVD04B
09/06/09
Split Spoon 1
~4.27 m depth

DHVD04B
09/06/09
Split Spoon 2
~8. m depth

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Fermeture de la mine Faro

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

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Faro Mine Complex

DATE: July 2009
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FIGURE: 10

Light brown silty sand, cutting observed at ~10.4m depth

Weathered phyllite (a silty dark grey with rock fragments) encountered at ~13.41m depth

DHVD04B

09/06/09

Split Spoon attempt ~13.41 m depth

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Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

DATE: July 2009
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FIGURE: 11

DHVD05
07/06/09
Split Spoon 1
~1.22 m depth

DHVD05
08/06/09
BOX 1
289.5 to 808 cm

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Faro Mine Closure
Fermeture de la mine Faro
Faro Mine Complex

Vangorda Creek Diversion
Core Box/Split Spoon Photographs
DATE: July 2009
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FIGURE: 12

DHVD05

08/06/09

BOX 2

808 to 1319 cm

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Fermeture de la mine Faro

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

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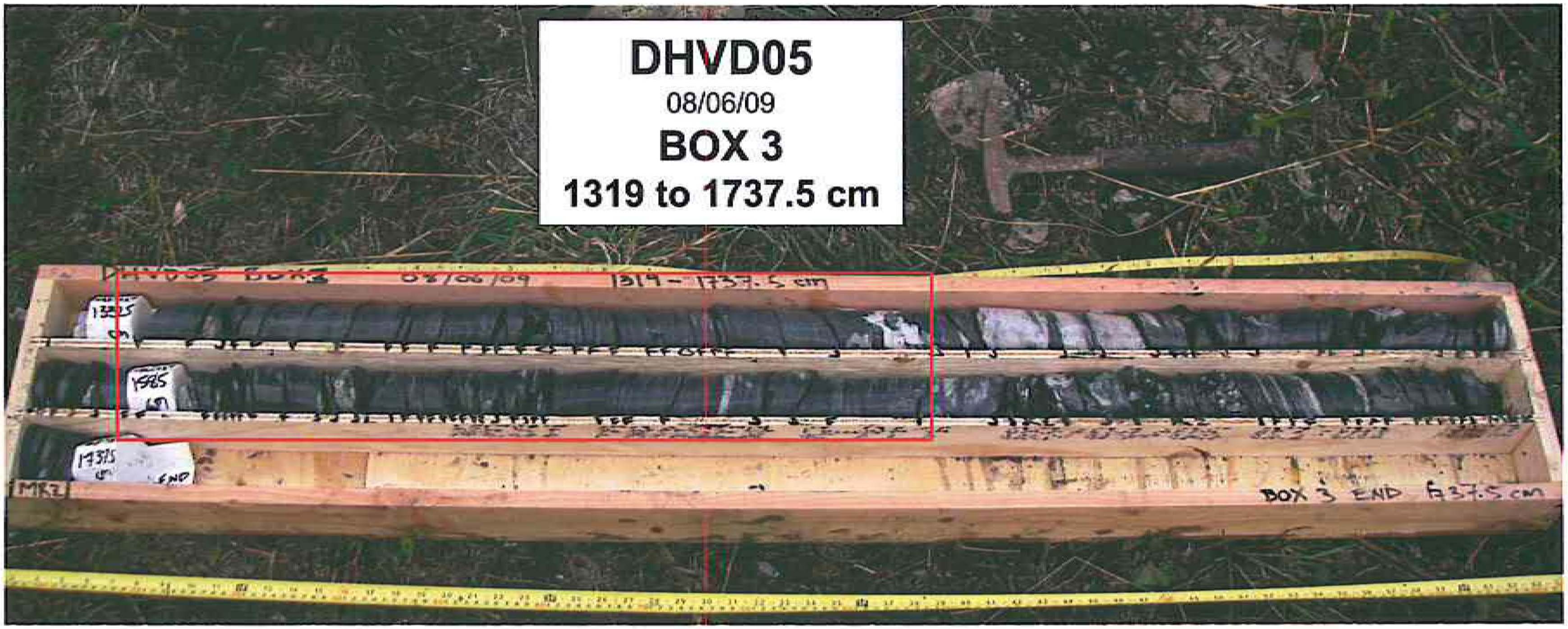
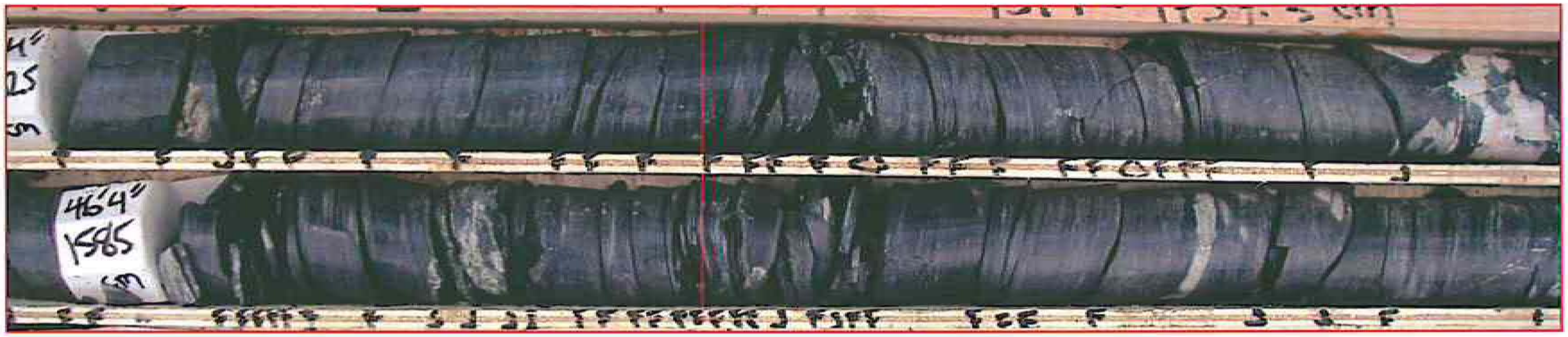
Faro Mine Complex

DATE: July 2009
APPROVED: JBK
FIGURE: 13



DHVD05
08/06/09
BOX 3
1319 to 1737.5 cm



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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

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Faro Mine Complex

DATE: July 2009
APPROVED: JBK
FIGURE: 14

DHVD06

08/06/09

Cutting Samples

Sandy silty till cutting observed at ~3.96 m depth.

Igneous (granodiorite like) boulder encountered at ~4.27m depth.

Sandy silty rocky (cobble and boulder rich) till cutting observed at ~6.40 m depth.

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Faro Mine Closure
Fermeture de la mine Faro
Faro Mine Complex

Vangorda Creek Diversion
Core Box/Split Spoon Photographs
DATE: July 2009
APPROVED: JBK
FIGURE: 15

DHVD06

08/06/09

Cutting Samples

Wet till, cuttings comprised of sand and gravel with some silt at ~12.19 m depth.

Sandy silty till with gravel, cutting observed at ~16.76 m depth

Weathered phyllite (a silty dark grey with rock fragments) encountered at ~17.37m depth.

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Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex

Vangorda Creek Diversion

Core Box/Split Spoon Photographs

DATE: July 2009
APPROVED: JBK
FIGURE: 16

Appendix E

Laboratory Test Results

PARTICLE SIZE ANALYSIS TEST REPORT

ASTM D422 & C136

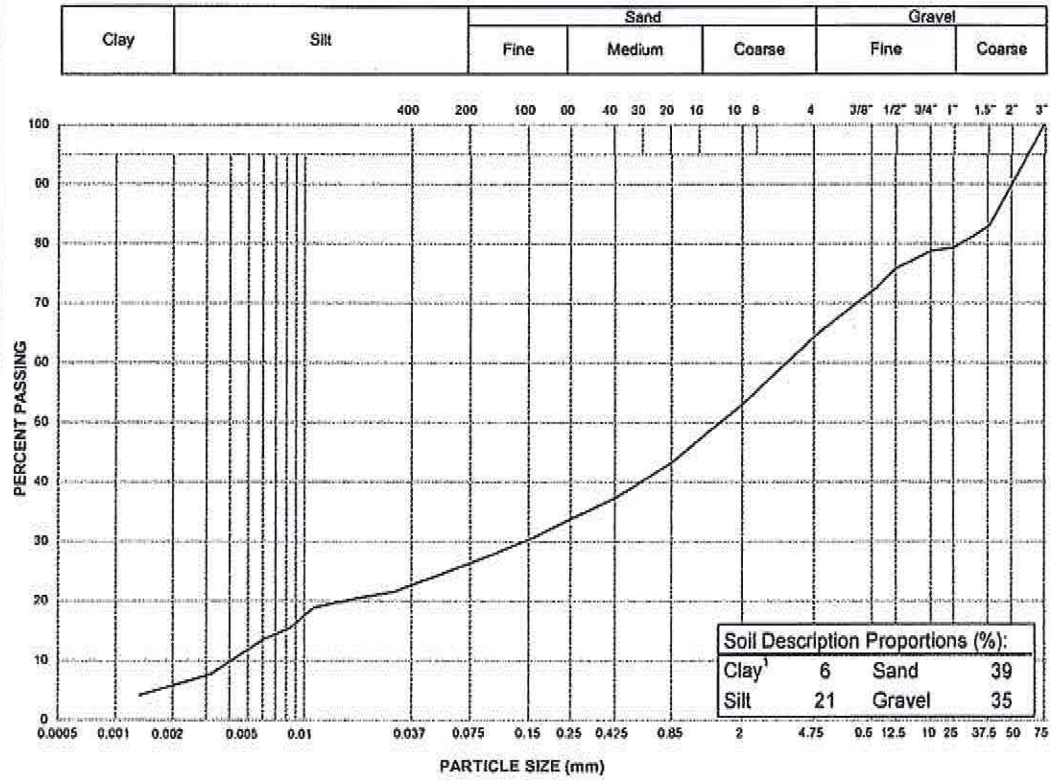
Project: SRK Project 1CY001.031
Project No.: W14101280
Site: Faro, YT

Client: SRK Consulting Inc.
Client Rep.:

Material Type:
Sample No.: TP02
Sample Loc.:
Sample Depth: 4.0 m
Sampling Method:
Date sampled: 23-Jun-2009 By: BS

Date Tested: By: BS
Soil Description[2]: SAND TILL - gravelly, some silt to silty, trace of clay
USC Classification: SM
Cu: 871.0
Cc: 1.3
Moisture Content: 9.5

Particle Size (mm)	Percent Passing
75	100
50	
38	83
25	79
19	79
12.5	76
10	73
5	65
2	53
0.85	43
0.425	37
0.25	34
0.15	30
0.075	26
0.0294	21.5
0.0189	20.5
0.0112	18.9
0.0084	15.5
0.0061	13.7
0.0031	7.6
0.0013	4.2



Soil Description Proportions (%):

Clay[1]	6	Sand	39
Silt	21	Gravel	35

Notes:

[1] The upper clay size of 2 um, per the Canadian Foundation Engineering Manual

[2] The description is visually based & subject to EBA description protocols

Specification:

Remarks:

Reviewed By:

EBA Engineering Consultants Ltd.

PARTICLE SIZE ANALYSIS TEST REPORT

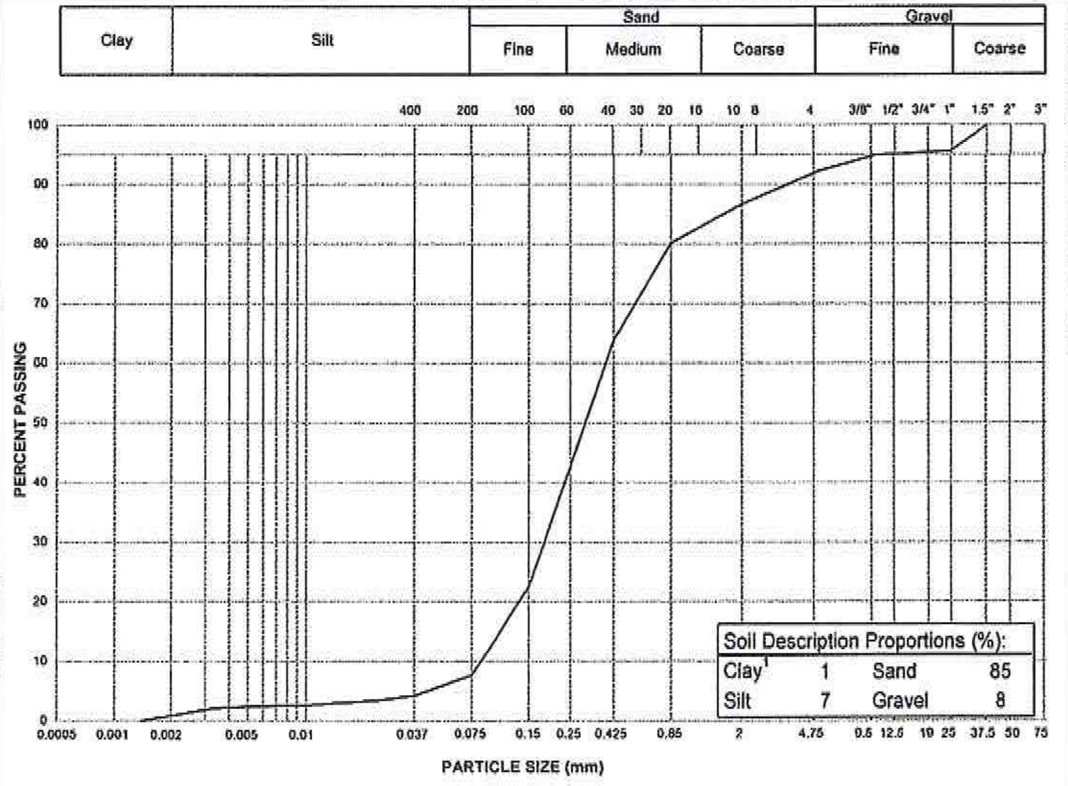
ASTM D422 & C136

Project:	SRK Project 1CY001.031	Client:	SRK Consulting Inc.
Project No.:	W14101280	Client Rep.:	
Site:	Faro, YT		

Material Type:
Sample No.: DHVD 04
Sample Loc.:
Sample Depth: 7.32
Sampling Method:
Date sampled: 23-Jun-2009 By: BS

Date Tested: By: BS
Soil Description[2]: SAND - trace of gravel, trace of silt trace of clay
USC Classification: SM Cu: 4.5
Cc: 1.0
Moisture Content: 6.9

Particle Size (mm)	Percent Passing
75	
50	
38	100
25	96
19	
12.5	
10	95
5	92
2	87
0.85	80
0.425	64
0.25	43
0.15	23
0.075	8
0.0374	4.3
0.0237	3.4
0.0138	3.0
0.0099	2.6
0.0070	2.6
0.0033	2.1
0.0014	0.0



Soil Description Proportions (%):

Clay[1]	1	Sand	85
Silt	7	Gravel	8

Notes: [1] The upper clay size of 2 um, per the Canadian Foundation Engineering Manual
[2] The description is visually based & subject to EBA description protocols

Specification:

Remarks:

Reviewed By:

EBA Engineering
Consultants Ltd.

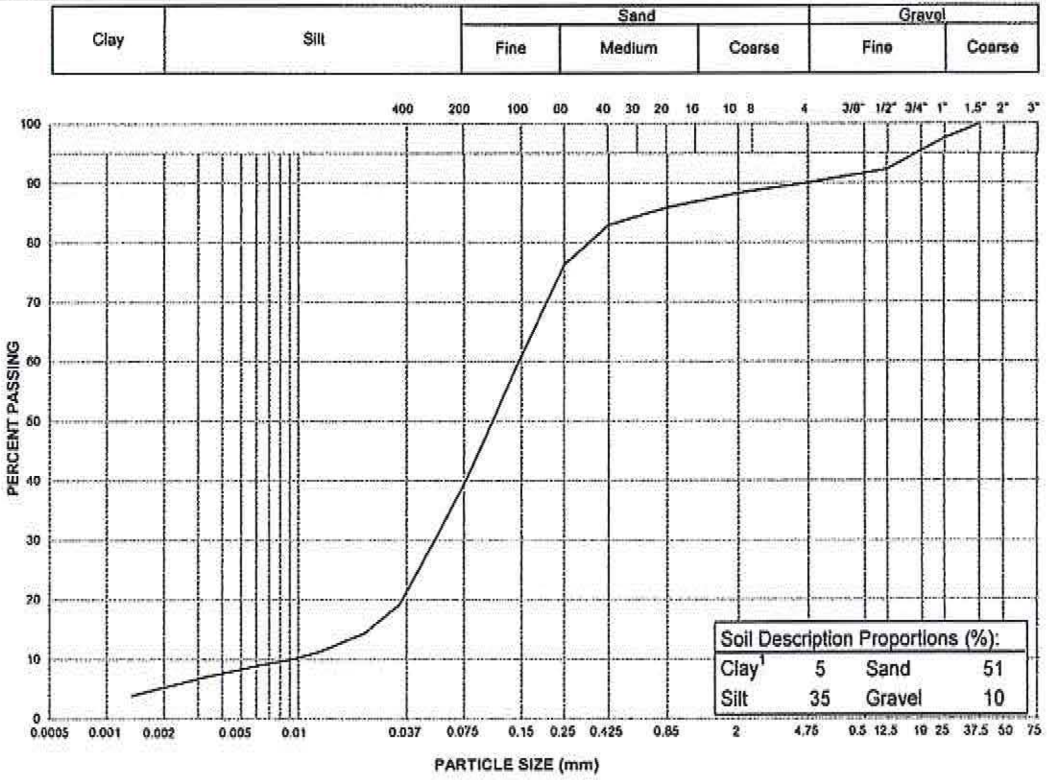
PARTICLE SIZE ANALYSIS TEST REPORT

ASTM D422 & C136

Project:	SRK Project 1CY001.031	Client:	SRK Consulting Inc.
Project No.:	W14101280	Client Rep.:	
Site:	Faro, YT		

Material Type:			Date Tested:		By:	BS
Sample No.:	DHVD 04B		Soil Description[2]:	SAND and SILT - trace to some gravel, trace of clay		
Sample Loc.:						
Sample Depth:	8.85		USC Classification:	SM	Cu:	16.0
Sampling Method:					Cc:	2.3
Date sampled:	23-Jun-2009	By: BS	Moisture Content:	8.0		

Particle Size (mm)	Percent Passing
75	
50	
38	100
25	97
19	95
12.5	92
10	92
5	90
2	88
0.85	86
0.425	83
0.25	76
0.15	61
0.075	39
0.0339	19.2
0.0222	14.4
0.0131	11.4
0.0093	10.1
0.0066	9.2
0.0033	7.0
0.0013	3.9



Soil Description Proportions (%):

Clay[1]	5	Sand	51
Silt	35	Gravel	10

Notes: [1] The upper clay size of 2 um, per the Canadian Foundation Engineering Manual

[2] The description is visually based & subject to EBA description protocols

Specification:

Remarks:

Reviewed By:

Appendix F

Site Photographs

Note: All photographs displayed in this Appendix were taken between May 28th, 2009 and June 12th, 2009. Photographs are presented progressing from the NE to the SW extents of the Vangorda Diversion alignment.

Photo 1 & 2: NE end of Alignment - at Vangorda Creek edge. Close-up view of phyllitic bedrock observed on Diversion opposing (more N) side of Vangorda Creek.

Photo 3 & 4: NE end of diversion alignment - looking ~SW down alignment back towards DHVDO1 (the red outline shows close up of bedrock observed near the W limits of photo 13).

Photo 5: Near DHVD01 – NNE view looking towards Vangorda Creek (blue).

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VANCOUVER

Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion Field Investigation

Site Photographs

Job No: 1CY001.031
Filename: APPENDIX F – Site Photographs

Faro Mine Complex

Date: July 2009
Approved: JBK
Photo Page: 1

Photo 6 & 7: ESE of DHVC06/ down slope from Test Pit #10 – Phyllitic bedrock noted in existing drainage ditch; note that the slope shown is at ~25° to the horizontal.

Photo 8: In Between TP#1 and TP#2- Evidence of Minor mass movement. Note head scarp, ~25m SW of DHVD01.

Photo 9: Near Looking ~SSW down cleared drill access route from TP#4 towards location of DHVD04A drilling activities. Photo Taken before drilling activities commenced.

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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion Field Investigation

Site Photographs

Job No: 1CY001.031
Filename: APPENDIX F – Site Photographs

Faro Mine Complex

Date: July 2009
Approved: JBK
Photo Page: 2

Photo 10: SW view towards location of TP#11 excavation (taken prior to excavation). Note main haul access road in the distance.

Photo 11: Looking NE from main haul road towards the Vangorda Pit. Near expected SW end of diversion alignment.

Photo 12: SE view, over bank adjacent to main haul access road. Note the flow through the existing culvert re-entering the original flow path of Vangorda Creek .

Photo 12: WNW view, of bank adjacent to main haul access road. Note the large volume of waste rock/road fill material which has been placed.

Photo 13: NNE view, of bank adjacent to main haul access road.

Photo 14: SSW view, of culvert flow entering Vangorda Creek, as well as the toe of bank adjacent to main haul access road.

Photo 15: NW view, of toe of bank adjacent to main haul access road.

Photo 16: Close up of silty glacial till overburden observed under a thin layer of organic rich topsoil. W of Vangorda Creek and S of the toe of the main haul road.

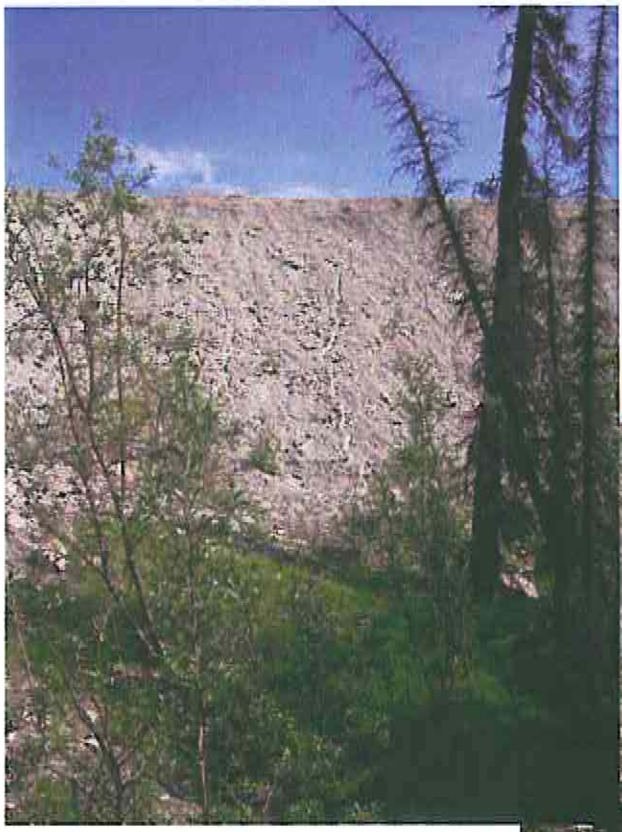

Photo 17: N view, of toe of bank adjacent to main haul access road and transition again into vegetated soil overlying expected phyllitic bedrock.

Photo 18: NNE view, of culvert flow entering Vangorda Creek, as well as the toe of bank adjacent to main haul access road.

Appendix II

ML/ARD Assessment of the Excavated Rock

SRK Consulting
Engineers and Scientists

SRK Consulting (Canada) Inc.
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Copper Cliff, Ontario P0M 1N0
Canada

sudbury@srk.com
www.srk.com

Tel: 705.682.3270
Fax: 705.682.9301

Memo

To:	Peter Healey	**Date:**	December 9, 2009
cc:		**From:**	Madeleine Corriveau Kelly Sexsmith
Subject:	ARD Potential for Rock from the Vangorda Creek Diversion Channel	**Project #:**	1CY001.031

1 Introduction

In order to determine the acid rock draining (ARD) potential of rock encountered during the construction of the Vangorda Creek Diversion Channel, samples were tested from five holes drilled during SRK's Vangorda Diversion Field Investigation in June 2009.

2 Methods

A total of 17 samples were collected representing three types of material: granodiorite, phyllite and till. Sample descriptions are provided in Table 1. The rock samples were collected from geotechnical drill core, and were typically 0.1 m in length. Samples of till were collected from bagged material from the upper portions of the drill holes.

All samples were submitted to CEMI in Vancouver, BC for:

- Acid base accounting (ABA) including modified Sobek neutralization potential (NP), total inorganic carbon (TIC) and sulphur species; and
- Elemental analysis by ICP.

A subset of the weathered rock samples were also submitted for shake flask extraction tests. Leach extraction tests were performed at a 3 to 1 liquid to solids ratio, with a contact time of 24 hours.

3 ARD Potential

A summary of ABA results are provided in Table 2.

Total sulphur content of granodiorite and till samples was near or below the detection limit of 0.02%. These samples also had low NP ranging from 2.7 to 5.7 kg $CaCO_3$/tonne. All NP/AP ratios were greater than 2 indicating negligible potential for ARD (Figure 1).

Phyllite samples on the other hand, had a wide range of total sulphur content ranging from <0.02 to 1% (median 0.17%). Nearly all sulphur was present as sulphide-sulphur with sulphate-sulphur content for all samples near or below the detection limit of 0.01%. Neutralization potential was low (ranging 1.9 to 3.7 kg $CaCO_3$/tonne), with the exception of one sample that had a moderate NP of 26 kg $CaCO_3$/tonne. Five of the ten phyllite samples had NP/AP ratios of less than 1 (Figure 1) indicating they are potentially acid generating (PAG). However, given the relatively low acid potential of most samples, the net amount of acidity that is likely to be produced is relatively low.

Results for the phyllite samples were examined in greater detail to see if there were any variations in the lithology that would be indicative of increased potential for ARD (Table 3). Samples with higher sulphur content tended to be associated with quartz veining. However, the sample with the highest sulphur content did not have quartz veins recorded in the sample description. There were no strong spatial patterns in the data, with PAG intervals noted in two of the three drillholes with phyllite, and non-PAG intervals occurring in close proximity to the PAG intervals. Therefore, segragation of PAG is not expected to be feasible at this site.

4 Metal Leaching Potential

Shake flask extraction tests were completed on five of the weathered phyllite samples to determine the potential for metal leaching. A summary of leachate chemistry is provided in Table 4.

Leachate pH for three of the samples was circum-neutral ranging from 7.2 to 7.4. Two of the samples (DHVD05 3.4-3.5m and DHVD05 5.4-5.5m) had mildly acidic leachate pH (6.3 and 5.5, respectively). Leachate conductivity was loosely negatively correlated to pH with the highest conductivity (193 uS/cm) reported for the sample with the lowest pH. The highest sulphate concentration (87 mg/L) was also reported for this sample.

Shake flask extraction results were compared to ten times the CCME guidelines for the protection of aquatic life (CCME 1999) to screen for parameters that were elevated in the test leachate. Dissolved metal concentrations were generally low with only one sample (DHVD05 5.4-5.5m) slightly exceeding the screening criteria for aluminum, cadmium, and iron.

5 Conclusions and Recommendations

The results of the static testing indicate that the granodiorite, the till, and half of the phyllite samples would be classified as potentially acid consuming. The remaining phyllites were classified as potentially acid generating. Most of the PAG samples have AP of less than 10 kg $CaCO_3$/tonne, indicating that the total amount of acidity that could be produced from these materials is relatively low.

Phyllite is the dominant type of rock in the walls of diversion ditch. The test results suggest that localized ARD and metal leaching is possible in some of this material. However, given the relatively small surface area of the ditch walls, the low sulphur content and modest amounts of soluble metal found in this material, it is unlikely that the exposed ditch walls are would have an appreciable effect on water quality. Inspection of the pit walls and seepage monitoring is recommended to confirm these findings.
Phyllite will also be the dominant type of waste rock that will be produced during construction. Given the much larger surface area expected for waste rock, some management plans are appropriate. Potential options for disposal of this material include:

1) Disposal in a random location in the Grum dump,
2) Disposal above the sulphide cell of the Grum dump, where it would ultimately be covered; or,
3) the upper part of the Vangorda pit where it could be used as a cover for highly sulphidic waste rock that is currently stored in the pit.

All three options would provide adequate control of seepage water quality and would not be expected to have an appreciable effect on the net loading from these areas.

6 References

Canadian Council of Ministries of the Environment 2007. Canadian Water Quality Guidelines for the Protection of Aquatic Life Update 7.0 September 2007.

Table 1: Sample Descriptions

Sample ID	Rock Type	Description
DHVD02 13.0-13.1 m	Granodiorite	Granodiorite, hematite alteration/slightly weathered
DHVD03 10.0-10.1 m	Granodiorite	Grey granodiorite, ferric/hemitite altered
DHVD01 2.8-2.9 m	Phyllite	Grey green phyllite with minor veining
DHVD01 5.0-5.1 m	Phyllite	Grey green phyllite with minor/sparse veining
DHVD01 9.7-9.8 m	Phyllite	Heavily quartz veined/intruded phyllite
DHVD02 13.5-13.6 m	Phyllite	Weathered phyllite bedrock
DHVD03 11.7-11.8 m	Phyllite	Dark grey phyllite bedrock. Lots of foliation breaks.
DHVD05 11.8-11.9 m	Phyllite	Dark grey soft highly weathered phyllite. Breaks easily on foliation
DHVD05 15.4-15.5 m	Phyllite	Green grey weathered chlorite altered phyllite.
DHVD05 3.4-3.5 m	Phyllite	Grey chlorite altered phyllite, lots of evidence of fluid flow/quartz vein intrusions
DHVD05 5.4-5.5 m	Phyllite	Grey-green phyllite, abundant, large quartz veins, chlorite alteration and sulphides observed.
DHVD05 8.2-8.3 m	Phyllite	Grey-green white phyllite, thin quartz veins, chlorite alteration, highly fractured, some sulphides observed.
DHVD03 1.22 m (-3/8")	Till	Grey brown sand and gravels, some silt; gravel is comprised mainly of phyllite
DHVD03 2.74 m (-3/8")	Till	Light brown silty sand with some gravel
DHVD03 7.31 m (-3/8")	Till	Medium to light brown sandy silt with some gravel and some to trace clay
DHVD04B 2.74 m (-3/8")	Till	Light brown sand, some silt
DHVD05 1.22 m (-3/8")	Till	Topsoil, then black organic rich silt, sand, gravel, some clay, then dark brown silt, clay, gravel (phyllite chips) some sand and trace organics

Table 2: Summary of ABA Results

Sample ID	Rock Type	Paste pH	Total S	Sulphate	Sulphur Diff.	AP	Modified NP	Equiv. CaCO3	NP/AP
		Std. Units	% S	% S	% S	kg CaCO3/t	kg CaCO3/t	kg CaCO3/t	Ratio
LOD		0.01	0.02	0.01	#N/A	#N/A	0.2	#N/A	#N/A
Method Code		Sobek	Leco	HCl Leach	Calc.	Calc.	Modified NP	Calc.	Calc.
DHVD02 13.0-13.1 m	Granodiorite	9.15	<0.02	<0.01	<0.02	<0.6	3.2	0.5	5.3
DHVD03 10.0-10.1 m	Granodiorite	7.85	<0.02	<0.01	<0.02	<0.6	3.3	0.7	5.5
DHVD01 2.8-2.9 m	Phyllite	8.02	0.15	<0.01	0.15	4.7	2.7	1.4	0.6
DHVD01 5.0-5.1 m	Phyllite	6.91	0.05	0.01	0.04	1.3	2.7	<0.5	2.2
DHVD01 9.7-9.8 m	Phyllite	8.37	0.31	<0.01	0.31	9.7	3.6	4.3	0.4
DHVD02 13.5-13.6 m	Phyllite	7.78	<0.02	<0.01	<0.02	<0.6	2.8	<0.5	4.7
DHVD03 11.7-11.8 m	Phyllite	7.93	<0.02	<0.01	<0.02	<0.6	1.9	<0.5	3.2
DHVD05 11.8-11.9 m	Phyllite	8.25	0.22	<0.01	0.22	6.9	3.3	<0.5	0.5
DHVD05 15.4-15.5 m	Phyllite	8.30	0.19	<0.01	0.19	5.9	26.2	30.5	4.4
DHVD05 3.4-3.5 m	Phyllite	6.49	0.03	0.03	<0.02	<0.6	1.9	<0.5	3.2
DHVD05 5.4-5.5 m	Phyllite	6.58	1.00	0.02	0.98	30.6	3.7	0.9	0.1
DHVD05 8.2-8.3 m	Phyllite	7.46	0.26	<0.01	0.26	8.1	3.5	<0.5	0.4
DHVD03 1.22 m (-3/8")	Till	7.02	<0.02	<0.01	<0.02	<0.6	4.1	<0.5	6.8
DHVD03 2.74 m (-3/8")	Till	7.10	<0.02	0.01	<0.02	<0.6	5.7	0.7	9.5
DHVD03 7.31 m (-3/8")	Till	7.98	<0.02	<0.01	<0.02	<0.6	2.4	0.7	4.0
DHVD04B 2.74 m (-3/8")	Till	8.19	<0.02	<0.01	<0.02	<0.6	2.7	0.9	4.5
DHVD05 1.22 m (-3/8")	Till	7.17	0.03	0.02	<0.02	<0.6	5.1	1.6	8.5

Table 3: Summary of Phyllite Results

	Sample ID	Description	Total S	NP/AP
			% S	Ratio
NP/AP<1	DHVD05 5.4-5.5 m	Green grey weathered chlorite altered phyllite.	1.00	0.1
	DHVD01 9.7-9.8 m	Heavily quartz veined/intruded phyllite	0.31	0.4
	DHVD05 8.2-8.3 m	Grey chlorite altered phyllite, lots of evidence of fluid flow/quartz vein intrusions	0.26	0.4
	DHVD05 11.8-11.9 m	Grey-green phyllite, abundant, large quartz veins, chlorite alteration and sulphides observed.	0.22	0.5
	DHVD01 2.8-2.9 m	Grey green phyllite with minor veining	0.15	0.6
NP/AP>2	DHVD01 5.0-5.1 m	Grey green phyllite with minor/sparse veining	0.05	2.2
	DHVD03 11.7-11.8 m	Dark grey phyllite bedrock. Lots of foliation breaks.	<0.02	3.2
	DHVD05 3.4-3.5 m	Dark grey soft highly weathered phyllite. Breaks easily on foliation	0.03	3.2
	DHVD05 15.4-15.5 m	Grey-green white phyllite, thin quartz veins, chlorite alteration, highly fractured, some sulphides observed.	0.19	4.4
	DHVD02 13.5-13.6 m	Weathered phyllite bedrock	<0.02	4.7

Table 4: Summary of SFE Results Compared to 10x CCME Guidelines

Sample ID			10x CCME Guideline	DHVD01 9.7-9.8 m	DHVD02 13.5-13.6 m	DHVD05 3.4-3.5 m	DHVD05 5.4-5.5 m	DHVD05 11.8-11.9 m
Parameter	**Method**	**Units**						
pH	meter			7.31	7.17	6.34	5.48	7.38
Conductivity	meter	uS/cm		49	27	54	193	15
Total Acidity (to pH 8.3)	titration	mg CaCO3/L		4.5	3.7	4.7	16.5	3.9
Alkalinity	titration	mg CaCO3/L		11.4	5.7	3.5	1.5	6.9
Sulphate	Turbidity	mg/L		12	7	17	87	3
Dissolved Metals								
Aluminum Al	ICP-MS	mg/L	0.05 - 1	0.0177	0.0112	0.0104	**0.0796**	0.0156
Antimony Sb	ICP-MS	mg/L		0.00019	0.00003	0.00002	0.00024	0.00369
Arsenic As	ICP-MS	mg/L	0.05	0.00088	0.00013	<0.00002	0.00042	0.0006
Barium Ba	ICP-MS	mg/L		0.00088	0.0142	0.00433	0.0211	0.00271
Cadmium Cd	ICP-MS	mg/L	0.00017	0.000008	0.000005	0.000033	**0.00154**	0.000014
Calcium Ca	ICP-MS	mg/L		4.12	3.86	6.71	26.8	1.61
Cobalt Co	ICP-MS	mg/L		0.000747	0.00308	0.0019	0.179	0.000185
Copper Cu	ICP-MS	mg/L	0.02	0.00066	0.00042	0.0011	0.00459	0.00052
Iron Fe	ICP-MS	mg/L	3	0.006	0.011	0.004	**4.34**	0.004
Lead Pb	ICP-MS	mg/L	0.01	0.00119	0.00002	0.000013	0.00165	0.00225
Magnesium Mg	ICP-MS	mg/L		0.89	0.47	0.93	2.12	0.18
Manganese Mn	ICP-MS	mg/L		0.515	0.0114	0.0335	0.0949	0.00339
Mercury Hg	ICP-MS	ug/L	0.26	<0.01	<0.01	<0.01	0.03	<0.01
Molybdenum Mo	ICP-MS	mg/L	0.73	0.00018	0.00047	<0.00005	0.00008	0.00034
Nickel Ni	ICP-MS	mg/L	0.25	0.00687	0.00397	0.00639	0.537	0.0017
Potassium K	ICP-MS	mg/L		3.06	0.78	1.09	1.98	1.86
Selenium Se	ICP-MS	mg/L	0.01	0.00011	0.00006	0.00006	0.00294	0.00011
Silicon Si	ICP-MS	mg/L		1.01	1.65	2.53	3.43	1.07
Sodium Na	ICP-MS	mg/L		1.79	0.79	0.82	0.78	0.62
Strontium Sr	ICP-MS	mg/L		0.0163	0.021	0.0253	0.101	0.00555
Thallium Tl	ICP-MS	mg/L	0.008	0.000012	<0.000002	0.000008	0.000017	0.000014
Uranium U	ICP-MS	mg/L		0.000291	0.000016	0.000006	0.00513	0.000086
Zinc Zn	ICP-MS	mg/L	0.3	0.0031	0.0004	0.0029	0.0658	0.0035

Notes: 1. SFE results are compared to CCME Guidelines for illustration purposes only. CCME Guidelines apply only to receiving environments and do not apply to mine water.

2. Values that exceed the screening criteria are in **bold**.

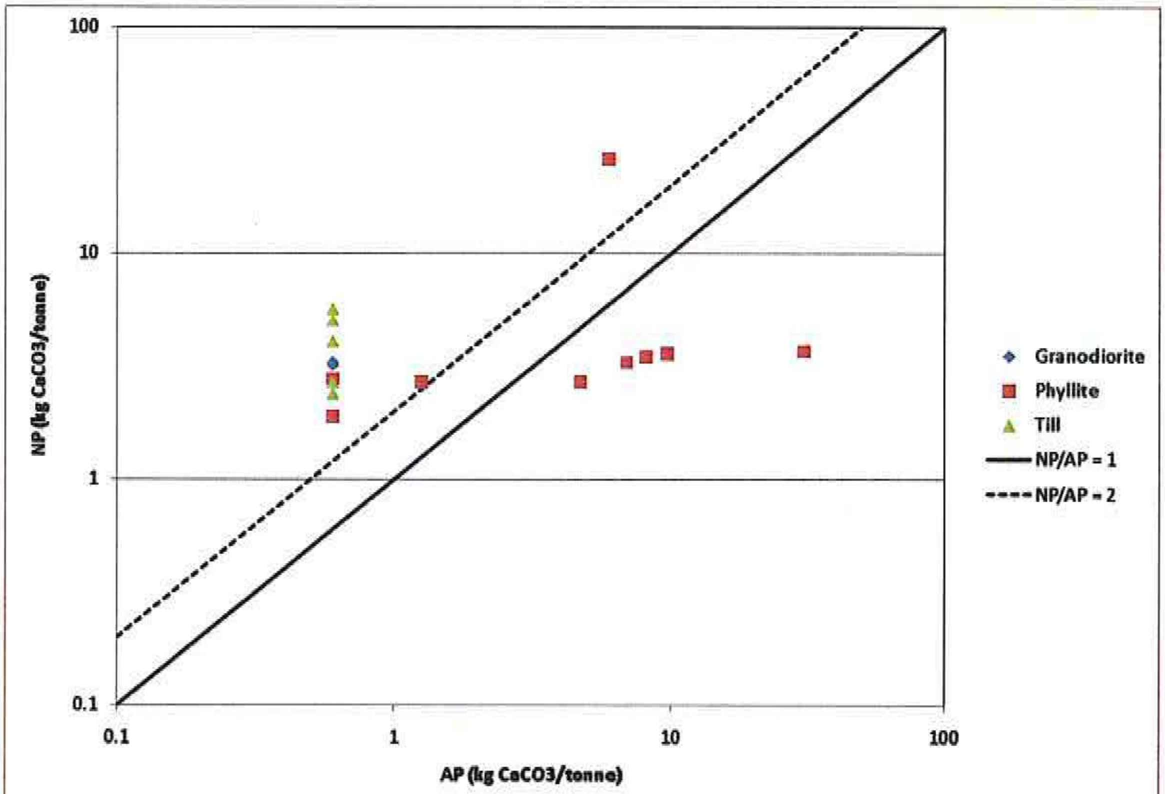

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Figure 1: NP Compared to AP by Rock

Appendix III

Erodibility Potential of the Bedrock

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Memo

To:	Peter Healy, SRK	**Date:**	November 16, 2009
cc:		**From:**	Megan Kinsey
Subject:	Vangorda Creek Diversion Channel Erosion	**Project #:**	1CY001.031.004

1 Introduction

SRK Consulting (Canada) Inc has prepared this memo to address concerns presented by North West Hydraulic Consultants (NHC) in regard to step-pool scour of the steep slope reach in the draft design for the proposed Vangorda Creek Diversion. This memo will discuss literature on the common approaches to calculate bedrock scour, the methods applicability to the problem presented and estimate if bedrock erosion due to water forces is likely to occur.

2 Background

2.1 Design

The proposed Vangorda Creek Diversion is consists of three reaches, the third reach which has a slope of 13.0% is the cause of the design concerns. A stepped channel design is to be used for the steep reach to dissipate energy. The step pool design itself in not cause concern rather the method of step pool lining. In the draft design report for the Vangorda Creek Diversion it was assumed that the phyllite and weathered phyllite bedrock underlying a large portion of the proposed diversion would be highly susceptible to erosion (SRK, 2009a). Therefore the stepped channel was designed with riprap lining to prevent channel erosion. Concerns presented regarding scour and movement of the riprap lining lead to further consideration of a stepped bedrock chute design. Therefore further analysis into the erodibility of the phyllite bed rock was required.

The stepped chute system is to be designed for a 500 year, 15 m^3/s flood. Key design parameters include:

- Design Discharge = 15.0 m^3/s
- Step height = 1.0 m
- Channel Width = 6.0 m

2.2 Bedrock Properties

The phyllite bedrock was generally moderately to heavily fractured and jointed displaying heavy micro defects and soft red brown staining. Typically the phyllite rock was of poor quality, with Rock Quality Designation (RQD) in the range of 29 to 46% (SRK, 2009b). The jointing and foliation breaks were found to be orientated sub horizontally, to horizontally with near horizontal planar cleavage. The intact rock strength observed varied between R1 for heavily weathered phyllite to R4 for un- weathered phyllite; R2 was the most common intact rock strength for weathered phyllite.

3 Determination of Bedrock Erodibility

Bedrock erosion is the product of several processes including freeze-thaw, scour due to water forces and scour due to entrained bed load. This section will only discuss methods to determine bedrock erosion from water forces. Bedrock erosion due to freeze thaw will only be considered in the most peripheral sense for the role it plays in bedrock weathering. Methods to quantify erosion and erosion rate due to water forces are limited and commonly assume that erosion rate is proportional to either shear stress or stream power. These assumptions are often too simple to describe the full erosional process of bedrock, but are commonly used to quantify erosion; several methods are discussed below (Sklar and Dietrich, 2001).

Sklar and Dietrich (1998) present a stream power law that equates erosion rate to the product of rock erodibility, and channel slope and area to the power of numerical constants. Values are assigned to the numerical constants by observation assuming that the coefficient representing efficiency of erosion is constant for the reach being studied (Sklar and Dietrich, 1998). The applications of Sklar and Dietrich's stream power law are limited, and only apply to fluvial dominated channels with a near constant slope of less than 20% (Sklar and Dietrich, 1998). Due to the simplifying assumptions inherent in the Sklar and Dietrich (1998) stream power model, the application of the model to a stepped chute system would provide limited results.

The Annandale stream power method does not directly calculate erosion rate, however it uses the erodibility index method is used to determine scour potential. Scour potential is determined by comparing the stream power of the water, to a critical stream power of the rock calculated from the erodibility index (Annandale, 2006). The equation used to correlate erodibility index is determined empirically, however a similar correlation between critical stream power and erodibility index is presented in an independent study conducted by van Schalkwyk in 1995 (Annandale, 2006). Stream power is calculated from the flow properties of the system. If stream power of the water is determined to be greater than the critical stream power of erosion, erosion is expected to occur.

The USDA has come up with a method to calculate headcut erosion of a vegetated earth spillways based on shear strength and makes use of several simplifying assumptions (USDA, 1997a). This method assumes all spillway exit channels are long enough for flow to approach normal depth, as well channel shape and width are assumed constant for all reaches. The model breaks down the erosion process into three phases. The first phase consists of the development of concentrated flow and the destruction of vegetal cover (USDA, 1997a). Phase two is downward downstream erosion resulting in a vertical headcut; once a vertical headcut is created the flow enters the third phase (Temple and Hanson, 1994). For the purposes of determining erosion rate in a stepped chute the third phase would be the only phase of concern, since the chute would not be vegetated and the vertical headcut would already be established by step creation. However, the USDA method assumes normal flow before the head cut and the stepped chute system never reaches normal flow, so this method might not be applicable. As well erosion rate is dependent on the critical shear stress of the material being eroded, for granular materials this stress would be calculated through and iterative application of the Shields method. Shields method is dependent on particle size and not applicable to solid rock faces, therefore the shear stress of phyllite would have to be determined from using another method before erosion rate could be calculated.

Both Annandale (2006) and the USDA (1997b) take into account the erosive properties of a soil or rock with the erodibility index. The erodibility index or headcut erodibility index (USDA, 2007b), is a factor representing the relative resistance of a material to erosion (Annandale, 2006 and USDA, 2007b). This factor takes into account material strength, discontinuities, material orientation, faults and fissures. The methods used to calculate erodibility index can be found in Annandale (2006) or USDA (1997b).

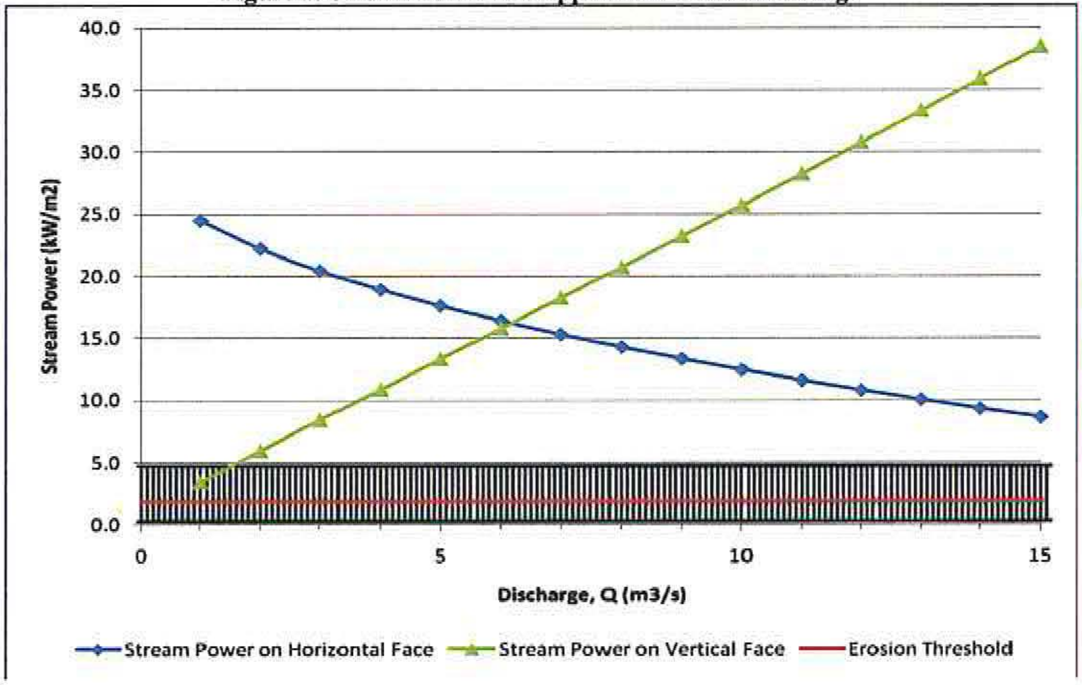
4 Scour Potential of Bedrock

Using the stream power methodology presented by Annandale (2006) and the bedrock properties presented above the scour potential of phyllite bedrock in a stepped chute was evaluated. In this evaluation the hydraulic regime of a step was taken as that of a shallow nappe undergoing headcut erosion. Therefore erosion of bedrock was evaluated for vertical face of the step as well as the horizontal face of the chute bed.

The critical stream power of the phyllite bedrock was determined to be between 0.31 kW/m^2 and 4.70 kW/m^2. The calculated critical steam powers were calculated using the best estimates given the data available, for a more accurate prediction specific field observations are required. Therefore, these critical stream power values should be used cautiously, since assumptions were made when applying the erodibility index criteria. In the field investigation report the phyllite was described as being highly fractured and folliated, therefore the maximum number of joint sets was assumed and that the ratio of joint width to joint length is large. As well, for the purposes of evaluating bedrock scour it is best to assume that all phyllite is significantly weathered. This assumption is to account for the fact that if the phyllite bedrock at the base of the channel is not weathered at the time of construction it will likely become weathered by freeze thaw within the design life of the diversion channel.

The stream power from a shallow nappe was calculated for both the vertical and horizontal faces of the stepped chute at various different discharge values, using the equations presented in Annandale (2006). The water stream power, for various flow rates in comparison to the threshold (critical) stream power for erosion are presented in Figure 1.

Figure 1: Stream Power of Stepped Chute with Discharge



From Figure 1it can be seen that the best estimate of critical stream power of erosion is less than the stream power on both the horizontal and vertical chute faces. Therefore erosion of the bedrock chute is expected to occur during all flow conditions.

5 References

Annandale, G.W. (2006). Scour Technology: Mechanics and Engineering Practice. McGraw-Hill, New York, 2006

Meyer, M. (ed) (2005) *Chapter 11: Freejet and Straight Drop Spillways*. Hydraulics of Spillways and Energy Dissipators. Marcel Dekker, New York, 2005

SRK (2009a). *Faro Mine Complex Vangorda Creek Diversion Design* Report-*Draft*. Report prepared for Government of Yukon. Project No. 1CY001.031. September 2009

SRK (2009b). *Faro Mine Complex Vangorda Creek Diversion Geotechnical Field Investigation-Draft*. Report prepared for Government of Yukon. Project No. 1CY001.031. August 2009

Sklar, L., Dietrich, W. (1998) *River Longitudinal Profiles and Bedrock Incicion Models: Stream Power and the Influence of Sediment Supply.* Rivers over Rock, Geophysical Monograph 107, 1998, pp.237-260

Sklar, L., Dietrich, W. (2001) *Sediment and Rock Strength Controls on River Incision into Bedrock.* Geology, Vol 29, December 2001, pp.1087-1090

Temple, D. And hanson, G. (1994) *Headcut Development in Vegetated Earth Spillways.* Applied Engineering in Agriculture, Vol 10(5) pp. 677-682

United States Department of Agriculture (USDA) (1997a). *Earth Spillway Erosion Model.* Part 628 of National Engineering Handbook. 210-VI-NEH, August 1997.

United States Department of Agriculture (USDA) (1997b). *Field Procedures Guide for Headcut Erodibility Index.* Part 628 of National Engineering Handbook. 210-VI-NEH, August 1997.

Appendix IV

Technical Specifications (Draft) and Gradation Requirements

Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex
Vangorda Creek Diversion – Phase 1

Technical Specifications – Draft Rev B

Prepared for:

Indian and Northern Affairs Canada
Affaires indiennes et du Nord Canada

and

Yukon
Government

Prepared by:

SRK Consulting
Engineers and Scientists

Project Reference Number
SRK 1CY001.031

April 2010

Table of Contents

List of Tables

1 General Requirements

1.1 Part 1 – General

1.1.1 Documents

This section of the Specification forms part of the Contract Documents and is to be read, interpreted and coordinated with all other parts.

1.1.2 Revision Summary

Table 1.1 below provides a summary of the revision history of these Technical Specifications.

Table 1.1: Revision Summary

Revision	Status	Issue Date	Major Changes
A	Issued for Review	November 2009	N/A
B	Issued for Review	February 2010	Stepped Chute

1.1.3 Definitions

The following definitions and interpretations shall apply to these Technical Specifications:

1. PROJECT means Phase 1 of the Vangorda Creek Diversion Channel Construction, of which the Works described in the Document may be the whole or part.

2. WORKS is defined as the entire completed construction as defined by this Document, or the various separately identifiable parts thereof, required to be furnished under the Contract Documents. Works is the results of performing services, furnishings labour, and furnishing and incorporating materials and equipment into the construction, all as required by the Contract Documents. CONTRACT DOCUMENTS are defined as the agreement, addenda (which pertain to the Contract Document), Contractor's bid (including documentation accompanying the bid and any post-bid addenda submitted) when attached as an exhibit to the agreement, Contractor's proposed plans and schedule, the bonds, the general conditions, the supplementary conditions, these Specifications, the Drawings, together with all Modifications issued after the execution of the agreement.

3. SPECIFICATIONS as defined as the Technical Specifications herein prepared by SRK Consulting (Canada) Inc. on behalf of the Yukon Government. These Specifications are to be read, interpreted and coordinated with all the Drawings, Modifications, updated Revisions, or any other relevant documents produced by the Engineer, and the Yukon Government.

4. DRAWINGS are defined as all Engineering Drawings, plans, sketches and maps issued for construction with these Specifications, or subsequently, as deemed necessary by the Engineer.

5. MODIFICATIONS are defined as changes made to the Specifications and/or Drawings, which have been approved by the Engineer in writing. These modifications can be issued at any time, including after issuance of these Specifications and any accompanying Drawings and/or other Modifications.

6. Responsible Parties:

    a. GOVERNMENT is defined as the Yukon Government (YG) is also the Owner of the Faro Mine Site.

    b. CONTRACT MANAGER (CM) is defined as YG's representative responsible for the administration and management of the works. The Designated Contract Manager for the works is Denison Environmental Services (DES). The ENGINEER-OF-RECORD is defined as an engineering representative appointed and authorized by YG for the Works described in this Document. The Engineer shall be a registered Professional Engineer in Yukon, or a designated site representative under his/her direct supervision during construction. At the time of issuing this Document, the Engineer-of-Record is a designated employee of SRK Consulting (Canada) Inc. (SRK).

    c. CONTRACTOR is defined as the party or appointed representative of the party that has an agreement with the Yukon Government to execute the Works defined in this Document.

    d. SUB-CONTRACTOR is defined as the party of appointed representative of the party that has an agreement with the Contractor to execute specialized components of the Works defined in this Document that cannot be carried out by the Contractor.

    e. SURVEYOR is defined as the party or appointed representative of the party that has an agreement with the Contractor to act as Site Surveyor for the execution of the Works defined in this Document. The Surveyor shall have equipment and means on site to carry out horizontal and vertical ground surveys with an accuracy of 10 mm. The Surveyor shall also have the equipment and means to prepare Digital Terrain Models and Drawings on site that is compatible with AutoCAD 2008 or AutoCAD 2010. The Surveyor reports to the Contractor, but will be available for use by the Engineer as required, provided the Engineer has requested such needs through the Contractor.

    f. QUALITY CONTROL AND ASSURANCE TEAM is defined as the individual(s) working under the Engineer to perform on site quality control and assurance for the works defined in this Document.

    g. LAND OWNER is defined as the party or appointed representative of the party that has the right of land outside of the mine vicinity.

7. ON-SITE MATERIAL is defined as borrow materials obtained from within designated on site excavations.

8. OFF-SITE MATERIAL is defined as material obtained from sources other than on-site.

9. RECORD DOCUMENTS are defined as the document prepared and certified by a Land Surveyor, Material Testing Technician, Quality Control and Assurance Personnel, Specialist Professional, or any other parties documenting any aspect of the Works.

10. PRODUCTS are defined as processed fill material, synthetic products, machines, components, equipment, fixtures, and systems forming the Works. This does not include machinery and equipment used for preparation, fabrication, conveying, and erection of the Works. Products may also include existing material or components required for reuse.

11. SLOPES are defined in all instances in these Specifications and on Drawings in term of horizontal distance to vertical distance (i.e. 2H:1V shall be read as 2 horizontal unit distance to 1 vertical unit distance).

12. EQUIPMENT means all construction mobile equipment that will be used to complete the Works.

1.1.4 Summary of Works

1. The Contractor will be responsible for ensuring that all the Works defined in this Document be executed in accordance with all appropriate permits and approvals. Furthermore, the Contractor is responsible for ensuring that all the Works are carried out in accordance with the Government's environmental and safety standards.

2. The Works in this document covered by this Specification is defined as Phase 1 of the Vangorda Creek Diversion and includes, but is not limited to the following:
    a. Clearing, stripping in work areas from STA 0+00 to STA 1+325.
    b. Construction of access roads.
    c. Construction of headworks at STA 0+00.
    d. Construction of the Re-aligned Vangorda Creek Diversion – Phase 1.
    e. Construction of an expansion to the existing plunge pool.
    f. Construction of earthworks components of stepped chute.
    g. Construction of surface water management measures along the channel alignment.
    h. Construction and removal of a temporary cofferdam.
    i. Construction of an erodible plug.
    j. Breaching of Blind Creek Road.
    k. Drilling and blasting in sections along the channel alignment.

l. Grading final surfaces and removal of temporary facilities and structures.

1.1.5 Contradictions

1. Should any contradictions, either implied or read, exist between the Specification and the Drawings, the Contractor shall:
   a. Notify the Engineer.
   b. Stop all works that concern the contradiction until the contradiction is remedied or clarified by the Engineer.
2. The decision of the Engineer is final.

1.1.6 Contractor's Responsibilities

1. The Contractor, in the context of the Works defined in this Document shall:
   a. Comply with Yukon Workers' Compensation Health and Safety Regulations and any other relevant required health and safety regulations.
   b. Provide the Engineer with a copy of their Health and Safety Plan, which has been specifically prepared for this Project.
   c. Become familiar with the relevant regional and site-specific conditions that deviate from the Specifications and Drawings, and inform the Engineer when a problem or delay is anticipated.
   d. Be responsible for making its own measurements and installing the Works to fit the condition encountered.
   e. Before proceeding with the Works, examine all Drawings and report to the Engineer any apparent discrepancies or interferences. The Engineer shall have the privilege of making minor alterations to the Drawings and the Specifications. All alternations shall be issued under a covering Works order signed and authorized by the Engineer prior to the start of alternation, if the alternation will affect the terms of Contract.

1.1.7 Testing by the Contractor and the Engineer

1. Testing the Works:
   a. The Engineer will carry out Quality Assurance for the Works defined in this Document, and will undertake testing at a frequency and at the location specified in the various sections of these Specifications. The Engineer may undertake any addition testing which he deems necessary on any part of the Works.

b. Performance testing by the Engineer shall in no way relieve the Contractor of its sole responsibility for completing the Works in accordance with the specified requirements.

c. The Contractor shall undertake his own quality control and quality assurance, and shall submit a copy of his Quality Assurance/Quality Control (QA/QC) program for review by the Engineer at least seven days prior to commencement of the Works.

d. All quality control or other test data, survey data or the like, collected by the Contractor shall be made available to INAC, the Government and the Engineer on request.

1.1.8 Submittals

1. The Contractor shall submit information as specified and requested from the Engineer. All submittals required by the Engineer will be requested through formal transmittal to the Contractor.

2. The Engineer has the right to request as a Submittal any other information deemed necessary throughout execution of the Works. This includes information not currently defined as Submittal information on the Drawings and Specifications.

1.1.9 Changes

1. Any changes that are outside of the Contract agreement shall be submitted to the Contract Manager for approval via a Change Order. The Contractor shall submit the Change Out with Engineer's approval to the Contract Manager.

1.1.10 Construction Schedule

1. The Contractor shall submit a detailed schedule of construction to the Engineer 28 days prior to the commencement of construction. The Engineer reserves the right to halt the commencement of specific construction components, if in his opinion there is any risk that the construction cannot be completed under the optimum weather conditions.

2. The Contractor is responsible for updating and modifying the construction schedule according to the ongoing progress and delays. The construction schedule shall be made available to the Contract Manager, and the Engineer upon request. The Contractor shall notify the Engineer and the Contract Manager 7 days before the scheduled component start date if and when major delay in the schedule is anticipated.

1.1.11 Construction Drawings

1. Drawings specific to construction will be issued by the Engineer prior to commencement of the Work. Drawings shall be reviewed by the Contractor to ensure all aspects of the construction are covered and report to the Engineer any discrepancies and interferences. The Contractor shall

notify and inform the Engineer of construction progress and Drawing requirements four weeks prior to commencement of any Works.

2. Only Drawings specifically marked with the Following words are considered acceptable for Construction: ISSUED FOR CONSTRUCTION, or IFC.

3. The following is the list of Drawings which accompany this Document:

Table 1.2: List of Drawings

Drawing ID	Title	Date of Issue	Revision
V-01	General Location	February 2010	D
V-02	Drawing Index and Notes	February 2010	D
V-03	General Arrangement (with Orthophoto)	February 2010	D
V-04	General Arrangement	February 2010	D
V-05	Profile Along Centreline of Channel (Section A-A')	February 2010	D
V-06	Section B-B' (0+025) and Section C-C' (0+075)	February 2010	D
V-07	Section D-D' (0+350) and Section E-E' (0+550)	February 2010	D
V-08	Section F-F' (0+870)	February 2010	D
V-09	Section G-G' (1+025)	February 2010	D
V-10	Headworks Site Plan	February 2010	A
V-11	Headworks Sections	February 2010	D
V-12	Step Pool Plan and Sections	February 2010	D
V-13	Step Pool Typical Profile	February 2010	D
V-14	Plunge Pool Plan and Sections	February 2010	D
V-15	Stake-Out Points (TBD)	February 2010	C

1.1.12 Survey and As-built Drawings

1. The Contractor is responsible for all construction surveying. Construction survey data will be made available to the Contract Manager, and the Engineer upon request. The Contractor will provide as-built drawings and surveys to the Engineer for completion approval. The as-built drawings and surveys must be provided to the Engineer within 28 days of project completion.

END OF SECTION 1

2 Mobilization and Demobilization

2.1 Part 1 – General

2.1.1 Documents

1. This section of the Specifications forms part of the Contract Document and is to be read, interpreted and coordinated with all other parts.

2.1.2 Description

1. The work covered by this section consists of supplying all plants, labours, materials and equipment, and performing all work necessary for the Contractor's mobilization and demobilization.

2. Mobilization shall be to the mine site and shall include all cost required to:

   a. Provide the Contract Manager, and the Engineer with a complete list of plants, equipment, tools, supplies and material that will be required for the Works. This list must be completed with individual piece shipping dimensions and weight and the highway and access road limitations.

   b. Mobilize all labour, supervision, technical personnel and other services required for completion of the work to the mine site.

   c. Furnish and install temporary facilities and utilities required for the construction including Engineer's work station.

   d. Setup and assemble plant and equipment and move to the specific work and staging locations.

   e. Provide the Engineer with office space and communication including radio, phone and internet within the site office area.

   f. Throughout the duration of the work, each new person shall , attend an initial and all subsequent Health and Safety site briefings as specified in Contractor's and Yukon Workers' Compensation Health and Safety requirements and standards.

3. Demobilization shall be regarded as completed when all labourers, equipment, temporary facilities, surplus and waste material resulting from the Contractor's operations have been removed from site and the work areas has been cleaned, reclaimed and graded to the satisfaction of the Engineer.

notify and inform the Engineer of construction progress and Drawing requirements four weeks prior to commencement of any Works.

2. Only Drawings specifically marked with the Following words are considered acceptable for Construction: ISSUED FOR CONSTRUCTION, or IFC.

3. The following is the list of Drawings which accompany this Document:

Table 1.2: List of Drawings

Drawing ID	Title	Date of Issue	Revision
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V-04	General Arrangement	February 2010	D
V-05	Profile Along Centreline of Channel (Section A-A')	February 2010	D
V-06	Section B-B' (0+025) and Section C-C' (0+075)	February 2010	D
V-07	Section D-D' (0+350) and Section E-E' (0+550)	February 2010	D
V-08	Section F-F' (0+870)	February 2010	D
V-09	Section G-G' (1+025)	February 2010	D
V-10	Headworks Site Plan	February 2010	A
V-11	Headworks Sections	February 2010	D
V-12	Step Pool Plan and Sections	February 2010	D
V-13	Step Pool Typical Profile	February 2010	D
V-14	Plunge Pool Plan and Sections	February 2010	D
V-15	Stake-Out Points (TBD)	February 2010	C

1.1.12 Survey and As-built Drawings

1. The Contractor is responsible for all construction surveying. Construction survey data will be made available to the Contract Manager, and the Engineer upon request. The Contractor will provide as-built drawings and surveys to the Engineer for completion approval. The as-built drawings and surveys must be provided to the Engineer within 28 days of project completion.

END OF SECTION 1

2 Mobilization and Demobilization

2.1 Part 1 – General

2.1.1 Documents

1. This section of the Specifications forms part of the Contract Document and is to be read, interpreted and coordinated with all other parts.

2.1.2 Description

1. The work covered by this section consists of supplying all plants, labours, materials and equipment, and performing all work necessary for the Contractor's mobilization and demobilization.

2. Mobilization shall be to the mine site and shall include all cost required to:

    a. Provide the Contract Manager, and the Engineer with a complete list of plants, equipment, tools, supplies and material that will be required for the Works. This list must be completed with individual piece shipping dimensions and weight and the highway and access road limitations.

    b. Mobilize all labour, supervision, technical personnel and other services required for completion of the work to the mine site.

    c. Furnish and install temporary facilities and utilities required for the construction including Engineer's work station.

    d. Setup and assemble plant and equipment and move to the specific work and staging locations.

    e. Provide the Engineer with office space and communication including radio, phone and internet within the site office area.

    f. Throughout the duration of the work, each new person shall , attend an initial and all subsequent Health and Safety site briefings as specified in Contractor's and Yukon Workers' Compensation Health and Safety requirements and standards.

3. Demobilization shall be regarded as completed when all labourers, equipment, temporary facilities, surplus and waste material resulting from the Contractor's operations have been removed from site and the work areas has been cleaned, reclaimed and graded to the satisfaction of the Engineer.

2.1.3 Submittals

1. Within 28 days after award of the Contract, the Contractor shall submit a mobilization plan including:

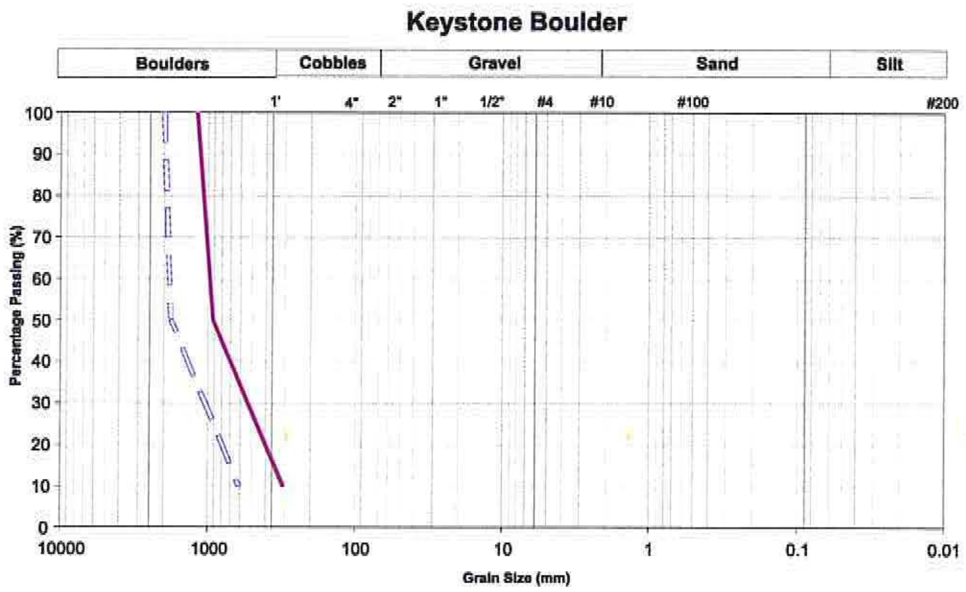
    a. Shipping dimensions and weight of all plants, supplies, tools, equipment, material and facilities.

    b. Shipping schedule for mobilization and demobilization.

    c. A layout drawing of the Contractor's temporary facilities, staging areas, including potable water source for the Works.

    d. Contractor Health and Safety Plan.

END OF SECTION 2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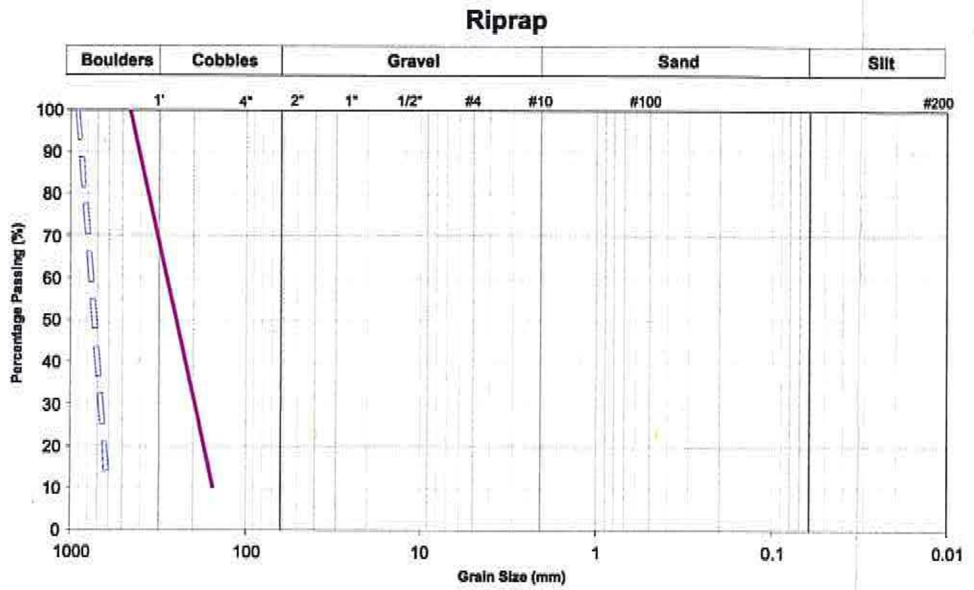
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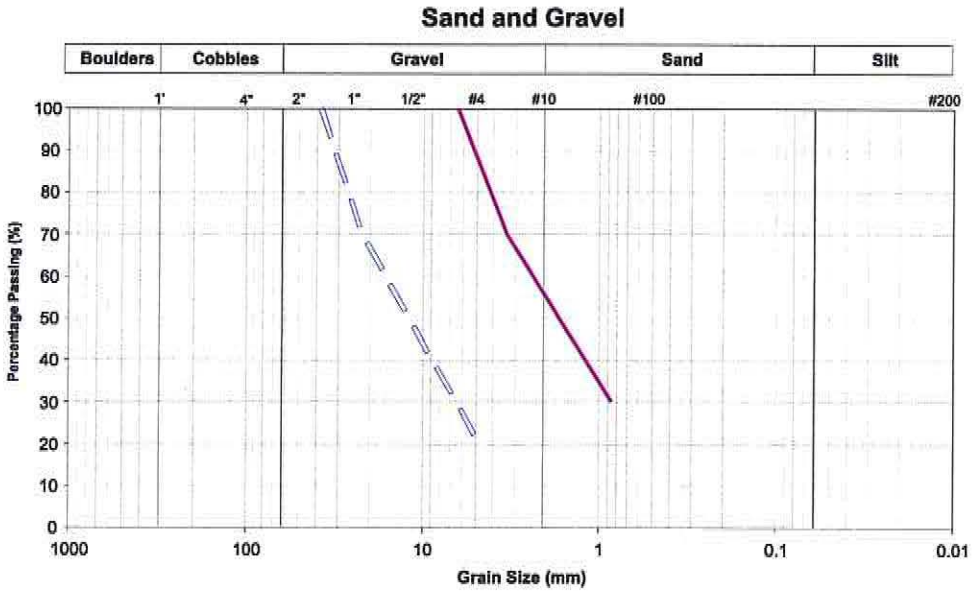
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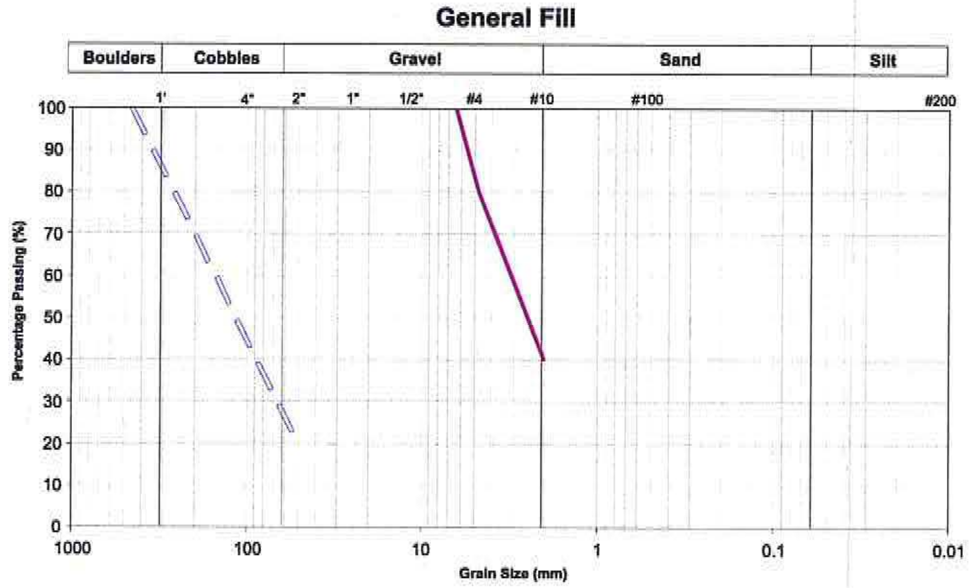
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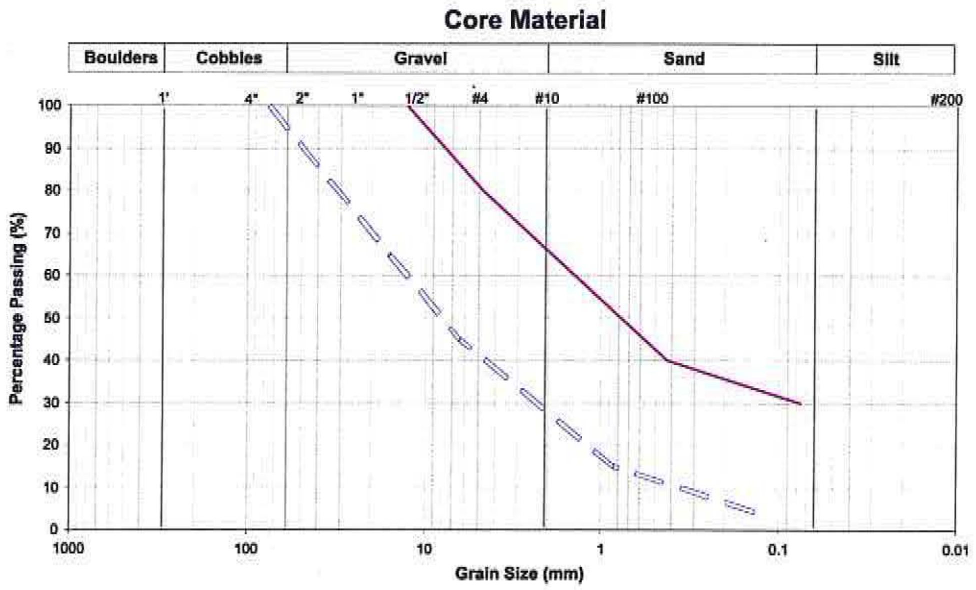

Core Material

Legend

- - Coarse Limits

— Fine Limits

J:\01_SITES\FARO\1.CY001.031_Vangorda Creek Diversion Channel\Acad\Rev C\1CY001_031-2-C.dwg

SRK Consulting
Engineers and Scientists
Vancouver B.C.

Faro Mine Closure
Fermeture de la mine Faro

Vangorda Diversion Channel

Gradation Requirements

SRK JOB NO.: 1CY001.031
FILE NAME: 1CY001_031-2-C.dwg

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

DATE: Nov. 09
APPROVED: PMH
FIGURE: App-4

3 General Construction and Site Preparations

3.1 Part 1 – General

3.1.1 Documents

This section of the Specification forms part of the Contract Document and is to be read, interpreted and coordinated with all other parts.

3.1.2 Work Description

1. The following tasks are part of initial works required prior to the main reclamation work in the mine site.

   a. PERMITS and REGULATIONS – the Contract Manager is to acquire all necessary permits required for the works for the scheduled construction and adhere to all safety and environmental regulations outlined by the applicable Federal, Provincial, Local and/or the Land Owner.

   b. PROTECTION – unless otherwise instructed, the Contractor is to take all necessary precautions to prevent damage to natural and man-made features, including but not limited to wildlife habitat, survey monuments, and instrumentations outside of project areas. The Contractor may not perform any Works outside of the permitted and pre-approved construction areas.

   c. PREPARATION – The Contractor shall confirm the Works limits by having his surveyor layout the extents of all works, prior to commencement of earthworks or surface works. The Engineer will inspect these demarcated areas and confirm all limits before giving written approval to proceed. The Contractor shall inspect the Works site and verify with the Engineer any restrictions within or adjacent to the work limits.

   d. SITE OFFICES AND LAYDOWN – the Contractor is to maintain a site office where all new personnel should report to prior entering and working on site. The site office should be facilitated according to the Contractor's daily needs with a work station for the Engineer. Laydown area(s) should be identified in the project area for staging of the materials and equipment, where it will not affect construction progress and traffic. Any cutting, clearing and stripping shall be done according to the Specifications. The laydown area(s) shall be fully reclaimed upon completion of the Works.

   e. CUTTING – is defined as cutting of any standing timber in the project area. The Contractor is to contact the Contract Manager for permission prior to any cutting of trees and timbers. All cutting of trees must be approval by the Contract Manager and carried out in accordance to directions and specifications from the Contract Manager.

f. CLEARING – it is defined as clearing of any vegetation and/or stumps, excluding standing trees, in the project area. The removed waste shall be stockpiled for reclamation use.

g. STRIPPING – it is defined as excavation and removal of unsuitable material and organic topsoil in the project area. The removed waste and topsoil shall be stockpiled for reclamation use.

h. STOCKPILES and DISPOSAL – the Contractor will submit plans and location for handling and operating the unsuitable and organic stockpiles to the Contract Manager for approval. The stockpile should have proper sediment and surface water control, and be free standing without short term stability issues.

i. SNOW and ICE REMOVAL – it is defined as clearing and removal of any snow that hampers construction activity and cover works. The removed snow shall be stockpiled in area(s) where subsequent melting will not affect the Works nor other mine site operations. The Contractor will submit plans and location for handling the snow removal to the Contract Manager for approval.

3.1.3 Submittals

1. The Contractor shall notify the Contract Manager outlining his intended methods for site preparation within a given area at least seven days prior to the commencement of Work, including, but not limited to the following details:

   a. Typical equipment deployment.

   b. Work schedule including work area(s), volume, handling and operating procedures, stockpile area(s) and typical sections, and traffic pattern(s).

   c. Contingency plan for change in weather conditions and other foreseeable risks.

   d. Sediment and surface water controls around the intended work area(s).

END OF SECTION 3

4 Soil Excavation

4.1 Part 1 – General

4.1.1 Documents

1. This section of the Specification forms part of the Contract Document and is to be read, interpreted and coordinated with all other parts.

4.1.2 Description

1. The excavation Works entail excavation of soil and other material below the original ground surface to neat lines as indicated on the Drawings.
2. The Works to be carried out under this Section consists of furnishing all labour, material, equipment and the performance of all Works necessary to carry out rock and soil excavation as shown on the Drawings, and as specified herein, which will include, but is not limited to the following:
    a. Excavation of new channel alignment.
    b. Excavation of a plunge pool.
    c. Excavation of thestepped chute in the steeper section of the channel alignment.
    d. Breaching of the Blind Creek Road as part of the channel excavation.
3. The Works shall also include the loading, transportation and permanent disposal of all excavated materials which are deemed by the Engineer to be surplus, or unsuitable for use as a construction material, and the loading, transportation and possible temporary stockpiling and re-handling of acceptable material to location to where they can either be used as part of the temporary or permanent structures, or stockpile in readiness for future temporary or permanent use.
4. The Contractor will be responsible for identifying suitable stockpile locations for any excavated material, whether temporary or permanent. The Engineer will, however, have the right to reject any identified site, if in his opinion it may interfere with any of the Works.

4.1.3 Exclusions

1. Contractor will submit plan(s), location(s), handling and operating procedure(s) to Contract Manage for approval for any excavation that is listed in the work description.

4.1.4 Definitions

1. The following words and terms, unless the context otherwise requires, in this Specifications, shall have the meaning set out below:

    a. SOIL means general overburden material free of organics which can be used in part as bedding material for liners and approved by the Engineer.

    b. ROCK means bedrock material which forms part of the foundation of the Works or from a designated foundation excavation.

    c. UNSUITABLE MATERIAL means any soil or rock that does not meet the Specifications for the use of this project.

    d. NEAT LINE means the final line or grade to which excavation is to be performed.

    e. COMMON EXCAVATION means excavation of all materials, included blasted rock weathered bedrock, soil, and unsuitable material by mechanical means.

4.1.5 Procedures

1. The details of the surface excavation shown on the Drawings represent an engineered design encompassing drainage under particular assumed conditions. Variations in site conditions may require field adjustments to the excavation shape, slope reinforcement and drainage under the Engineer's discretion.

2. If, in a specific area, a plan that has been previously adopted does not fit the site conditions in accordance with the requirement of these Specifications, the Engineer shall submit a revised plan to the Contractor before continuing excavation in identified areas.

3. Surface water management measures shall be constructed and implemented prior to the Work and emergency adjustment shall be made to accommodate any change in site conditions.

4.1.6 Submittals

2. The Contractor shall notify the Engineer outlining his intended methods for excavation within a given area at least seven days prior to the commencement of Work, including, but not limited to the following details:

    a. Typical equipment deployment.

    b. Work schedule including work area(s), volume, stockpile area(s), traffic pattern and hours of operations.

    c. Contingency plan for change in weather conditions and other foreseeable risks.

    d. Sediment and surface water controls around the intended work area(s).

4.2 Part 2 – Execution

4.2.1 Preparation

1. Prior to beginning a grading or excavation operation in any area, all necessary clearing and stripping in that area shall have been performed in accordance with the Specifications.
2. The Contractor shall satisfy with himself as to the character, quantity and distribution of all the material to be excavated.
3. The Contractor shall have a contingency plan for sudden unforeseeable change of weather condition in place prior to excavation commencement.
4. The Contractor shall be responsible for sediment and surface water runoff control around the construction area to ensure there is minimal impact on the natural state of the surrounding environment in accordance to all issued regulations and permits.

4.2.2 Common Excavation Methods

1. Common excavation of soil and blasted rock shall be performed as indicated on the Drawings, or as directed by the Engineer to the lines, grades, and elevations, and shall be finished to a reasonable smooth and uniform surface.
2. Should the Contractor, through carelessness or other fault, not excavate to the designated grades, he shall replace the excavation in an approved method, in accordance with the Specifications, or any modification thereof as directed by the Engineer.
3. At all times during the construction, the Contractor shall adopt excavation procedures such that at no time shall the stability of any slope be impaired. The Engineer reserves the right to stop work if he deems conditions to be unsafe.

4.2.3 Control of Surficial Water

1. Surface water flows during the precipitation events shall be directed away from the Works by means of temporary diversion berms, channels, or other acceptable means and, in any case, all surface flow on the Works area(s) shall be satisfactorily controlled, and to the environmental standard specified.

4.2.4 Slope Stability and Safety

1. Immediately following excavation and at any time during the Project, all loose material on slopes that appears to be unsafe or to endanger workmen, structures or equipment, shall be removed.
2. All slope stability measures will be considered incidental to the Works, and will be the responsibility of the Contractor, and done according to the required government safety standards.

END OF SECTION 4

5 Drilling and Blasting

5.1 Part 1 – General

5.1.1 Documents

1. This section of the Specification forms part of the Contractor Document and is to be read, interpreted and coordinated with all other parts.

5.1.2 Description

1. All blasting operations must be performed in accordance with all Federal and Yukon Government Regulations.
2. The Contractor will be responsible for familiarizing himself with all appropriate conditions that would apply to blasting.
3. The Works to be done under this Section consists of supplying all labour, materials, plants, and equipment and performing all Works necessary to carry out drilling and blasting with certified personnel and chemical agents as shown on the Drawings and specified herein.
4. The Work shall include but is not limited to:
    a. Providing a typical list of safety protocols and typical operation procedures that will be suitable for carrying out the Works.
    b. Providing suitably qualified personnel(s), with current blasting certifications, and chemical reagents for the specified Works.
    c. Drilling and blasting at the channel intake and sections of the channel alignment, as well as along the stepped chute sections where competent bedrock is found and mechanical excavation is not achievable.
    d. Drilling and blasting bedrock at the head works.

5.1.3 Definitions

1. The following words and terms, unless the context otherwise requires, in this Specification, shall have the meaning set out below:
    a. CERTIFIED PERSONNEL means a suitably qualified person that hold current blasting certificates issue by all necessary Federal and Yukon Government Regulatory agencies for the Project.

b. CHEMICAL BLASTING REAGENT means any form of reagent, and components that are suitable for use in the Project.

c. EXPLOSIVE MAGAZINE means a certified storage vault(s) or area(s) for storing unused explosive prior to work.

5.1.4 Submittals

1. The Contractor shall submit a drilling and blasting plan describing the schedule, and proposed methods for blasting operations.
2. The Contractor shall submit a plan and location for the explosive magazine(s) that meets the Federal and Yukon Government Regulations.

5.2 Part 2 – Products and Personnel

1. The Contractor is responsible for the procurement of all necessary supplies, personnel, equipment and acquire all necessary licenses, and notification to all Yukon Government and Federal Agencies.
2. The Contractor is responsible for management, maintenance, operation and security of the Explosive Facility, whether temporary or permanent.

5.3 Part 3 – Execution

5.3.1 Drilling

1. It is Constructor's responsibility to survey exposed bedrock along the channel alignment to clearly identify necessary drill depth.
2. The Contractor will lay out an appropriate blast depth and pattern for the specified grade or material size, taking due care to prevent over-breaking.

5.3.2 Blasting

1. The Contractor's Health and Safety Plan, list of blasting reagents, technician's certifications, and proposed methods shall be in place prior to blasting operations.
2. The Contractor will be responsible for notifying all air and land traffic of the time and location of any blast according to Federal and Yukon Government Regulations.
3. The Contractor will be responsible for putting in place all protocols and physical barriers to warn and prevent land and air traffic from entering the designated blast zone, according to all applicable Regulations.

4. The Contractor should use controlled blasting methods to minimize fly rocks and satisfy minimum safe distance requirements.
5. Certified Personnel must inspect the blast pattern post blasting to ensure there are no unconsumed agents and explosives left behind prior to continuing the Work. If unconsumed agents are found, the Certified Personnel shall remove them according to standard procedures.
6. The Contractor will be responsible for over break or fracture and cost to remediate the breakage as directed by and deemed necessary by the Engineer.

END OF SECTION 5

6 Fill Material Specifications

6.1 Part 1 – General

6.1.1 Documents

1. This section of the Specification forms part of the Contract Document and is to be read, interpreted and coordinated with all other parts.

6.1.2 Description

1. The sources and borrow area(s) of all fill are shown on the Drawing or as designed by the Engineer. For the types of material and related Specifications, see the gradation requirement herein or as shown on the drawings. The material types required for completion of the Works are labelled as:

    a. Sand and Gravel

    b. Riprap

    c. Boulders

    d. General Fill

    e. Core Material

2. All construction material shall be non acid generating, free of organic matter or similar impurities.

6.2 Part 2 – Product

6.2.1 General

1. Borrow area(s):

    a. The Contractor is responsible for borrow development and operations in the designated borrow area(s). The Contractor shall submit borrow development plans, procedures, operations, surface water and sediment management to Contract Manager for approval for specific borrow area development.

    b. Fill, required for the Works, shall be obtained from the designed borrow area as shown on the Drawings.

c. Unsuitable material from the borrow area shall be disposed of or stockpiled on site in a designated onsite disposal or stockpile area as specified in this Specification. All topsoil or organic material shall be stockpiled.

d. If the Contractor proposes to obtain any fill from an area not within the excavation or designated area shown on the Drawings, he shall communicate his intention to the Contract Manager. Then the Contractor shall obtain the necessary approval and permits to carry out sub-surface investigations and obtain samples to enable the Engineer to assess the suitability of the material for the Works.

e. The Contractor shall give the Contract Manager no less than 28 days notice of his intention to develop any potential borrow area(s) not shown on the Drawings.

f. The Contractor shall make his own determination of the adequacy of any borrow source he intends to exploit.

6.2.2 Sand and Gravel

1. The sand and gravel is a granular material obtained from approved borrow area(s) that is free of organics, well graded and heterogeneous with a grain size distribution that meets the Specifications.

2. Sand and gravel will be used for road surfacing and bedding for liner deployment. The material may require sorting and screening to meet specifications.

6.2.3 Riprap

1. Riprap material is a hard sub-angular to rounded durable rock material that is homogeneous and free from deleterious material, obtained from approved borrow area(s) with a grain size distribution that meets the Specifications.

2. Riprap will be used as channel armouring surfaces as erosion protection and on the upstream and downstream faces of the Diversion headworks embankment.

6.2.4 Boulders

1. Boulder material is a hard angular durable rock material that is homogeneous and free from deleterious material, obtained from an approved borrow area(s) with an open grade grain size distribution and overall nominal dimension of 1.3m as shown in the Specification.

2. These anchor boulders will be used as erosion protection and as the primary means of energy dissipation in the stepped chute section of the diversion.

6.2.5 General Fill

1. General fill material is a granular material that is obtained from either borrow area(s) or from works excavation that is free of organics, well graded and heterogeneous with a grain size distribution that meets the Specification.

2. General fill will be use to construct access roads along the channel realignment.

6.2.6 Core Material

1. Core material is a fine grain material that is obtained from designated borrow area(s) that is free of organics, well graded and heterogeneous with a grain size distribution that meets the Specification.

2. Core material will be used in the dam embankment and erodible plug at the head works.

END OF SECTION 6

7 Materials

7.1 Part 1 – Geosynthetics Specifications

7.1.1 Documents

1. This section of the Specification along with manufacturer's technical documents form part of the Contract Documents and is to be read, interpreted and coordinate with all other parts.

7.1.2 Description

1. The Works to be done under this section consists of furnishing all labour, material and equipment and the performance of all Works necessary to carry out Geosynthetics installation as shown on the Drawings and as specified herein, which will include, but is not limited to the following:
   a. Install HDPE pipe underneath the diversion access road for runoff drainage.
   b. Install a 60mil thick textured LLDPE liner on a section of the new channel and in the head works embankment.
   c. Install geotextile on a section of the new channel and in the headworks embankment.

7.1.3 Products

Submittals

1. The Contractor will submit the following information at least 28 days prior to material arrival at site:
   a. Manufacturer's written certification that the materials to be used meet the Specifications and have been continuously inspected.
   b. All manufacturer's in house quality control and assurance certification on material testing.
   c. Manufacturer's installation and specification documents.
2. The Contractor will submit an as-built report on all Geosynthetics installation showing the specified information/data herein, which will include, but is not limited to the following:
   a. Manufacturer's batch number associated with each panel and pipe installed.
   b. All quality control test done on during the installation.
   c. A certification of completion and warranty of the installation.

Manufacturer Quality Control and Assurance

1. The Contractor is responsible to ensure that all Geosynthetics materials delivered to site meet the Specifications.

2. Geosynthetics that do not meet the Specification will be rejected. The Contractor will replace any rejected material with new material that meets the Specifications.

3. The Contractor must ensure that the geosythetics installations are carried out by a suitably qualified and experience team or subcontractor.

4. Delivery, storage and handling:

    a. Supply geosynthetics in rolls or bundle with straps for unloading.

    b. Supply geosynthetics marked or tagged with the following information:

        i. Manufacturer's name;

        ii. Product information;

        iii. Roll/Pipe serial number;

        iv. Batch or lot number; and

        v. Roll/Pipe dimensions.

    c. Ensure Geosynthetics are handled with care to prevent damages during transit and handling.

    d. Protect geosynthetics from excessive cold, heat, puncture, cutting, or other damaging or deleterious conditions.

    e. Ensure personnel responsible for loading, transport and unloading of Geosynthetics are familiar with the handling and transport constraints imposed by the manufacturer.

5. Acceptance at Work site:

    a. Engineer may perform inventory and surface inspection for defects and damages of Geosynthetics upon delivery.

    b. The Contractor will repair damages resulting from handling and transport of Geosynthetics. If irreparable, in the opinion of the Engineer, the Contractor will replace the damaged materials.

6. Storage and protection

    a. Prepare storage area so that the Geosynthetics products are stored off the ground and protected from the elements and damages.

    b. Preserve integrity and readability of the Geosynthetics labels and store in a fashion that the Engineer have access to the package slips or labels for each product for verifications and acceptance.

7. Subgrade preparation

    a. Prior to liner deployment, all subgrade surfaces shall be compacted when specified, and free of sharp protruding debris. All subgrade must be approved by the Engineer prior to Geosynthetics deployment.

8. The Contractor shall supply all testing technicians and equipment required for the Quality Control and Assurance Program.

9. The Contractor or subcontractor's testing technicians shall be responsible for all quality control protocol such as: panel labelling, destructive testing, repair labelling, control inspections and record keeping.

Product Specifications

1. The Textured Linear Low Density Polythylene (LLDPE) geomemeber shall satisfy the Specification as listed in Table 7.1.

Table 7.1: Texture Linear Low Density Polythylene Specification (GSE UltraFlex Textured or equivalent)

Parameter	Standard	Value
Thickness	ASTM D5994	1.5mm
Density	ASTM D1505	0.92 g/cm3
Grab Tensile	ASTM D6693, Type IV	29 N/mm
Elongation	ASTM D6693, Type IV	500%
Tear	ASTM D1004	169 N
Puncture	ASTM D4833	422 N
Carbon Black Content	ASTM D1603	2.0-3.0 %
Asperity Height	ASTM D7466	0.45 mm

2. The Geotextile shall satisfy the Specification as listed in Table 7.2.

Table 7.2: Geotextile Specification (GSE NW16 or equivalent)

Parameter	Standard	Value
Grab Tensile	ASTM D4632	1.735kN
Elongation	ASTM D4632	50%
Tear	ASTM D4533	665 N
Puncture	ASTM D4833	1055 N
AOS	ASTM D4751	150 microns
Permittivity	ASTM D4491	0.6 sec-1
Water Flow	ASTM D4491	1,830 l/min/m2
Weight	ASTM D5261	540 g/m2 (Nominal)
Thickness	ASTM D5199	3.0mm (Nominal)
UV (500hrs)	ASTM D4355	70%

3. The HDPE pipe shall satisfy the Specification as listed on Table 7.3.

Table 7.3: HDPE Pipe Specification (Sclairpipe 150mm DR32.5)

Parameter	Standard	Value
Average Inside Diameter	ASTM F714	157mm
Average Outside Diameter	ASTM F714	168mm
Minimum Wall Thickness	ASTM F714	5.182mm
Average Weight	PPI's TR7	2.68kg/m
Maximum Continuous Operating Pressure	ASTM D3350	344.7kPa
Carbon Black Content	ASTM D1603	2%

7.1.4 LLDPE Installation

Installation

1. The Contractor is responsible for preparing and maintaining a subgrade surface that is compacted, smooth and free of all rocks, sticks, roots, sharp objects, or debris of any kind. High contrast undulation should be filled in to create a general even surface.

2. The Contractor shall have sufficient amount of ballast weights (ie. sand bags) during the deployment to keep the liner in place.

3. An anchor shall be excavated at dimensions of 1m by 1m to secure the liner limits or as directed by Engineer or as recommended by the manufacturer.

4. The liner shall be seamed according to the manufacturer's recommendations and guidance, and as directed by the Engineer.

5. The liner shall be unrolled as smoothly as possible on the prepared subgrade in the longitudinal direction parallel to the slope.

6. Seams of between panels shall be done with fusion or extrusion welding with manufacturer recommended equipment and supplies.

   a. Weld rods for extrusion welding shall be the same resin from the same batch of the liner.

7. Horizontal seams shall have a shingle type overlap toward the down slope direction. Transversal seam shall be avoided 5m uphill pass the toe a slope.

Field Quality Control and Assurance

1. The Contractor is responsible for field destructive testing including but not limited to: manual peel tests, Tensometer Peel tests, and tensometer tensile tests.

2. The Contractor is responsible for field non-destructive testing such as but not limited to: visual inspection, air test on fusion welded seams, and vacuum box test on extrusion welded seams.

   a. Visual inspection is done to identify any flaws, imperfections, damages, and other variances on liner that could compromise the liner installation.

3. The Engineer will observe all installation, field destructive and non-destructive tests and will sign off approved air tests.

4. As a minimum, the testing program on the compacted fill shown on Table 7.3. The Engineer will at his discretion, carry out tests on areas where listed installation quantities are not met.

Table 7.3: Liner Field Testing Schedule

Test	Frequency	Value
Manual Peel Test	End of each fusion seam	N/A
Tensometer Peel Test	1 per 250m	40 N
Tensometer Tear Test	1 per 250m	147 N
Air Test	1 per fusion seam	200 kPa for 3 minutes
Vacuum Box Test	All extrusion seams	N/A

5. Engineer will sign an acceptance for qualified QA/QC protocols on the installed liner.

6. The Contractor will take all precautions to prevent foreseeable damages on installed and approved liner and will be responsible for damage repair when such preventable incident occurs.

7.1.5 Geotextile Installation

Installation

1. The Contractor shall have sufficient amount of ballast weights (ie. sand bags) during the deployment to keep the textile in place.

2. The geotextile shall be unrolled as smoothly as possible on the prepared subgrade in the longitudinal direction parallel to the slope.

3. Horizontal seams shall have a shingle type overlap toward the down slope direction.

4. The geotextile shall have 300mm minimum overlap and stitched or heat bonded together. The Engineer will inspect the stitching or heat bonding to ensure quality of Work. If the seams come undone during fill placement, the Contractor must repair the aperture prior continuing the fill placement. The Engineer will determine the seaming method, rather sewing or heat bonding, on site.

5. Damaged geotextile, as identified by the Engineer, shall be repair immediately. The damaged area plus an additional 1m around the damage area shall be clear of all fill material. A geotextile patch extending 1m beyond the perimeter of the damage shall be installed as directed by the Engineer.

Field Quality Control and Assurance

1. The Quality Control Personnel shall visually inspect all geotextile and monitor the fill placement procedure to ensure no damage is done or no aperture in geotextile.

2. The Contractor must acquire Engineer's approval prior to fill placement.

7.2 Part 2 – Concrete and Steel

7.2.1 Documents

1. This section of the Specification along with manufacturer's technical documents form part of the Contract Documents and is to be read, interpreted and coordinate with all other parts.

7.2.2 Description

a. The Works to be carried out under this section consists of furnishing all labour, material and equipment and the performance of all Works necessary to carry out the installation of steel pipe posts infilled with concrete located immediately downstream of each of the keystone boulder steps as shown on the Drawings and as specified herein.

7.2.3 Products

Submittals

1. The Contractor will submit the following information at least 28 days prior to material arrival at site:
    a. The manufacturer specification for the steel pipes.
    b. The manufacturer specification for the concrete mixture.

Product Specifications

1. The steel posts shall be an 88.9mm (4-inch) OD schedule 40 steel pipe.
2. The concrete shall have a 28 days breaking strength of 34.5 MPa with maximum 12.7 mm aggregate and quick set cement.

7.2.4 Concrete Mixture

1. The contractor shall provide the specified concrete meeting the Specifications ready for use on site according to the construction progress.

END OF SECTION 7

8 Fill Placement

8.1 Part 1 – General

8.1.1 Documents

1. This section of the Specification forms part of the Contract Document and is to be read interpreted and coordinated with all other parts.

8.1.2 Description

1. The Works specified in this section includes furnishing all supervision, labour, materials, tools and equipment for placement of fill material to the lines and grades shown on the drawings and specified herein.
2. The Work shall include, but is not limited to the following:
    a. Foundation preparation to receive the fill.
    b. The supply, hauling, placing, and compacting of the specified fill material as shown on the Drawings.
    c. All related surveys for layout and control of the Works.
    d. Assist and provide the Engineer with QA/QC testing and results.
    e. Maintenance of haul roads, when and where applicable.
    f. The development, maintenance and restoration of fill material borrow area(s).
    g. Any other related Works not covered elsewhere.
3. Fill material required to be placed include, but are not limited to the following:
    a. Haul, place and compact general fill as foundation for access roads.
    b. Haul, place and compact sand and gravel on access road surface as road and dam surface caps.
    c. Haul, place and compact general fill material as the primary embankment dam material.
    d. Haul and place riprap along realigned channel, dam, erosion plug and step pools.
    e. Haul and place boulders in stepped chute section.

f. Haul, place and compact core material in the headworks embankment and in the erodible plug within the embankment.

8.1.3 Codes and Standards

1. The Quality Control and Assurance Program (QA/QC) shall use testing procedure from, but not limited to the list of American Society of Testing and Material Standard in Table 8.1.

Table 8.1: List of QA/QC Testing Standards

Test	Protocol
ASTM D2216	Water (Moisture) Content in Soil and Rock
ASTM D422	Particle Size Analysis of Soils
ASTM D698, Procedure A, B or C	Laboratory Compaction Characteristics of Soil Using Standard Effort (Standard Proctor Test)
ASTM D2922	Density of Soil in Place by Nuclear Methods

8.1.4 Submittals

1. The Contractor shall submit all QC data and document at the end of project and upon request by the Engineer.

2. Testing responsibilities

   a. Quality Control testing will be done by the Contractor.

   b. Quality Assurance observation and testing will be done by the Engineer.

   c. The Engineer's testing shall not relieve the Contractor of his sole responsibility to construct the Works in accordance with specified requirements.

8.2 Part 2 – Execution

8.2.1 Compaction Equipment

1. The compaction equipment shall be the appropriate size and type to achieve the specified densities of respective fill materials.

2. Where compaction procedure (lift thickness, number of passes, compactor type) is specified, the Contractor shall ensure the work done meet or exceed those described in the Specification.

3. A vibrator plate jumper jack temper will be required for compaction on narrow filter material. The hand compactor shall be rated to provide sufficient pressure to meet compaction requirements.

4. A sheep foot/padded foot vibratory compactor is recommended in addition the smooth drum compactor. The sheep foot will be for compacting the core material, silty/clayey general fill whilst the smooth drum will be for sealing the surface to provide a smooth surface for water shed and workable surface.

5. Notwithstanding the requirements stated above, the equipment and compaction procedures employed by the Contractor shall be subject to approval from the Engineer.

8.2.2 Fill Placement

1. The Contractor shall prepare an acceptable foundation surface to receive the specified fill material. An acceptable foundation surface is a clean, sound, firm and does not contain any snow, ice, organic, loose, softened or disturbed material as determined by the Engineer.

2. Fill shall not be placed on the prepared foundations until they have been inspected and approved by the Engineer.

3. The Contractor shall dump, spread and level fill in such a manner as to avoid multiple work zone and crossing traffic pattern.

4. The direction of fill placement and construction equipment traffic shall be parallel to the long axis of the structure being built, or as directed by the Engineer.

5. The compaction operation for fill shall be conducted within the same work day to provide a smooth compacted surface and meet the density requirement shown in Table 8.2. Adjacent individual passes of the compactor shall overlap by approximately 1/3 of the width of the compactor's drum. New fill shall be "keyed" into the existing approved fill. Keying in is by placing new fill adjacent to exposed compacted fill. The Contractor is responsible to repair all damages on unfinished work from previous work day. Moisture conditioning might be required on the granular type materials to achieve optimum moisture conditions.

6. Any placed material, which does not meet the specified requirement, shall be reworked to produce a material which does satisfy the specified requirement, or shall be remove and disposed of accordingly.

7. Construction material maximum lift thickness and compaction requirement shall be as indicated in Table 8.2.

Table 8.2: Compaction Requirements

Fill Description	Maximum Lift Thickness Before Compaction (mm)	Minimum Density % of the Standard Proctor Maximum Dry Density	Placed Consolidated Density[1] Tonne/m^3	Minimum Passes/Lift & Compactor Type[2]	Construction Tolerance
Sand and gravel	300	95	1.8	5, 15t vibratory equivalent	±30mm
General fill	500	95	2.0	5, 15t vibratory equivalent	±50mm
Core Material	300	95	2.1	5, 15t vibratory sheep foot or equivalent	±15mm

1. Density herein is assumed. Field tests will confirm the actual densities from borrow material.
2. Compaction effort might be adjusted by field compaction trial result. The Engineer will determine on site if the compaction specification needs to be adjusted to reflect the results.

8.2.3 Tolerances

1. Fill shall be place in horizontal lifts to the lines and levels shown on the Drawings or as directed by the Engineer, and to the tolerances as shown in Table 8.2, in elevation and horizontal dimension determined by survey.

8.2.4 Compaction Trials

1. Compaction trials shall be performed upon production of fill material to determine site specific parameter such as density and compaction standards. The trial shall be carried out as part of the fill placing operation.

2. The Engineer may request the Contractor to periodically conduct field trials to optimize moisture conditioning, lift thickness and compaction effort.

3. The compaction trials on the material in question shall be done using a survey method according to the general procedure listed below, and as specified by the Engineer.

   a. A pad made with approved material in approximately 7 m by 20 m with specified thickness associated with specified material with placement method according to this Document.

   b. A set of survey points with accuracy of ±5mm shall be laid out as specified by the Engineer in a grid pattern.

   c. The elevation of each survey points shall be recorded immediately after placement and after each compaction effort.

   d. Compaction is to be done upwards of 10 passes in accordance with this Document or otherwise specified by the Engineer. Survey will be done after each pass.

   c. This process shall be repeated to simulate construction as directed by the Engineer.

4. The Contractor shall obtain Engineer's approval before implementing any change to the Specifications.

8.2.5 Restrictions due to Weather and Suspension of Operations

1. The Contractor shall not place any fill when condition for such operation are unsatisfactory due to snow, freezing condition, heavy rainfall, or any other reason determined by the Engineer.

2. Where operation have been discontinued by the Contractor or suspended by the Engineer, the effects of adverse conditions shall be assessed by the Engineer and the surficial layer of fill reworked or replaced to the satisfaction of the Engineer before resumption of fill placement.

3. Before suspension of operation each day, or each construction shift, as described in this section, and before suspension due to inclement weather, the till material shall be:

   a. Surface shaped to drain excess water.

   b. Rolled smooth to seal against water infiltration.

   c. Clear snow between material placement during heavy snow pending on haul rotations.

   d. The Engineer will examine the quality of surficial fill to determine if rework is required to meet Specifications.

8.2.6 Sediment and Runoff Control

1. The Contractor shall provide construction facilities such as diversion berms, ditches, sediment control measures, and other measures to prevent the release of fines from the construction areas and to prevent these fines from entering any natural water courses downstream of the Works. The Engineer will review the measures and notify the Contractor if the measures are inadequate.

2. In general, when placing fill material, the Contractor shall slope the surface toward collection channels for surface water management. The Contractor is responsible to ensure downstream work area(s) will not be affected or damaged by runoff water.

8.2.7 Quality Control and Assurance

1. The Quality Control and Assurance team will conduct testing according to Table 8.3 or as specified by the Engineer.

2. The Contractor shall performer regular quality tests to ensure the quality of the work is done according to the Specifications.

3. Testing shall be performed in accordance with the principles and methods prescribed by the American Society for Testing and Materials (ASTM) and other such recognized authorities.

4. Testing shall be carried out across the full length, width and depth of the various fill zones so as to fully represent the overall quality of the structure.

5. The Contractor shall conduct regular topographic surveys to demonstrate the placement of fill to the specified lines, levels, grades and tolerances. The Engineer may from time to time conduct check surveys. Survey results shall be reported to the Engineer within 24 hours of the completion of each survey.

6. As a minimum, the testing program on the compacted fill shown on Table 8.3. The Engineer will at his discretion, carry out tests on areas where listed placement volume are not met.

Table 8.3: Testing Schedule

Fill	Tests and Frequency (1 test per Vol. in m^3)(1)				
Description	**Moisture Content**	**In-situ Density**	**Gradation**	**Standard Proctor**	**Durability**
Sand and gravel	400	100	400	400	N/A
Riprap	N/A	N/A	5000	N/A	Min. 1 per borrow
Boulders	N/A	N/A	Min. 1	N/A	Min. 1 per borrow
General fill	4000	500	4000	4000	N/A
Core Material	Min. 1	Min. 10	Min. 1	Min. 1	N/A

1. Volume is measure in placed volume.

8.2.8 Acceptance

1. Final acceptance of earthworks will be made only after fill materials have been dumped, spread, moisture conditioned, and compacted, and tests and surveys have demonstrated compliance with the Specifications.

2. If on the basis of the sampling and testing, or if in the opinion of the Engineer, an area of the fill does not meet the specified requirements, such fill shall be removed and replaced with conforming material. Rejection of fill material by the Engineer maybe made at source, in transporting vehicles, or in place.

3. The Engineer will re-inspect previously approved areas for damages and instruct the Contractor to repair said damages in accordance with the Specifications.

END OF SECTION 8

9 Construction

9.1 Part 1 – General

9.1.1 Documents

1. This section of the Specifications forms part of the Contract Documents and is to be read, interpreted and coordinated with all other parts.

9.1.2 Description

1. The Works to be carried out under this Section consists of performing all Works necessary to carry out the Works as shown on the Drawings and specified herein. The Works shall include, but is not limited to:
    a. Access Roads Construction.
    b. Realigned Channel Excavation and Construction.
    c. Stepped Chute Excavation and Construction.
    d. Breaching of the Blind Creek road.
    e. Headworks Embankment and Erodible Plug Construction.

9.2 Part 2 – Access Roads Construction

9.2.1 Construction

1. The access roads shall be constructed of general fill and alignment according to the Drawings, Specifications, and as directed by the Engineer. The general fill will be from materials that meet the Specifications and are either excavated from the slope and channel alignment.

2. The road surface will be capped with 0.3m thick of sand and gravel as traffic wear layer.

3. The side slope of the access road will be minimum two horizontal to one vertical (2H:1V) where soil material (overburden/topsoil) is exposed and minimum one and a half horizontal to one to one vertical (1.5H:1V).

4. The surfaces of the access roads will be grade minimum 0.5% as directed on the Drawings to shed water.

5. A minimum 1m wide and 0.5 m deep runoff ditch will be constructed along the upstream access road with specified herein HDPE pipes to route the runoff into the channel. The pipes shall have a minimum 0.5% grade for drainage and spaced at approximately 100 m along the access road in low spots.

6. Flow in each of the culverts shall be directed into half-round CMP culverts constructed down the channel slopes to the base of the channel.

9.3 Part 3 – New Channel Excavation and Construction

9.3.1 Excavation

1. The excavation of the new Vangorda Creek diversion channel to the base level (11% grade line), shall be carried out in accordance with the lines and grades shown on the Drawings and as directed by the Engineer.

2. Soil and weathered rock excavation shall be carried out in accordance with the Soil Excavation section herein. Where competent bedrock is encountered, excavation shall be in accordance with the Drilling and Blasting section herein.

3. The Contractor will be responsible for backfilling or remediating over excavation and breakage as deemed by the survey or the Engineer.

9.3.2 Construction

1. The construction of the realigned channel shall be carried out in accordance with the Drawings and as directed by the Engineer.

2. Riprap erosion protection shall be placed by equipment to the depth and stone size specified. It shall be installed to the full course thickness in one operation and in such a manner as to avoid serious displacement of the underlying material. The rock for riprap shall be delivered and placed in a manner that ensures the riprap in place is reasonably homogeneous with the larger rocks uniformly distributed and firmly in contact one to another with the smaller rocks and spalls filling the voids between the larger rocks. Some hand placing may be required to provide a neat and uniform surface.

3. Addition work procedures will be required in the following conditions:

    a. A layer of geotextile shall be placed prior to riprap installation along the channel surface where natural dense till is exposed.

    b. LLDPE and geotextile shall be place along the channel surface where sandy or silty material is encountered. The LLDPE shall be installed according to the specification herein or as directed by the manufacturer. A layer geotextile shall be deployed on top of the LLDPE as a cushioning layer prior to riprap placement. The Contractor will be responsible to repair any damage on the liners cause by ongoing construction.

    c. Both the LLDPE and geotextile have to extend minimum 5 m pass silty sand section into bedrock or natural dense competent till as directed by the Engineer.

9.4 Part 4 – Stepped Chute Excavation and Construction

9.4.1 Excavation

1. The excavation of the stepped chute section shall be carried out in accordance with the Drawings and as directed by the Engineer.

2. Soil and weathered rock excavation shall be carried out in accordance with the Soil Excavation section herein. Where competent bedrock is encountered, the material shall be removed in accordance with the Drilling and Blasting Specification section herein.

3. The Contractor will be responsible for backfilling or remediating over excavation and breakage as deemed by the survey and Engineer.

4. The over-excavation for the steped chute and the boulder seating shall be carried out with due care to maintain the overall design slope gradient of 11% between steps.

9.4.2 Construction

1. The construction of the stepped chute shall be carried out in accordance with the Drawings and as directed by the Engineer.

2. Steel post barriers shall be installed immediately downstream of each of the boulder at the steps, at the approximate locations as shown on the drawings. Each pipe shall be set in a drilled hole to a minimum depth of 3 m in the weathered bedrock and to a min depth of 5m in the overburden. Each pipe shall be spaced no more than 1 m across the base width of the channel. Concrete shall be placed in each pipe to the top of the pipe as directed by the Engineer.

3. Riprap erosion protection shall be placed in overburden sections to the depth and stone size specified. It shall be installed to the full course thickness in one operation and in such a manner as to avoid serious displacement of the underlying material. The rock for riprap shall be delivered and placed in a manner that ensures the riprap in place is reasonably homogeneous with the larger rocks uniformly distributed and firmly in contact one to another with the smaller rocks and spalls filling the voids between the larger rocks. Some hand placing may be required to provide a neat and uniform surface.

4. In accordance with the Drawings and as directed by the Engineer, riprap erosion protection placed during the stepped chute construction shall be grouted or mortored in place to adjoin the riprap particles.

9.5 Part 5 – Blind Creek Road

9.5.1 Excavation

1. The excavation of the new diversion channel will require cutting off the existing Blind Creek access road. Breaching of the road will be carried out during the channel excavation. Re-establishing the access road after the channel has been constructed using a culvert(s) will be decided prior to construction start. Contractor shall include a price to install the culvert and re-establish road access in the bid document.

9.6 Part 6 – Head Works Dam and Erodible Plug Construction

9.6.1 Construction

1. The headworks embankment including the erodible plug shall be constructed in accordance with the Drawings and as directed by the Engineer.
2. The embankment foundation and base of the erodible plug shall be prepared in accordance with Specifications herein and as directed by the Engineer.
3. General Fill, Sand and Gravel, Core Material and Riprap shall be placed in accordance with the Specifications herein.
4. The base of the erodible plug shall consist of two layers of geotextile and one layer of LLDPE. This combination of synthetic material shall extend over the entire area of the embankment, extending a minimum of 3 m below grade at the upstream toe of the embankment as shown on the drawings and as directed by the Engineer.
5. Contractor will be responsible for any damage done to the liners when placing and compacting materials for erodible plug construction.
6. The erodible plug shall consist of sand and gravel and an impervious till core placed over the synthetic liner.
7. The bulk of the embankment shall consist of compacted general fill obtained from the channel excavation or sourced from quarry material.
8. Riprap erosion protection shall be placed by equipment on the downstream and upstream faces of the embankment to the depth and stone size specified. It shall be installed to the full course thickness in one operation and in such a manner as to avoid serious displacement of the underlying material. The rock for riprap shall be delivered and placed in a manner that ensures the riprap in place is reasonably homogeneous with the larger rocks uniformly distributed and firmly in contact one to another with the smaller rocks and spalls filling the voids between the larger rocks. Some hand placing may be required to provide a neat and uniform surface.

END OF SECTION 9

These document "**Vangorda Creek Diversion - Phase 1, Technical Specifications – Draft Rev B**", was prepared by SRK Consulting (Canada) Inc.

Prepared by

Alvin Tong, EIT
Consultant

Reviewed by

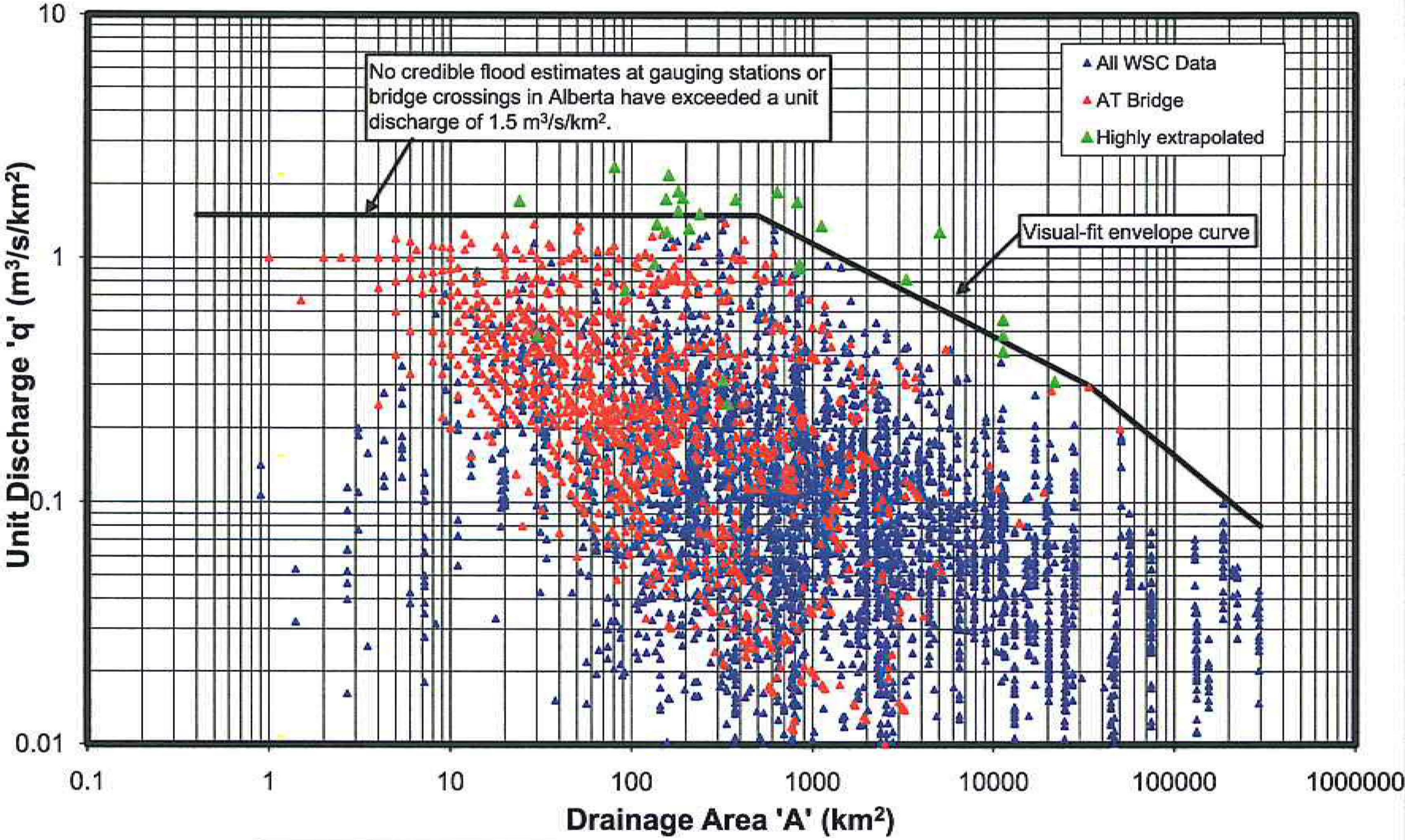
Peter Healey, P.Eng.
Principal

10 References

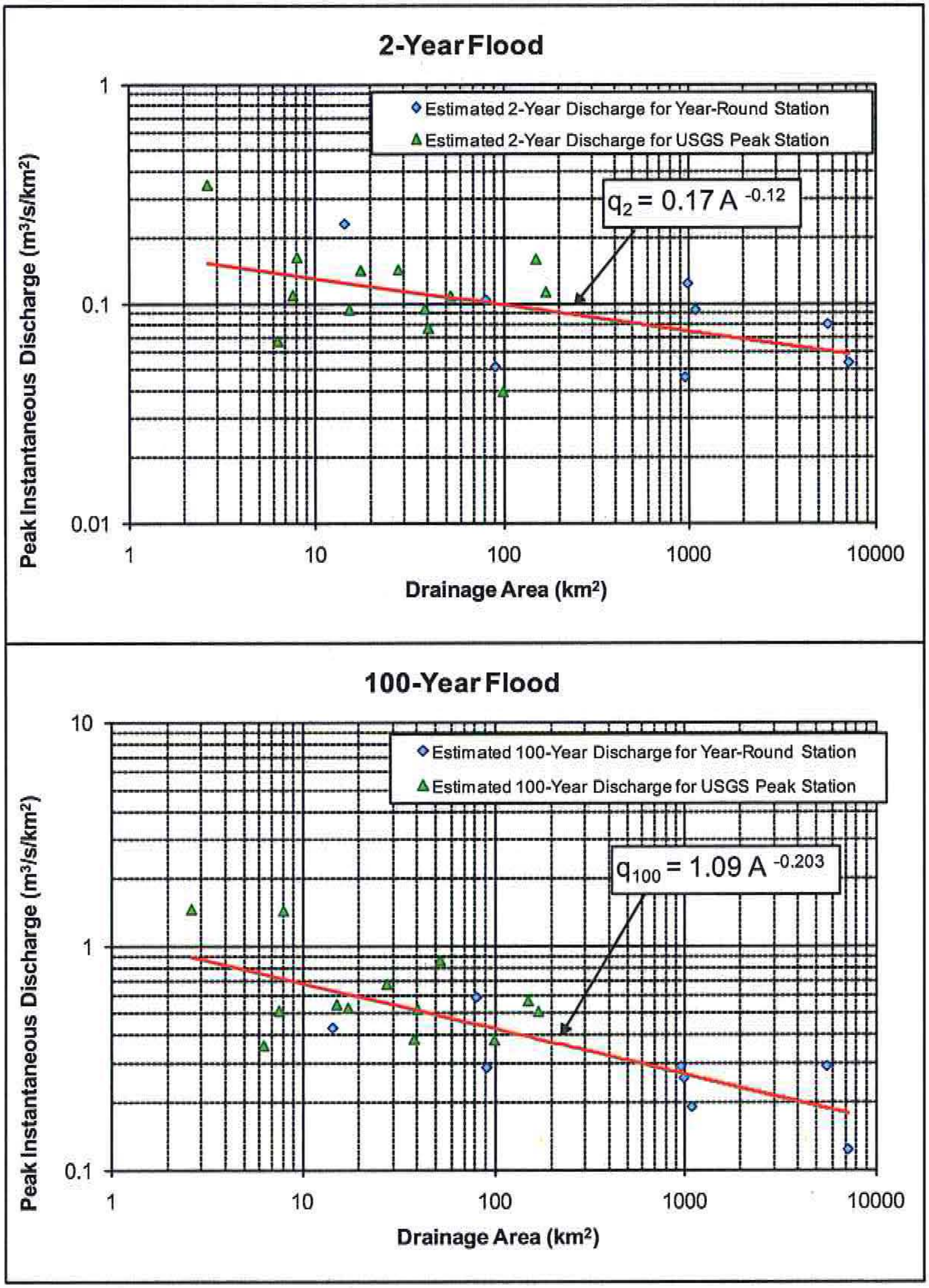
SRK (2010b). Draft Design Report, Vangorda Creek Diversion, Vangorda Plateau Development. Report prepared for Yukon Government. February, 2010.

SRK (2009b). Draft Field Investigation Report, Vangorda Creek Diversion, Vangorda Plateau Development. Report prepared for Yukon Government. September, 2009.

Appendix V
Hydrological Analyses



SRK Consulting Engineers and Scientists VANCOUVER	Faro Mine Closure Fermeture de la mine Faro	Vangorda Creek Diversion **Government of Alberta** **Flood Analysis Study**
Job No: 1CY001.031 Filename: Gov't of Alberta Flood Analysis Fig_2009.12.ppt	Faro Mine Complex Final Closure and Remediation Plan	Date: February 2010 Approved: PB Figure: **Appx V**



SRK Consulting
Engineers and Scientists
VANCOUVER

Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion
Alternatives Analysis

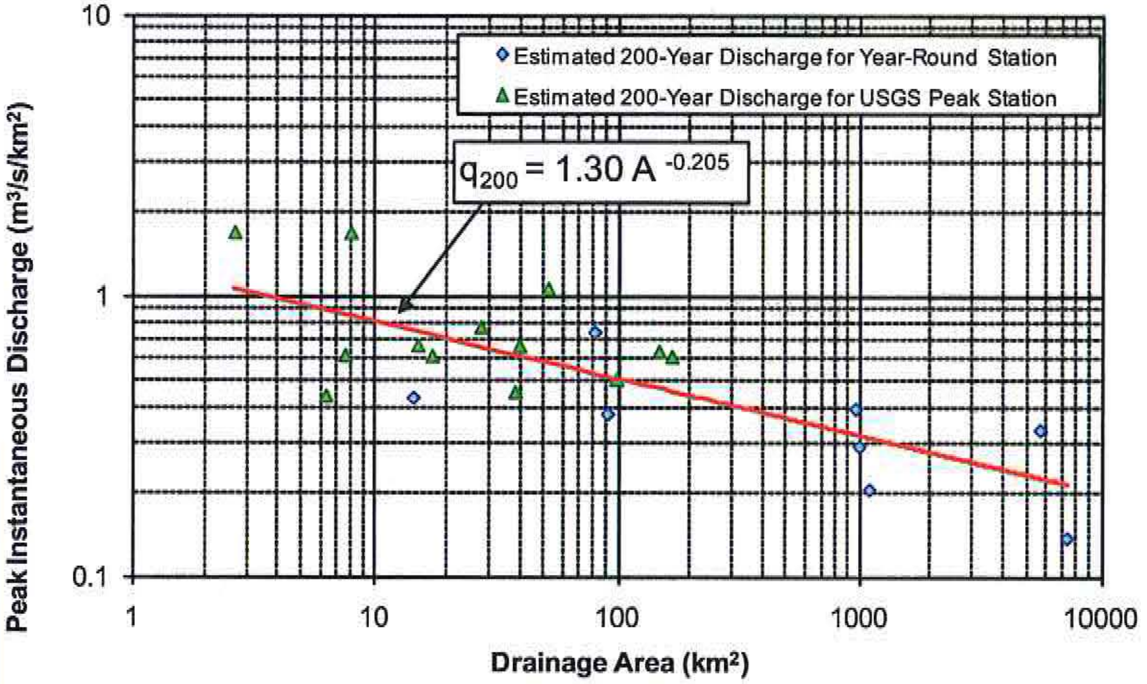
Regional Relationships for Estimating Peak Instantaneous Discharges of 2-Year and 100-Year Floods

Job No: 1CY001.031
Filename: Figures_5_6_Floods_20091216.ppt

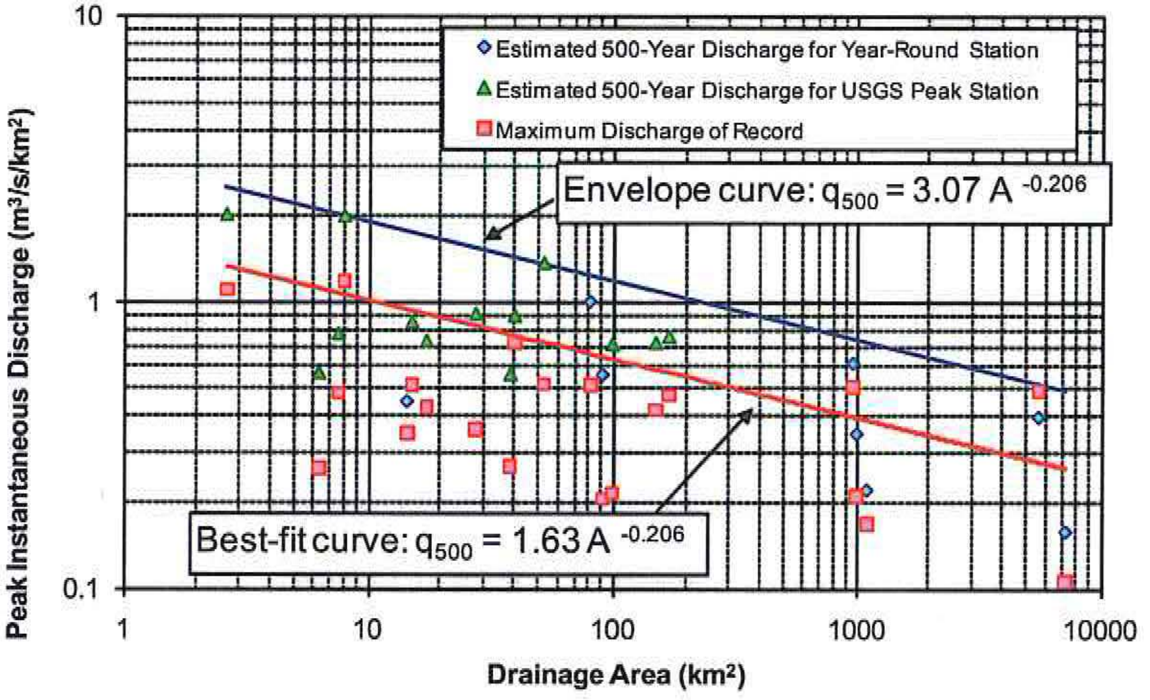
Faro Mine Complex
Final Closure and Remediation Plan

Date:	Approved:	Figure:
December 2009		5

200-Year Flood



500-Year Flood



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Faro Mine Closure
Fermeture de la mine Faro

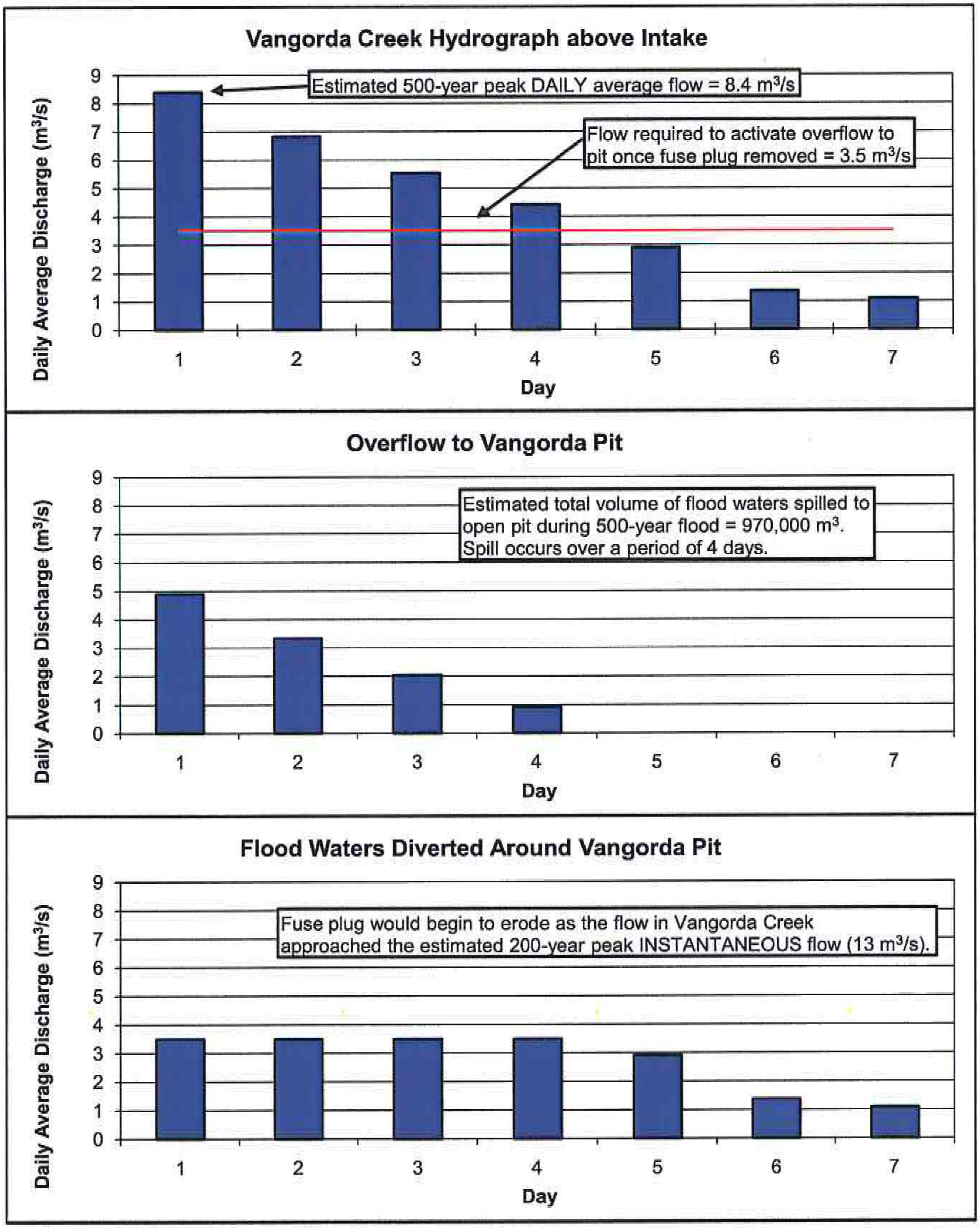
Vangorda Creek Diversion
Alternatives Analysis

Regional Relationships for Estimating Peak Instantaneous Discharges of 200-Year and 500-Year Floods

Job No:	1CY001.031
Filename:	Figures_5_6_Floods_20091216.ppt

Faro Mine Complex
Final Closure and Remediation Plan

Date:	Approved:	Figure:
December 2009		6



Vangorda Creek Hydrograph above Intake

Estimated 500-year peak DAILY average flow = 8.4 m³/s

Flow required to activate overflow to pit once fuse plug removed = 3.5 m³/s

Overflow to Vangorda Pit

Estimated total volume of flood waters spilled to open pit during 500-year flood = 970,000 m³. Spill occurs over a period of 4 days.

Flood Waters Diverted Around Vangorda Pit

Fuse plug would begin to erode as the flow in Vangorda Creek approached the estimated 200-year peak INSTANTANEOUS flow (13 m³/s).

Figure 7: Estimated Behaviour of Headworks during Highest Seven Days of 500-Year Flood

Appendix VII

Step-Pool Review (NHC)

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memorandum

To:	Peter Healey, P.Eng. SRK Consulting (Canada) Inc.	**Date:**	01-Oct-2009
From:	Dr. N. Rajaratnam, P. Eng. E.K. Yaremko, P. Eng.	**No. Pages:**	
		Project No.:	17384
		Ref. No.:	6266

Re: **Vangorda Creek Diversion**
Review Comments on Section 3.4:
Energy Dissipation Utilizing A Step-Pools Design

1 BACKGROUND AND GENERAL COMMENTS

The Vangorda Creek Diversion is designed for a discharge of 15 m^3/s. In this open channel diversion, the bed slope of the diversion channel is equal to 1.5% in the early portion (upstream part); increases to 5.22% in the middle reach and has a steep reach of length of about 660 m with a slope of 13.00%. It is this steep reach which is of primary concern in this letter report. In the early reach, for the cross-section shown in Fig. 6 (of the SRK Consulting Sept 09 Report) for section B, the mean velocity V is about 2.9 m/s with a depth of about 1.2 m (our calculations). We estimated the Manning n by the equation in Chow: Open channel Hydraulics (1959) & Bray (1991) as:

$$n = 0.047(d_{50})^{1/6}$$

where d_{50} is the 50% finer size.

For Section C, V=2.49 m/s with a depth of 1.14 m. For Section F, with a slope of 5.2%, V=3.3 m/s with a depth of 0.63 m.

For Section G with a slope of 13.0% and a bed width of 6 m, V=2.16 m/s with a depth of 0.47 m.

For the steep slope reach, a stepped chute, cut into bed rock, would have been an ideal solution but this idea appears to have been abandoned because of the erodibility of the bed rock. The option of constructing concrete steps was apparently not considered. Perhaps this option was deemed to be impractical or too expensive for this site.

We have reviewed the adopted design approach of using a series of step-pools in this steep reach, which has been selected as the preferred option.

2 STEP-POOLS

From the literature in the areas of Geomorphology, Geography, and Stream remediation (Church & Zimmerman 2009; Chin & Wohl 2005; Comiti et al. 2009), step-pools have been studied in the last 10 years. Step-pools have been observed to exist as a natural flow element in mountain streams. Most of these studies are somewhat qualitative and many essential details like the regime of flow in the pools and the extent of energy dissipation appear to have not been treated in sufficient detail. The length of the pools is estimated by empirical rules (Thomas et al. 2004). It is not known as to whether the flow entering the pool from the step will be of the impinging jet or in the surface flow modes. In the first case, the energy dissipation in the pool will be very effective but the flow impinging on the bed of the pools will be very turbulent. Under these circumstances, the stability of the rocks forming the bed will be uncertain. Perhaps the rocks on the bed should be tied together or one should use interconnected gabions. In the surface jet regime, the stress on the bed will be reduced but at the same time the energy dissipation in the pools might be reduced. These questions can be answered by a Froudian scale model in the Laboratory.

A further difficulty could be the stability of the rocks forming the step. It would be easy to estimate the forces on these rocks as well as the force or moment that will take to topple them, once the type of flow is known. But the resistance of the rocks to this type of forces and moment is difficult to calculate. Further, if there is erosion in the pools, which itself is difficult to estimate in a reliable manner, these rocks could simply tip into these scour holes. Percolation beneath the rocks could encourage the sliding of these rocks. There exists the risk that failure of a single step-pool structure could result in the progressive (cascading) failure of the step system downstream.

Even though a number of studies have been carried out in recent years on erosion below drops and similar structures, our knowledge in this area is still not very satisfactory. Of course, it is also difficult to predict the scour from model studies conducted to study the flow regimes in the pools. Some care must also be used in using a discharge-head relation for the steps, which could be obtained from the physical model studies.

In conclusion, even though the concept of step-pool is a good idea for this project, it appears that there are not enough quantitative engineering studies in the general literature to calculate essential quantities. As a result, we advocate caution in proceeding with the construction of these 86 step-pools. Even a relatively short physical model study could predict the flow regimes that would exist in the pools as well as the energy dissipation. As a minimum, it would be prudent to perform some calculations to estimate the forces on the rocks forming the steps and analyse the stability of these rocks.

This structure supposedly must have long term integrity, at least up to the 200-year design flood level. It is unsure what the design life must be but presumably it could be 100 or more years - there is a 40 percent chance of the design flood occurring in a 100-year period. The secondary concern has to do with progressive damage and lack of proper monitoring and timely maintenance. It seems that this structure must be designed to a higher standard because of this, which is a further argument to be cautious about accepting the proposed design on the basis of available design criteria.

3 MITIGATION OF DESIGN RISKS

Given the general lack of engineering-based criteria for step-pool design, a possible approach to mitigate the level of risk associated with the proposed design might be to provide measures which would address those risks directly. Some ideas which come to mind include:

- concrete grouting of the rock material making up the step and pool bed.
- use galvanized steel cable to join rocks together;
- utilize gabion wire or gabion baskets to secure rock material in the pool area
- utilize larger rock material
- conduct additional hydraulic analysis: principally with regard to potential for flow to move or rotate step rocks; and, potential for pool rock material to become mobilized.

Feel free to ask questions or request additional information. Perhaps a teleconference call meeting could be organized to review the proposed design in relation to our conclusions and recommendations.

Respectively submitted,

northwest hydraulic consultants

Dr. N. Rajaratnam, P. Eng.

E.K. Yaremko, P. Eng.

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Memorandum

To:	Stephen Mead	**Date:**	October 21, 2009
cc:	John Brodie	**From:**	Peter Healey
Subject:	Vangorda Creek Diversion	**Project #:**	1CY001.031

1 Introduction

SRK Consulting (Canada) Inc has prepared this memo as a follow-up to a conference call held on October 5, 2009 to discuss review comments by Northwest Hydraulic Consultants on the draft design report for the proposed Vangorda Creek Diversion. The focus of the call was the proposed approach to energy dissipation utilizing a step-pool design. Participants on the call included Gene Yaremko and Dr. Rajaratnum both from NHC, John Brodie, Pat Bryan (Hydrologist) and Peter Healey.

The NHC memo is attached to this memo.

2 Design Issues

The approach to the proposed realignment of Vangorda Creek Diversion involves three main components: an erodible fuse plug at the headworks of the dam, a diversion channel with a bed slope ranging from 1.5 to 13 percent and a plunge pool at the outlet. The main focus of the NHC review and the meeting on October 5, was the steep (13%) lower reach over the last 500m of the diversion. The proposed approach in the SRK design is a series of step-pools (about 86) which were incorporated into the design to provide energy dissipation. While we agree with NHC that a stepped chute cut into the bedrock would have been an ideal solution, as NHC correctly assumed, the foundation soils at the base excavation in the steep reach is expected to consist of highly erodible bedrock and in part, glacial till. Both these soils will likely require extensive erosion protection. The option to concrete this entire section of the channel was considered but discounted based on the high cost and the short life span of the concrete.

NHC concluded that although they felt the concept of a step pool for energy dissipation is a "good idea" there remains a number of risks associated with the current design. The first concern is the uncertainty of the stability of the rocks forming pools. The flow impinging on the bed of the pools will be very turbulent and there is a risk that during high flow events, the flow could scour out the rocks that make up the bedding of the pools. In addition, there is risk that the large rocks that make up the step could topple under high flow with a combined result of a progressive failure downstream.

3 Conclusions

SRK agrees with NHC in their comment that within the industry there is a general lack of engineering based criteria for step-pool design. Although further studies could be carried out to answer some of the question surrounding the step-pool approach, in the interest of moving ahead with the project given the time constraints, it is SRK's opinion that effort be given to assessing

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Step-Pool Design Memo

To:	Peter Healy, SRK	**Date:**	September 9th, 2009, edited February 8th, 2010
cc:		**From:**	John Kurylo
Subject:	Vangorda Creek Diversion Channel Step-Pool Design Concept	**Project #:**	1CY001.031.002

1 Introduction

SRK Consulting (Canada) Inc has prepared this memo to detail the step-pool design concept originally developed in the early stages of the draft design report for the proposed Vangorda Creek Diversion.

2 Background

Until recently, little was known about the forms and processes of step-pool channels. However, over the past two decades significant advances in the theory of step pool sequences have been made (Chin et al., 2008). In evaluating this option SRK conducted an extensive literary review for a Step-pool design (See reference section of the 2010 Vangorda Creek Diversion Draft Design Report for more details/a detailed list).

3 Step-Pool Design Criteria

To achieve an optimized alignment for the proposed Vangorda Creek Diversion, a large number of arrangement and grade iterations were completed. The volume of excavation, hydraulics, constructability, static and geotechnical stability were incorporated into the assessment. Consistently the most favourable diversion arrangements exhibited a low grade section near the diversion intake then the diversion transitioned to a steeper final grade of 13% (approximately 11% in Phase 1 then originally proposed to be taken to 13% by the end of phase 2).

Conveying the diversion design discharge of 13-15m^3/s down an approximately 660m long channel with a 13% grade requires some form of energy dissipation. The option of a rock lined stepped chute was first adopted as the preferred method of energy dissipation. As shown in Figure 5, only a portion of the steep slope section is expected to be in bedrock while the remainder is expected to be in overburden. Due to the weak friable nature of the phyllitic bedrock encountered during the field investigation, it was determined that steps entirely carved out of the bedrock would be inadequate and that the energy dissipation in the expected overburden sections beyond that of a simple rock lined chute would have to be investigated. To provide a diversion design that would be more statically stable over the long-term, to mimic nature and to satisfy the energy dissipation requirements, a design for step pool morphology was created for the steep section of the diversion.

On milder slopes experiencing uniform flow typically subcritical conditions result (Froude conditions that $F < 1$). In contrast supercritical flow is observed on steep slopes ($F > 1$). If a supercritical flow state was to results throughout the steep section of the diversion, significant channel destabilization would be expected. By utilizing a step pool morphology uniform flow does not result. As described by Church et al. 2006 "oscillation between supercritical and subcritical flows results in large energy losses to spill resistance and it is supposed that this is a key control on channel stability. On steep slopes at higher stages, flow is launched vigorously off the step and a recirculation cell develops in the head of the pool under the drop". In simply terms step pool energy dissipation happens when water readily transitions from pressure head to velocity head. However, water does not as readily convert back into pressure head leading to significantly larger coefficients for expansion losses than those for contraction losses.

The design procedure developed by Thompson, Abt, Mussetter, and Harvey was used as a preliminary base for the design criteria. The design procedure developed by Thompson et al., 2000 documents the morphology of step pools in Colorado both natural and man-made, however the steepest channels utilized in this design procedure only extended to grades on the order of 8%. Additional gemorphological work, most notably the work of Church et al., as well as the work from Lenzi, performed in the Alps on slopes similar those designed for in the Vangorda Creek Diversion, was incorporated to extend the design criteria to a 13% or 7.7H:1V slope. The specific design criteria adopted herein for the step pool design are:

- The step geometry is required to handle and dissipate the energy derived from a 10-15m^3/s discharge on a 13% slope.
- Boulder size (minimum diameter) and arrangement utilized for step construction should be modified to ensure that the steps are functional and statically stable and are resistant to motion during the large flow events.
- Riprap must be present in the pools downstream of the steps to 'armour' the channel against erosion and to limit the effects of scouring.
- To maximize the flow resistance for steep channels the pools are required to fill the entire space between steps. The geomorphologic ratio 1< Height of the Step (H)/ Step Length (L)/ Slope (S) < 2 should be satisfied.
- Excessive sediment inputs into the system should be avoided.
- Lateral erosion around the steps should be minimized to prevent step pool failure due to outflanking.
- Foundation requirements are required to withstand the erosive action of flow over the steps. Measures are to be implemented to reduce the likelihood of step undermining and piping through the steps.
- Minimum requirements for long-term maintenance.

4 Step-Pool Design

Figure 11, shows a plan and sections through a typical step pool structure, as well Figure 12 shows a typical profile through a set of step pools. The step pools introduced will cause the dissipation of energy and create grade control during high flows while mimicking what nature is observed to geomorphically do. When developing the step pool design the risks and benefits associated with each change to the step pool arrangements, geometry, and material sizing were taking into account.

4.1 Step Boulder Sizing

The sizing of the steps and arrangement of the steps is centred around a few key particles termed anchor boulder (Thomas et al, 2000) or referred to in this report as 'keystone' boulders. These keystone boulders are the largest size gradation found within the diversion channel. These boulders can provide stability and flow confinement over the 'boulder weir', difference in sequential step crest elevations, and play a key role in the function of the step. The size of the step boulders is largely dictated by the step pool geometry. As demonstrated by Lenzi in steep, up to approximately 20% slopes in Northern Italy, the structure height to size ratio is typically kept within the range of step height (H)/D_{90} = 1-4 ; where step height is defined the depth from the pool to the elevation of the upstream boulder crest, see Figure 11 for details. Further, as a general guideline the geomorphology literature reviewed suggests that, based on stable step pool environments observed in the field, the size of the material in the step is approximately proportional to the size of the step height, 1:1 ratio (Church, 2006, Chin, 2008).

In addition to basic geometric relationships the boulders in the steps have been sized with the "moment stability analysis". This analysis assumes that the keystone boulder would be positioned in isolation and submerged in a stream (Fischenich, Seal, 2000). For the keystone boulders it was determined that a minimum boulder diameter of 1.3m is required for the steps.

The forces of the boulder exerted by its' weight in addition to the frictional forces are computed to be greater than the lift, drag and buoyancy forces. This moment stability analysis assumes that the boulder will move first by rolling. To simplify the problem the analysis was performed through utilizing a sole contact point with any downstream boulder to conduct the moment analysis. In this manner the friction forces can be ignored as the moment arm passes through the contact point. The shear forces exerted by the boulder on the steam are then estimated from the critical depth and an overall channel slope of 13-15% to derive an applied shear stress of 102.5 kg/m^2 (21 lb/ft^2). From this technique a keystone boulder with a diameter of 1.3m, FOS = 1, is derived. This method was then compared against the results of Lenzi stability assessments step boulders against sliding. By checking the determined boulder diameter of 1.3m value against the USDA envelope curve sizing equation and by applying a FOS of 1.2 comparable results are found. To further reinforce this sizing method the "moment stability analysis" performed was contrasted against a method developed in 1942 (Allen) for determining when isolated cubes would overturn in a stream. For conditions which would be at the brink of a step, the Allen method suggests that a 1.2 m diameter block would be required (this is approximately mass equivalent to a boulder with a 1.3m diameter).

Thomas's et al suggests in their design procedure that the minimum size of boulder should be assess as being greater than that determined by the U.S. Army Corps of Engineers (USCOE) "steep slope riprap design" method. Coupling the USCOE method with the geometric consideration a D_{50} of 1.0m was adopted for the steps.

A FOS equal to 1 was assumed adequate when determining the step material size for the primary reasons:

- The boulder size is believed to be overestimated as they are a function of the shields parameter which is assumed to be constant under all flow rates; this is unlikely to be observed for the step pool structures.
- As explained in Church's work these step pool structures can withstand more stress than predicted by Shield's parameter, perhaps up to three times as much. This is because of the added strength gained by contacts between the rock and stream side slopes (referred to by Church as the "jammed state"). The benefits from friction would therefore be much greater in an interacting boulder system then that assumed for the boulders assessed in isolation as the interactions can lead to additional structural strength.

To stabilize these step or 'keystone' boulders against toppling or rotation, it is proposed that 100mm diameter concrete filled galvanized steel pipes be installed up against the boulders. The pipes would be buried to depths of about 3m into the bedrock and the embedment depth increased to about 5m in areas exhibiting an overburden (expected to be mainly comprised of till) foundation. Details of the support pipe installations are detailed in Figures 12 attached.

4.2 Boulder Placement

Four to six keystone boulders are required per step. The boulders should be arranged in a broad upstream facing U or V or crescent shape with pointed, curved or apex pointing upstream. The curvature of the boulder is expected to be field fit however it should follow the general guideline that the curvature of the step is equal to the D_{50}/channel width (W); curvature approximately 1.3m/6m or 0.22. This crescent shape step curvature takes advantage of the additional strength gained from the "jammed state" while promoting the alignment of the flow towards the centre of the downstream pool , thereby helping to maintain a constant downstream scour position and limiting excessive bank erosion.

The keystone boulders should be placed along the side of the channel and between these boulders on the outside channel flanks. Transverse boulder work is required to limit lateral erosion around the steps. Step boulders should be ideally be 'seated', dug, or cut preferable into the phyllitic bedrock if possible and will be stabilized with embedded steel pipes installed against the boulders. In the overburden unit to offer additional stability the boulders should as well be seated or dug into the foundation with steel pipes again installed against the boulders. To derive additional stability it is suggested that the boulders as well be concreted or grouted into the foundation at the base of each keystone boulder.

In the overburden section additional riprap and boulder material may need to be placed against the downstream face of the step to provide additional buttressing support and to reduce the likelihood of scour undermining the step. The latter would have to be reassessed during construction. Some preferable boulder arrangements are detailed in Figure 11 and 12. Note that the long or A-axis of the rock should ideally be orientated parallel to the direction of flow to obtain the most stable configuration.

4.3 Step Pool Geometry

The weir height or the difference in elevation of crests of neighbouring steps should be 1m. This arrangement has been designed for to satisfy the geomorphological requirements and is a function of the size of boulders specified for the step construction. Steps should be created consecutively with a crest to crest spacing or step length (L) of approximately 7.7m (1m weir height/ 0.13 slope grade = 7.7m) to allow for maximum resistance to occur. This value is comparable to the values that are calculated for step pool spacing when the Thomas et al. procedure is adopted. The profile detailed in

Figure 12 better illustrates the arrangement of the dimensions required to derive the desired 13% slope grade. On average the channel slopes between steps are expected to be constructed with slight reverse slopes to promote the formation of a pool at lower discharges and to possibly reduce the flow passing through the voids of the steps.

Church provides many, mainly geomorphologically derived, relationships for estimating the step height (H), or the pool depth relative to the crest of the upstream step. In addition Thomas's design criterion outlines a method to calculate the step height based on the weir height and calculated scour. In addition Lenzi, 2002 and D'Agostino et al, 2004 provide methods to estimate scour depth. To limit the quantity of over excavation, to provide additional resistance against undermining of the steps and utilizing the more reliable field based recursion equation, developed by Thomas et al, a step height of 1.6m was determined. First proven by Abrahams and then later confirmed by many other step pool researchers the maximum resistance occurs when $1 < H/L/S < 2$ with a typically value being closer to 1.5 (Lenzi, Comti, Church, Chin, etc...). Doing a quick check on the derived step pool geometry it can be seen that $H/L/S = 1.6/7.7/0.13 = 1.6$ for the proposed step pool design which is in the ranged observed exhibit maximum resistance.

4.4 Step Pool Lining

Based on the USDA method a chute with a 13% slope, 6m base width and a design discharge of $15m^3/s$ would require a D_{50} of 0.5m with an apparent FOS = 1.2. If round rather than angular rock is assumed then a D_{50} of 0.5m with an apparent FOS of 1.0 is derived. Riprap protection with a D_{50} of 0.5m has been determined to remain stable on a chute. The addition of the steps into the reach will effectively cause an increase in the FOS for the bed material against moment as a portion of the energy is dissipated in water cushioning, thereby reducing the shear stress on the riprap. Further evidence that the selected D_{50} of 0.5 wouldn't readily mobilize during the typical normal high flow can be gained by utilizing the Abt and Johnson as well as the Whittaker and Haggi riprap sizing methods which generate similar results to the pool riprap design specification.

To prevent piping non-woven geotextile should be placed on the upstream side of the boulder in both bedrock and overburden. The upstream side of the step should be backfilled first with riprap then with riprap and a bedding material (with a D_{50} approximately 0.04-0.06m in diameter). Backfilling on the upstream side of the step is done to provide a tighter packing matrix so that flow is promoted over the step boulder rather than through the step boulders at low flow. In overburden section the entire structure will be under laid by geotextile to provide further protection against undermining, mitigating against possible piping and associated sediment influxes into the system. In order to prevent the pool rip-rap lining from migrating downstream in high flows, grout or mortar should be utilized to adhere the rip-rap together as a more coherent mass.

4.5 General Hydraulic Calculations

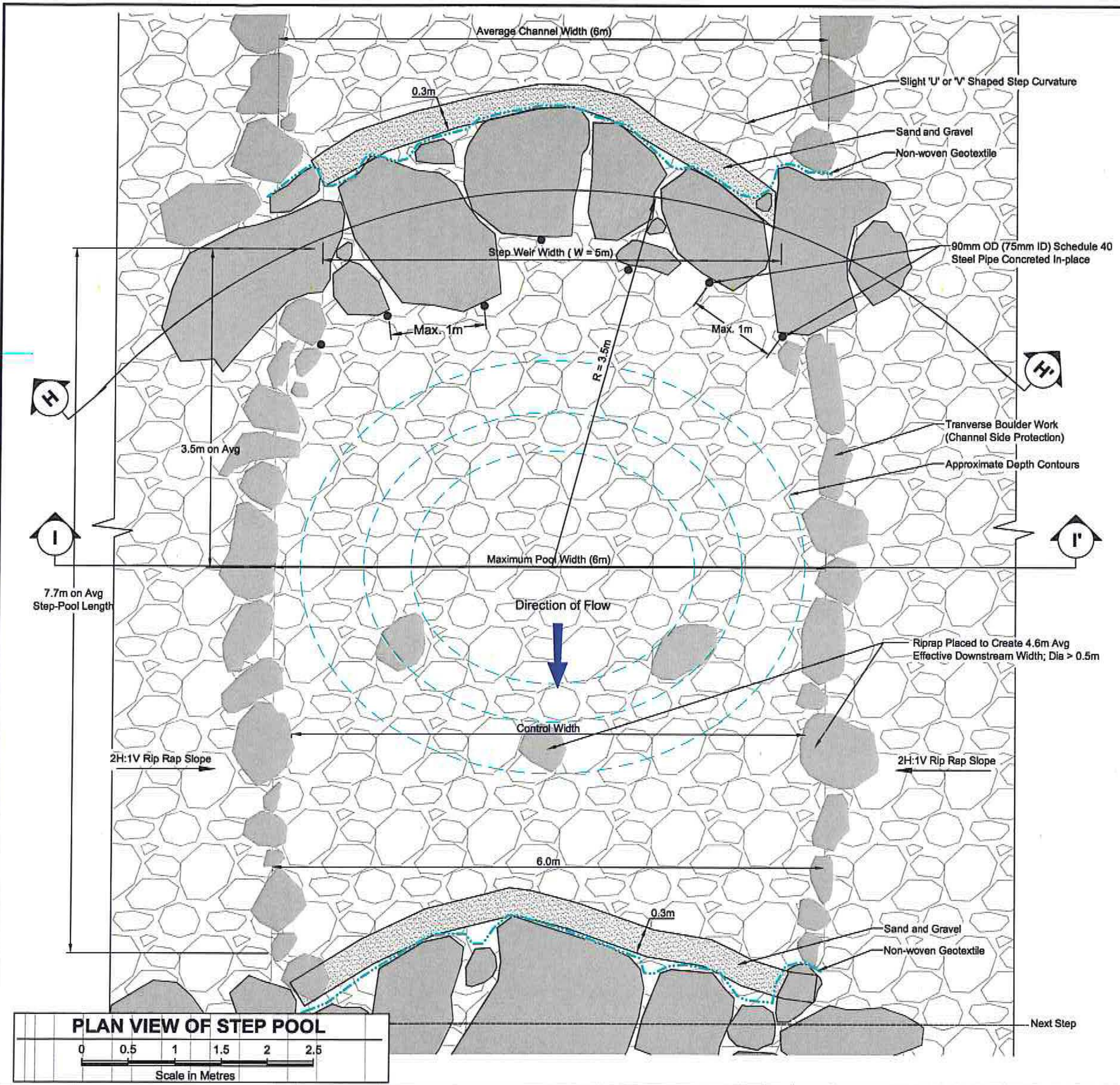
Utilizing the hydraulics of drop structures as an approximation it was found that the jets travelling from the steps would impact the bottom of the pool about 3.8m downstream of the step. Accordingly it has been specified that the deepest part of the pool be constructed 3.8m from the base of the step or approximately at the half way point between steps.

The approximation of the step pool hydraulics was preliminary done to depict the water surface. The latter was done by computing flow values base on a broad-crested trapezoidal weirs and vertical drop structures. By computing the head required to pass $15m^3/s$ over these weirs (steps) a preliminary determination of the water in the upstream pool relative to the crest elevation of the step is obtained. The vertical drop structure dissipates less energy than the drop height. The stream is believed to have velocities faster than what is depicted. To account for this 1m of freeboard has been specified.

4.6 Sediment Inputs

The downstream tailwater control regulated the water-surface elevation of the pool which in turn affects the energy dissipation (Thomas et all, 2000). Small amount of tailwater control produce little control and generate higher velocities while too much narrowing may lead to increased sediment deposit due to increased tailwater. To adequately control the tail water the contraction at the downstream end of each pool has been designed to be 4.6m on average.

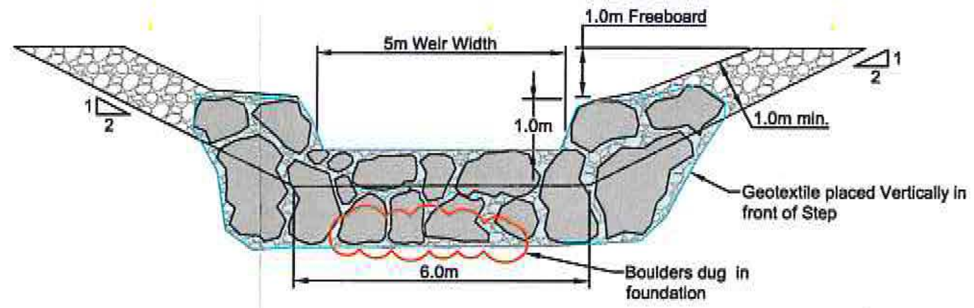
Step pool stability decreases as bedload transport increases. Vangorda Creek is believed to be in a low bedload transport environment so sediment inputs were not considered to be a significant factor. The proposed design was tested against Church's Jammed State Hypothesis and it was found that, provided there is no sediment transport the design specified above is shown to pass. This being stated after larger normal and extreme flow events it is advised that the step pool structure be visually examined to determine if excessive sediment has being trapped in the pools and any unwarranted accumulation of sediment be removed from the pools to ensure proper step pool function.



PLAN VIEW OF STEP POOL

LEGEND

- ▬·▬ Geotextile Filter Fabric
- ▨ Sand and Gravel
- ◌ Rip Rap
- ⬢ Boulder



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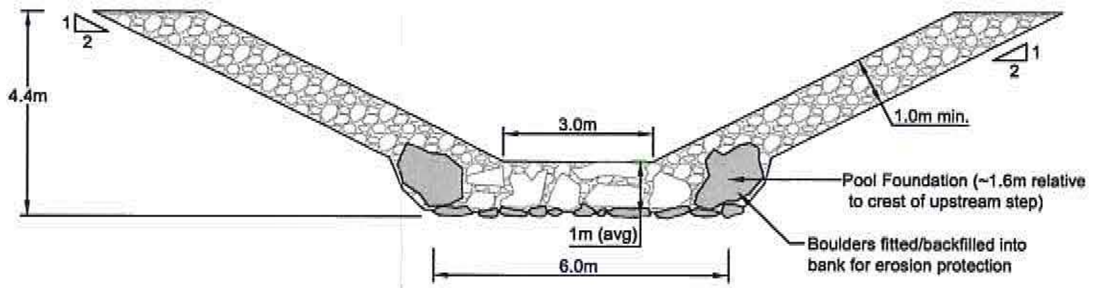

I TYPICAL SECTION THROUGH POOL

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REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
B	Issued for Review	AT	PMH	30Oct09
A	Issued for Discussion	PMH	PMH	31Aug09

REVISIONS

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SRK Consulting
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Vancouver

DESIGN: AT
DRAWN: NV / DC
REVIEWED: PMH
CHECKED: AT
APPROVED: PMH
DATE: October 2009
FILE NAME: 1CY001_031-7-B.dwg

Faro Mine Closure
Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

SRK JOB NO.: 1CY001.031

Vangorda Diversion Channel
DRAWING TITLE:
Step Pool
Plan and Sections

DRAWING NO.: V-11
SHEET 11 OF 14
REVISION NO.: B

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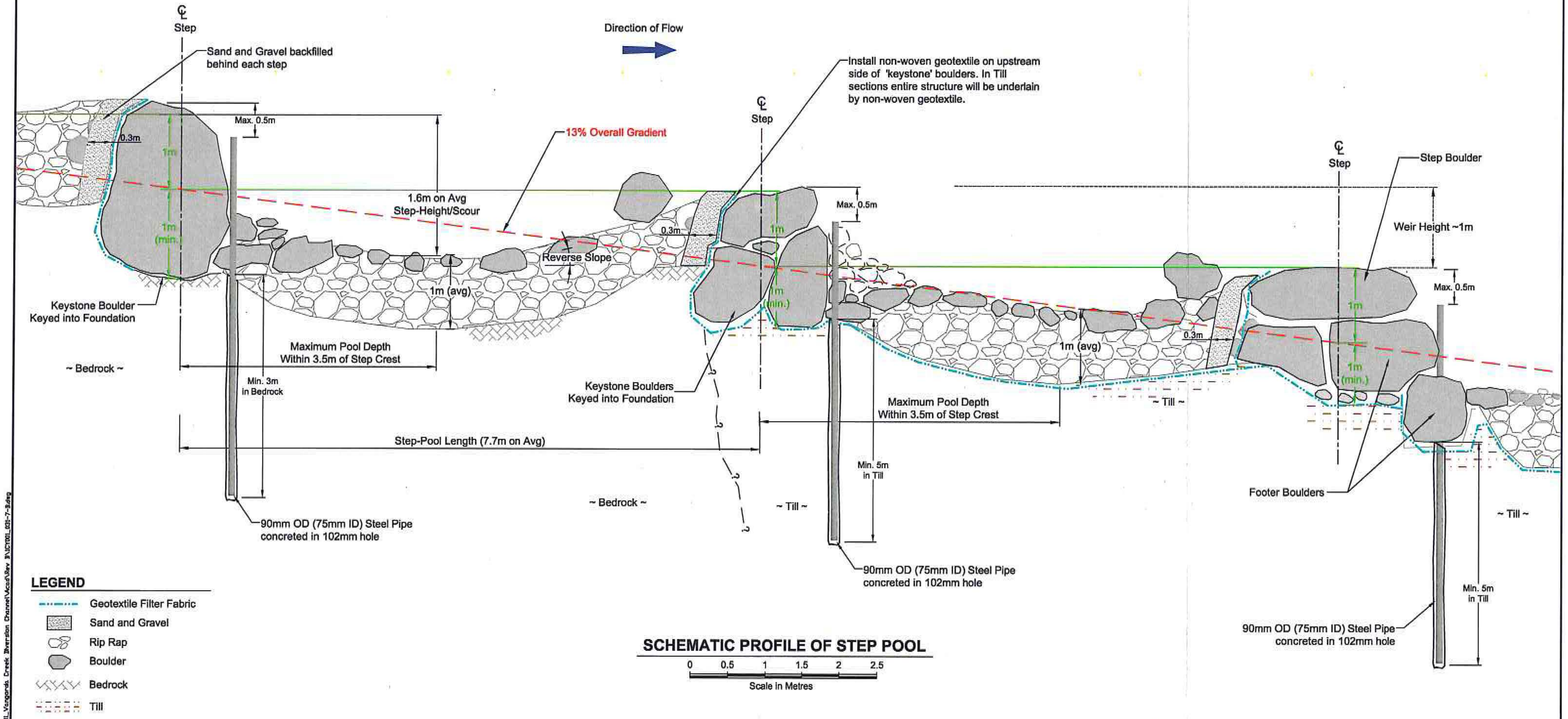

SCHEMATIC PROFILE OF STEP POOL

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REFERENCE DRAWINGS

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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Fermeture de la mine Faro

FARO MINE COMPLEX
FINAL CLOSURE AND REMEDIATION PLAN

SRK JOB NO.: 1CY001.031

Vangorda Diversion Channel

DRAWING TITLE:
Step Pool
Typical Profile

DRAWING NO. V-12
SHEET 12 OF 14
REVISION NO. B

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Appendix VIII

Alternative Alignment Evaluation

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Memorandum

To:	Dustin Rainey	**Date:**	December 18, 2009
cc:	John Brodie	**From:**	Peter Healey
Subject:	Vangorda Creek Diversion Alternative Analysis	**Project #:**	1CY001.031

1 Introduction

SRK was requested to investigate variations and alternatives to the current design for the permanent post closure of the Vangorda Creek Diversion. This memo presents a summary of the analysis and recommendations for moving forward with this project.

2 Background and Scope

The current design (as of November 2009) for the new diversion follows a near-surface route varying in grade from 1.5 percent in the upper reaches to a relatively steep grade of 11 percent in the lower reach before entering the existing plunge pool and dropbox system by the haul road. A plan view of this alignment is shown on Figures 1 and 2. The current profile is shown on Figure 3. The overall diversion system was designed to accommodate the 500-year flood (peak instantaneous flow of 30m^3/s). The current alignment in the steeper section passes through erodible phyllitic bedrock and a section of glacial till. Given the challenges of designing a channel to safely pass large flows down a slope of 11 percent with the given foundation conditions, a decision was made early in the design process to handle the flood with a combination of diversion and temporary storage of water in the Vangorda Pit. The new diversion would be designed to accommodate a lesser event such as the 200-year flood event (peak instantaneous discharge of 13m^3/s). To accommodate rarer floods with return periods between 200 and 500 years, the current design incorporated a feature in the headworks of the diversion that will direct the excess flow down the old Vangorda Creek channel. This flow would then pass through the existing flume diversion, which will be retained after closure. If the flows exceed the capacity of the existing flume, flow will be directed into the Vangorda Pit. This excess volume in the pit would eventually be treated prior to discharge.

Conveying the diversion design discharge of 13 m^3/s down an approximately 660 m long channel with a 11% grade requires some form of energy dissipation and erosion protection. A number of alternative solutions for energy dissipation and erosion protection were evaluated. These included: i) a uniform-sloped, riprap-lined chute; ii) a series of step-pool structures; and iii) a stepped chute. With the latter option, the steps cut in glacial till would have to be lined with concrete or grouted riprap. The steps cut in bedrock may also require a lining to prevent scour.

The evaluation primarily focused on two of the solutions: step-pool structures and a stepped chute.

Step Pools

Step-pool structures are a common morphology on steep streams and, in theory, would appear to be an ideal solution. Until recently, little was known about the forms and processes of step-pool channels. However, over the past two decades significant advances in the theory of step pool sequences have been made (Chin et al., 2008). In evaluating this option, SRK conducted an extensive literature review for a step-pool design. As commented by NHC in their review of the step-pool concept, most of the studies are qualitative and lack the essential detail to provide an acceptable comfort level for a final design. It was concluded that there is a general lack of engineering based criteria for step-pools.

The step-pool approach would involve the construction of a series of pools cut into the channel bottom on the steep slope of the diversion. The vertical height between each pool would be about 1 m and the average length of each pool would be about 9 m. However, the stability of any riprap lining that would be used as erosion protection in the pools is difficult to determine. The other concern is the stability of the large boulders that would be used to form the steps between the pools against rotation or sliding. Furthermore if there is a failure of one step-pool, a progressive failure of the subsequent pools (56 in Phase 1) could occur. As a result the step-pool approach was abandoned.

Stepped Chute

The stepped chute approach would involve cutting a series of steps into the channel base that have a rise of 1m and a tread of 9 m. The main difference between the step-pools and the stepped chute is the latter does not require pools. The primary concern with this approach is the erodibility of the bedrock and the glacial till overburden. The phyllitic bedrock would typically not erode significantly as a result of water flow, but it is felt that the freeze-thaw cycles would accelerate the erosion process. However, there is no best available technology in general literature that would provide the basis for a reliable estimate of the rate of bedrock erosion.

It was then proposed that an adaptive management approach be taken for the bedrock erosion potential. At each step, large boulders (with intermediate axis dimension of 1.3 m) would be concreted into the foundation. These boulders will be placed in an arch formation to provide additional stability. To stabilize these boulders against rotation or sliding, it was proposed to install 100 mm diameter concrete filled, galvanized steel pipes up against the boulders at each step. The pipes would be buried to depths of about 3 m into the bedrock. The bench sections of the stepped chute in bedrock would be excavated with a relatively flat grade. No measures to prevent erosion of the phyllite bedrock exposed in the flat bench sections of the chute would be applied. The rate of erosion would be carefully monitored during the first few years of operation and if it was found that the rate of bedrock erosion was impacting the structural integrity of the diversion, mitigative measures such as placing a concrete surface over the flat section of stepped chute would be implemented.

The steps cut in till would require scour protection. This would be accomplished by placing a 0.5m thick layer of riprap on each flat bench section of the chute. The riprap would be concrete grouted in place. Concrete filled galvanized pipes would also required be installed to stabilize the step boulders, but the embedment depth would be increased to about 5 m.

However, the proposed design with the stepped chute is considered a high risk option and is not believed by the design team to be the best strategy to move forward because of the following concerns:

- erosion of the till and the phyllitic bedrock remains a key issue;
- failure of the erosion protection over the till could result in a progressive failure of a significant portion of the channel and/or a complete rerouting of the stream; and
- a high degree of site management and supervision would be required to ensure that the energy dissipation measures were constructed in accordance with the specifications, particularly for the section of the steep channel founded on till.

Consequently, YG and INAC decided to put a hold on the project pending more detailed evaluation of other options.

As a result, SRK was asked to investigate the following variations to the current design to address the energy dissipation in the 11 percent slope section of the channel:

- Assess a channel alignment which would provide a flatter grade. It was felt that a grade of say 6 to 7 percent would be more acceptable and manageable;
- Review bedrock geology to see if an alignment in granodiorite, which is more erosion resistant than graphitic phyllite, is possible; and
- Assess the hydraulics of a conventional riprapped lined channel along the current alignment to determine whether smaller design flows on a uniform sloped chute on the steep grade would provide more acceptable flow conditions. It is recognized that this would lead to a greater flow being diverted to the Vangorda Pit during a 500 year event. SRK would estimate the additional volume of water that would be directed into the pit and eventually need to be treated.

In addition to the above options, SRK would evaluate the overall strategy for the diversion, including an overview of the following:

- Constructing a conventional concrete lined channel (stepped or unstepped) for the steep reach from STA 800 to STA 1+300; and
- Expand the existing channel alignment further to the NW, with the excavated material disposed in the pit to buttress the pit wall. This would essentially be the alignment that was proposed by SRK in the 2003 alternative evaluation.

In addition to the above, SRK was also asked to prepare a preliminary assessment for a flow attenuation structure located somewhere upstream of the current location of the proposed intake and fuse plug. Details of this assessment are provided below with possible locations of the structure and an estimate of quantities.

SRK understands that the option to backfill the Vangorda pit with waste from the Vangorda dump was rejected as a closure option early in the closure option assessment study. Backfilling the pit would have provided the project with opportunity to re-divert Vangorda Creek over the pit in its original alignment over the backfilled rock. This option eliminates the concern with steep grades but would only be reconsidered as a last resort.

3 Alternatives Analyses

3.1 Alignment along toe of Grum Dump

SRK looked at a number of alternative alignments along the corridor between the toe of the Grum Dump and the Vangorda Creek. The alignments were optimized to limit the gradient of the channel base to no more than 7% and to minimize the excavation volume by keeping the cut depth to less than 10m. An alignment that meets the above criteria is discussed below.

Figures 1 and 2 show a plan view of an alignment that would avoid the steep grade currently needed in the lower reaches of the new diversion. The alignment would follow the current proposed alignment to about STA 0+800 and the deviate north along the hillside staying parallel with the existing access road at the toe of the Grum waste rock dump. The alignment would continue until it intercepts Tributary B at which point the flow would discharge into a plunge pool and be redirected down the gully to a second plunge pool before entering Vangorda Creek. A profile along the centreline of the alignment is shown on Figure 3.

Table 1 provides a comparison of key dimensions and excavation quantities.

Table 1: Quantities Comparison of Alignment

Alignment	Length (m)	Excavation Volume (m^3)	Maximum Grade	
			Grade	Distance (m)
Current	1,506	206,234	11.0%	650
Alternative	2,459	362,340	10.5%	240

While the channel gradient over much of the route is less than 7%, the overall length of the diversion is significant longer that the current proposed route as shown in Table 1. Furthermore, the length of the 7% reach is over 1400m long and diversion would require a 10% drop at the end to intercept Vangorda Creek. The total volume of excavation would be about 50,000m^3 more than the currently proposed alignment and so would be significantly more costly.

SRK does not consider this option as a serious contender for the new diversion.

3.2 Review of Bedrock Geology

A review of the regional geology in the area to determine the extent of the more competent Granodiorite was carried out to assess whether a different alignment could be found that was bedded in this rock type avoiding the issue of erodibility of the phyllitic bedrock encountered along the existing alignment.

As shown on Figure 4 most of the area in the vicinity of the Vangorda Creek Diversion is phyllite, commonly interbanded with Gabbaro dykes and sills or thin quartzoze siltstone interbeds (Pigage, 2005). Granodorite bedrock was found in several of the test pits and boreholes during the 2009 Vangorda Creek diversion site investigation (SRK, 2009) adjacent to the proposed headworks of the diversion. However, it appears that most of the granodiorite bedrock is located several hundred metres to the north. Hence any further evaluation of this approach is not recommended.

3.3 Review Hydraulics of a Uniform-Sloped Chute

The stepped chute approach to energy dissipation proposed in the current design for the steep section of channel raised a number concerns. SRK has evaluated an alternative to the stepped chute approach on the same alignment using a uniform slope. The uniform chute approach essentially involves the excavation of a uniform slope of 11% without steps and with riprap erosion protection along the entire length of the steep section.

The evaluation involved an assessment of the hydraulics of this option along the current alignment to determine whether smaller design flows on a uniform-sloped chute on the steep grade would provide more acceptable stability conditions. It was recognized that adoption of a smaller design flow for the diversion channel would be associated with an increase in the frequency and amount of water diverted to the Vangorda Pit during extreme floods. For example, designing the diversion to convey the 100-year peak instantaneous flood would mean that overflow events to the Vangorda Pit would be expected to occur 5 times in a 500-year period, on average. To examine the tradeoff between diversion design discharge and temporary storage of flood waters in Vangorda Pit, a hydrological study was carried out to estimate the volume of water that would spill to the pit during a 500-year flood.

The first step in the analysis was to characterize both the peak and volume characteristics of floods expected on Vangorda Creek. All the design options were aimed at handling the 500-year peak instantaneous discharge by a combination of diversion and temporary storage. The regional relationships for estimating peak instantaneous discharges for return periods of 2, 100, 200 and 500 years are shown on Figures 5 and 6. Table 2 summarizes estimates of 500-year flood volumes for streams in the Yukon and east-central Alaska. To facilitate comparison, the estimates are presented as average unit flows (L/s/km2). Information in Table 2 was used to construct a hydrograph representing the most intense seven days of the 500-year flood on Vangorda Creek. Figure 7 illustrates how the flood volume data were used to simulate the behaviour of the diversion headworks during passage of the 500-year flood.

Table 2: Estimated 500-Year Floods at Regional Streamflow Gauging Stations

Streamflow Gauging Station		Length of Record	Drainage Area	Mean Annual Runoff	Authority	Average discharge in L/s/km^2 for the following number of consecutive days:						
ID No.	Name	(years)	(km^2)	(mm)		1	2	3	4	5	6	7
29AB006	Upper Wolf Creek	12	14.5	179	EY	303	255	187	160	141	132	125
15439800	Boulder Creek near Central	20	81.0	131	USGS	423	430	339	268	229	204	194
29BC003	Vangorda Creek at Faro Townsite Road	28	91.2	235	EY	250	190	151	134	122	114	108
15511000	Little Chena River near Fairbanks	42	963	199	USGS	426	415	391	356	318	278	246
09BB001	South MacMillan River at km 407 Canol Road	22	997	624	WSC	327	305	304	282	271	252	247
09EA004	North Klondike River near the mouth	33	1100	379	WSC	192	175	161	154	146	137	127
15484000	Salcha River near Salchaket	60	5618	261	USGS	474	396	333	298	262	235	209
09BA001	Ross River at Ross River	46	7250	293	WSC	148	145	141	137	131	127	122

The second step involved characterizing the hydraulics of the chute and estimating the required riprap size. A wide variety of techniques are available for estimating the stability of rock on steep slopes. These techniques generally produce a wide range of size estimates, with this range becoming ever wider with increasing channel slope. To illustrate this range, riprap sizing techniques developed by the US Department of Agriculture (USDA) and the US Army Corps of Engineers (USACE) were employed in this analysis.

The flood hydrology and hydraulics analyses are summarized in Table 3. Three different design flows were evaluated for the diversion channel (viz., 30m^3/s, 13 m^3/s and 11 m^3/s). The following key observations can be drawn from the table:

- The required median size of the riprap lining would be large (with estimates ranging from 0.45 m to 0.63 m for the assumed channel dimensions).
- The uncertainty in assessing rock sizes for steep chutes is illustrated by the large difference in rock sizes estimated by the USDA and USACE techniques. This difference is particularly emphasized when the masses of the riprap sizes are compared (i.e., 180 kg vs. 500 kg).
- The penalty for designing the diversion channel to convey a flow less than the 500-year peak instantaneous flood would be small. If the channel was designed for the 200-year peak instantaneous discharge, then the long-term average additional treatment requirements by the WTP would only be about 5,000 m^3/y. This compares to the average annual inflow to the Vangorda Pit of 420,000 m^3. Adoption of a 100-year design flow for the diversion, would roughly triple the amount of water inflowing to the pit over a 500-year period.
- If the diversion was designed for the 200-year flood, then the Vangorda Pit would receive an estimated 970,000 m^3 of water during passage of the 500-year flood. This would correspond with a 12 m rise in the pit water level. If the diversion was designed for the 100-year flood, the estimated volume of spill is 1,500,000 m^3, corresponding to an 18 m rise in pit water level.

Table 3: Comparison of Channel Parameters for Different Flood Events over the Steep Section of the Channel

Item	Units	Design Event for Diversion Channel		
		500-Year Flood (conservative estimate)	**200-Year Flood (best estimate)**	**100-Year Flood (best estimate)**
Design Discharge for Diversion Channel	m^3/s	30	13	11
Channel Configuration using USDA Design Procedure				
Gradient	m/m	0.11	0.11	0.11
Base Width	m	14	6	5
Depth of Flow	m	0.51	0.51	0.51
Average Velocity	m/s	3.7	3.7	3.7
Froude Number		1.8	1.8	1.8
Riprap D_{50}	m	0.45	0.45	0.45
Riprap D_{50}	kg	180	180	180
Channel Configuration using USACE Design Procedure				
Base Width	m	14	6	5
Depth of Flow	m	0.52	0.52	0.52
Average Velocity	m/s	3.6	3.6	3.6
Froude Number		1.7	1.7	1.7
Riprap D_{50}	m	0.63	0.63	0.63
Riprap D_{50}	kg	500	500	500
Flow Required to Activate Headworks Spillway After Fuse Plug Eroded	m^3/s	n/a	3.5	2.2
Volume Spilled to Vangorda Pit During 500-Year Flood	1000 m^3	0	970	1500
Average Number of Spill Events in a 500-Year Period		~ 0	2.5	5
Long-Term Average Annual Spill to Vangorda Pit	1000 m^3/y	~ 0	5	15

For all three design flow options examined in Table 3, the flow would run supercritical. It is good practice to use straight reaches where supercritical flow occurs in order to avoid uneven distribution of the flow across the channel width. To keep the flow regime subcritical, the slope of the chute would have to be no more than about 3%. Given the geometry of this area, creating a channel with a maximum grade that mild is not feasible.

Generally, rock chutes are short and handle relatively small drops in channel elevation. The riprap lining gains a portion of its strength through the buttressing action of the riprap-lined apron at the downstream end of the chute.

The following items should be considered in evaluating a uniform-sloped chute for conveying Vangorda Creek flows down the 11% slope:

- The riprap sizing should be based on a method that provides conservative rock sizes, such as the USACE method. There are two main reasons for this. Firstly, it is unclear how much the strength of the rock lining can be attributed to the apron. The relative support of the apron diminishes as the chute length increases. Secondly, the stability of the riprap relies on friction at the base of each rock, together with interlocking forces from neighbouring rocks. The larger rock estimated by conservative methods such as the USACE method will be less susceptible to partial failures of the riprap lining.

- Without a significant increase in excavation, it is not possible to make the steep section of the diversion straight. A slight curvature in the upper portion of the chute will be required. Owing to the occurrence of supercritical flow, there will be a tendency for concentration of flow at the outside bend. A larger rock size will have to be used in the curved section of the diversion to accommodate the flow concentration.
- It would be absolutely imperative that the riprap specifications (D_{50} and gradation) be consistent throughout the full length of the chute. Owing to the importance of interlocking forces between rocks, a weak zone in the chute could lead to a catastrophic failure of the chute. As for the stepped chute option, a high degree of site management and supervision would be required to ensure that the energy dissipation measures were constructed in accordance with the specifications.

3.4 Review a Conventional Concrete Lining

Riprap is the conventional method for erosion protection in stream or river channels. A possible alternative to riprap on the stepped chute approach is concrete.

Concrete channels have long standing performance records and proven design procedures. However, there are a number of design considerations for a concrete channel including: the impact of freeze/thaw cycles on the concrete, the cost and availability of the concrete, and long term maintenance. As the Faro mine is in a rural region where all the material and plants would have to be mobilized to site for construction, the construction and material costs of the concrete could be significant.

A preliminary estimate for the concrete structure is \$850,000, based on a 10m channel width, 0.3m thickness and a distance of 630m at \$450/m^3 placed concrete, in addition to the excavation. Furthermore, concrete channels would require periodic maintenance and repairs to ensure performance and structural integrity. In Faro where ice jacking and heavy freeze-thaw cycles, one would expect maintenance to repair flaws and damages over time.

While this option would eliminate the concern with the riprap lining, this benefit would have to be weighed against the high cost and contract management issues.

4 Further Diversion Options

4.1 Upgrading the Existing Flume Diversion (North Wall Pushback)

In April 2003, SRK completed an alternatives analysis for the design of the Vangorda Creek Diversion. One of the options considered involved upgrading the existing flume diversion to a riprap lined open channel around the north perimeter of the Vangorda Pit. The design was based on the 500-year flood event (30m^3/s) and involved the excavation of a wider channel and riprap armouring to prevent scouring and erosion. A site plan for this option is provided in Figure 8. A profile along the centreline of the alignment is provided in Figure 9.

The prime advantage of this option is the maximum channel gradient for Phase 1 of the project would be limited to no steeper than 7% over a distance of about 220 m, which compares to the 11% slope over a distance of 500 m for the current option. In Phase 2, the gradient of the channel from the plunge pool to Vangorda Creek would be in the order of 10% over a much shorter distance and in a less critical section of the diversion. The estimated volume of the excavated material would be comparable to the current proposed alignment, or about 190,000 m3. This option would also eliminate the need for flow control at the intake. Another advantage of this option is that the steep section of the channel would be completely in rock and would be relatively straight. Although the flow would still be supercritical, the reach is straight and the flow distribution would remain even. The till section of the alignment would be in the milder reaches of the diversion and erosion protection would be less critical. Overall this option reduces the grade issue that is common to the other options. Of all options considered, the pushback option involves the smallest elevational distance between the intake and outlet of the diversion.

The main disadvantage of this option is the need to construct this diversion in the winter as Vangorda Creek would need to be diverted around the construction zone using pumps and pipes. It may also be necessary to buttress the pit wall below the diversion to provide an increased factor of safety against failure in the long term. This option also removes the flexibility of retaining the existing flume as a back up in the event flows exceed the design event.

Despite these disadvantages, this option should be given serious consideration. The tradeoff study outlined above in Section 3.3 would also apply to the pushback option. It may prove cost effective to adopt a lower design discharge for the diversion channel than the 500-year peak instantaneous discharge. The penalty would be a slight increase in the amount of water treated by the WTP, when averaged over a long time period.

4.2 Flow Attenuation Structure Upstream

As the high flows are a key design consideration for this diversion, methods to attenuate the peak flow were evaluated. One option considered was the construction of a flow attenuation structure located further upstream of the currently proposed intake structure. The concept involves the construction of a rock filled embankment with a rock drain at the base of the embankment. The embankment would be designed to attenuate the peak flow of a 500-year flood event by temporarily storing a portion of the incoming flood waters.

The analysis looked at two potential locations for the embankment, as shown on Figures 10 and 11. Figure 12 also shows a typical configuration of the embankment with 2H:1V sideslopes. Results of the analysis are shown in Table 4.

The analysis looked at three different embankment sizes at each location and made a preliminary estimate of the storage volume available behind each dam. The behaviour of each dam during passage of the 500-year flood was approximated using a simplified flood-routing technique. In this technique, outflows through the rock drain were assumed to equal inflows until a specified capacity was reached. Beyond that point, inflows in excess of the rock drain capacity were simulated to be stored in the reservoir. This method provides a reasonable first approximation of the ability of the reservoir to attenuate flood peaks. A more accurate representation of the flood routing would require the development of a rating curve for the rock drain based on hydraulic analysis. Such an analysis was beyond the scope of this memorandum. Figure 13 illustrates application of the simplified flood-routing technique. The technique was modified somewhat for those dam options with small storage capacities in which the peak reservoir storage would occur during the first day of the storm.

To determine a design flow for the diversion, the simulated peak outflow from the dam was added to the peak instantaneous discharge generated by the incremental catchment between the dam and the diversion intake.

The results of the analysis indicate that the storage capacity of the dam has to be approximately 400,000 m^3 or more to have a significant impact on the incoming flood peak. This is an expected result: to significantly modify the flood hydrograph, the available storage has to be large relative to the volume of the flood. A reservoir capacity of 400,000 m^3 represents about 60% of the estimated daily inflow volume during the single largest day of the flood. At location 2, the dam would have to be about 23 m high to create a storage of 400,000 m^3. This equates to embankment fill volumes of about 93,000 m^3. This volume is significant and further optimization will be required before this approach is considered feasible.

Table 4: Handle 500 Year Flood by Attenuating the Flood Peak in a Reservoir Upstream of the Diversion

Item	Units	Flood Attenuation Reservoir	
		Location 1	Location 2
Catchment Area Controlled by Dam	km^2	17.3	16.2
Incremental Catchment Area Between Dam and Diversion Intake	km^2	0.4	1.5
Peak 500-Year Discharge Generated by Incremental Catchment	m^3/s	1.5	4.2
Small Dam Option			
Height of Embankment	m	11.0	18.0
Length of Embankment	m	80.0	156.0
Volume of Dam	m^3	10,200	49,000
Capacity of Reservoir	1000 m^3	44.0	212.0
Capacity of Reservoir (as proportion of peak daily inflow volume)	%	6.2	32.0
Peak Outflow from Dam (Via culvert or rock drain)	m^3/s	22.2	12.2
Required Design Discharge of Diversion	m^3/s	23.7	16.4
Medium Dam Option			
Height of Embankment	m	15.0	23.0
Length of Embankment	m	106	197
Volume of Dam	m^3	22,900	93,350
Capacity of Reservoir	1000 m^3	95.0	432.0
Capacity of Reservoir (as proportion of peak daily inflow volume)	%	13.4	65.1
Peak Outflow from Dam (Via culvert or rock drain)	m^3/s	18.7	5.4
Required Design Discharge of Diversion	m^3/s	20.2	9.6
Large Dam Option			
Height of Embankment	m	17.0	27.5
Length of Embankment	m	120	244
Volume of Dam	m^3	31,900	151,700
Capacity of Reservoir	1000 m^3	130.0	653.0
Capacity of Reservoir (as proportion of peak daily inflow volume)	%	18.3	98.4
Peak Outflow from Dam (Via culvert or rock drain)	m^3/s	16.9	3.9
Required Design Discharge of Diversion	m^3/s	18.4	8.1

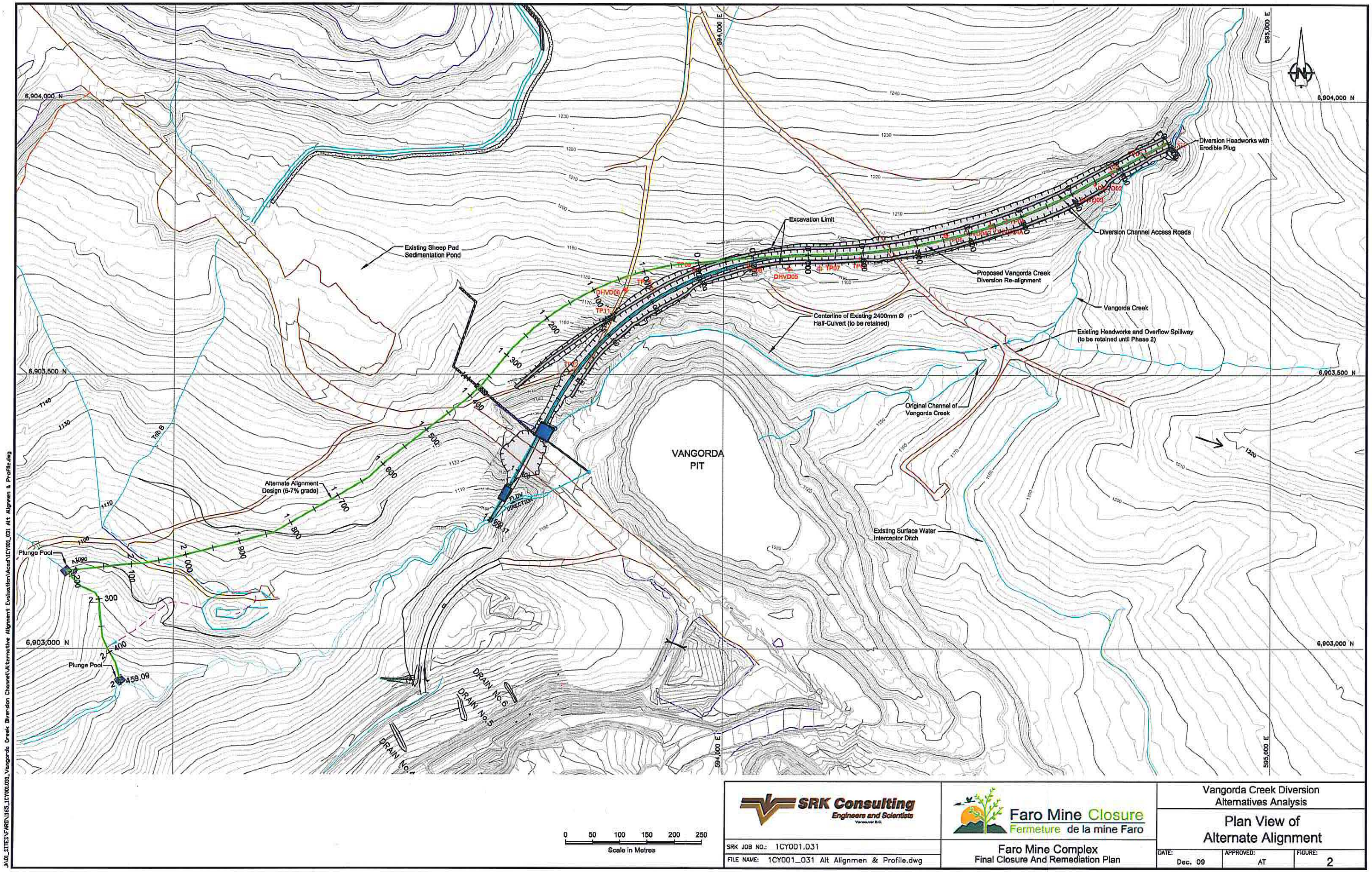
5 Reference

SRK (2009), Faro Mine Complex, Vangorda Creek Diversion – Geotechnical Investigation. Report prepared for Yukon Government. 1CY001.031, August 2009.

Pigage, L.C. (2005), Bulletin 15; Bedrock Geology Compilation of the Anvil District (Parts of NTS 105K / 2, 3, 4, 5, 6, 7 and 11), Central Yukon. Yukon Geological Survey, Digital Compilation. January 2005.

SRK (2003), Alternate Assessment for the Vangorda Creek Diversion. Report prepared for Deloitte & Touche Inc. 1CD003.015, April 2003.

Figures



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Faro Mine Closure
Fermeture de la mine Faro

Vangorda Creek Diversion
Alternatives Analysis

Plan View of Alternate Alignment

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Faro Mine Complex
Final Closure And Remediation Plan

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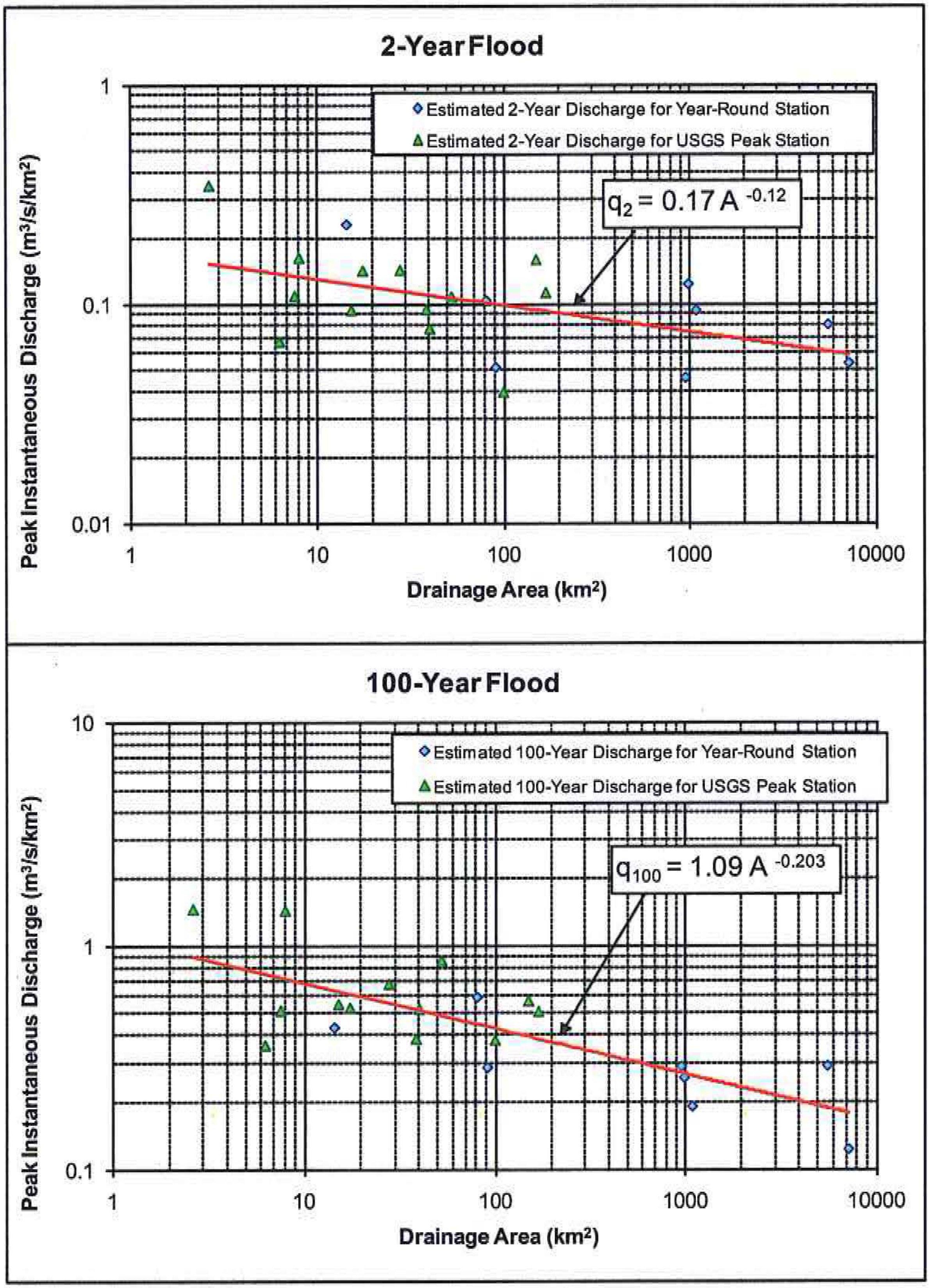

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Vangorda Creek Diversion
Alternatives Analysis

Regional Relationships for Estimating Peak Instantaneous Discharges of 2-Year and 100-Year Floods

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Final Closure and Remediation Plan

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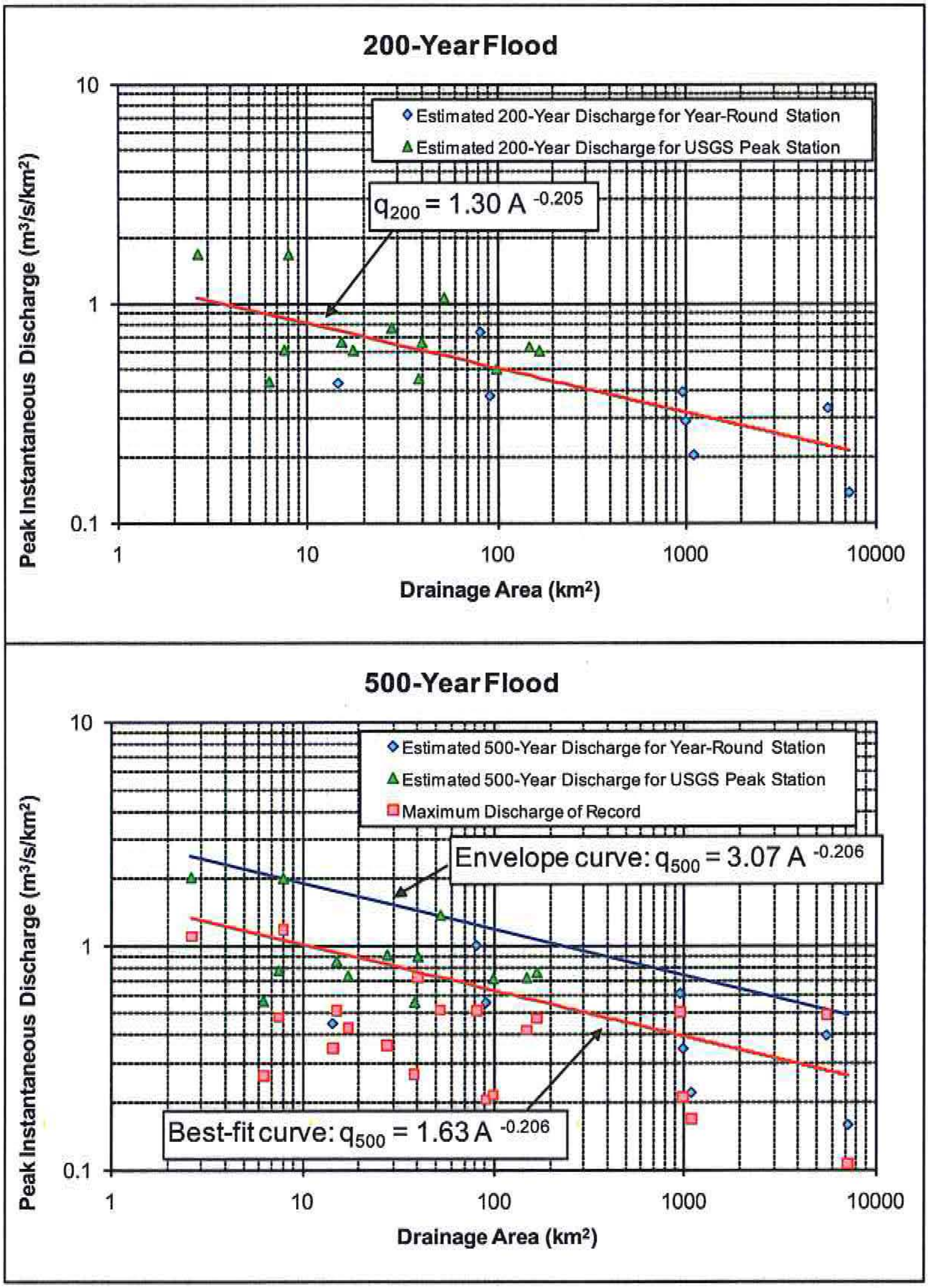

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Alternatives Analysis

Regional Relationships for Estimating Peak Instantaneous Discharges of 200-Year and 500-Year Floods

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Faro Mine Complex
Final Closure and Remediation Plan

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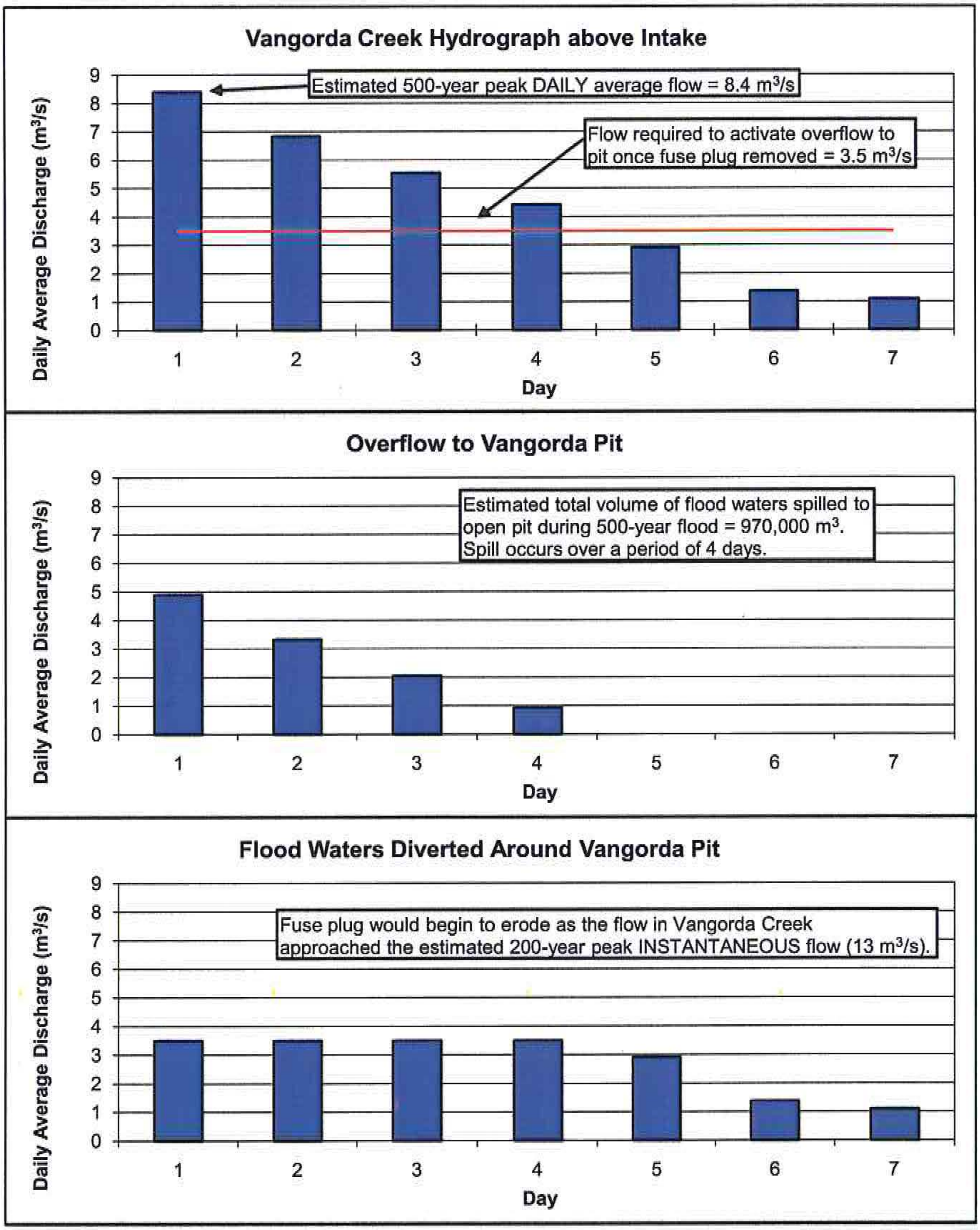


Vangorda Creek Hydrograph above Intake

Estimated 500-year peak DAILY average flow = 8.4 m³/s

Flow required to activate overflow to pit once fuse plug removed = 3.5 m³/s

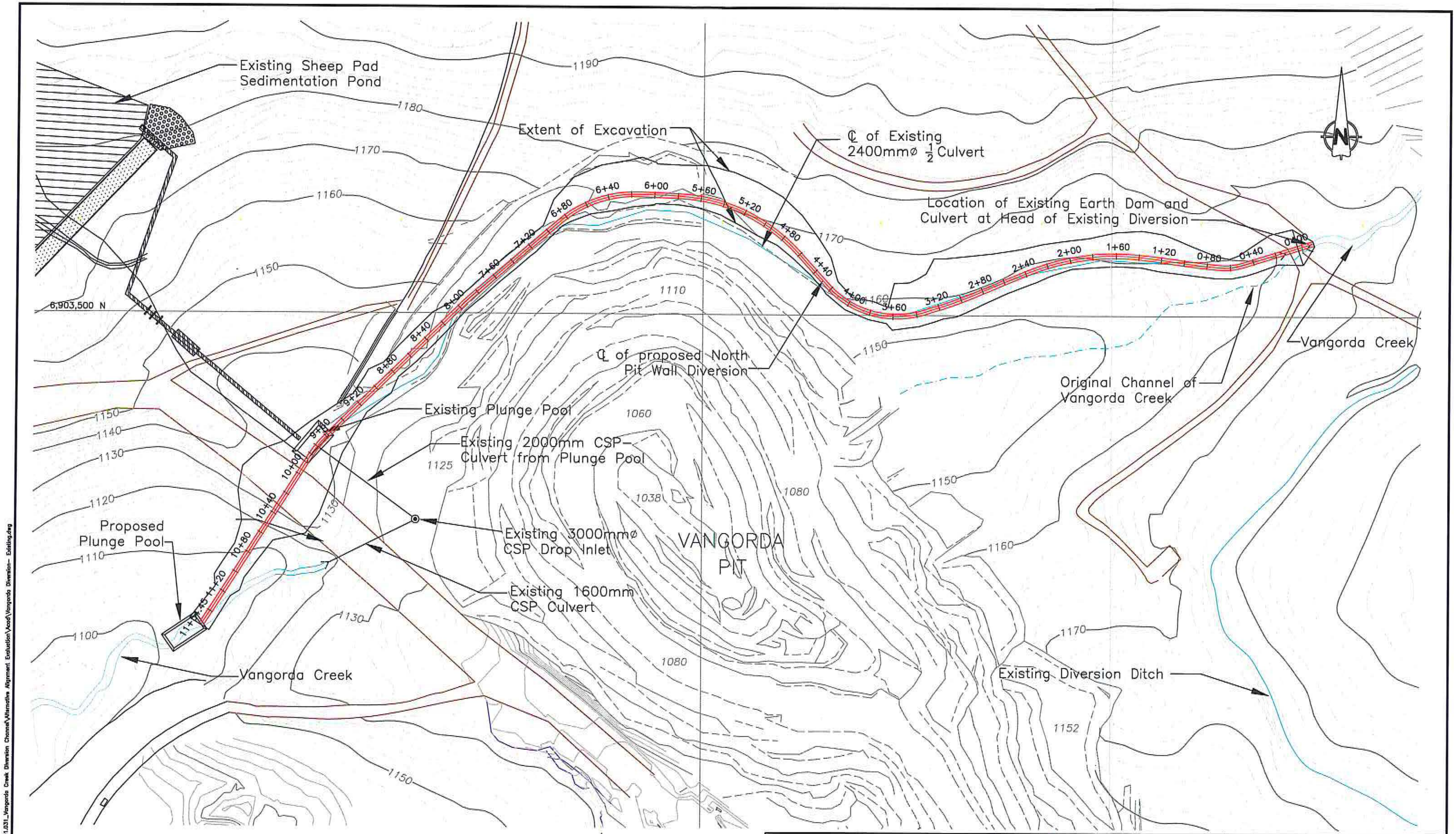
Overflow to Vangorda Pit

Estimated total volume of flood waters spilled to open pit during 500-year flood = 970,000 m³. Spill occurs over a period of 4 days.

Flood Waters Diverted Around Vangorda Pit

Fuse plug would begin to erode as the flow in Vangorda Creek approached the estimated 200-year peak INSTANTANEOUS flow (13 m³/s).

Figure 7: Estimated Behaviour of Headworks during Highest Seven Days of 500-Year Flood



Notes 1) Datum = NAD 27
2) Coordinates = UTM
3) Contour Interval = 2m

0 30 60 90 120 150
Scale in Metres

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Vangorda Creek Diversion
Alternatives Analysis

North Pit Wall Alignment Option

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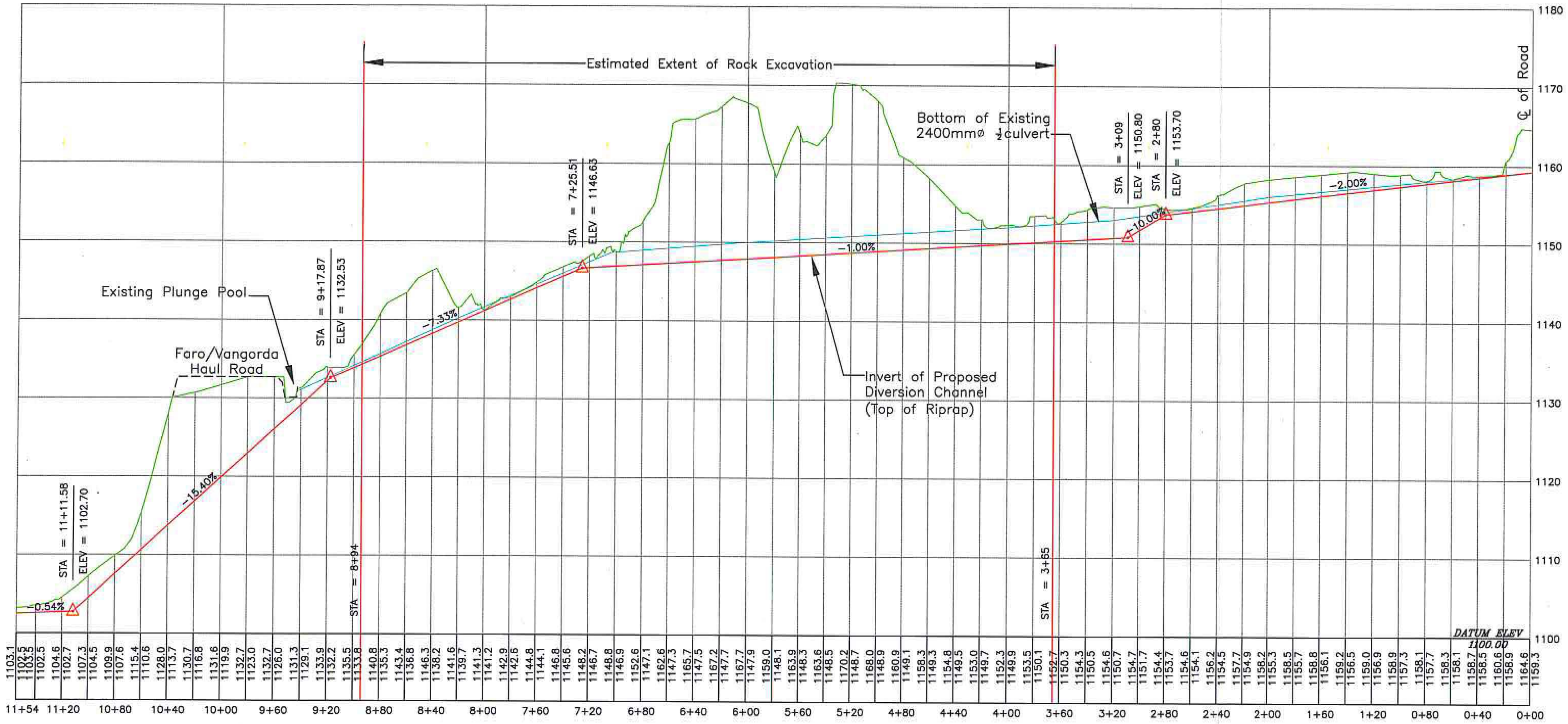

Scale

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Vert: 1:500 0 10

Vertical Exaggeration x 6

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Fermeture de la mine Faro

Faro Mine Complex
Final Closure And Remediation Plan

Vangorda Creek Diversion
Alternatives Analysis

North Pit Wall Profile Option

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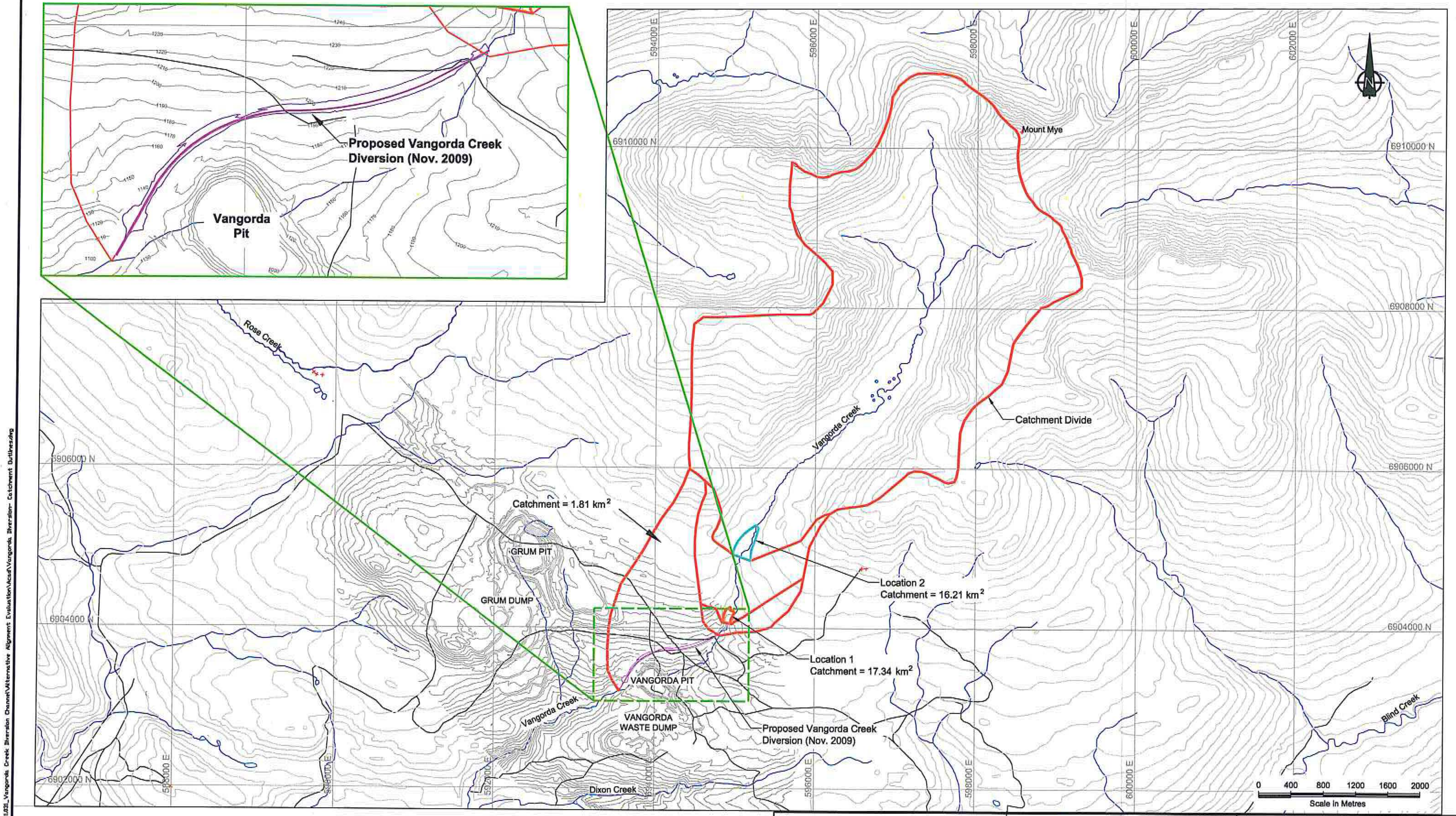

LEGEND

Catchment Area

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FILE NAME: Vangorda Diversion– Catchment Outlines.dwg

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Vangorda Creek Diversion
Alternatives Analysis

Location Plan
Flow Attenuation Structure

DATE:	APPROVED:	FIGURE:
Dec. 09	PMH	10

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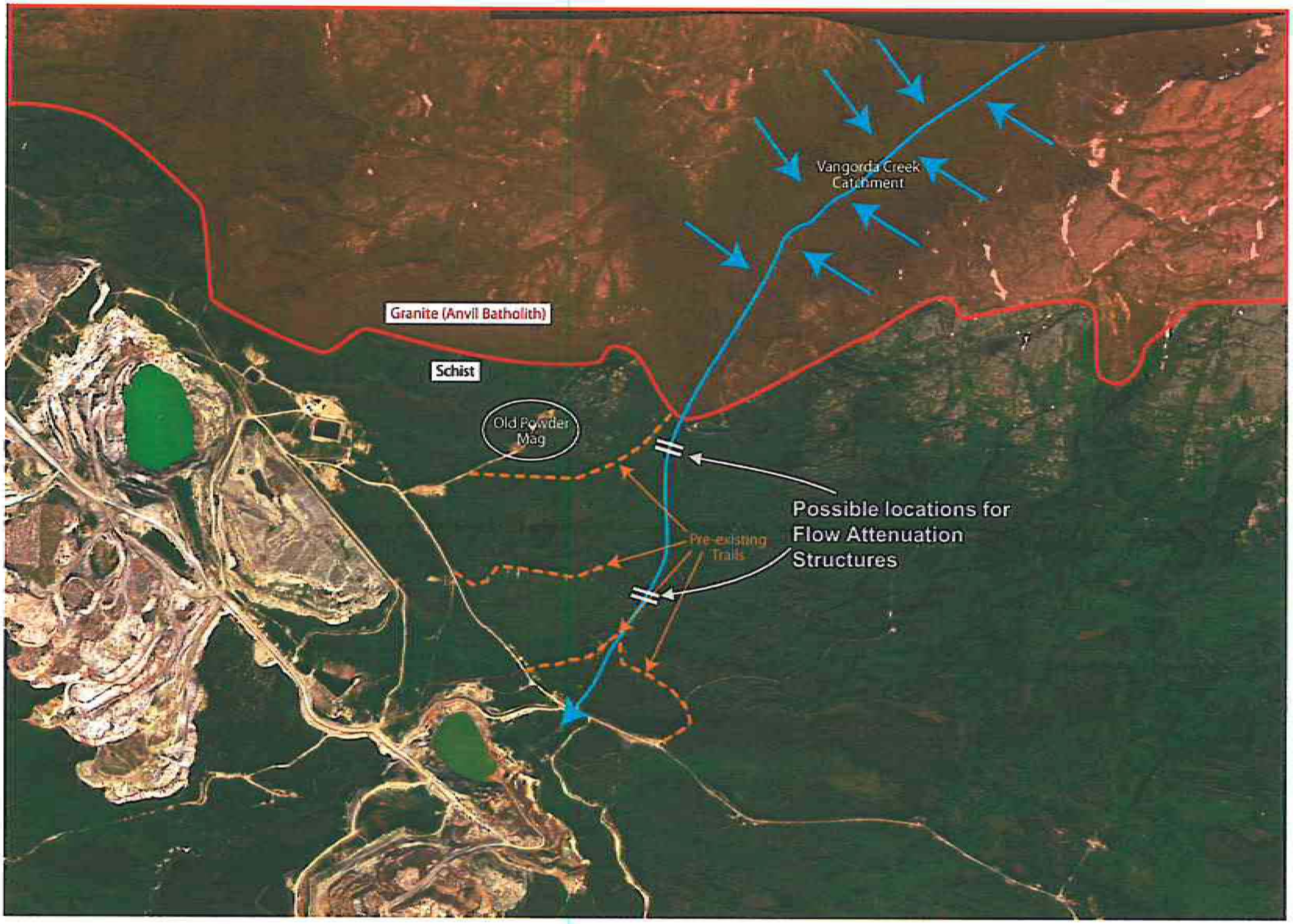

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VANCOUVER

Job No: 1CY001.031
Filename: Figures_4_10_Geology_Trails_20091216.ppt

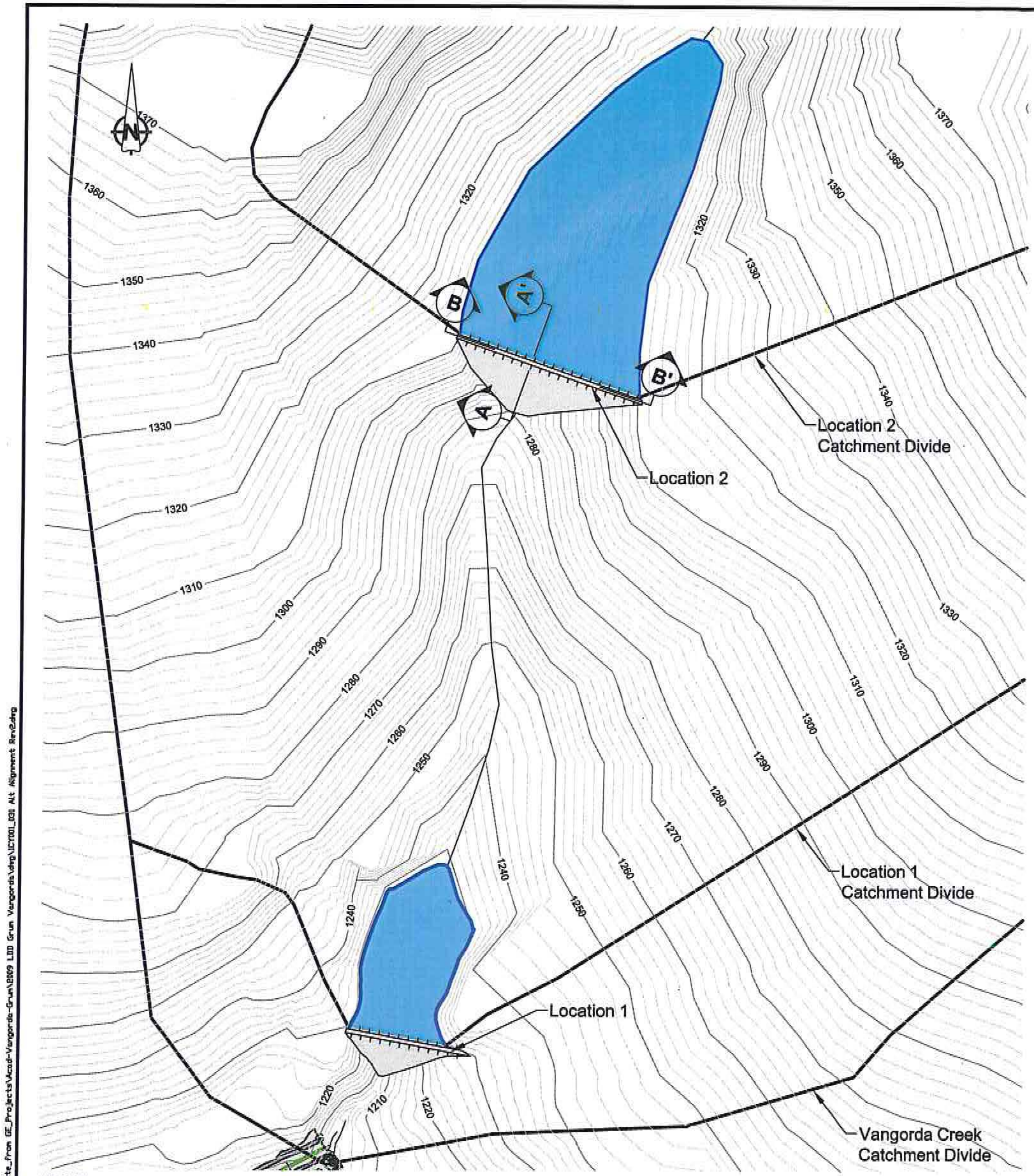
Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex
Final Closure and Remediation Plan

Vangorda Creek Diversion
Alternatives Analysis

Access Trails adjacent to Flow Attenuation Structures

Date:	Approved:	Figure:
December 2009		11



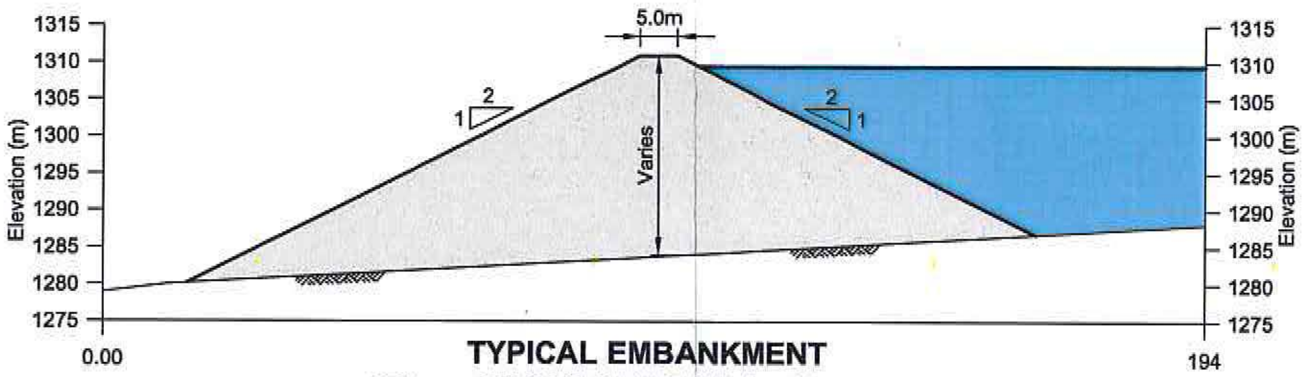
NOTE

Contours shown are derived from regional 1:50,000 topography.

Flow Attenuation Structures

0 50 100 150 200 250

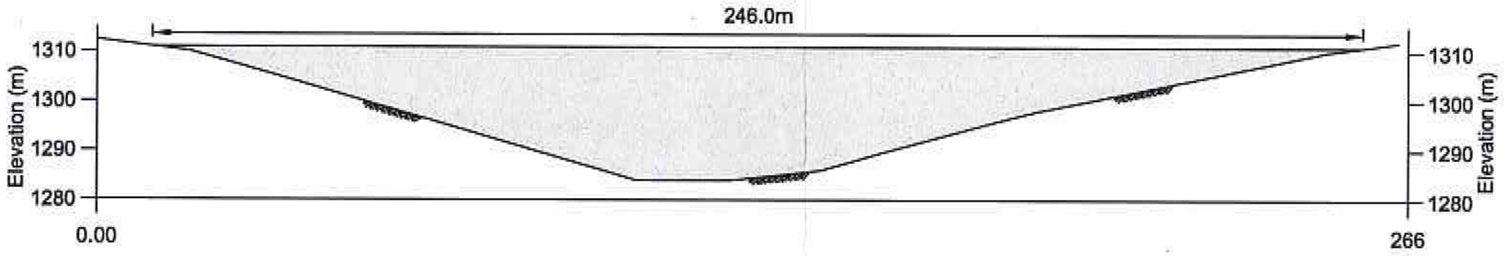
Scale in Metres



A **TYPICAL EMBANKMENT CROSS SECTION A - A'**

0 10 20 30 40 50

Scale in Metres



B **CROSS SECTION B - B'**

0 10 20 30 40 50

Scale in Metres

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SRK JOB NO.: 1CY001.031
FILE NAME: 1CY001_031 Alt Alignment Rev2.dwg

Faro Mine Closure
Fermeture de la mine Faro

Faro Mine Complex
Final Closure And Remediation Plan

Vangorda Creek Diversion
Alternatives Analysis

Vangorda Creek Flow Attenuation Structures

DATE: Dec. 09
APPROVED: AT
FIGURE: 12

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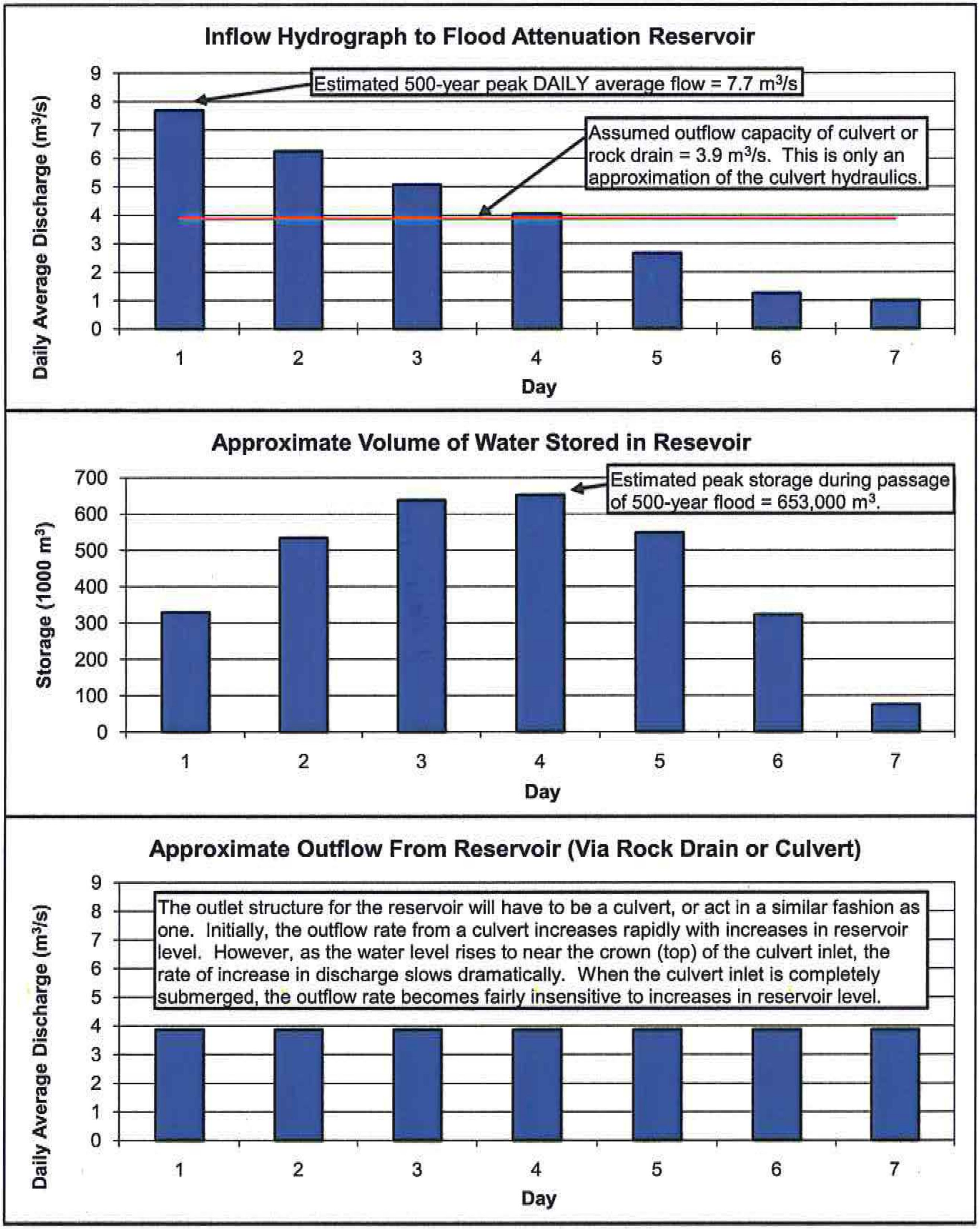

Inflow Hydrograph to Flood Attenuation Reservoir

Estimated 500-year peak DAILY average flow = 7.7 m³/s

Assumed outflow capacity of culvert or rock drain = 3.9 m³/s. This is only an approximation of the culvert hydraulics.

Approximate Volume of Water Stored in Resevoir

Estimated peak storage during passage of 500-year flood = 653,000 m³.

Approximate Outflow From Reservoir (Via Rock Drain or Culvert)

The outlet structure for the reservoir will have to be a culvert, or act in a similar fashion as one. Initially, the outflow rate from a culvert increases rapidly with increases in reservoir level. However, as the water level rises to near the crown (top) of the culvert inlet, the rate of increase in discharge slows dramatically. When the culvert inlet is completely submerged, the outflow rate becomes fairly insensitive to increases in reservoir level.

Figure 13: Estimated Behaviour of Flood Attenuation Reservoir during Highest Seven Days of 500-Year Flood

Vangorda Creek Diversion (November 2009 Design) - Preliminary Cost Estimate

Work Item	Activity	Description	Quantity	Unit	Unit Cost	Sub Task Total	Activity Total	Subtotals
Directs								
Mobilization and Demobilizatoin								$ 119,792
	Mobilization	Mobilization to site	1	unit	$ 59,895.82	$59,895.82	$ 59,896	
	Demobilization	Demobilization off site	1	unit	$ 59,895.82	$59,895.82	$ 59,896	
Site Preparation								$ 389,473
	Site preparation	Clear and grub the work area	92,750	m2	$2.07	$191,946.49	$ 389,473	
		Strip and stockpile topsoil	30,592	Bm3	$5.58	$170,780.36		
		Snow clearing	100	hrs	$253.78	$25,378.16		
		Install Sediment Control	500	m	$2.74	$1,367.59		
Access Roads Construction								$ 311,743
	Excavate material	Excavate overburden material	6,773	Bm3	$5.64	$38,218.03	$ 130,598	
		Excavate weathered rock and dense till	6,773	Bm3	$3.50	$23,708.23		
		Drill and blast competent bedrock	2,580	Bm3	$20.43	$52,716.34		
		Excavate blasted material	2,580	Bm3	$6.18	$15,955.34		
	Construct roads	Load, place, dump and compact General Fill	13,353	Bm3	$8.49	$113,411.54	$ 181,145	
		Load, place, dump and compact Sand and Gravel	3,750	Bm3	$13.69	$51,337.77		
		Excavate uphill runoff collection ditch	1,250	Bm3	$9.96	$12,444.16		
		Install drain pipe for uphill runoff ditch	150	m	$26.35	$3,951.99		
Diversion Channel Construction								$ 1,440,260
	Excavate channel alignment	Excavate overburden material	76,916	Bm3	$5.64	$434,047.61	$ 1,112,715	
		Excavate weathered rock and dense till	47,700	Bm3	$3.50	$166,981.55		
		Drill and blast competent bedrock	25,043	Bm3	$20.43	$511,685.61		
	Construct Channel	Deploy LLDPE liner over sandy section of channels	2,400	m2	$9.47	$22,739.59	$ 327,545	
		Deploy geotextile over liner for protection	2,400	m2	$9.50	$22,792.28		
		Load, haul, dump and place riprap along channel alignment	8,540	Bm3	$33.02	$282,012.94		
Step Pools Construction								$ 480,083
	Excavate step polls	Excavate overburden material for step pools	1,706	Bm3	$3.41	$5,824.19	$ 44,545	
		Excavate weathered rock and dense till for step pools	1,706	Bm3	$5.64	$9,629.42		
		Drill and blast competent bedrock for step pools	853	Bm3	$6.18	$5,276.39		
		Load, haul, dump and place excavated material to stockpile	4,266	Bm3	$5.58	$23,815.02		
	Steel Post installation	Drill and concrete steel in place	1,950	m	$95.96	$187,114.04	$ 187,114	
	Construct step pools	Place and secure keystone boulders	511	Bm3	$49.09	$25,084.40	$ 248,424	
		Deploy geotextile in front of keystone boulders	1,500	m2	$9.50	$14,245.17		
		Load, haul, dump, place and compact Sand and Gravel in front of boulders	4,500	Bm3	$13.69	$61,605.33		
		Load, haul, dump and place riprap in step pools	4,466	Bm3	$33.02	$147,489.53		
Head Works Construction								$ 46,597
	Excavate headworks areas	Excavate overburden material for foundation and key trench	150	Bm3	$5.64	$846.47	$ 2,043	
		Excavate weathered rock and dense till for foundation and key trench	50	Bm3	$3.50	$175.03		
		Drill and blast competent bedrock for foundation and key trench	50	Bm3	$20.43	$1,021.63		
	Construct dam and erodible plug	Load, haul, dump, place and compact Dam shell material	1,272	Bm3	$8.49	$10,803.53	$ 44,554	
		Deploy LLDPE over the dam	960	m2	$9.47	$9,095.84		
		Deploy geotextile over the liner as protection	960	m2	$9.50	$9,116.91		
		Load, haul, dump and place riprap	360	Bm3	$33.02	$11,888.43		
		Load, haul, dump, place and compact Sand and Gravel	92	Bm3	$13.69	$1,256.75		
		Load, haul, dump, place and compact Core material	18	Bm3	$132.89	$2,392.09		
Quarry Development								$ 386,610
	Develop quarry for riprap	Clear and grub the work area	2,500	m2	$2.07	$5,173.76	$ 386,473	
		Strip and stockpile topsoil	1,250	Bm3	$5.58	$6,978.15		
		Drill and Blast competent bedrock	7,002	Bm3	$20.43	$143,077.86		
		Load, haul, dump and stockpile riprap	7,002	Bm3	$33.02	$231,243.13		
	Sediment control	Install Sediment Control	50	m	$2.74	$136.76	$ 137	
								$ 3,174,557
Indirects								
Administrative and Supervison								$ 1,242,700
	Contractor Adminstrative and Supervison	CGL Insurance	5%		$3,174,557.05	$ 158,727.85	$ 772,089.38	
		Office Overhead	3%		$3,174,557.05	$ 95,236.71		

Vangorda Creek Diversion (November 2009 Design) - Preliminary Cost Estimate

Work Item	Activity	Description	Quantity	Unit	Unit Cost	Sub Task Total	Activity Total	Subtotals
		Communications (Radios, satellite phones, etc.)	15	week	$250.00	$ 3,750.00		
		Adminstrative Assistance	1050	hrs	$36.46	$ 38,283.64		
		Medic	1050	hrs	$72.96	$ 76,605.59		
		Contractor Profit	10%		$3,174,557.05	$ 317,455.70		
		Contractor Site Supervision	1,050	hrs	$63.84	$ 67,029.89		
		Site Office and administration	1	unit	$15,000.00	$ 15,000.00		
	Room and Board	Room and Board	1,155	day	$247.50	$ 285,862.50	$ 285,863	
	Engineer's Site QA	Site Vehicle Rental	105	day	$125.00	$ 13,125.00	$ 184,748	
		Room and Board	105	day	$247.50	$ 25,987.50		
		Engineer's Site Office Supplies	1	L.S.	$2,000.00	$ 2,000.00		
		Engineering site Supervision	1,050	hrs	$136.80	$ 143,635.47		
								$ 1,242,700
Contingency								
	Contingency		20%	%	20%	$634,911.41		
							Direct	**$ 3,174,557**
							Indirect	**$ 1,242,700**
							Contingency	**$ 634,911**
							Total	**$ 5,052,168**