HYDROTECHNICAL STUDY FOR CLOSURE PLANNING FARO MINE SITE AREA, YUKON

Draft Report

Submitted to:

SRK Consulting Inc.
Vancouver, BC

Submitted by:

Northwest Hydraulic Consultants Ltd.
Edmonton, AB

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1. INTRODUCTION

1.1 BACKGROUND

This report provides an assessment of information to complete hydrotechnical studies in relation to closure planning for the Faro Mine Area, Anvil Range Mining Complex in Yukon, and updates a preliminary assessment by Northwest Hydraulic Consultants Ltd. (**nhc**, 2001). BGC Engineering Inc. (**BGC**) provided geotechnical input for various closure scenarios for routing extreme floods up to PMF down the Rose Creek Diversion Channel.

The Faro mine is located approximately 20 km northwest of the town of Faro (see Figures 1).

1.2 SCOPE OF WORK

The scope of work as outlined in the request for proposal was described in two tasks:

Task 1 Assess Faro and Vangorda Creek Hydrology.

1.1 Obtain and review all available flow data for Faro and Vangorda Creeks. Update flood estimates for the mine site sub-basins for events up to the 1000-year flood, and comment as to the confidence level of flood predictions.

1.2 Assess whether or not additional flow monitoring is required on the two creeks to better knowledge of runoff characteristics through correlation with Rose Creek flow data. The assessment to be made in the context of improving the level of flood predictions.

1.3 If deemed necessary, install flow monitoring stations on Faro and/or Vangorda Creeks.

1.4 In the event that additional flow monitoring is installed, provide a task list and cost estimate for:

- Developing correlations with Rose Creek flows; and
- Continuing flow monitoring over six years or so.

Task 2 Assess Rose Creek Diversion Options.

2.1 Review the probable maximum flood (PMF) estimates for Rose Creek, and provide an opinion as to the confidence level of flood predictions.

2.2 Examine possibility for routing extreme floods up to the PMF through a modified Rose Creek Diversion using the following three scenarios[1]:

Scenario 1. Increase size of Rose Creek Diversion channel along the south side of the tailings facility to convey the PMF.

Scenario 2. Abandon the Rose Creek Diversion channel downstream of the plug dam. From the plug dam, convey the PMF over the tailings (covered with a soil cover) in swale lined to prevent erosion of the cover/tailings to a new spillway located in the bedrock on the south abutment of the Intermediate Dam (see Figure 2). This requires the spillway be sized to pass the PMF.

Scenario 3. Remove tailings from the Original, Second and Intermediate Impoundments to El. 1042 m. Rose Creek flow to enter the impoundments immediately downstream of the Pumphouse Pond. The attenuated PMF to pass over the spillway sited in the south abutment of the Intermediate Dam.

2.3 Assess requirements for fish passage and energy dissipation.

2.4 Produce nominal designs for the Diversion based on existing geotechnical information.

1.3 SITE VISIT

The Faro Mine Site was visited by Barry Evans of **nhc** on September 25-26, 2003 to view the characteristics of the site streams and their watersheds. This included the Faro Creek Diversion, North and South Forks of Rose Creek, and the Rose Creek Diversion around the tailings ponds. Specific attention was paid to the three streamflow monitoring stations operated by mine site personnel on; North Fork Rose Creek, Rose Creek downstream of the tailings ponds, and Vangorda Creek. Photos 1 to 28 illustrate conditions in the area.

[1] The three scenarios were discussed and agreed upon during a conference call between Cam Scott of SRK Consulting Inc., Jim Cassie of BGC Engineering Inc. and **nhc** personnel on October 31, 2003.

2. HYDROLOGY

2.1 FARO AND VANGORDA CREEKS

2.1.1 Background

Faro Creek is an ungauged stream that, prior to mine development, used to pass through the mine site before flowing into Rose Creek in the vicinity of the abandoned Water Survey of Canada gauge (Stn. 09BC003). With development of the mine, Faro Creek flows were diverted into a channel immediately to the northeast of the Main Pit and released into North Fork Rose Creek at Loc. 1 (see Figure 2, and Photos 1 and 2). Faro Creek has a drainage area of 16 km^2 at Loc. 1.

Faro Creek water levels and flows have not been measured on a regular basis. It is our understanding that the only flow measurements of Faro Creek were made in September 2002 as part of a three-day mine site hydrometric survey.[2]

Vangorda[3] Creek passes to the southeast of the mine site area, and has been gauged by DIAND since 1977 (Stn. 29BC003, Figure 1). Mine site personnel established a second gauge on Vangorda Creek (Stn. V8, Figure 1) approximately 500 m downstream of the DIAND gauge in 1999. The DIAND gauge records summer flows only and does not always catch the annual peak. Some winter data have been collected at Stn. V8. Photo 27 shows that Stn. V8 is located on a steep-sloped, boulder-lined reach: discharge measurements are difficult at such locations, particularly at low flows. Water quality samples are collected by mine site personnel 60 m downstream of Stn. V8 (see Photo 28).

Extreme snowmelt/rainfall flood flows for mine site locations were estimated in the earlier study (**nhc**, 2001) from a regional analysis of the annual flood data of seven gauging stations, including the DIAND station on Vangorda Creek. Average monthly flows were estimated from the gauging records of two mine site streams: Stn. R7 on North Fork Rose Creek (see Figure 2 and Photo 4) and Stn. X14 on Rose Creek downstream of the tailings complex (see Figure 2 and Photo 26).

The runoff characteristics of the mine site with respect to monthly flows and flood events are updated in the following section. See Section 2.1.2 below.

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[2] Survey conducted by Laberge Environmental Services, Whitehorse for Gartner Lee Ltd. The objectives of the survey were to provide flow measurements at a number of locations along the streams passing through the mine site area, thereby allowing determination of seepage losses.
[3] Shows on the Figure 1 NTS map as Van Gorder.

2.1.2 Update of Mine Site Area Hydrology

Average Monthly Flows.

Two more years of flow data are now available for the two gauged mine site streams: Stn. R7 (drainage area 95 km^2) on North Fork Rose Creek upstream of the Faro Creek diversion inflow; and Stn. X14 (drainage area 230 km^2) on Rose Creek downstream of the tailings complex (see Figures 1 & 2).

Figures 3 and 4 present updated daily-flow hydrographs for the two stations over their periods of record[4].

Significant revisions have been made to the Stn. R7 record since 2001. Discharge values have been revised as a result of changes to the rating curves used to convert recorded water levels to flows, and ice effects have been recognized, resulting in blanks in the record - primarily in the fall and winter periods.

Only minor revisions have been made to the earlier Stn. X14 data record. Unfortunately, no data were collected at this station in 2003 due to equipment malfunction.

Tables 1 and 2 list monthly flows at both mine stations. There are significant data gaps in both records.

The flow at Stn. X14 includes 2 to 4×10^6 m^3 of treated effluent water that are released from the Polishing Pond during the summer months.[5] The average effluent volume accounts for approximately 5 percent of the 59.5×10^6 m^3 annual flow volume at Stn. X14.

Snowmelt/Rainfall Floods.

In the earlier hydrotechnical study (**nhc**, 2001) a regional analysis approach was used to estimate annual maximum discharges for return periods up to the 500-year event. Log-log plots of 2-and 100-year flood estimates versus gross drainage area were produced from frequency analyses of annual flood peaks of seven streamflow gauging stations in the Faro region. The log-log regression lines were used to generate synthetic flood frequencies for mine site locations. This procedure (referred to herein as **Method 1**) is repeated herein and the synthetic plots are extended up to the 1000-year event.

A second procedure involving the generation of a dimensionless frequency curve for the Faro region (ratios of extreme flood estimates to mean annual flood) has

[4] Data provided by Gartner Lee Ltd., Yellowknife.
[5] Verbal communication with Eric Denholm of Gartner Lee Ltd.

also been followed in the present study. This procedure is referred to as **Method 2**, and includes error band estimates.

Initially, a homogeneity test was first performed on the annual flood peak series of eight gauging stations (Table 3) within about 150 km of Faro to determine the conforming station records. The homogeneity test used is based on the assumption of a 3LN (3-parameter lognormal) distribution and is described in the publication "Hydrology of Floods in Canada" (Watt et al. 1989). On the basis of the test, the records of seven of the eight gauging stations were accepted as homogeneous (see Figure 5).

For each of the seven gauging stations, frequency analyses were conducted of annual maximum (daily) discharges. The 3LN distribution was mainly used to derive flood frequency estimates up to the 1000-year event. Table 4 lists selected flood estimates.

The plotted frequency curves for the seven stations are presented in Appendix A (Figures A.1 to A.7). Approximate 95% upper and lower error limits have been placed about the frequency curves using a method proposed by Beard (1962) and described in Viessman (1977). The error limits are listed in Table A.1.

The error limits plotted in Figures A.1 to A.7 cover only "sampling uncertainty" associated with the short length of record, assuming that the form of statistical distribution used to fit the data points would also fit a much longer series of data from the same station. Further sources of uncertainty not covered by these plots arise from possible errors in raw data, and from lack of knowledge as to the best form of distribution for a long series.

The frequency plots of Figures A.1 to A.7 illustrate the considerable degree of uncertainty associated with estimating flood values of long return periods by extrapolating curves fitted to short-period data sets[6]. For example, the 3LN frequency curve adequately fits the 15-point Vangorda Creek data set (Figure A.1) but a straight line provides a better fit to the four largest flood data points. The extension of the straight line gives a 1000-year flood estimate of about 30 m^3/s as opposed to 43 m^3/s for the 3LN curve. It can be seen that 30 m^3/s lies below the lower 95% error limit for the 3LN curve.

In conclusion, the flood estimates provided for the seven stations in Table 4 are based on limited data and a frequency distribution that may not be appropriate for extrapolation to long return periods. For the present study, however, these

[6] The length of station records range from 14 to 40 years with a mean of 23 years.

estimates are used for the two regional analysis methods that follow, and from which flood frequency values are estimated for the mine site.

Regional Analysis - Method 1. Log-log plots of the mean annual and 100-year flood estimates versus gross drainage area are shown in Figure 5. The log-log regression fitting lines for the plots are:

For mean annual floods

$$Q_{MAF} = 0.134 \times (DA)^{0.87} \quad \text{..(1)}$$

$(R^2 = 0.84)$

For 100-year floods

$$Q_{100} = 1.01 \times (DA)^{0.73} \quad \text{..(2)}$$

$(R^2 = 0.87)$

Where
Q_{MAF} = mean annual flood (daily) in m^3/s
Q_{100} = 100-year flood (daily) in m^3/s
DA = gross drainage area in km^2
R^2 = logarithmic coefficient of determination

Equations 1 and 2 were used to compute mean annual and 100-year flood (daily) estimates for six sub-basins in the vicinity of the mine site. Instantaneous to daily ratios of 1.3 and 1.8 were used to convert the mean annual and 100-year daily flood estimates to instantaneous equivalents, and flood frequency plots were synthesized for the sites (see Figures 7a through 7f).

Regional Analysis - Method 2. The dimensionless regional frequency curve concept is referred to by Watt (1989) and described in details by Mutreja (1986).

The analytical procedure starts with the computation of the flood ratios (ratio of flood frequency estimates to mean annual flood) for the seven stations in the Faro region. The regional frequency curve is developed from a frequency analysis of the mean of the flood ratios for various return periods (10- to 1000-years).

Figure 8 shows the developed regional frequency curve. The approximate error limits were derived from the dimensionless error band widths of the individual station frequency curves (see Table A.1 and Figures A.1 to A.7).

Figure 8 was used to compute 10- to 1000-year flood (daily) and error band estimates. Instantaneous to daily ratios of 1.3 to 1.95 were used to convert the mean annual to 1000-year daily flood estimates to instantaneous equivalents, and the results were superimposed on the **Method 1** site flood frequency plots of Figures 7a through 7f.

Recommended Flood Frequency Curves. Figures 7a through 7f show that the Method 2 dimensionless frequency curve results in smaller flood frequency estimates than Method 1, and that the difference decreases with increasing drainage area.

The recommended flood frequency curves are shown on Figures 7a to 7f, as straight lines drawn on the log-probability graph from the mean annual flood to the upper 95% error limit for the 1000-year flood of Method 2. Recommended values for the mean annual to 1000-year instantaneous flood discharges, as read from Figures 7a through 7b, are listed in Table 5 for the six sub-basins in the mine site area.

Comparison of the Table 5 flood estimates with the estimates from the earlier study (**nhc**, 2001; Table 8) show:

- a significant reduction in flood estimates for the smaller sub-basins; and
- essentially identical values for the larger basins.

For example, the ratios of the 500-year flood estimates of Table 5 to the earlier values range from 59% for Faro Creek Diversion, Loc. 1 (drainage area 16 km^2; see Figure 2) to 100% for Rose Creek, Stn. X14 (drainage area 230 km^2).

Confidence Level of Flood Estimates. The adopted mine site estimates of Table 5 are a compromise between the results of the Methods 1 and 2 regional analyses.

The following error bands are tentatively suggested for the mean annual and 1000-year flood estimates, but these have no objective statistical basis:

- mean annual flood: ±10% of adopted value; to
- 1000-year flood: ±25% of adopted value.

2.1.3 Flow Monitoring of Faro and Vangorda Creeks

Overview

The need for additional flow monitoring is assessed here in the context of improving the confidence of extreme flood estimates for the Faro Mine Site area. Flood estimates presented above were derived using a regional analysis.

Faro Creek, as noted in Section 2.1.1, is not gauged. Faro Creek has a drainage area of 16 km^2 at its confluence with North Fork Rose Creek (Loc. 1) which is approximately 60 m downstream of the North Fork gauging Stn. R7 - drainage area 95 km^2 (see Figure 2 and Photos 1 and 2).

The establishment of a flow monitoring station for Faro Creek is not considered worthwhile. Collection of six years of data would only provide a preliminary estimate of the mean annual flood, and probably would not significantly improve estimation of extreme flood values.[7]

Vangorda Creek has been gauged by DIAND since 1977 (Stn. 29BC003) and by mine site personnel since 1999 (Stn. V8). The reported drainage area at the DIAND station is 91 km^2. The drainage area at V8 is approximately the same, as it is located only about 500 m farther downstream (see Figure 1). Photo 27 shows Vangorda Creek at the V8 gauge, and Photo 28 the footbridge approximately 60 m downstream where water samples are collected by mine site personnel.

Figure 9 plots Vangorda Creek hydrographs for 1999 to 2003. Annual flow peaks were probably missed in some years. There are few data for the fall/winter period. Significant differences are evident for the certain periods when data were collected at both stations - June to July 1999 and May to June 2000. For 1999, reported daily discharges for V8 are always greater than for 29BC003. The converse is true for 2000.

This comparison of the two records indicates that either one or both are incorrect. Review of the gauging procedures, equipment, calibration, and data compilation used at both stations is suggested.

The possible error in the 29BC003 data is of concern, since the annual maxima at this station were used in the regional frequency analysis to develop extreme flood values for the mine site.

[7] Six years is the stated length of additional flow monitoring in the request for proposal from SRK Consulting Inc.

Given their close proximity, the question arises, why are there two stations on Vangorda Creek? We understand that V8 is required under the Mine Site Water Licence, but shifting the discharge measurements to 29BC003 should be acceptable to the regulatory authorities[8].

Recommendations

1. A gauging station on Faro Creek is **not recommended**.
2. A third gauging station on Vangorda Creek is **not recommended**.
3. The discrepancies between the two Vangorda Creek stations should be investigate by reviewing the field measurement procedures, data collection and computation of discharges. A program outline and cost estimate are given below.
4. Simultaneous discharge measurements should be made in the spring of 2004 at the same time at the two Vangorda Creek stations.
5. Consideration should be given to terminating discharge data collection at Stn. V8, and to having mine site personnel assist in the operation of the DIAND Stn. 29BC003.

Proposed Review of Vangorda Creek Records

An office review using a two-phase approach is recommended.

Phase 1. Check gauging procedures and data for Stn. V8 and re-compute the discharge record if needed. The review will be terminated at this point if:

- gauging procedures and/or data do not allow accurate estimation of daily discharges, or
- re-computed discharge data agree reasonably with Stn. 29BC003.

Phase 2. Check Stn. 29BC003 in a similar manner to Stn. V8.

The review will require access to all original field notes, channel survey data (if collected), staff gauge and data logger records, and related information. Agencies and persons responsible for the installation and operation of the gauging stations will be expected to assist by supplying information in a format that can be readily understood and manipulated.

[8] Communication with Eric Denholm of Gartner Lee Ltd., on November 12, 2003.

It is estimated that the review can be carried out in three weeks after receiving all field data and information, including the preparation of a letter report summarizing findings. The preliminary cost estimate is $11,000 exclusive of GST.

2.2 ROSE CREEK DIVERSION

2.2.1 Background

Probable Maximum Flood (PMF) Estimates

The two most important inputs to the computation of PMF estimates are:

- probable maximum precipitation (PMP); and
- time to peak, i.e., the time it takes for the entire watershed to contribute flow and runoff to reach a peak at the downstream location.

The earlier study (**nhc**, 2001) adopted a 24-hour point PMP of 250 mm extrapolated from a U.S. Weather Bureau (1963) map of Alaska. A time to peak of about 2 hours was computed using the procedures of Kirpich and of Watt and Chow (Watt et al. 1989).

Revised Probable Maximum Precipitation (PMP)

PMP has recently been determined for the Mayo area 200 km northwest of Faro, as part of Yukon Energy Corporation (YEC) program to review the performance of the Wareham Dam spillway under PMF conditions. A 24-hour point PMP of 133.5 mm was estimated for the Mayo area (Hogg, W.D, November 2002; see Appendix B) from maximization of the five largest single-day rainfall events in the Yukon, and, specifically, the 61.5 mm of rain that fell at Boundary[9] 420 km northwest from Faro on June 31, 1971.

The watersheds of Faro Mine Site and Mayo lie in rugged terrain and at somewhat similar elevations; mine site watersheds range between El.1100 and1800 m, compared to El. 600 to 2060 m for the Mayo watershed.

The PMP for Faro mine site was estimated from the Mayo value by comparing the maximum one-day rainfall records of Faro and Mayo. One-day rainfall maxima for Mayo and Faro town site are listed below:

[9] Met Stn. 2100165, El. 1036 m, 12-year record (1967-78).

Location	Station Elev. (m)	Max. 1-day Rainfall (mm)	Date
Mayo[10]	500	31.8	August 27, 1932
Faro town[11]	700	46.7	July 13, 1975

A 24-hour point PMP of 200 mm was adopted for Faro mine site area. This was obtained by applying the ratio of the one-day maximum rainfalls at Faro town and Mayo to the Mayo PMP (46.7/31.8 x 133.5 = 196 mm). The Faro town site data were used because Faro town and Mayo are at similar elevations.

Time to Peak
Times may be deduced from discharge and rainfall records where these data are collected hourly. At Faro, discharge and rainfall data are reported only as one-day averages. Nonetheless an attempt was made to determine typical times by comparing daily discharge records of mine site streams and Vangorda Creek - converted approximately to continuous hydrographs - with Faro town daily rainfall data. Apparent times to peak of about 24 hours were obtained, but these are considered unrealistically high given the steep terrain, rock outcrops and discontinuous permafrost.

Times to peak adopted herein have been increased somewhat over the earlier value of 2 hours for all locations (**nhc**, 2001), recognizing that undergrowth and tree cover is well established over significant areas of the watersheds (see Photos). Adopted times listed in Table 6 range from 3 hours for the Fresh Water Supply Dam catchment (drainage area 67 km^2) to 6 hours for the larger catchments of the Rose Creek Diversion Channel (200 km^2) [12]. Times were varied according to drainage area raised to the power of 0.6.

PMF Estimates
Estimated PMF peak discharges for four locations on Rose Creek (3, 4, 5 and X14 - see Figure 2) are listed in Table 7. Values range from 354 m^3/s for the FWSD catchment (Loc. 4) to 783 m^3/s downstream of the diversion channel (Stn. X14).

The average PMP runoff for each catchment was distributed over a 12- to 24-hour hydrograph (4 times the time to equilibrium) with a peak discharge of 3 times the

[10] Mayo Stn. 2100700, El. 504 m, 75-year record (1925-2000).
[11] Faro A Stn. 2100517, El. 717 m, and Faro Stn. 2100516, El. 694 m, 28-year record (1971-2000).
[12] It has been assumed that the flow-through rock drain through the haul road will be removed and so will not impede flow.

average discharge. PMF hydrographs for the four locations are presented in Figure 10.

Table 8 compares the revised and previous PMF estimates (**nhc**, 2001). PMF peaks have been reduced to between 47% and 64% of the earlier estimates. These revised estimates result in consist Creager C values of 22 to 24 for the four mine site locations[13].

Confidence Level of PMF Estimates

The two most important inputs in the PMF estimation procedure are the PMP and time to peak. Because the new PMP estimate is based on the 2002 PMP study for Mayo, an area of similar hydrologic characteristics, we are fairly confident about the adopted 24-hour point PMP of 200 mm for Faro.

The adopted times to peak of 3 to 6 hours are considered reasonable, but could probably be refined. Methods of doing so are addressed in the next section.

It is suggested that the overall error band for the PMF estimates is ±25% of adopted values but has no objective basis.

Recommendations to Check and Refine PMF Estimates

As adopted time to peak values probably account for the greatest uncertainty in the computation of PMF estimates, methods to refine them are recommended as follows.

1. Collect rainfall and streamflow data to enable computation of time to peak in future rainfall/flood events. This will require:

   i collection of short duration rainfall data at the mine site using a tipping bucket rain gauge; and

   ii computation of instantaneous discharge hydrographs from the streamflow data logger records of the existing gauging stations, for the rainfall flood period.

   Times computed from field data could be used to re-estimate PMF values used in the present study, and/or to calibrate a numerical watershed model.

2. Development a watershed runoff model to compute PMF values from the adopted PMP. The U.S. Army Corps of Engineers HEC HMS (Hydrologic Modeling System) program is recommended. The preliminary cost

[13] The Creager diagram plots peak discharge per unit area against drainage area and is a practical tool for comparing flood data and estimates (see Watt et al. 1989).

estimate for the modelling is $13,000 (not including GST) and would take about four weeks to complete.

3. CLOSURE SCENARIOS FOR PASSING PMF

3.1 INTRODUCTION

3.1.1 The Scenarios

Nominal designs for three scenarios to convey extreme floods up to the PMF through a modified Rose Creek Diversion Channel (RCDC) are presented as part of the long-term closure planning for the Anvil Range Mine Site. The three scenarios are as follows.

Scenario 1: Increase the size of RCDC along the south side of the tailings facility to convey the PMF.

Scenario 2: Abandon the RCDC downstream of the plug dam. From the plug dam, convey the PMF over the Intermediate Pond tailings (assumed covered with a soil cap) in a swale lined to prevent erosion of the cover/tailings to the Intermediate Dam, then over a new spillway by-passing the Intermediate and Cross Valley Dams.

Scenario 3: Remove tailings from the Original, Second and Intermediate Impoundments to El. 1042 m. The Rose Creek PMF to enter the impoundments immediately downstream of the Pumphouse Pond. The attenuated PMF passes over the spillway located at the Intermediate Dam.

The provision of effective fish passage and energy dissipation requirements are integral parts of the three scenarios.
BGC provided geotechnical design input, cost estimates for earthworks and details of assumptions regarding the major work items. The project memorandum of **BGC**'s input is included in Appendix C and extracts from this document are included herein.

nhc provided hydraulic design input, including fish passage and energy dissipation requirements, and cost estimates for concrete structures.

3.1.2 Site Conditions

The Figure 11 map of the tailings facility shows key features, including the location of 39 cross-sections (numbered 1 to 39) along the RCDC. The cross-sections were created by **nhc** from a 0.25 m interval contour map of the channel generated by Yukon Engineering Services, Inc. (YES) using land survey data collected during the summer of 2003 (**nhc**, October 2003).

The 2 m interval contours shown on Figure 11 were generated from 1:20,000 scale aerial photography dated 25 July 2003. Discrepancies were found between the 2 m contour map and the cross-sections based on land survey data.

Hydraulic computations of the RCDC used the land-survey based cross-sections in the one-dimensional hydraulic model HEC-RAS 3.1

Figure 12 shows bedrock surface contours along the south valley wall from the Intermediate Pond to below the Cross Valley Dam. The map was generated by **BGC** from available borehole log data.

3.1.2 Rose Creek Diversion Channel - Existing Conditions

The RCDC extends for a total length of 4.4 km along the south valley wall of Rose Creek. The side slope of the valley wall provides the left bank to the channel and a dike provides the right bank. The RCDC can be subdivided into the following reaches, based on hydraulic aspects:

- The farthest downstream reach from cross-section 1 to 3 (see Figure 11) is a mildly sloped section (slope: 0.0029) below the rock drop weir section where the diversion flow returns into the natural Rose Creek channel.
- The rock drop weir section from cross-sections 3 to 9 is a steeply sloped section (slope: 0.049) consisting of numerous rock weirs. This section compensates for the difference in grades between the RCDC (0.2%) and the original Rose Creek valley (2%).
- A mildly sloped section (slope: 0.0019) above the rock drop weir section from cross-sections 9 to 30, which was constructed in 1980 to divert Rose Creek around the expansion of the tailings facilities. A fuse plug dam is located within the original Rose Creek channel between cross-sections 29-31.
- The upper end of the RCDC is a flat sloped section (slope: 0.0008) that was in place prior to 1980 and is called the original diversion. This reach is located upstream of the fuse plug dam from cross-sections 30 to 39.

The hydraulic capacity and channel stability of the RCDC was recently assessed for the 500-year return period flood peak of 135 m^3/s (**nhc**, October 2003). In summary:

- Overtopping of the right bank dike would commence at discharges of 82 m^3/s (approximately the 100-year flood, see Table 5).

- Full bed movement would occur under the 500-year flood in the steeply sloped rock drop weir section (CS 3-9). The mildly sloped reaches immediately upstream and downstream of the rock drop weir would not be subject to bed movement. Confirmation that minimum bed material size requirements are met in the original diversion section was recommended.
- Upgrading of bank riprap is required in the rock drop weir section and in the mildly sloped section downstream. The mildly sloped section upstream of the rock drop weir has adequate bank protection, except for the original diversion, which likely needs upgrading.

For geotechnical considerations along the RCDC, refer to Appendix C, Section 2.3.

3.1.3 Fish By-Pass Channel Design

In order to safely provide capacity for large flood events and provide effective fish passage, a separate channel providing fish passage is required for the proposed Rose Creek diversion channel. A nature-like fishway should be constructed to ensure unrestricted fish access upstream and downstream in Rose Creek. A nature-like fishway channel has several benefits in this application to a retrofitted formal fishway structure. The hydraulics of these fishways provide access based on swimming in burst modes and resting as opposed to leaping ability in a typical pool-weir type fishway. This is energetically beneficial for migrating fish, and provides high passage efficiency for a wider range of fish sizes and species. The channel would also have a more natural appearance and provide additional habitats for rearing and spawning fish. These channels have been used extensively in Europe, especially Austria and Germany, for grayling migration around instream weir and dams (Jungwirth 1998).

Depending on the final configuration of conveyance channels, spillways and diversion works, the bypass channel could be separate from the existing Rose Creek Diversion channel or future PMF conveyance channel, or incorporated into the existing Rose Creek diversion channel. The bypass channel could be designed to convey all flows up to a pre-determined maximum flood event, above which flows would by diverted through other structures. If the channel utilized the existing diversion channel, the channel section, materials and profile would have to be re-engineered to provide the improved hydraulics for more efficient and effective passage of fish. Inspection of the current channel indicates that is heavily armoured with large rock and relatively trapezoidal. There are relatively few pools, and depth of flow are shallow with relatively high velocities. The re-worked morphology would include additional pools, a greater range of bed sediment sizes and a more refined channel structure.

If a new bypass channel was constructed it would utilize a series of stable pool-riffle structures that create natural hydraulic conditions suitable for fish passage. The length and grade of the structure is matched to the required elevation gain over the obstruction. The width, grade and morphology of the channel is also matched to the flow regime, and swimming characteristics and abilities of the target fish species. The channel could have a step-pool form, similar to the rock cascade fishway concept proposed for Wilsey Dam (**nhc** 2002).

The channel would incorporate natural materials – boulders and cobbles – that provide rearing areas for juvenile fish with potential spawning substrates in the pools and pool tail-outs. Roughness provided by the rounded boulder banks, bed and riffles provide optimum hydraulic conditions rough boundary hydraulics and turbulence – for small fish passage. Given the expected grades and flows, juvenile salmonid access upstream through the by-pass fishway is expected. Currently the gradient of the lower part of the diversion channel is 5%, and is reported to be passable by adult arctic grayling. Accordingly, the hydraulics would be designed around the swimming ability of mature grayling in the Rose Creek system.

3.2 SCENARIO 1

3.2.1 Design

The right bank dike is raised in this scenario to enable the RCDC to convey the PMF peak flow of 730 m^3/s. The dike raise was assumed to be made as a continuous extension of the existing dike slope of 2 horizontal:1 vertical in order to place the new dike within the existing dike footprint as much as possible.

Table 9 lists the computed water levels for 730 m^3/s at each of the 39 cross-sections. From these data the top of the impervious core or water retention element of the new dike was set a nominal 1.0 m above the 730 m^3/s water level. The physical crest of the dike was set 1.0 m above the top of the impervious water retention element. Along most of the channel, the dike height above the channel bed is over 10 m (8 m flow depth + 1 m hydraulic freeboard + 1 m to physical crest).

Figure 13 shows the typical design of the upgraded channel along the mildly sloped reaches. The design of riprap requirements to protect the bed and banks of the upgraded channel from erosion were channel PMF velocities were up to 3.6 m/s. For geotechnical details of the dike design refer to Appendix C, Section 3.1 (from **BGC**).

In the steeply sloped rock drop weir section (CS 3-9) velocities of 10 m/s were computed. A rock of at least 2 m is required to withstand this high velocity which is an impracticable size to use for channel protection on steep slopes. Imbedding riprap in concrete was considered, but is not a suitable at the site as the protection would not stand up to severe freeze-thaw action over a long term period

Finally, the design was modified to replace the rock lined channel with a concrete spillway down the drop rock weir section.

The Figure 14 plan shows a 30 m wide by 300 m long spillway starting at CS 9 and ends in a 45 m long stilling basin in the vicinity of CS 6. The spillway slopes at 0.73% (22 in 300; 1V:13.6H). Outflow from the stilling basin is directed in a rock lined channel back into the RCDC. At the spillway headworks an ogee weir is provided to control velocities in the upstream channel.

The Figure 15 centreline profile indicates that the stilling basin floor and outlet channel bed are close to the bedrock surface. Locating the stilling basin on bedrock will simplify construction. Energy dissipation of the outflow will also be easier to provide as the large boulders required to further dissipate energy may possibly be placed directly on the bedrock surface and not rock riprap underlay. The outflow re-enters the RCDC at CS 3 and the continuation of the improved channel down to CS 1.

A 550 m long fish by-pass is provided from CS 3 to immediately upstream of the spillway entrance at CS 9[14]. The upstream end of the fish by-pass channel will be constricted to restrict flow passing down the ladder to about 30 m^3/s (the 5-year flood) for all flow conditions. Figure 16 illustrates a suitable generic fish ladder design.

Other options were also considered, and are briefly described below (from Appendix C, sections 3.2 and 3.3).

Scenario 1a: This involves widening the existing channel invert by 5 m into the south bank. This would result in lower channel water levels, reducing the raised dike costs. These gains are offset by extra site preparation and excavation costs. Environmentally, this scenario increase the overall footprint of mine disturbance for at most a marginal savings over Scenario 1.

The main concern is the potential for long term degradation of permafrost-affected slopes in the left bank of the channel. The cost assessment in Appendix C did not

[14] In all three scenarios, a fish by-pass has been provided to enhance fish movement along the steeply sloped rock drop weir section from CS 3 upstream to at least CS 9.

include the cost of thermal protection measures for the excavated slopes, which would require over-excavating beyond the nominal 5 m width and covering the slope with thaw-stable thermal protection materials. These costs would definitely drive the cost above the cost estimated for Scenarios 1 and possibly1b.

Scenario 1b: This scenario is similar to Scenario 1 except that between CS 10 and 8, the existing channel would be replaced by a partially concrete lined approach channel (similar to the approach proposed for Scenario 2, see Section 3.2) leading to a spillway on the south abutment of the Cross Valley Dam. Outflow from the spillway stilling basin leading to a rock lined channel, which returns the PMF flow into Rose Creek downstream of the Cross Valley Dam.

The rough cost estimate for Scenario 1b is $38,000,000, or $3,00,000 higher than the cost estimate for Scenario 1.

3.2.2 Scenario 1 Costs

Table 10 summarizes preliminary Scenario 1 costs for earthworks, spillway structural concrete, downstream outlet channel and fish by-pass channel.

Preliminary cost estimate: **$34,800,000**.

3.3 SCENARIO 2

3.3.1 Design

In this scenario the Intermediate Pond is drained and the pond tailings covered with a protective soil cap. The RCDC is abandoned for flood conveyance purposes downstream of the plug dam (see Figure 11). From the plug dam, the PMF peak flow of 730 m^3/s is conveyed in new channel adjacent to the existing RCDC to the Intermediate Dam. Much of the PMF channel passes over the soil covered tailings of the Intermediate Pond. At the south abutment of the Intermediate Dam, the peak flows pass into an approach channel to a spillway located in the south abutment of the Cross Valley Dam, where flow discharges into the pre-mine site development Rose Creek channel (see Figures 17a, b and c).

The Figure 17a plan shows the arrangement whereby flow from the RCDC is diverted at the plug dam into the new downstream flood channel. The design concept, from upstream to downstream (east to west), is as follows:

- CS 39 to 31. Right dike raised along the RCDC and channel erosion protection upgraded as per Scenario 1.

- CS 31 to 28. Plug dam removed.
- CS 31 to 25. Right dike of existing RCDC removed, and swath of land to the right (north) of the RCDC levelled to 0.5 m above the invert of the existing channel bed to allow flow to expand to the 80 m bed width of the PMF channel. The PMF channel dike and portions of the right channel bottom will be constructed on the soil covered tailings.
- CS 25. Headwall constructed across the existing RCDC with a 20 m long conduit to allow flow down the RCDC for fish passage. Conduit sized to allow a maximum discharge of approximately 30 m^3/s into the existing RCDC.
- CS 25 to 13. PMF channel parallels the existing RCDC. The channel and dike will be mainly over soil covered tailings.

Continuing with Figure 17b plan:

- CS 14 to 12. Figure 18 details the typical design of the PMF channel.
- CS 13 to 11. PMF channel converges from a bed with of 80 m at CS 13 to 30 m at CS 11 where it merges with the spillway approach.
- CS 11 to 8. Figure 19 details the spillway approach at CS 10. The approach channel has a vertical concrete wall on the right, a 30 m wide concrete floor slab and a riprap covered left bank sloping at 5H:1V. The fish by-pass is shown towards the top of the left bank located in the existing RCDC. The fish by-pass starts upstream of CS 11, passes through the spillway headworks wingwall at CS 8 and continues downstream in the existing RCDC to the end of the rock drop weir section at CS 3 for a total length of 900 m.
- CS 8. At the spillway headworks an ogee weir is provided to control velocities in the upstream channel.
- The 30 m wide by 120 m long spillway chute slopes at 5H:1V and joins a 42 m long stilling basin that discharges into a riprap lined outflow channel in the centre of the Rose Creek valley (see Figures 17c and 20).

Table 11 lists computed hydraulic properties for 730 m^3/s at cross-sections along the initial section of the RCDC with raised right dike (CS 39 to 31, as in Scenario 1) and the downstream expanded PMF channel to the spillway headworks. These data were used to set the top of the impervious core of the new dikes a nominal 1.0 m above the 730 m^3/s water level, and channel bed and bank erosion protection requirements. The physical crest of the dike was set 1.0 m above the top of the impervious core. For geotechnical details of the dike and channel design (see Figure 18, for example) refer Section 3.4 of Appendix C (from **BGC**).

3.3.2 Scenario 2 Costs

Table 12 summarizes preliminary Scenario 2 costs for earthworks, spillway and approach structural concrete, downstream outlet channel and fish by-pass channel.

Preliminary cost estimate: **$59,900,000**.

3.4 SCENARIO 3

In this scenario the tailings are removed from the Original, Second and Intermediate Impoundments to El. 1042 and located in the Faro pit. Rose Creek PMF enters the impoundments immediately downstream of the Pumphouse Pond (see Figure 11). The attenuated PMF to pass over a spillway in the north abutment of the Intermediate Dam.

At the start of the study, the south abutment of the Intermediate Dam was the proposed location for the spillway as bedrock was thought to be close to the surface, providing a suitable base for founding the spillway. When bedrock was shown to be well below the south abutment (see Figure 12) the location for the spillway was switched to the north abutment. The north side has the added advantage that there is more space for construction and foundation conditions are expected to compromise a mixture of till, sand and gravel and colluvium (Appendix C, Section 3.5).

Unlined emergency spillways for both the Intermediate and Cross Valley Dams are currently located on the north abutment.

3.4.1 PMF Routing

For routing the PMF hydrograph through the dredged impoundment ponds, the following were assumed:

- Inflow - A 24-hour PMF hydrograph with the flow peak of 730 m^3/s occurring at hour 6.
- Outflow weir - Crest at El. 1045.0.
- Maximum pond level - El. 1048.0 (3 m above weir crest).
- Freeboard to top of impervious core - 1.2 m (to El. 1049.2).
- Pond elevation storage curve - see Table 13.

- Initial pond level - El. 1045.0 (outflow weir crest) at start of PMF hydrograph.

The routing computations resulted:

- Adopted width of weir - 55 m.
- Maximum pond water level - El. 1048.1 (0.1 m into freeboard).
- Peak outflow discharge - 610 m^3/s

3.4.2 Design

Rose Creek flood flows will be diverted into the impoundment pond immediately downstream of the Pumphouse Pond. A headwall will be constructed across the RCDC at CS 39. A conduit through the headwall will allow flow up to a maximum of 30 m^3/s for fish passage in the RCDC. This is similar to the Scenario 2 arrangement shown in Figure 17a.

A 550 m long fishway will be constructed in the steeply sloped rock drop weir section of the RCDC between CS 9 and 3 to enhance fish passage.

The Figure 21 plan shows a concrete spillway on the north abutment: headworks with a 55 m wide weir, crest at El. 1045.0 m; transition to a 30 m wide channel; 480 m long stepped spillway; 50 m long spillway chute ending in a 32 m long stilling basin. Outflow from the stilling basin is directed in a rock lined channel outflow channel into the Rose Creek valley.

Bedrock under the spillway alignment is at about El. 1040 m at the Intermediate Dam and at El. 1015 m at the Cross Valley Dam (Appendix C, Section 3.5). The Figure 22 spillway profile shows that bedrock is close to the surface at the spillway headworks.

3.4.3 Scenario 3 Costs

Table 14 summarizes preliminary Scenario 3 costs for CS 39 headwall, spillway structural concrete, downstream outlet channel and fish by-pass channel.

Preliminary cost estimate: **$32,600,000**.

3.5 SCENARIO COST SUMMARY

The preliminary capital cost estimates for the three scenarios are:

Scenario 1	$34,800,000.
Scenario 2	$59,900,000.
Scenario 3	**$32,600,000.**

Scenario 3 - passing extreme flood flows through the dredged tailings impoundment - is the least expensive option for conveying the PMF down the Rose Creek valley.

3.5.1 Concluding Remarks

The following is from Appendix C, Section 3.

It should be noted that for each of these Scenarios there are various levels of unknowns and assumptions, which have implications on cost. Therefore the cost estimates presented should not be compared as if they were based on equal levels of uncertainty. A common cost element for all Scenarios considered is the need to undertake a program of seismic upgrading of the entire Down Valley tailings disposal area, which would be in addition to the earthworks costs presented here. This would involve upgrading the Intermediate and Cross Valley dams to withstand a Maximum Credible Earthquake (MCE) event. The level of upgrading required under each of the above scenarios may vary, depending on design details. For the purposes of this study, the seismic upgrading costs were assumed to be equal for each scenario, and therefore could be ignored in preparing the relative costs for each scenario.

4. REFERENCES

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TABLES

Table 1
North Fork Rose Creek Stn. R7 monthly discharges
for 1996 - 2002

Month	Monthly Discharge in m^3/s for							Average Discharge
	1996	1997	1998	1999	2000	2001	2002	(m^3/s)
Jan		0.11	0.31	0.18	0.22		1.25	0.42
Feb		0.09	0.29	0.14	0.20		1.20	0.38
Mar		0.09	0.25	0.16	0.17		1.14	0.36
Apr		0.26	0.25		0.16		1.11	0.45
May		2.20	2.12	2.71	1.34	1.39	3.10	2.14
Jun			1.34	3.23	2.85	4.13	5.37	3.38
Jul			0.91		1.63	1.77	3.71	2.01
Aug		1.34	0.91		2.10	0.83	4.30	1.89
Sep		0.76	0.80		2.45			1.33
Oct								
Nov		0.39						0.39
Dec	0.17	0.30						0.24

Month	Monthly Discharge in 10^6 m^3							Average Volume
	1996	1997	1998	1999	2000	2001	2002	(10^6 m^3)
Jan		0.30	0.83	0.51	0.59		3.36	1.12
Feb		0.21	0.69	0.34	0.49		2.91	0.93
Mar		0.24	0.66	0.43	0.45		3.04	0.96
Apr		0.68	0.65		0.42		2.88	1.16
May		5.67	5.66	7.29	3.56	3.72	8.28	5.70
Jun			3.46	8.35	7.33	10.71	13.92	8.76
Jul			2.50		4.40	4.75	9.90	5.39
Aug		3.58	2.42		5.64	2.24	11.52	5.08
Sep		2.05	2.07		6.36			3.49
Oct								
Nov		0.98						0.98
Dec	0.45	0.81						0.63
Partial Totals								34.2

Notes:
1. Data provided by Gartner Lee Ltd., Yellowknife
2. Drainage area at gauge 95 km^2

Table 2
Rose Creek Stn. X14 monthly discharges
1994-2002

Month	Monthly Discharge in m^3/s for									Average Discharge (m^3/s)
	1994	1995	1996	1997	1998	1999	2000	2001	2002	
Jan		0.67	0.38	1.77	1.32		0.39	0.34	1.36	0.89
Feb			0.34	0.51			0.14	0.52	1.06	0.51
Mar			0.27	0.37			0.13	0.29	0.97	0.41
Apr			0.33	1.03			0.12	1.52	0.44	0.69
May	4.23		2.79	4.47			1.85	2.07	4.26	3.28
Jun	5.32		3.55	4.95			6.45	1.33	3.30	4.15
Jul	2.92		3.41	3.95		2.89	3.82			3.40
Aug	1.80		2.58	3.98		2.06	5.56			3.20
Sep	1.82		3.15	3.03		1.98	6.13			3.22
Oct	2.13		1.56	2.19			3.58			2.36
Nov	0.98		1.11	1.23			1.64			1.24
Dec	0.66		1.34	1.09			1.03	1.34		1.09

Month	Monthly Discharge in 10^6 m^3									Average Volume (10^6 m^3)
	1994	1995	1996	1997	1998	1999	2000	2001	2002	
Jan		1.78	0.4	4.8	0.1		1.0	0.9	3.7	1.80
Feb			0.7	1.2			0.4	1.2	2.6	1.22
Mar			0.7	1.0			0.4	0.7	2.6	1.08
Apr			0.9	2.7			0.3	3.9	1.1	1.78
May	11.3		7.5	12.0			4.9	5.5	11.7	8.81
Jun	13.8		9.2	12.8			16.7	3.4	8.5	10.74
Jul	7.8		8.9	10.1		7.8	10.3			8.96
Aug	4.8		6.9	10.4		5.5	15.0			8.52
Sep	4.7		8.2	0.3		5.2	16.1			6.88
Oct	5.7		4.2	0.2			9.6			4.91
Nov	2.5		2.9	0.1			4.36			2.47
Dec	1.8		3.5	0.1			2.75	3.52		2.34
Total			53.9	55.6			81.7			59.5

Notes:
1. Data provided by Gartner Lee Ltd., Yellowknife
2. Drainage area at gauge 230 km^2

Table 3
Stream gauging stations used in the homogeneity test

Station	No.	Record Period	Record Length (years)	Drainage Area (km^2)
Vangorda Creek	29BC003	1977-2002	15	91
South Big Salmon River below Livingstone Creek	09AG003	1983-1996	14	515
South MacMillan River at km 407 Canol Rd.	09BB001	1975-1996	22	997
Big Creek near the mouth	09AH003	1975-2002	27	1750
Pelly River below Fortin Creek	09BA002	1986-1994	9	5020
Nordenskiold River below Rowlinson Creek	09AH004	1983-2002	20	6370
Big Salmon River near Carmacks	09AG001	1953-1996	22	6760
Ross River at Ross River	09BA001	1962-2002	40	7250

Note: The Pelly River Stn. 09BA002 data did not pass the homogeneity test.

Table 4
Hydrologic data for stream gauging stations used in the regional analysis

Station	No.	Estimated Flood Discharge (Daily)						
		Mean Annual	10-year	50-year	100-year	200-year	500-year	1000-year
Vangorda Creek	29BC003	4.27	7.23	14.2	18.6	24.0	33.1	40.0
South Big Salmon River below Livingstone Creek	09AG003	33.7	59.3	92.0	108	124	148	165
South MacMillan River at km 407 Canol Rd.	09BB001	125	160	210	235	261	300	330
Big Creek near the mouth	09AH003	106	195	299	347	397	467	530
Nordenskiold River below Rowlinson Creek	09AH004	91.6	153	225	258	292	340	380
Big Salmon River near Carmacks	09AG001	335	470	621	689	760	857	940
Ross River at Ross River	09BA001	408	566	707	765	822	897	950

Table 5
Estimated 2-to 1000-year floods for the Faro Mine site

Mine Site Sub-basins	Drainage Area (km^2)	Flood Discharge (Instantaneous) Mean annual (m^3/s)	50-year (m^3/s)	100-year (m^3/s)	200-year (m^3/s)	500-year (m^3/s)	1000-year (m^3/s)
North Fork Rose Cr. above Faro Creek Diversion Channel (Stn. R7)	95	9.2	37	45	54	67	77
Faro Creek Diversion above North Fork Rose Creek (Loc.1)	16	1.9	7.7	9.4	11	14	16
North Fork Rose Creek at Flow-through Rock Drain (Loc.3)	118	11	44	54	65	81	93
Fresh Water Supply Dam (FWSD) catchment (Loc.4)	67	6.8	27	33	40	49	57
Rose Creek above Tailings Diversion Channel (Loc.5)	203	18	71	86	103	130	150
Rose Creek downstream of Tailings Diversion Channel (Stn. X 14)	230	20	79	96	115	145	167

Table 6
Adopted times to equilibrium for Faro Mine site PMF

Mine Site Sub-basins	Drainage Area (km^2)	Time to Peak (h)
North Fork Rose Creek at Flow-through Rock Drain (Loc.3)	118	4
Fresh Water Supply Dam (FWSD) catchment (Loc.4)	67	3
Rose Creek above Tailings Diversion Channel (Loc.5)	203	6
Rose Creek downstream of Tailings Diversion Channel (Stn. X 14)	230	6

Table 7
Estimated Probable Maximum Floods for the Faro Mine site

Mine Site Sub-basins	Drainage Area (km^2)	PMF Peak Discharge (m^3/s)
North Fork Rose Creek at Flow-through Rock Drain (Loc.3)	118	504
Fresh Water Supply Dam (FWSD) catchment (Loc.4)	67	354
Rose Creek above Tailings Diversion Channel (Loc.5)	203	690
Rose Creek downstream of Tailings Diversion Channel (Stn. X 14)	230	783

Table 8
Comparison of Probable Maximum Flood estimates

Mine Site Sub-basins	Drainage Area (km2)	PMF Peak Discharge **NHC**, 2001 (m3/s)	PMF Peak Discharge **NHC**, 2003 (This study) (m3/s)	Ratio of Discharges NHC, 2003/NHC, 2001 (%)
North Fork Rose Creek at Flow-through Rock Drain (Loc.3)	118	920	504	55
Fresh Water Supply Dam (FWSD) catchment (Loc.4)	67	550	354	64
Rose Creek above Tailings Diversion Channel (Loc.5)	203	1480	690	47
Rose Creek downstream of Tailings Diversion Channel (Stn. X 14)	230	1680	783	47

Table 9
Scenario 1: Modified Rose Creek Diversion Channel
Hydraulic properties for PMF of 730 m^3/s

Cross-Sect. No.	Thalweg Elevation (m)	Flow Depth (m)	Water Surface (m)
39	1054.00	8.91	1062.92
38	1054.50	8.39	1062.90
37	1055.00	7.70	1062.71
36	1054.25	8.34	1062.59
35	1054.00	8.56	1062.56
34	1053.75	8.78	1062.53
33	1053.50	9.00	1062.50
32	1053.25	9.28	1062.53
31	1053.00	9.52	1062.52
28	1053.50	8.33	1061.83
27	1053.00	8.55	1061.55
26	1052.75	8.54	1061.29
25	1052.25	8.84	1061.09
24	1052.50	8.26	1060.76
23	1052.00	8.22	1060.22
22	1051.75	8.09	1059.84
21	1051.50	8.33	1059.83
20	1051.25	8.31	1059.56
19	1050.75	8.61	1059.36
18	1050.75	8.43	1059.18
17	1050.50	8.35	1058.85
16	1050.25	8.26	1058.51
15	1050.00	8.04	1058.04
14	1049.75	7.99	1057.74
13	1049.50	8.05	1057.55
12	1049.25	7.93	1057.19
11	1049.00	7.12	1056.12
10	1048.25	7.50	1055.75
9	1048.00	5.86	1053.86
8	1042.75	3.77	1046.52
7	1035.75	5.07	1040.82
6	1030.50	4.56	1035.06
5	1026.50	4.09	1030.59
4	1022.50	4.57	1027.07
3	1021.00	6.28	1027.28
2	1020.25	7.13	1027.38
1	1020.25	7.09	1027.34

Table 10
Preliminary costs estimate for Scenario 1
with concrete spillway from CS 9 to 6

Earthworks	Spillway	Outlet Channel	Fish By-Pass	Total Cost
$16,100,000	$17,400,000	$400,000	$900,000	$34,800,000

Notes:

1. Earthwork costs include site clearing, excavation, disposal, fill, and dike construction including bank and bed riprap erosion protection (from BGC: Appendix C, Table 1).
2. Spillway costs are for structural concrete only.

Table 11
Scenario 2: PMF channel over Intermediate Pond tailings to spillway by-passing Intermediate & Cross Vally Dams
Hydraulic properties for PMF of 730 m^3/s

	Cross-Sect. No.	Thalweg Elevation (m)	Flow Depth (m)	Water Surface (m)	Channel Velocity (m/s)	Channel Froude No.
Existing RCDC	39	1054.0	7.23	1061.23	1.4	0.17
with raised right dike	38	1054.5	6.70	1061.20	1.4	0.19
	37	1055.0	5.63	1060.63	3.7	0.53
	36	1054.3	5.91	1060.16	4.0	0.59
	35	1054.0	5.72	1059.72	4.2	0.63
	34	1053.8	5.51	1059.26	4.4	0.64
	33	1053.5	5.33	1058.83	4.0	0.63
	32	1053.3	5.54	1058.79	2.7	0.44
Expansion of	31	1053.0	5.71	1058.71	2.4	0.36
channel width	28	1053.5	3.37	1056.87	3.4	0.66
	27	1053.0	3.41	1056.41	2.8	0.53
	26	1052.8	3.24	1055.99	2.6	0.51
	25	1052.8	2.69	1055.44	3.1	0.61
Start of PMF	24	1052.0	2.69	1054.64	3.1	0.61
channel	23	1050.9	2.69	1053.63	3.1	0.61
	22	1050.3	2.69	1053.02	3.0	0.61
	21	1049.7	2.72	1052.46	3.0	0.60
	20	1049.3	2.74	1052.03	3.0	0.59
	19	1048.6	2.87	1051.49	2.9	0.55
	18	1048.3	2.96	1051.23	2.8	0.53
	17	1047.8	3.19	1050.95	2.5	0.47
	16	1047.2	3.57	1050.72	2.3	0.40
	15	1046.4	4.12	1050.56	1.9	0.32
	14	1045.6	4.82	1050.46	1.6	0.25
	13	1045.4	5.04	1050.44	1.5	0.23
	12	1044.8	5.46	1050.23	2.0	0.30
	11.1	1044.2	5.46	1049.68	3.1	0.48
Spillway approach	11	1044.0	5.58	1049.58	3.0	0.46
	10	1043.1	6.16	1049.26	2.5	0.37
	9	1042.7	6.47	1049.17	2.4	0.34
	8	1042.1	6.95	1049.05	2.1	0.29
Spillway headworks	7.9	1042.0	6.60	1048.60	3.5	0.44

Table 12
Preliminary costs estimate for Scenario 2

Earthworks	Spillway	Outlet Channel	Fish By-Pass	Total Cost
$29,460,000	$28,700,000	$400,000	$1,350,000	$59,910,000

Notes:

1. Earthwork costs include site clearing, excavation, disposal, fill, and dike construction including bank and bed riprap erosion protection (from BGC: Appendix C, Table 3).
2. Spillway costs include approach and are for structural concrete only.

Table 13
Storage curve for dredged impoundment pond for Scenario 3 PMF routing

Geodetic Elevation (m)	Pond Volume (m^3)
1042	0
1043	1,950,000
1044	3,900,000
1045	5,850,000
1046	7,800,000
1047	9,750,000
1048	11,700,000
1049	13,650,000

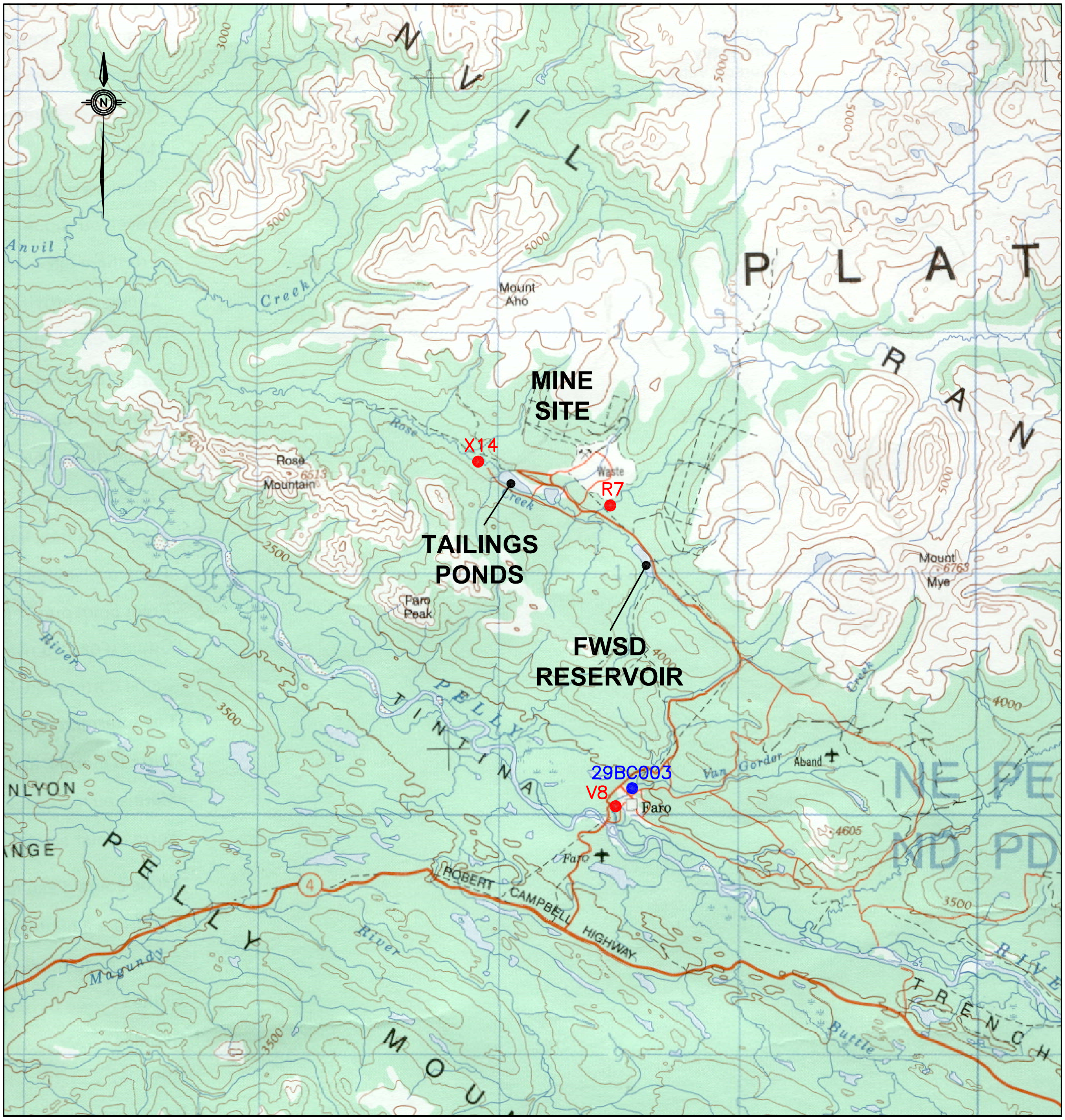
Table 14
Preliminary costs estimate for Scenario 3

CS 39 Headwall	Spillway	Outlet Channel	Fish By-Pass	Total Cost
$200,000	$31,100,000	$400,000	$900,000	$32,600,000

Notes:

1. Spillway costs include headworks, stepped & chute spillways, & stilling basin and are for structural concrete only.
2. Cost for seismic upgrading of Intermediate and Cross Valley Dams not included.

FIGURES



5 km 0 5 10 km

SCALE 1:250,000

LEGEND:

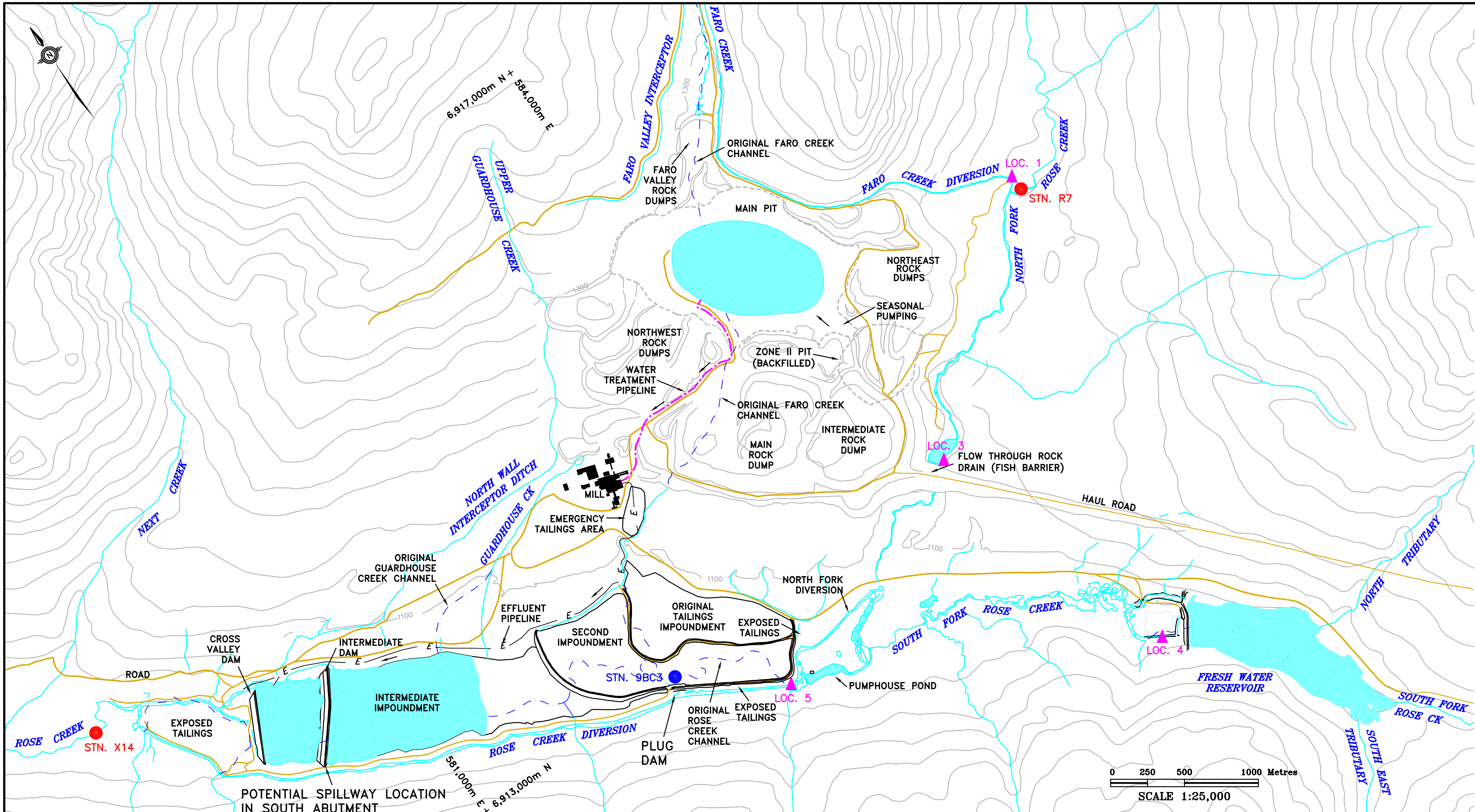
R7 GAUGING STATION – MINE SITE STAFF

29BC003 GAUGING STATION – DIAND

NOTES:

1. MAP SOURCE: NTS 105 K, EDITION 4
2. CONTOURS ARE IN FEET

SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
LOCATION PLAN		
Dwg. 6399–001	29 Oct 2003	**Figure 1**
northwest hydraulic consultants ltd.		



LEGEND:

ROADS	SURFACE WATER
EXISTING SURFACE DRAINAGE	STN. R7 — STREAMFLOW GAUGING STATION
PRE–MINE DRAINAGE	LOC. 1 — STREAM CATCHMENT LOCATION
E — EFFLUENT PIPELINE	STN. 9BC3 — DISCONTINUED WATER SURVEY OF CANADA STREAMFLOW GAUGING STATION 09BC003
WATER TREATMENT PIPELINE	

SOURCES OF INFORMATION:

1. DIGITAL COPY OF 1:50,000 TOPOGRAPHIC MAP SUPPLIED BY SRK CONSULTING.
2. MAP COORDINATES ARE UTM NAD83 ZONE 8; CONTOUR INTERVAL 100 FT.
3. FARO MINE DETAILS ADAPTED FROM DRAWINGS BY ROBERTSON GEOCONSULTANTS INC.

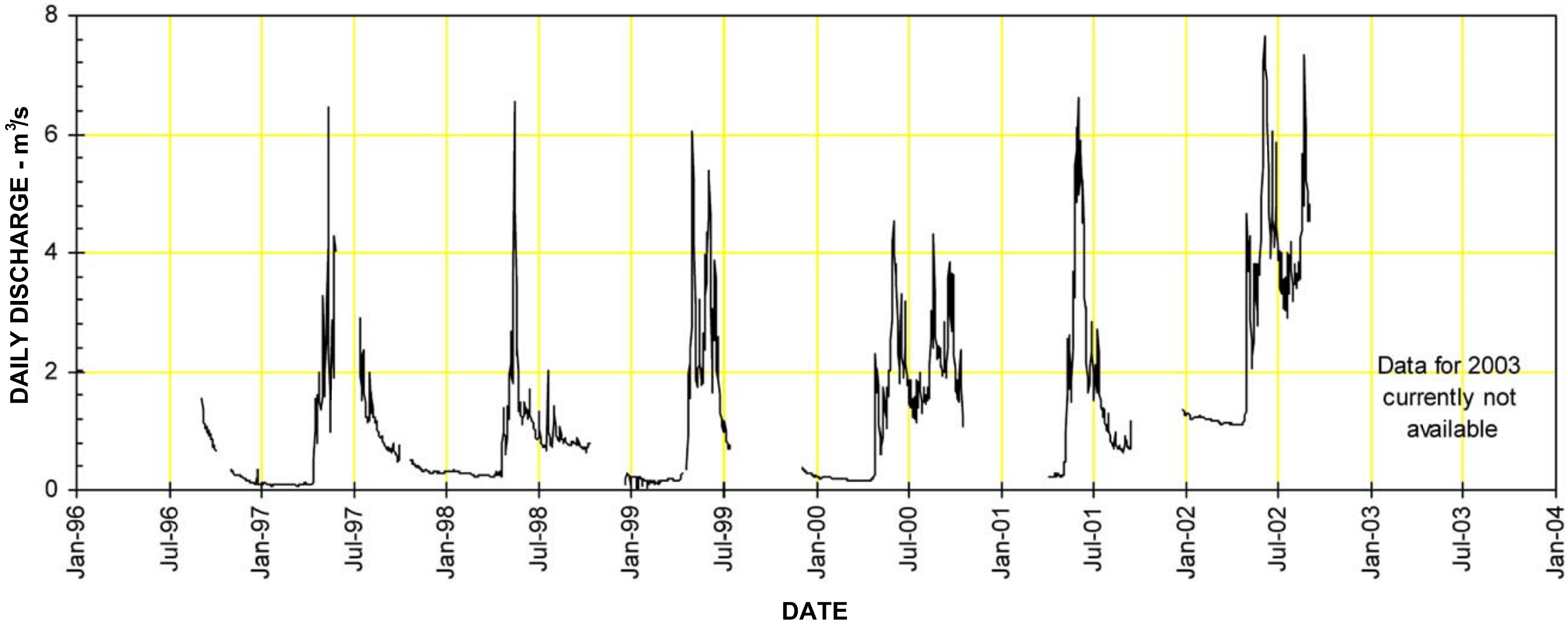
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

OVERVIEW OF FARO MINE SITE

Dwg. 6399–002	29 Oct 2003	**Figure 2**

northwest hydraulic consultants ltd.



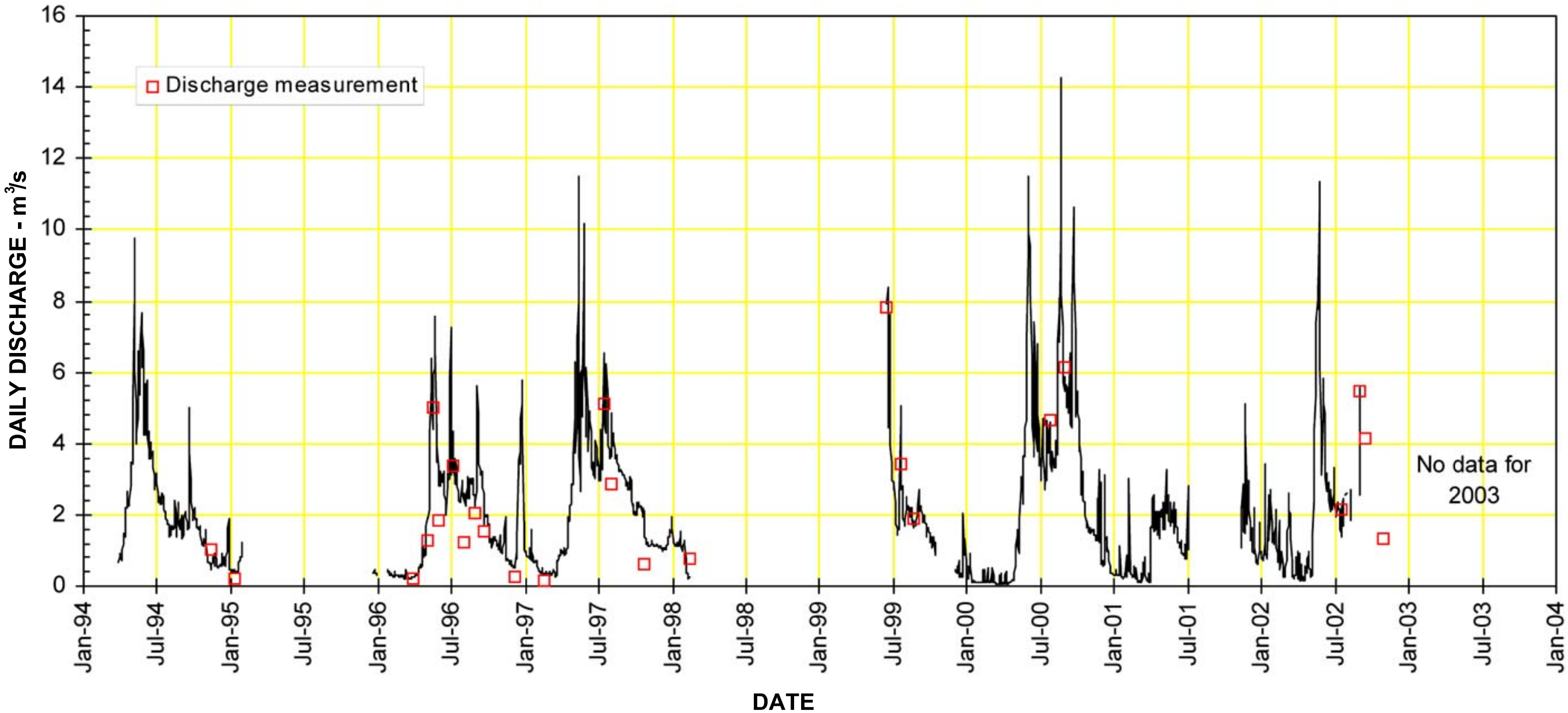
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

NORTH FORK ROSE CREEK HYDROGRAPH, STN. R7 1996-2002

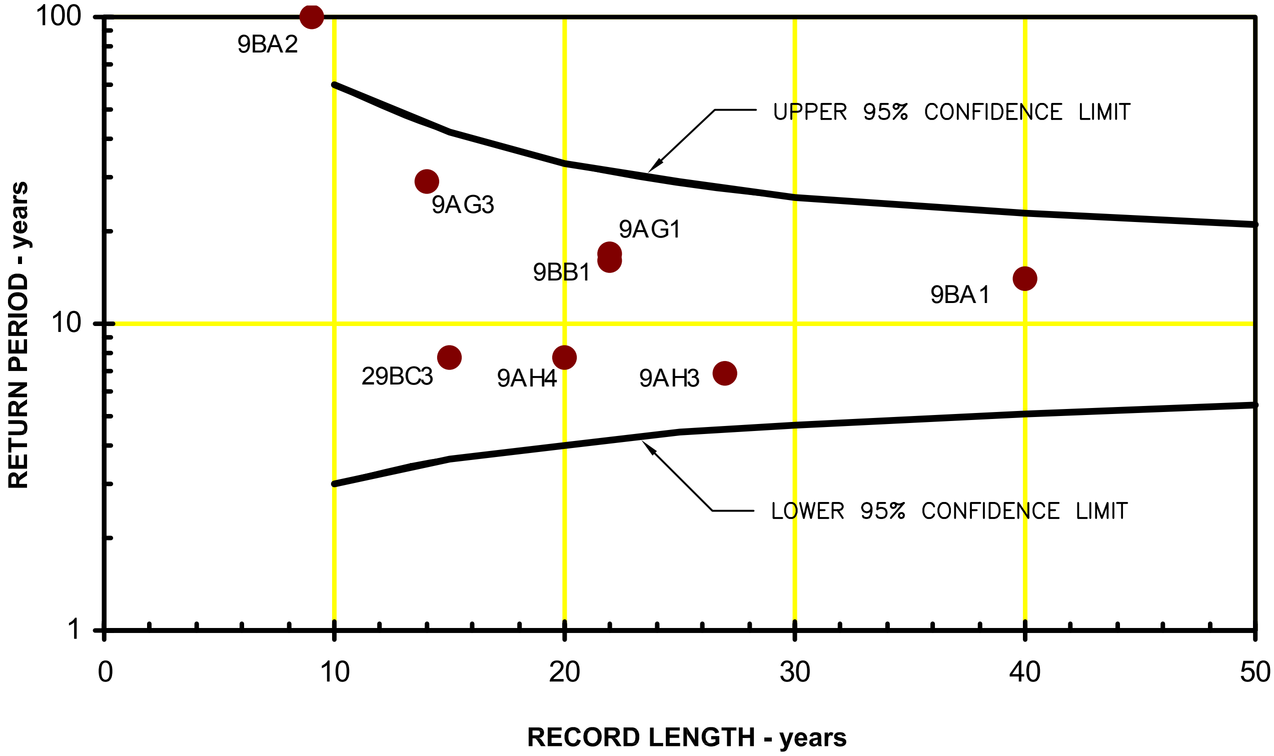
Dwg. 6399–003	29 Oct 2003	**Figure 3**

northwest hydraulic consultants ltd.



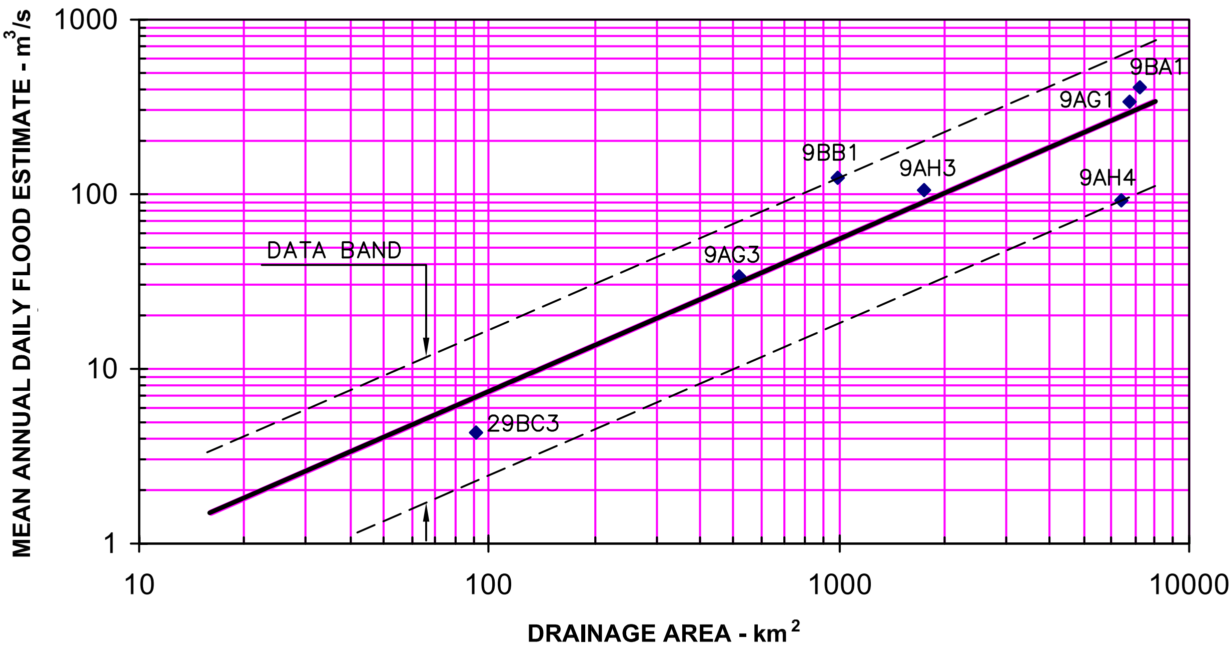
SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY **ROSE CREEK HYDROGRAPH, STN. X14 1994-2002**		
Dwg. 6399-004	29 Oct 2003	**Figure 4**
northwest hydraulic consultants ltd.		

HOMOGENEITY TEST FOR 10-YEAR FLOODS (3 LOGNORMAL)

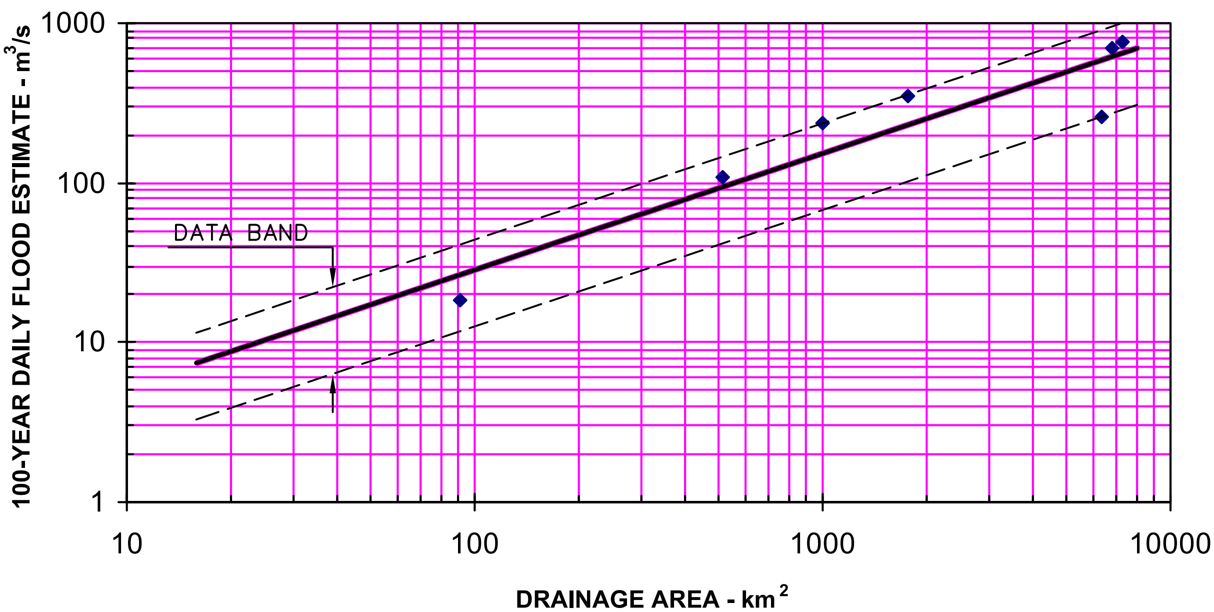


SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY **HOMOGENEITY TEST FOR GAUGING STATIONS NEAR FARO**		
Dwg. 6399-005	29 Oct 2003	**Figure 5**
northwest hydraulic consultants ltd.		

MEAN ANNUAL FLOOD (2.33-YEAR) VERSUS DRAINAGE AREA



100-YEAR FLOOD VERSUS DRAINAGE AREA



SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
MEAN ANNUAL & 100-YEAR FLOOD ESTIMATES VS DRAINAGE AREA FOR FARO REGION		
Dwg. 6399-006	29 Oct 2003	**Figure 6**
northwest hydraulic consultants ltd.		

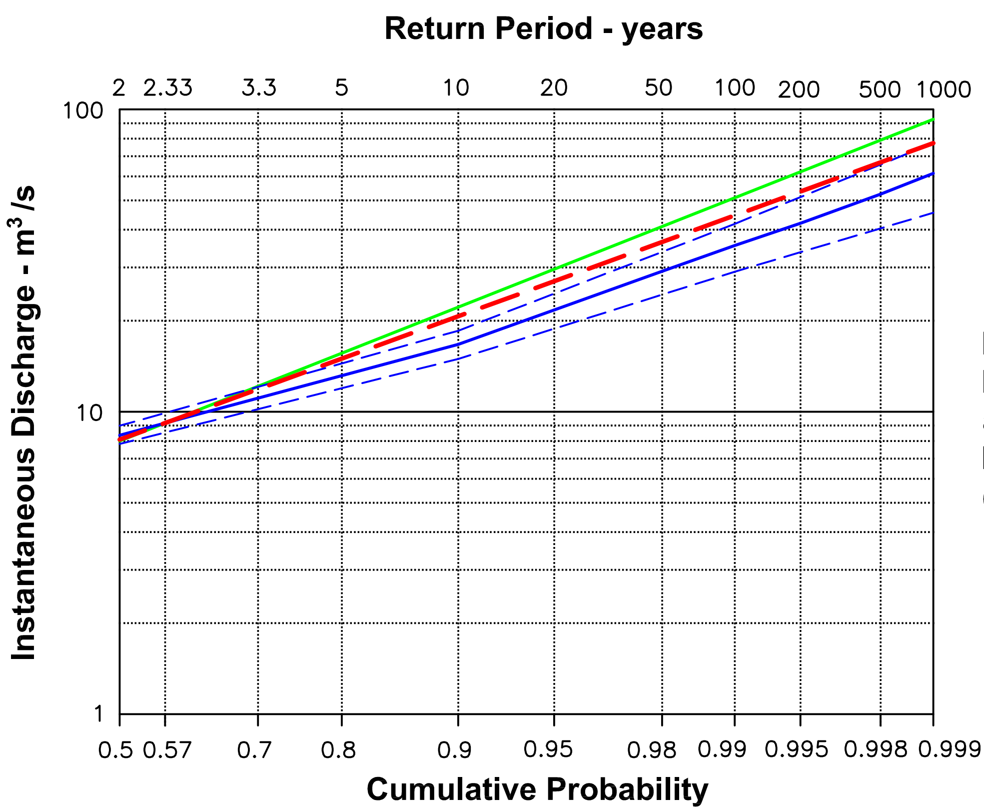


Fig. 7a
North Fork Rose Creek above Faro Creek Diversion Channel (Stn. R7)

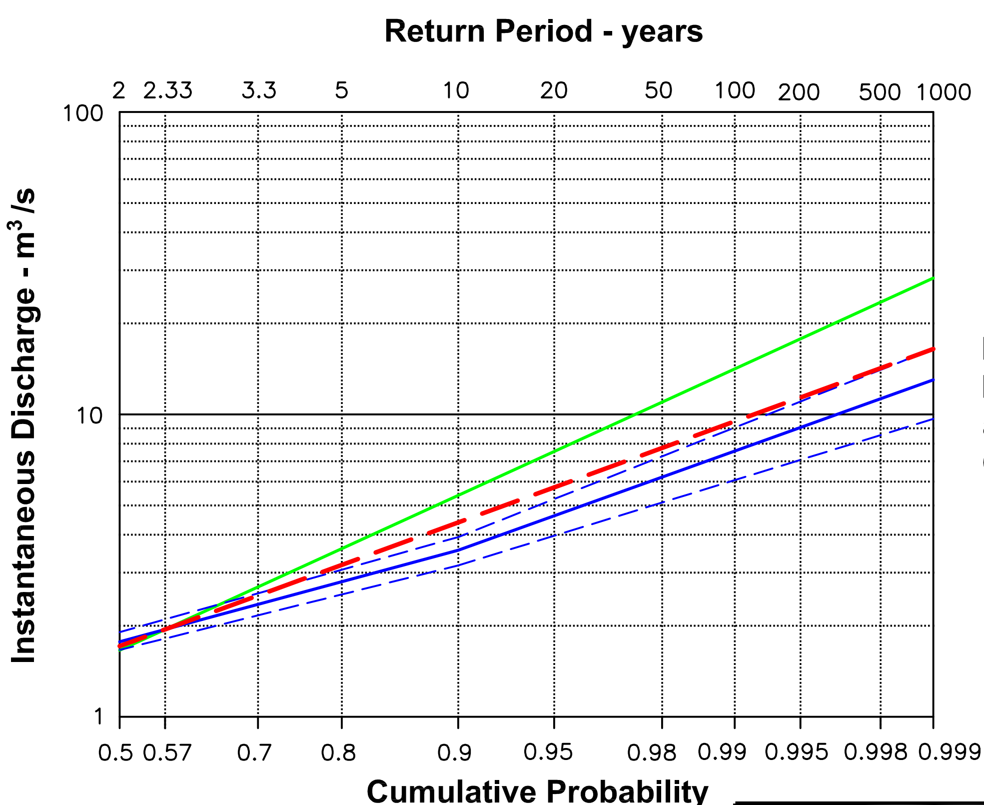


Fig. 7b
Faro Creek & Diversion above North Fork Rose Creek (Loc. 1)

LEGEND:

RECOMMENDED FLOOD FREQUENCY CURVE

METHOD 1 FLOOD FREQUENCY CURVE

METHOD 2 FLOOD FREQUENCY CURVE

UPPER AND LOWER 95% ERROR LIMITS (METHOD 2)

BGC ENGINEERING INC.		
FARO MINE HYDROLOGY **SYNTHESIZED FLOOD FREQUENCY PLOTS FOR MINE SITE**		
Dwg. 6399–010	5 Nov 2003	**Figure 7 a,b**
northwest hydraulic consultants ltd.		

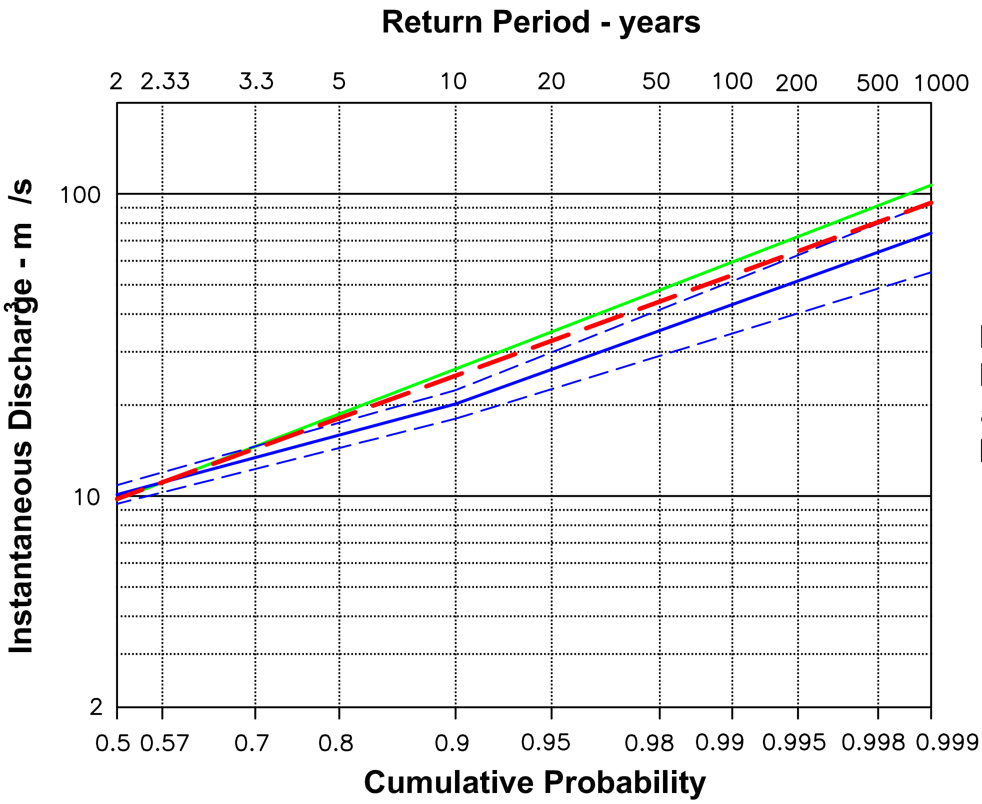


Fig. 7c
North Fork Rose Creek at Flow-through Rock Drain (Loc.3)

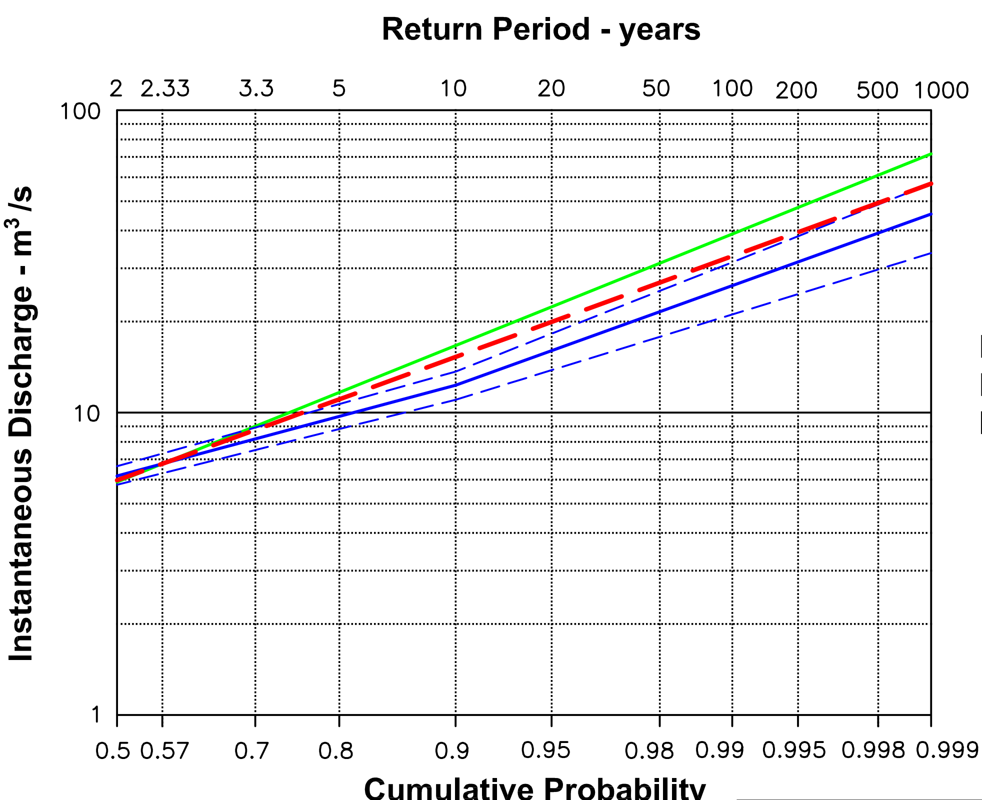


Fig. 7d
Fresh Water Supply Dam (FWSD) Catchment (Loc. 4)

LEGEND:

RECOMMENDED FLOOD FREQUENCY CURVE

METHOD 1 FLOOD FREQUENCY CURVE

METHOD 2 FLOOD FREQUENCY CURVE

UPPER AND LOWER 95% ERROR LIMITS (METHOD 2)

BGC ENGINEERING INC.		
FARO MINE HYDROLOGY **SYNTHESIZED FLOOD FREQUENCY PLOTS FOR MINE SITE**		
Dwg. 6399–010	5 Nov 2003	**Figure 7 c,d**
northwest hydraulic consultants ltd.		

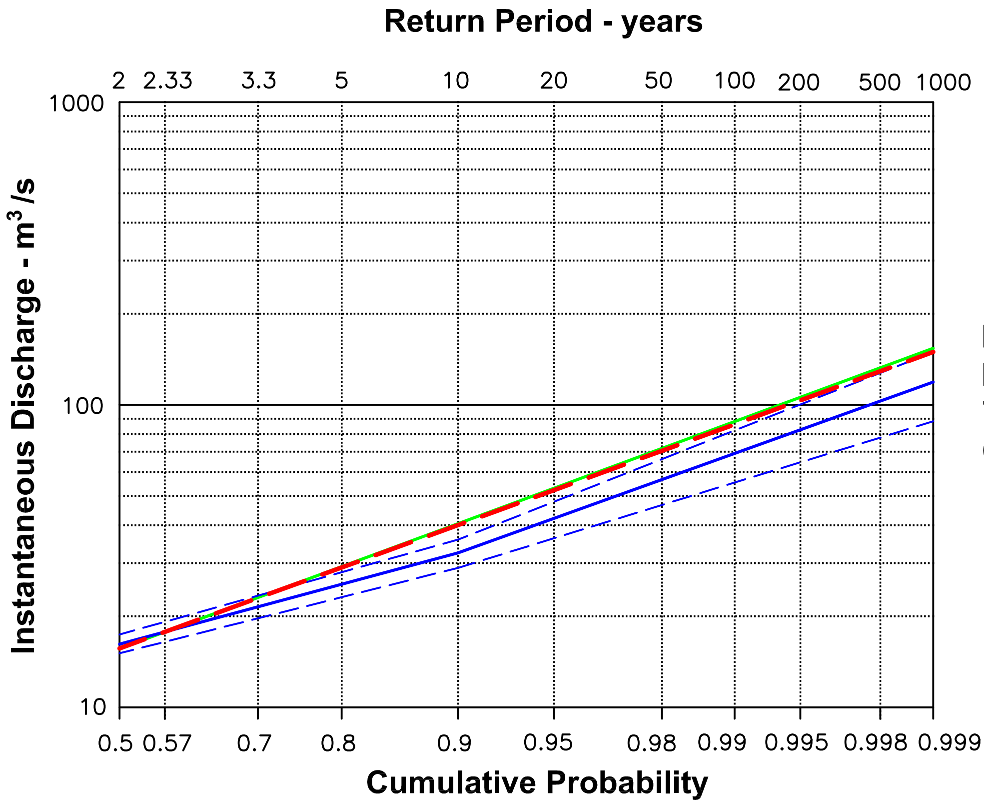


Fig. 7e
Rose Creek above Tailings Diversion Channel (Loc.5)

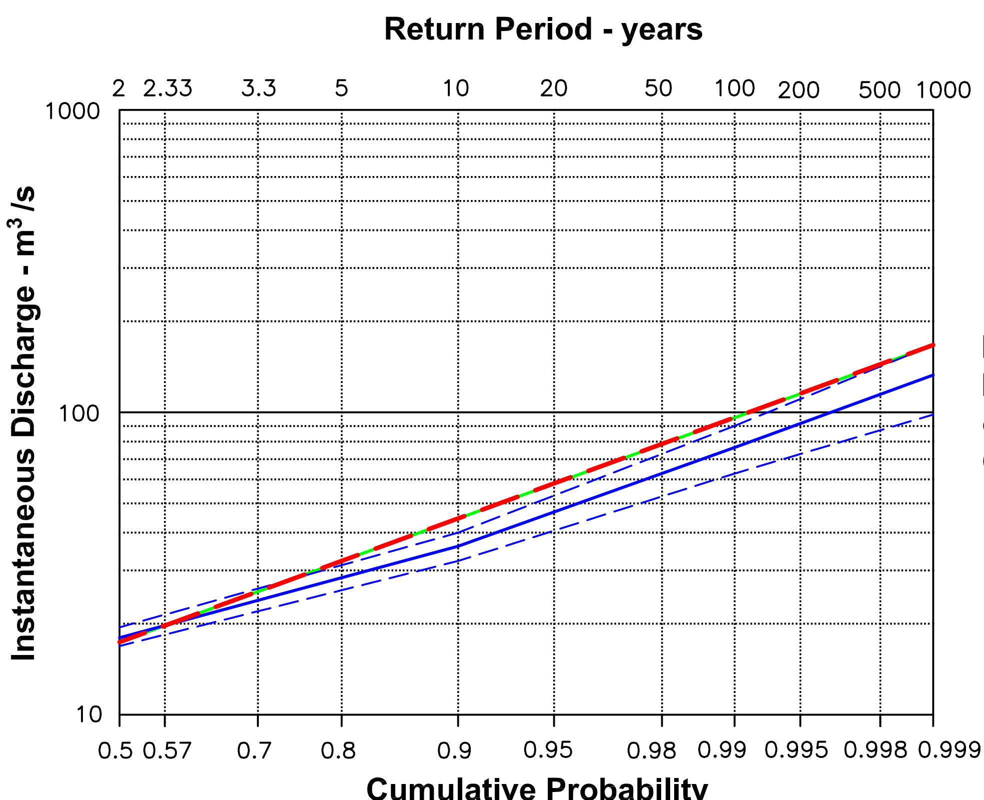


Fig. 7f
Rose Creek downstream of Tailings Diversion Channel (Stn. X14)

LEGEND:

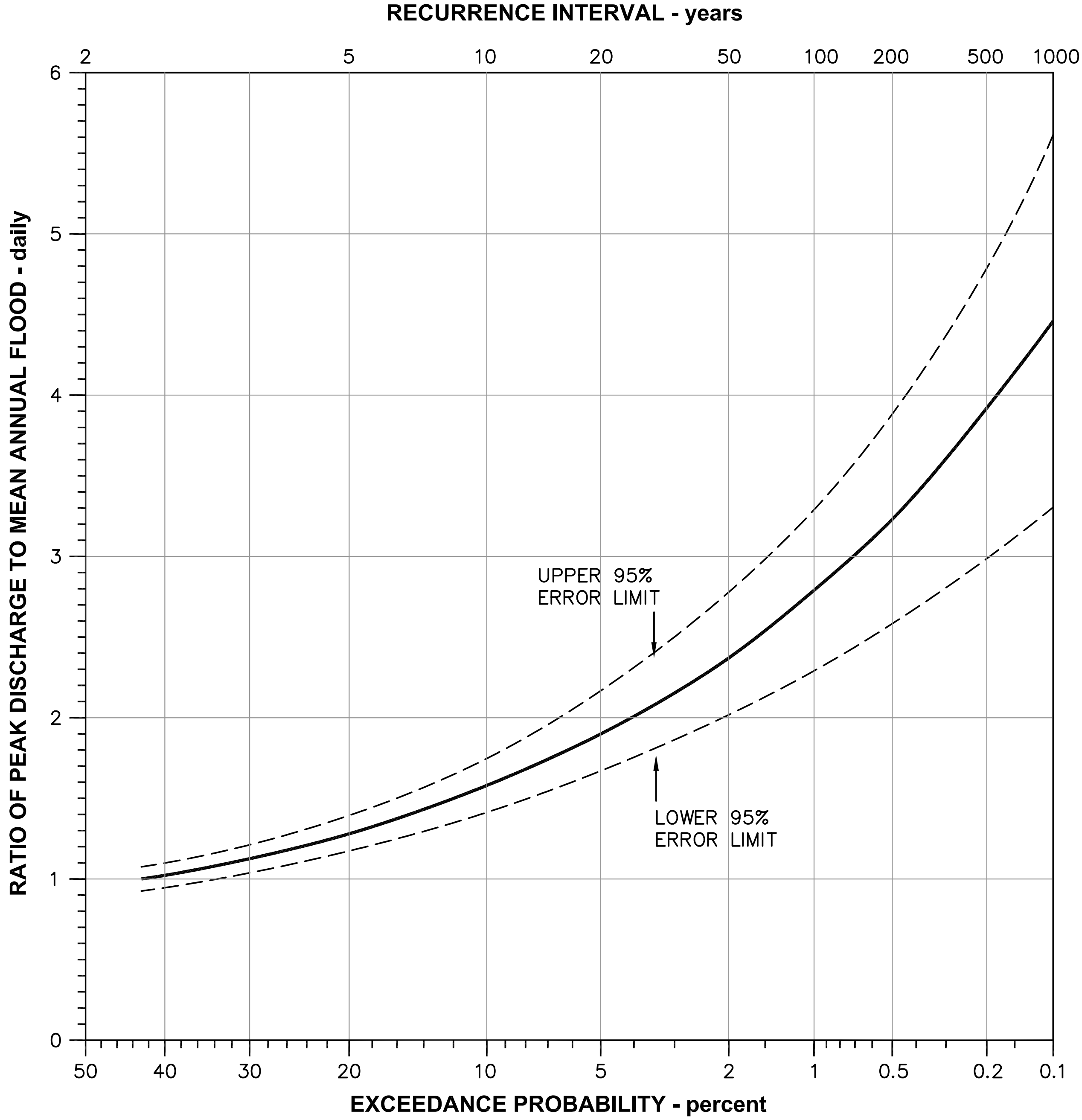
RECOMMENDED FLOOD FREQUENCY CURVE

METHOD 1 FLOOD FREQUENCY CURVE

METHOD 2 FLOOD FREQUENCY CURVE

UPPER AND LOWER 95% ERROR LIMITS (METHOD 2)

BGC ENGINEERING INC.		
FARO MINE HYDROLOGY **SYNTHESIZED FLOOD FREQUENCY PLOTS FOR MINE SITE**		
Dwg. 6399–010	5 Nov 2003	**Figure 7 e,f**
northwest hydraulic consultants ltd.		



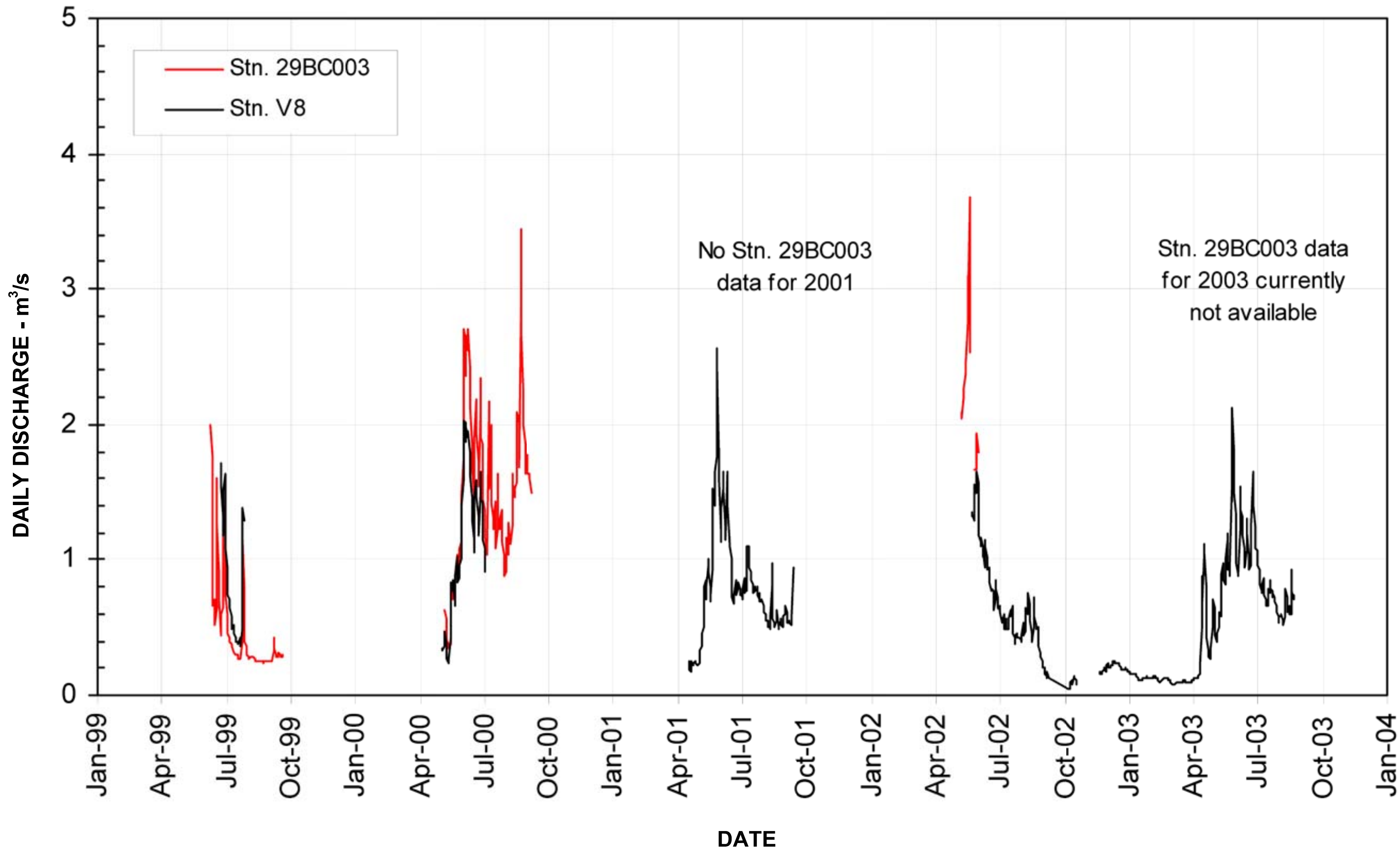
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

DIMENSIONLESS FLOOD FREQUENCY CURVE FOR FARO REGION

Dwg. 6399–011	5 Nov 2003	**Figure 8**

northwest hydraulic consultants ltd.



SRK CONSULTING/DELOITTE & TOUCHE

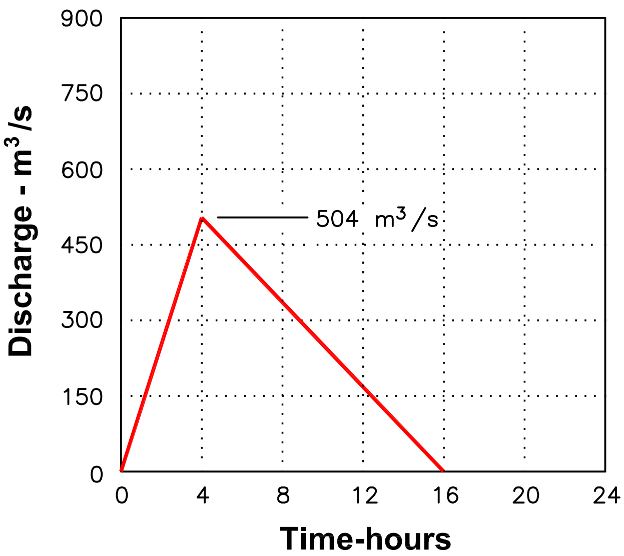
FARO MINE SITE HYDROTECHNICAL STUDY

COMPARISON OF VANGORD CREEK HYDROGRAPHS 1999-2003

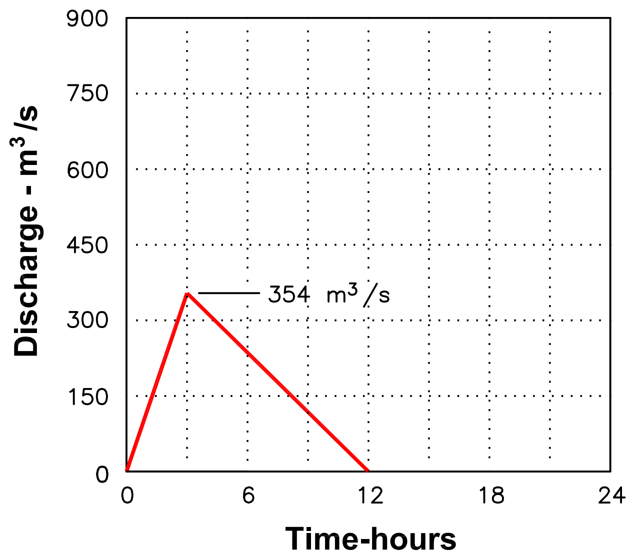
Dwg. 6399-009	29 Oct 2003	**Figure 9**

northwest hydraulic consultants ltd.

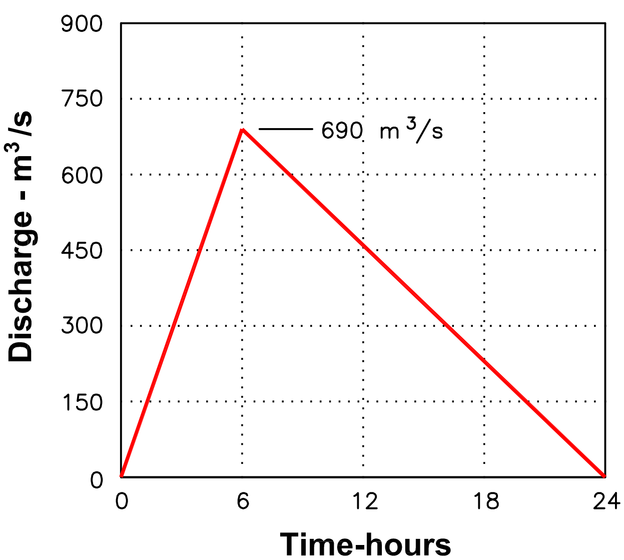
North Fork Rose Creek at Flow-through Rock Drain (Loc. 3)



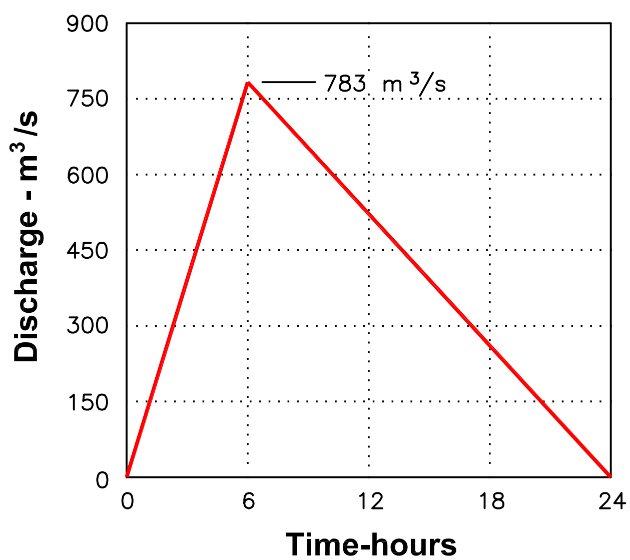
Fresh Water Supply Dam (FWSD) catchment (Loc. 4)



Rose Creek above Tailings Diversion Channel (Loc. 5)



Rose Creek downstream of Tailings Diversion Channel (Stn. X 14)



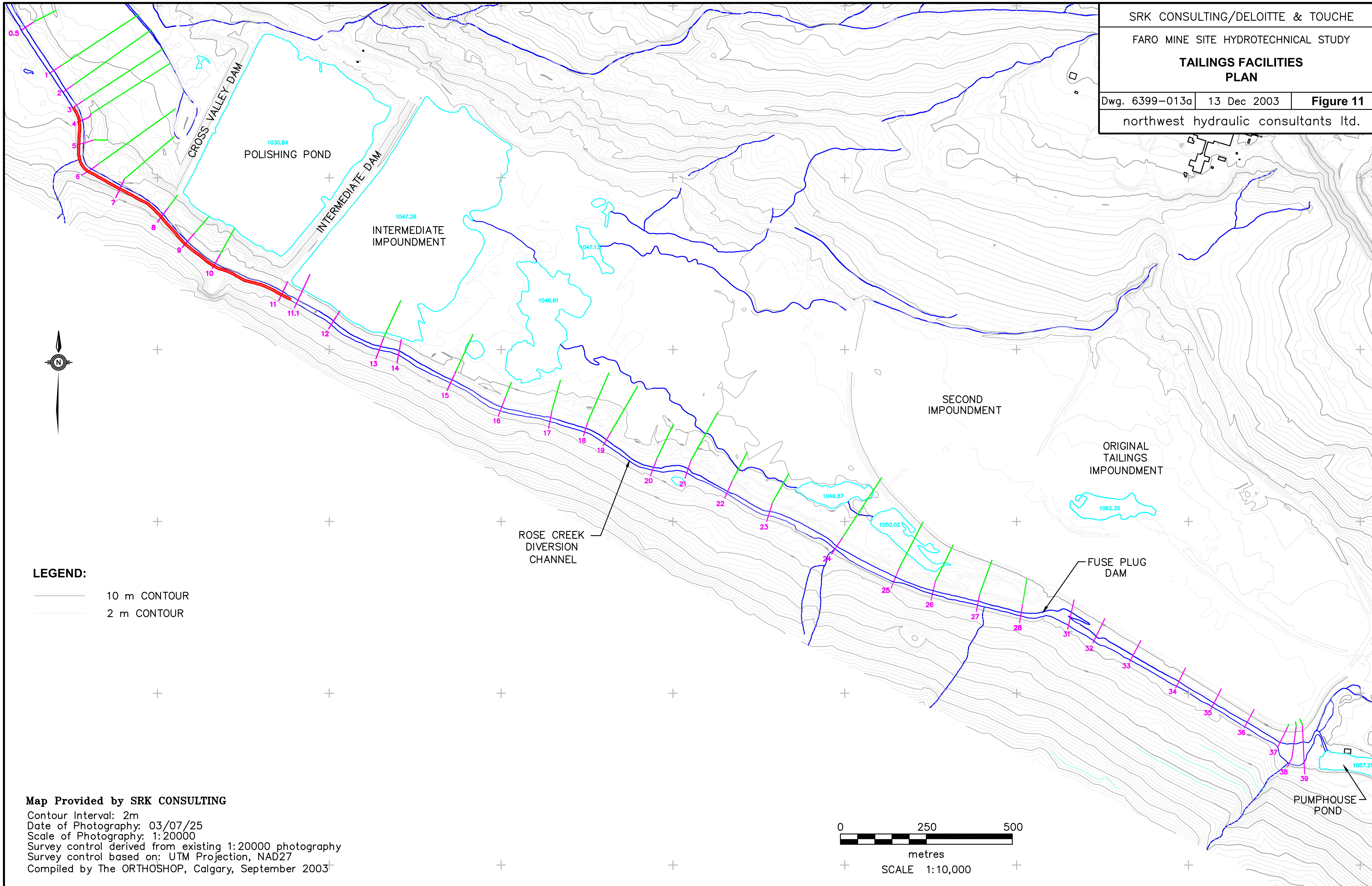
BGC ENGINEERING INC.

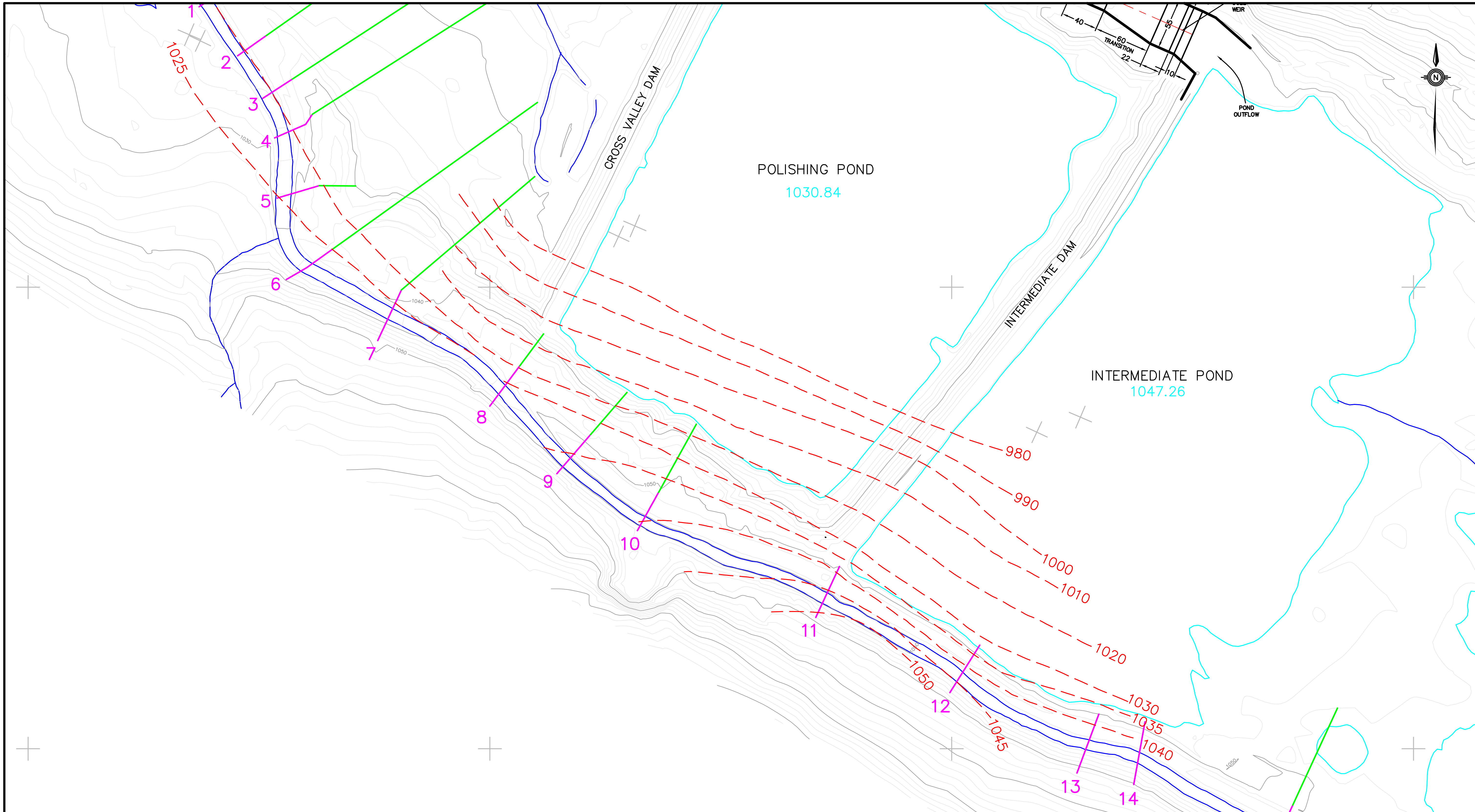
FARO MINE HYDROLOGY

PMF HYDROGRAPHS FOR MINE SITE

Dwg. 6399–015	19 Nov 2003	**Figure 10**

northwest hydraulic consultants ltd.


SRK CONSULTING/DELOITTE & TOUCHE
FARO MINE SITE HYDROTECHNICAL STUDY
TAILINGS FACILITIES
PLAN
Dwg. 6399-013a
13 Dec 2003
Figure 11
northwest hydraulic consultants ltd.
CROSS VALLEY DAM
POLISHING POND
INTERMEDIATE DAM
INTERMEDIATE IMPOUNDMENT
SECOND IMPOUNDMENT
ORIGINAL TAILINGS IMPOUNDMENT
ROSE CREEK DIVERSION CHANNEL
FUSE PLUG DAM
PUMPHOUSE POND
LEGEND:
10 m CONTOUR
2 m CONTOUR
Map Provided by SRK CONSULTING
Contour Interval: 2m
Date of Photography: 03/07/25
Scale of Photography: 1:20000
Survey control derived from existing 1:20000 photography
Survey control based on: UTM Projection, NAD27
Compiled by The ORTHOSHOP, Calgary, September 2003
0
250
500
metres
SCALE 1:10,000



0 100 200
metres
SCALE 1:4000

LEGEND:

1030 — — — GEODETIC ELEVATION OF BEDROCK SURFACE CONTOUR

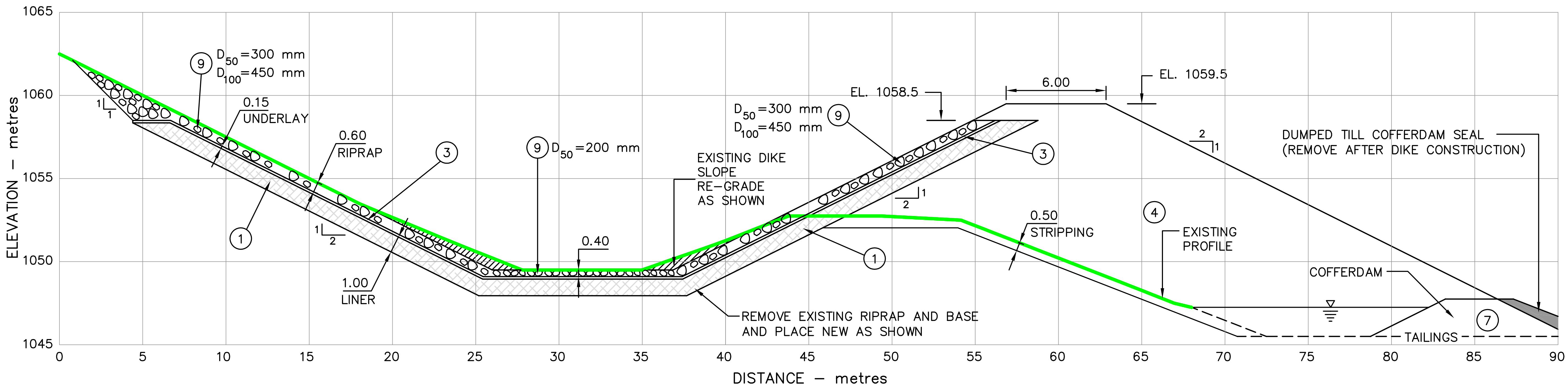
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

BEDROCK SURFACE CONTOUR MAP

Dwg. 6399–013	12 Dec 2003	**Figure 12**

northwest hydraulic consultants ltd.



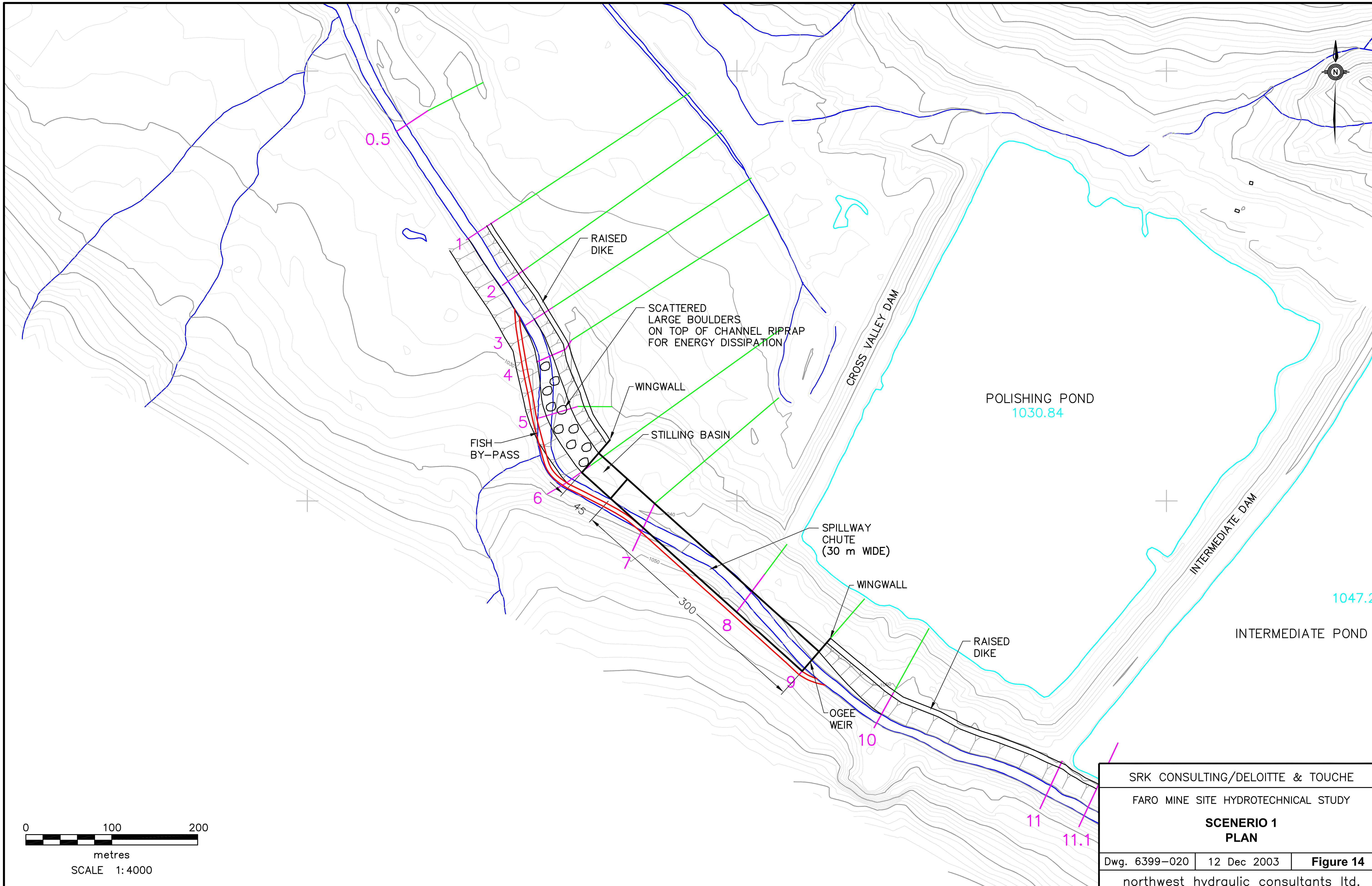
LEGEND:

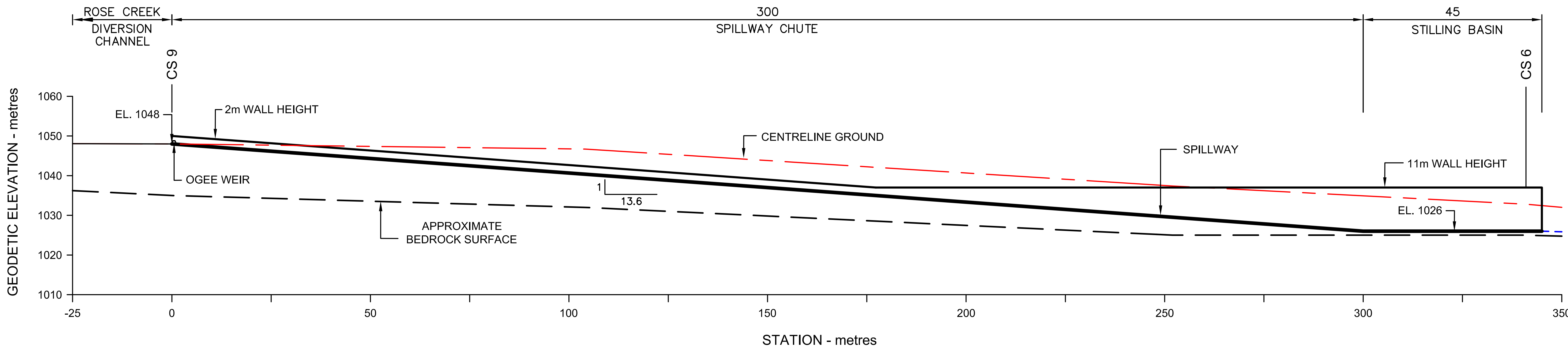
1. IMPERVIOUS CORE – COMPACTED TILL
3. RIPRAP BASE
4. COMPACTED SAND AND GRAVEL
7. FINE COMPACTED ROCK FILL (200 mm MIN.)
9. RIPRAP (D50/D100 TO SUIT LOCAL FLOW CONDITIONS)

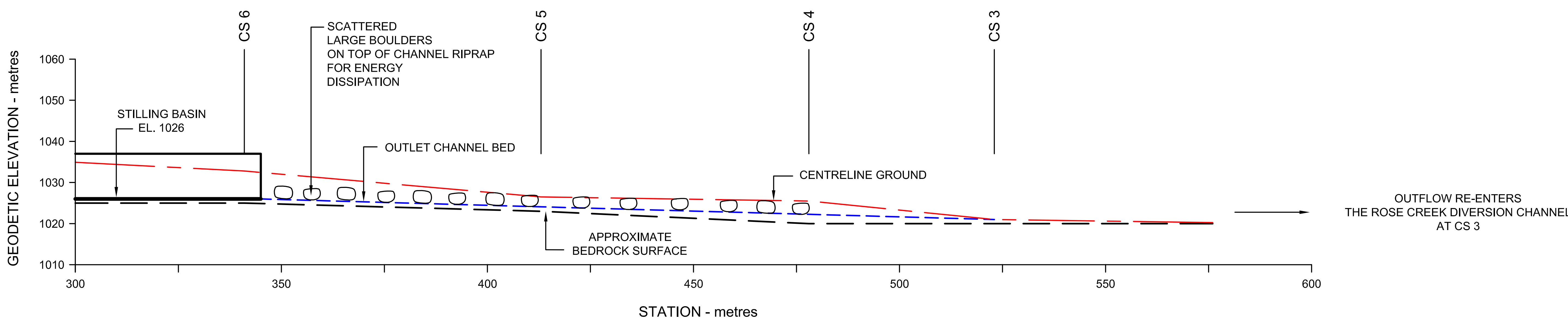
NOTES:

1. ALL DIMENSIONS ARE IN METRES UNLESS STATED OTHERWISE
2. ELEVATIONS FOR CS 13 ONLY
3. SCALE 1:250

SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY **SCENARIO 1** **CS 13** **(TYPICAL FOR CS 12-14)**		
Dwg. 6399–017	1 Dec 2003	**Figure 13**
northwest hydraulic consultants ltd.		


0.5
1
2
3
4
5
6
7
8
9
10
11
11.1
RAISED DIKE
SCATTERED LARGE BOULDERS ON TOP OF CHANNEL RIPRAP FOR ENERGY DISSIPATION
WINGWALL
STILLING BASIN
FISH BY-PASS
45
300
SPILLWAY CHUTE (30 m WIDE)
WINGWALL
RAISED DIKE
OGEE WEIR
CROSS VALLEY DAM
POLISHING POND
1030.84
INTERMEDIATE DAM
1047.26
INTERMEDIATE POND
N
0
100
200
metres
SCALE 1:4000
SRK CONSULTING/DELOITTE & TOUCHE
FARO MINE SITE HYDROTECHNICAL STUDY
SCENERIO 1
PLAN
Dwg. 6399-020
12 Dec 2003
Figure 14
northwest hydraulic consultants ltd.





SCALE 1:1000

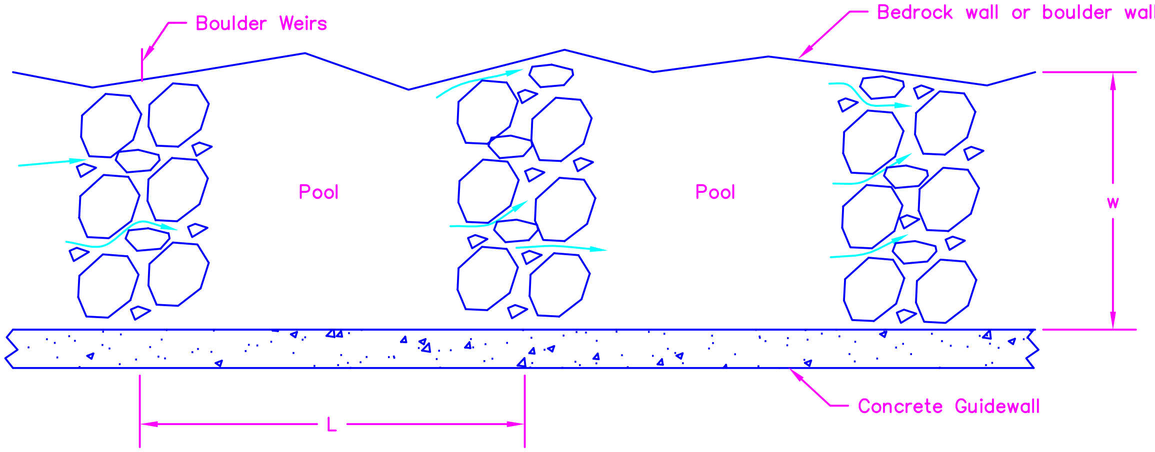
SRK CONSULTING/DELOITTE & TOUCHE

FAROMINE SITE HYDROTECHNICAL STUDY

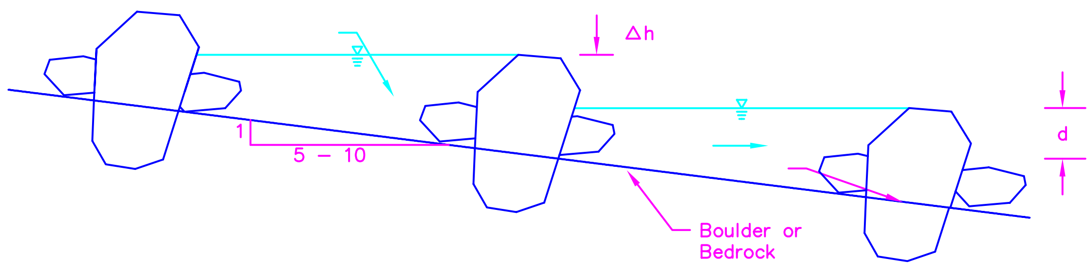
SCENERIO 1
SPILLWAY CENTRELINE PROFILE

Dwg. 6399–021	12 Dec 2003	**Figure 15**

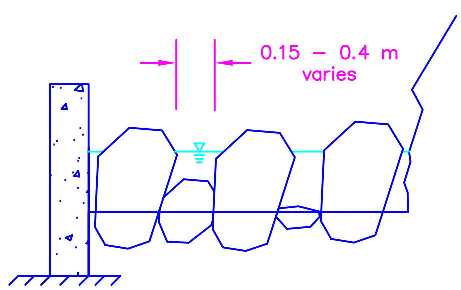
northwest hydraulic consultants ltd.



PLAN



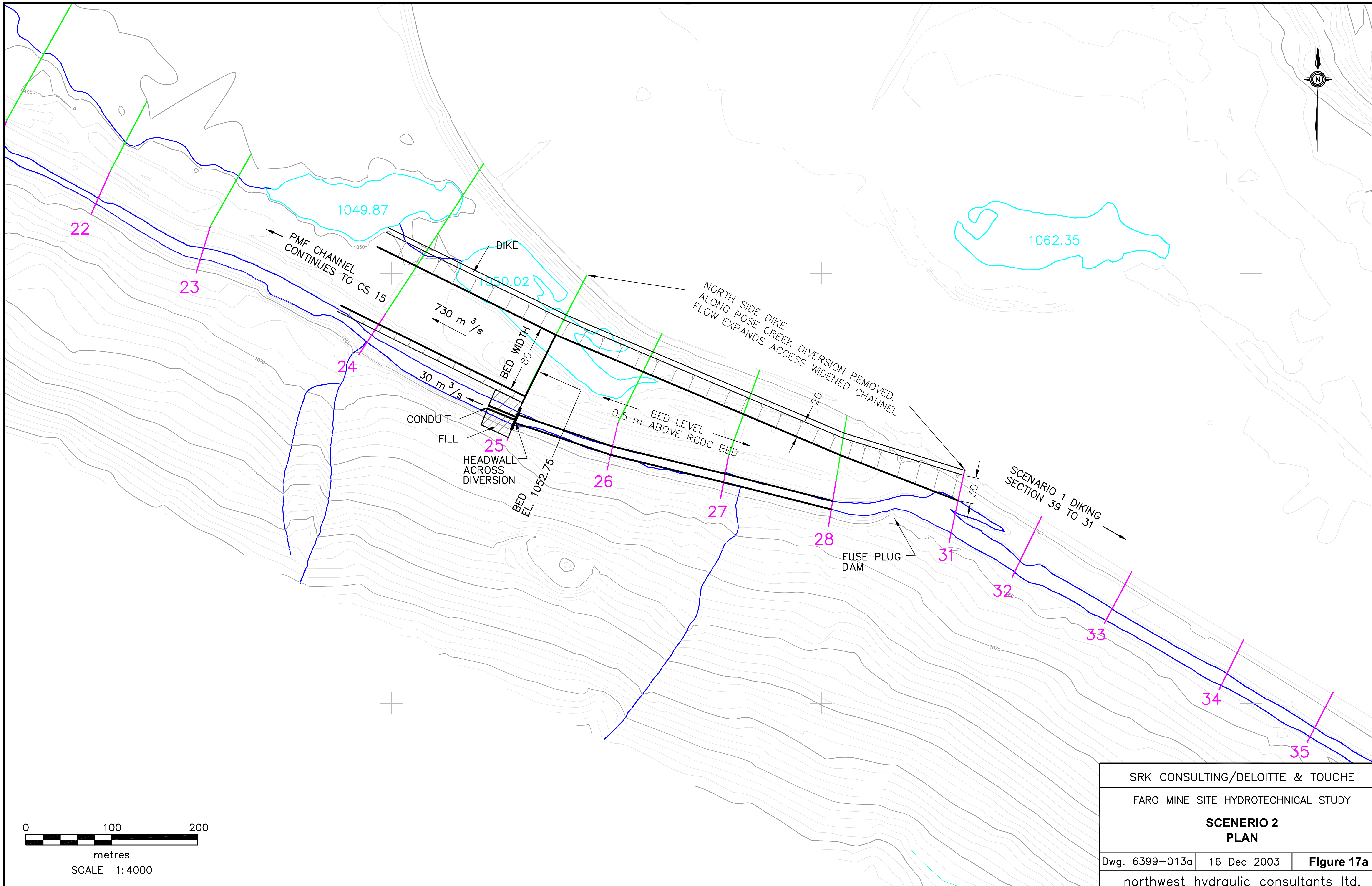
PROFILE



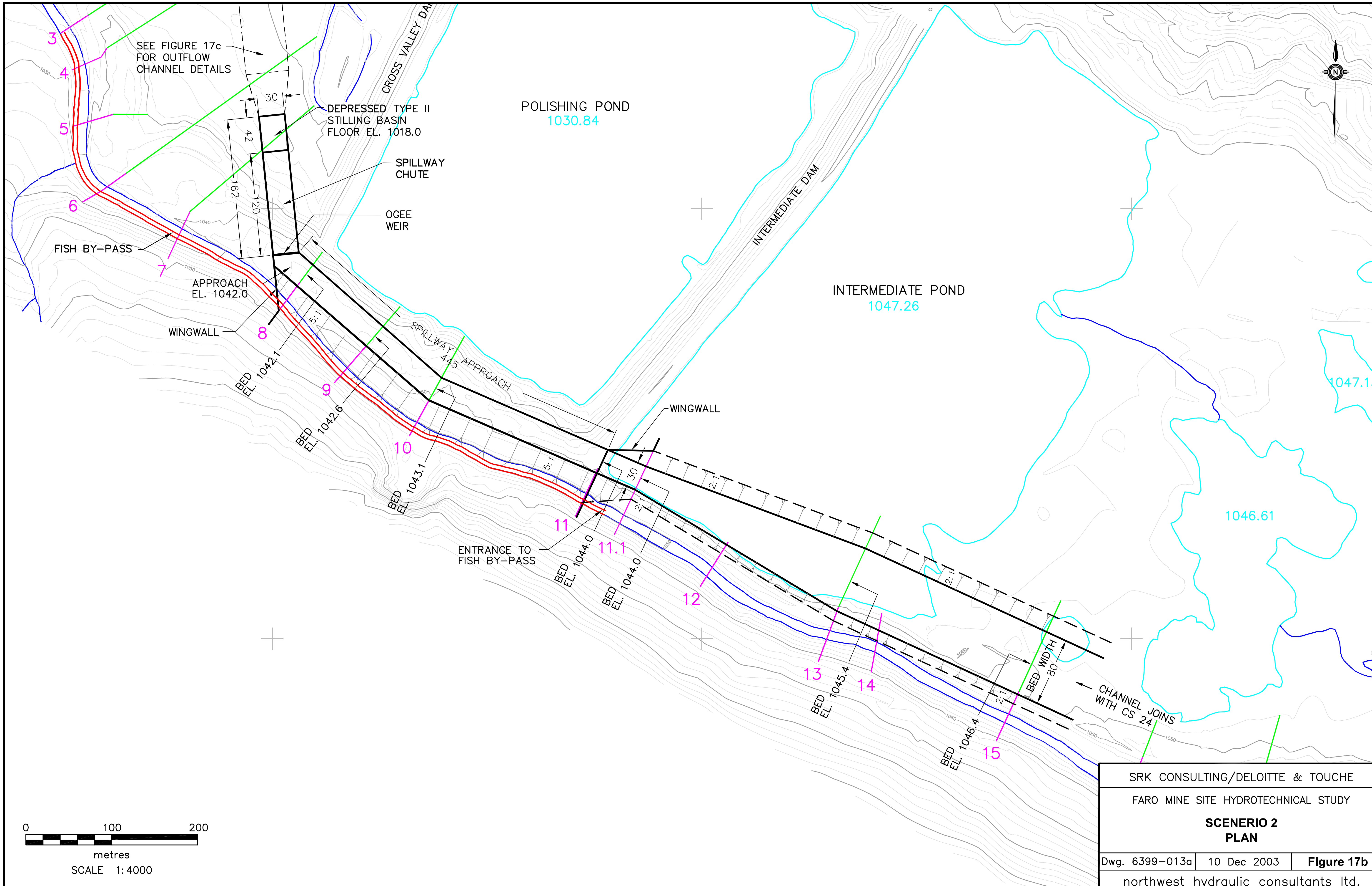
CROSS SECTION (D/S)

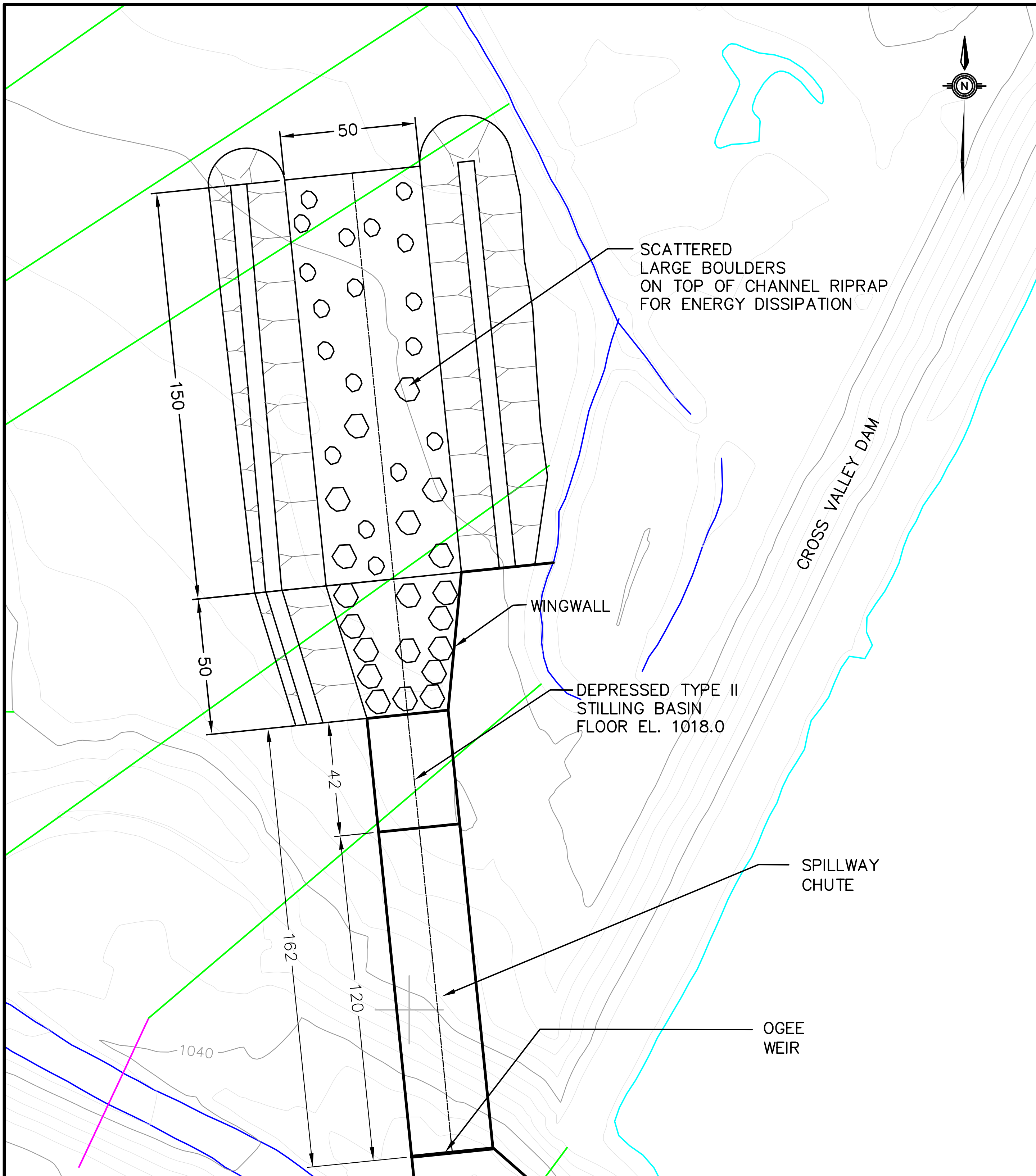
Notes:
1. w > 1.5 – 2.0 m
2. L > 1.5 w
3. Δ h ~ 0.3 – 0.5 m
4. d > 0.6 – 1.0 m

SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY **NATURAL CHANNEL FISHWAY** **STEP POOL DESIGN**		
Dwg. 6399–022	15 Dec 2003	**Figure 16**
northwest hydraulic consultants ltd.		



SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
SCENERIO 2 PLAN		
Dwg. 6399-013a	16 Dec 2003	**Figure 17a**
northwest hydraulic consultants ltd.		


SEE FIGURE 17c FOR OUTFLOW CHANNEL DETAILS
CROSS VALLEY DAM
POLISHING POND
1030.84
DEPRESSED TYPE II STILLING BASIN FLOOR EL. 1018.0
SPILLWAY CHUTE
OGEE WEIR
FISH BY-PASS
APPROACH EL. 1042.0
WINGWALL
SPILLWAY APPROACH
445
INTERMEDIATE DAM
INTERMEDIATE POND
1047.26
1047.13
1046.61
WINGWALL
BED EL. 1042.1
BED EL. 1042.6
BED EL. 1043.1
BED EL. 1044.0
BED EL. 1044.0
BED EL. 1045.4
BED EL. 1046.4
ENTRANCE TO FISH BY-PASS
BED WIDTH 80
CHANNEL JOINS WITH CS 24
3
4
5
6
7
8
9
10
11
11.1
12
13
14
15
30
42
162
120
5:1
2:1
0 100 200
metres
SCALE 1:4000
SRK CONSULTING/DELOITTE & TOUCHE
FARO MINE SITE HYDROTECHNICAL STUDY
SCENERIO 2
PLAN
Dwg. 6399–013a
10 Dec 2003
Figure 17b
northwest hydraulic consultants ltd.



0 50 100

metres

SCALE 1:2000

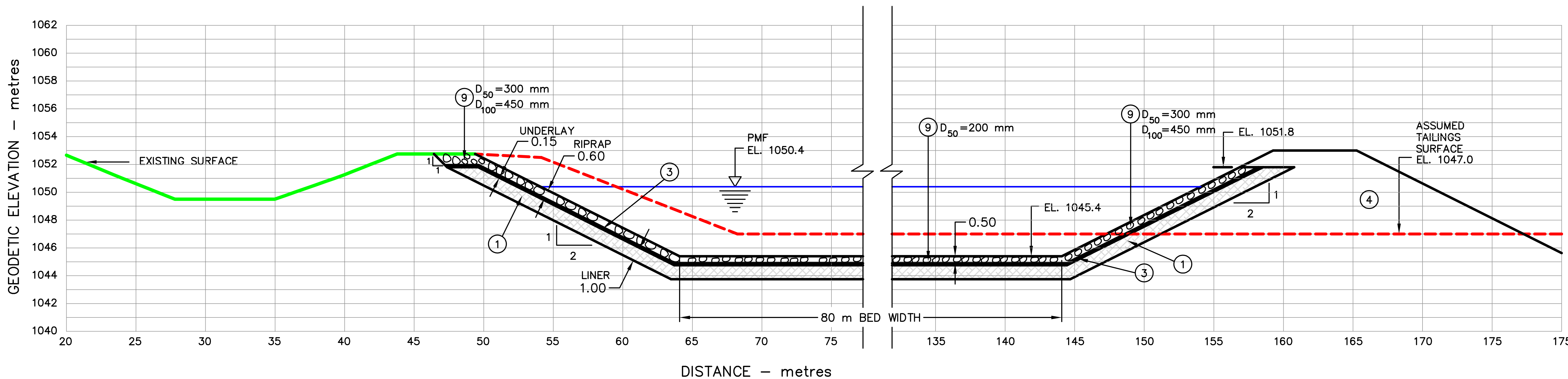
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

SCENERIO 2
PLAN

Dwg. 6399–018	10 Dec 2003	**Figure 17c**

northwest hydraulic consultants ltd.



LEGEND:

1. IMPERVIOUS CORE – COMPACTED TILL
3. RIPRAP BASE
4. COMPACTED SAND AND GRAVEL
7. FINE COMPACTED ROCK FILL (200 mm MIN.)
9. RIPRAP (D_{50}/D_{100} TO SUIT LOCAL FLOW CONDITIONS)

NOTES:

1. ALL DIMENSIONS ARE IN METRES UNLESS STATED OTHERWISE
2. ELEVATIONS FOR CS 13 ONLY
3. SCALE 1:300

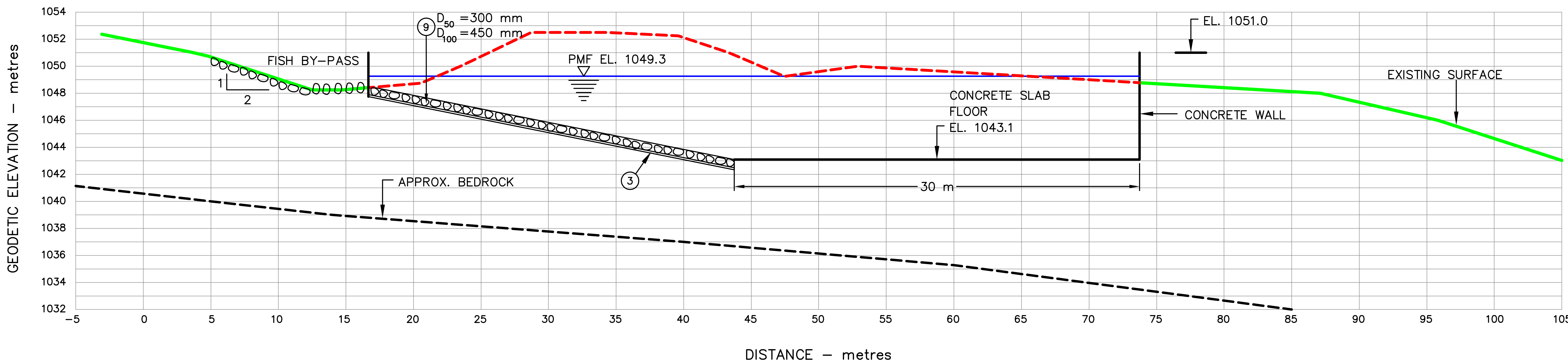
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

SCENARIO 2
CS 13
(TYPICAL FOR CS 12-14)

Dwg. 6399–017	15 Dec 2003	**Figure 18**

northwest hydraulic consultants ltd.



LEGEND:

1 IMPERVIOUS CORE – COMPACTED TILL

3 RIPRAP BASE

4 COMPACTED SAND AND GRAVEL

7 FINE COMPACTED ROCK FILL (200 mm MIN.)

9 RIPRAP (D_{50}/D_{100} TO SUIT LOCAL FLOW CONDITIONS)

NOTES:

1. ALL DIMENSIONS ARE IN METRES UNLESS STATED OTHERWISE
2. SCALE 1:300

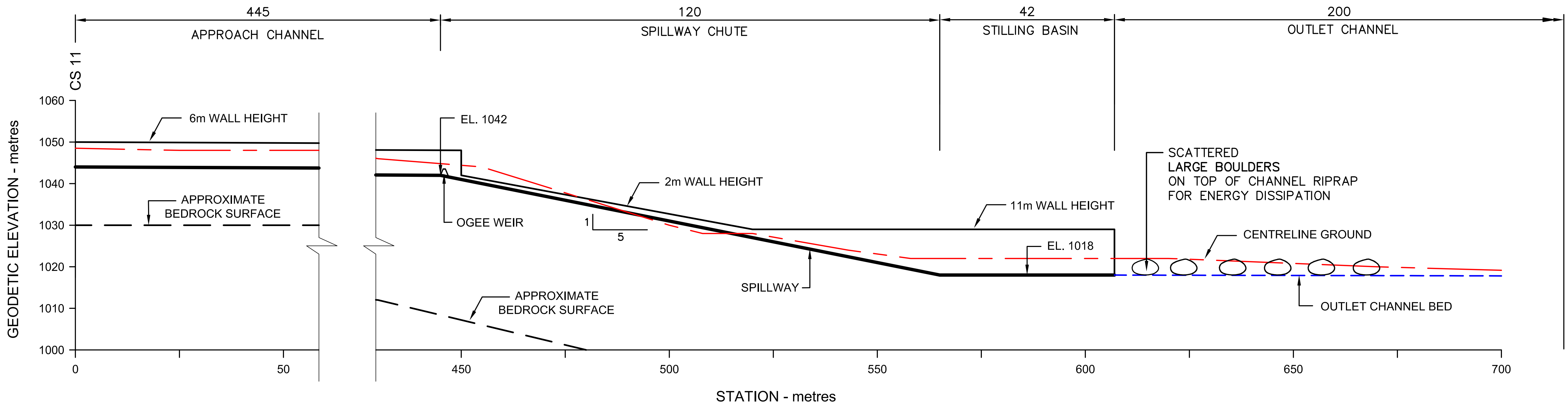
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

SCENARIO 2
CS 10

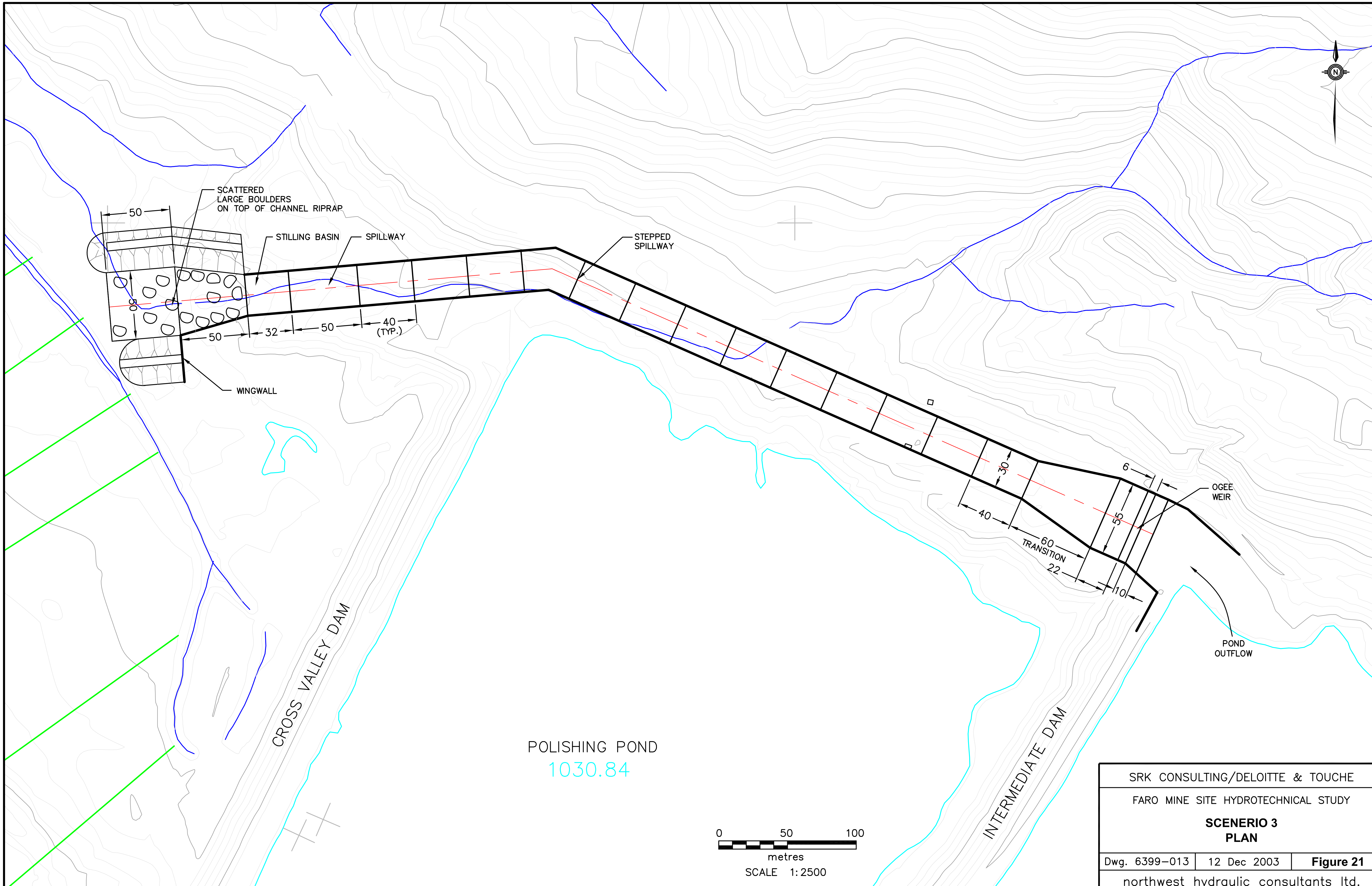
Dwg. 6399–017	16 Dec 2003	**Figure 19**

northwest hydraulic consultants ltd.



SCALE 1:1000

SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
SCENERIO 2 **SPILLWAY CENTRELINE PROFILE**		
Dwg. 6399–019	16 Dec 2003	**Figure 20**
northwest hydraulic consultants ltd.		



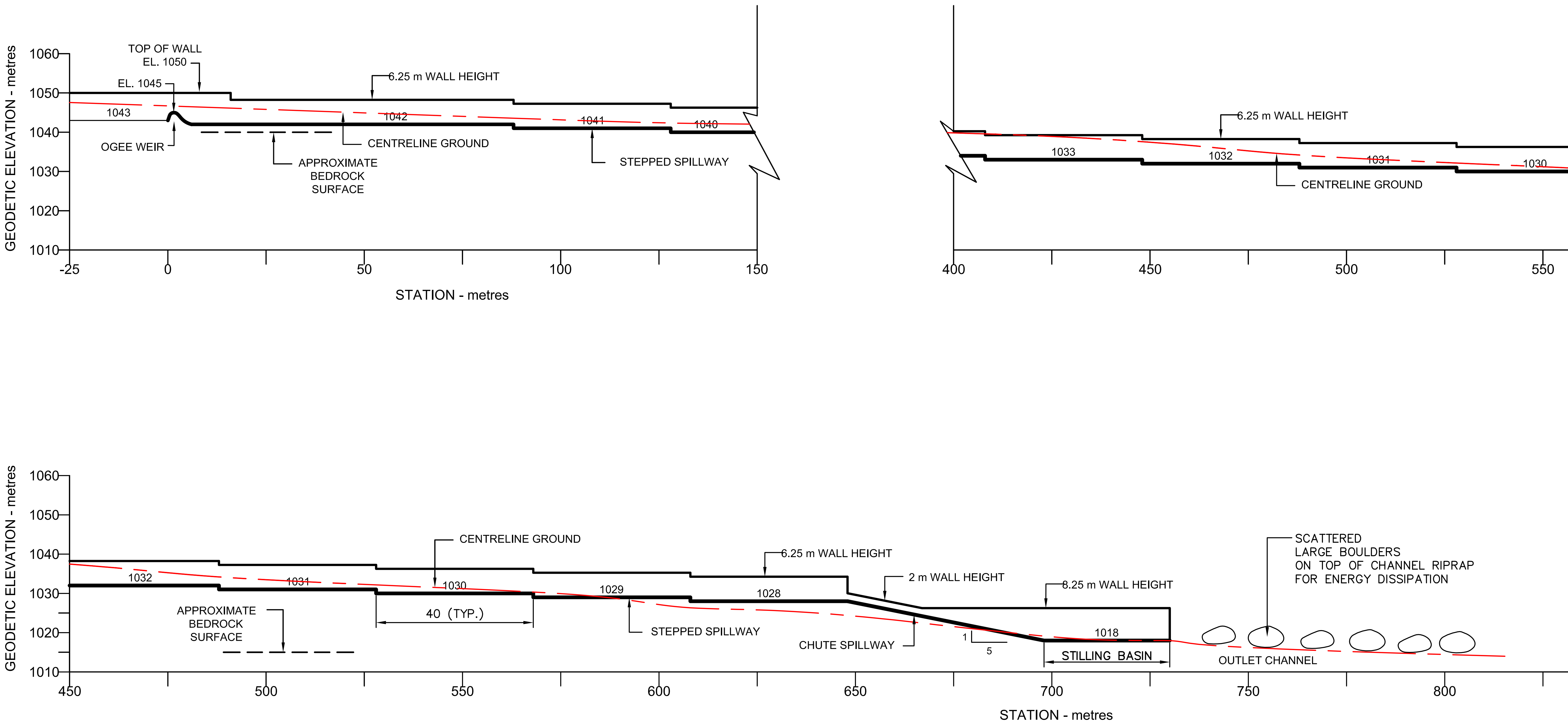
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

SCENERIO 3
PLAN

Dwg. 6399–013	12 Dec 2003	**Figure 21**

northwest hydraulic consultants ltd.



SCALE 1:1000

SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

SCENERIO 3
SPILLWAY CENTRELINE PROFILE

Dwg. 6399-701	15 Dec 2003	**Figure 22**

northwest hydraulic consultants ltd.

PHOTOGRAPHS

FARO MINE SITE AREA STREAMS

Photo 1. Overview of the Faro Creek Diversion flowing towards the confluence with the North Fork Rose Creek, which is visible just above the centre of the photo. Note the well established undergrowth and trees adjacent to the Diversion.

Photo 2. Faro Creek Diversion viewing upstream from the confluence with North Fork Rose Creek. The Faro Diversion is a braided channel flowing over a steep alluvial slope. This would be an impracticable reach for a gauging station.

Photo 3. Confluence of Faro Creek Diversion and North Fork Rose Creek. The photo shows the good vegetative cover adjacent to the creek.

Photo 4. North Fork Rose Creek viewing downstream to location of streamflow gauge Stn. R7. This is a good gauging site. Station is approximately 60 m upstream of the confluence with the Faro Creek Diversion.

Photo 5. North Fork Rose Creek viewing downstream approximately 80 m downstream of the confluence with the Faro Creek Diversion.

Photo 6. Overview of North Fork Rose Creek flowing towards the Haul Road, showing the well established undergrowth adjacent to most of the creek in this reach.

Photo 7. Upstream view of North Fork Rose Creek from the Haul Road with water ponded at entrance to flow-through rock drain. Mine rock dumps are visible along the left side of the photo.

Photo 8. Downstream view of North Fork Rose Creek from the haul road. Water exiting flow-through rock drain. The fresh water supply dam (FWSD) visible in the top lefthand corner of photo.

Photo 9. The Fresh Water Supply Dam (FWSD) reservoir. The water level was at El. 1085.14 at 12:15 h on 25 September 2003. This photograph was taken at approximately 17:00 h on the 25 September.

Photo 10. The Fresh Water Supply Dam (FWSD) spillway showing the syphon pipes used to lower the reservoir water level.

Photo 11. Riparian outflow channel from the Fresh Water Supply Dam (FWSD) to South Fork Rose Creek, with the valve house in the centre of the photo. A notch will be cut through the embankment to the left of the photo to permanently lower reservoir water levels.

Photo 12. Downstream view of North Fork Rose Creek from the Haul Road.
The original tailings impoundment is visible towards the top righthand corner of photo.
The photo shows extensive tree cover on the sloping terrain towards the top of the photo.

Photo 13. North Fork Rose Creek flowing through the 14 ft (4.3 m) culvert through the main access road to the Mine Site. The Mine Haul Road is in the background.

Photo 14. View across North Fork Rose Creek to the side overflow weir in the centre of the photo, that allows excess flow to enter the North Fork Diversion. Creek flow passes into the downstream series of four Recharge Ponds and finally the Pumphouse Pond.

Photo 15. This shows North Fork Rose Creek entering Recharge Pond # 1 towards the top of the photo.

Photo 16. View downstream across Recharge Pond # 2 with Ponds # 3 & 4 in the background. The outflow from Pond # 4 enters South Fork Rose Creek above the Pumphouse Pond.

Photo 17. Rose Creek flowing into the Pumphouse Pond. The confluence of North Fork Rose Creek flow from Recharge Pond # 4 and South Fork flow from the FWSD is about 200 m upstream of the Pumphouse Pond. The photo shows the dense vegetative cover along the creek and on adjacent slopes.

Photo 18. The start Impoundment.of Rose Creek Diversion viewing upstream towards the Pumphouse Pond approximately 200 m away. The photo is taken from the embankment that separates the Diversion from the Second Tailings

Photo 19. Rose Creek Diversion viewing downstream from start of the Diversion. Vegetation is well established along the far bank of the diversion channel and the moderately sloped hillside behind.

Photo 20. View to the northwest across the upper part of the Second Tailings Impoundment from the start of the Rose Creek Diversion channel.

Photo 21. Rose Creek Diversion viewing downstream (northwest) to the Plug Dam, approximately 800 m downstream of the start of the Diversion.

Photo 22. View downstream (northwest) to the Plug Dam. Rose Creek Diversion is to the left, and the Second Impoundment is visible on the right of the photo.

Photo 23. View upstream from the south abutment of the Intermediate Dam. The Rose Creek Diversion is to the right of the photo.

Photo 24. Rose Creek Diversion viewing upstream adjacent to the Cross-Valley Dam.

Photo 25. Rose Creek Diversion viewing downstream adjacent to the Cross-Valley Dam.

Photo 26. Rose Creek viewing upstream at gauging Stn. X14, downstream of Rose Creek Diversion and the tailings pond complex.

Photo 27. Vangorda Creek streamflow Stn. V8 viewing upstream. This gauge is approximately 500 m downstream from the DIAND streamflow Stn. 29BC003.

Photo 28. View upstream to the footbridge over Vangorda Creek where water samples are collected. The footbridge is approximately 60 m downstream of gauging Stn. V8.

APPENDIX A

FLOOD FREQUENCY ANALYSIS OF STREAMFLOW GAUGING STATION DATA IN THE FARO REGION

Flood Frequency Plot
Vangorda Creek 29BC003 (1977, 84-85, 89-98, 2000, 2002)
3 Parameter Lognormal Distribution with 95% Error Limits

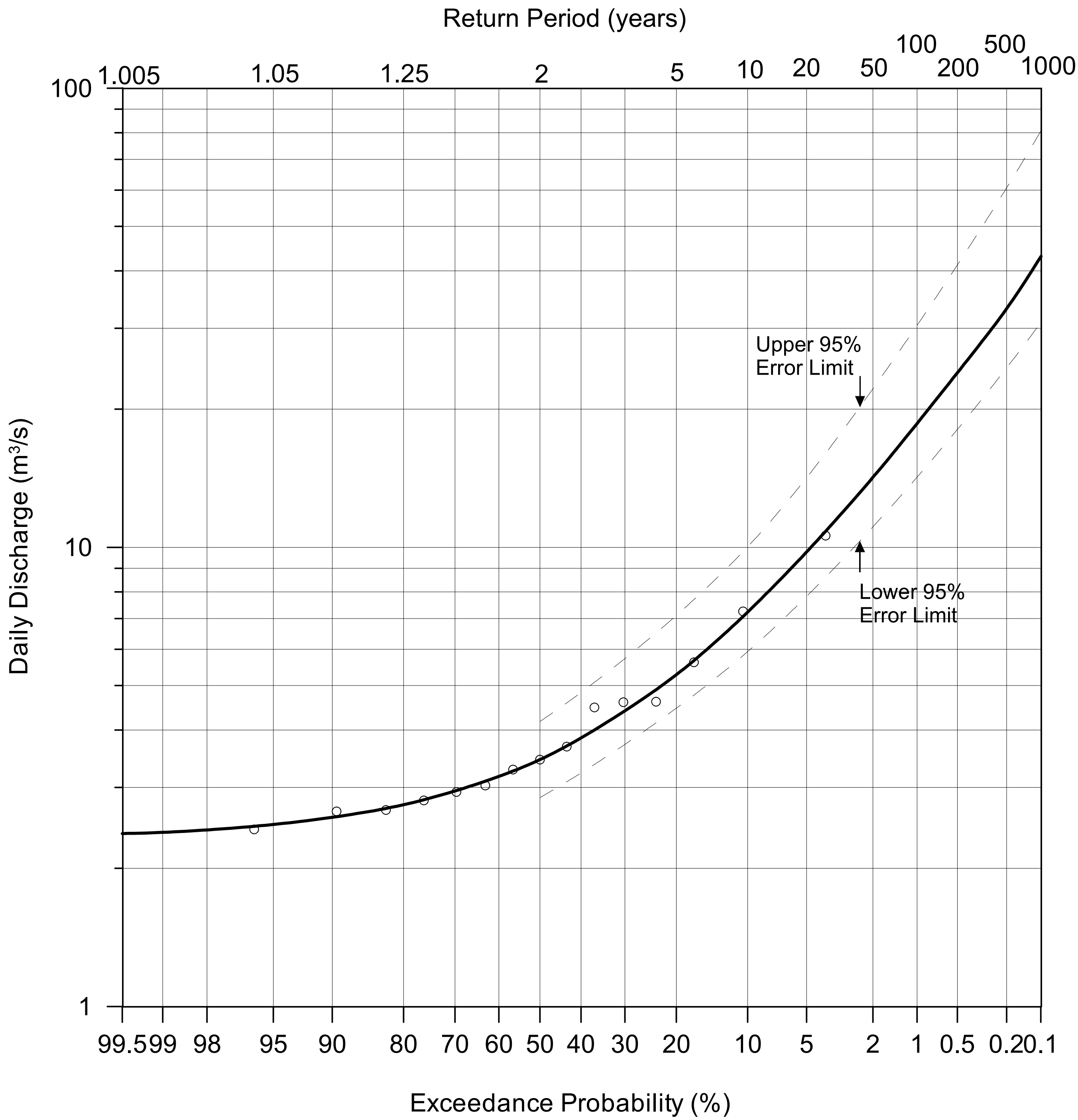


Figure A.1

Flood Frequency Plot
South Big Salmon River below Livingstone Ck. 09AG003 (1983-1986)
3 Parameter Lognormal Distribution with 95% Error Limits

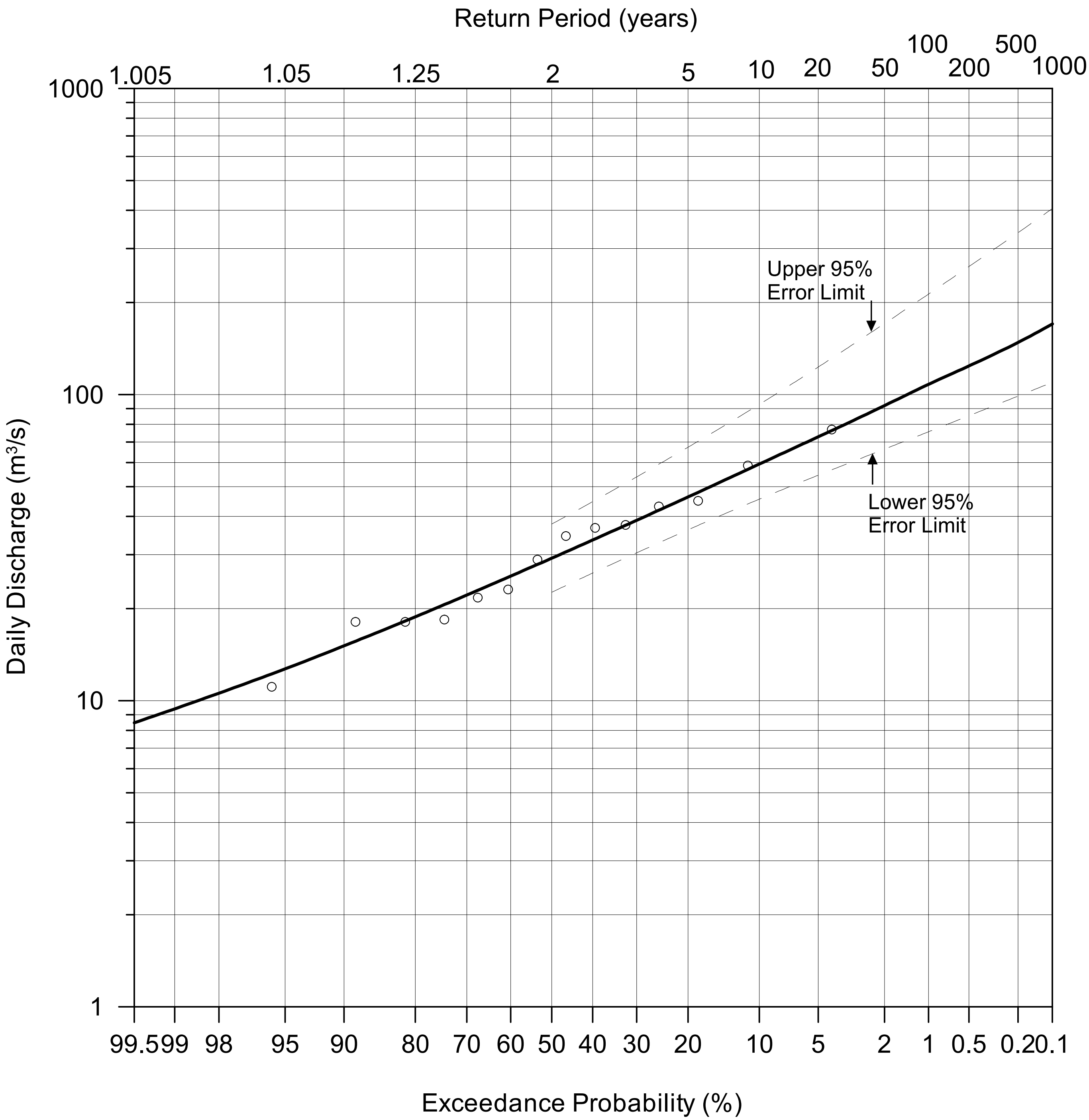


Figure A.2

Flood Frequency Plot
South MacMillan River at KM 407 Canol Road 09BB001 (1975-1996)
3 Parameter Lognormal Distribution with 95% Error Limits

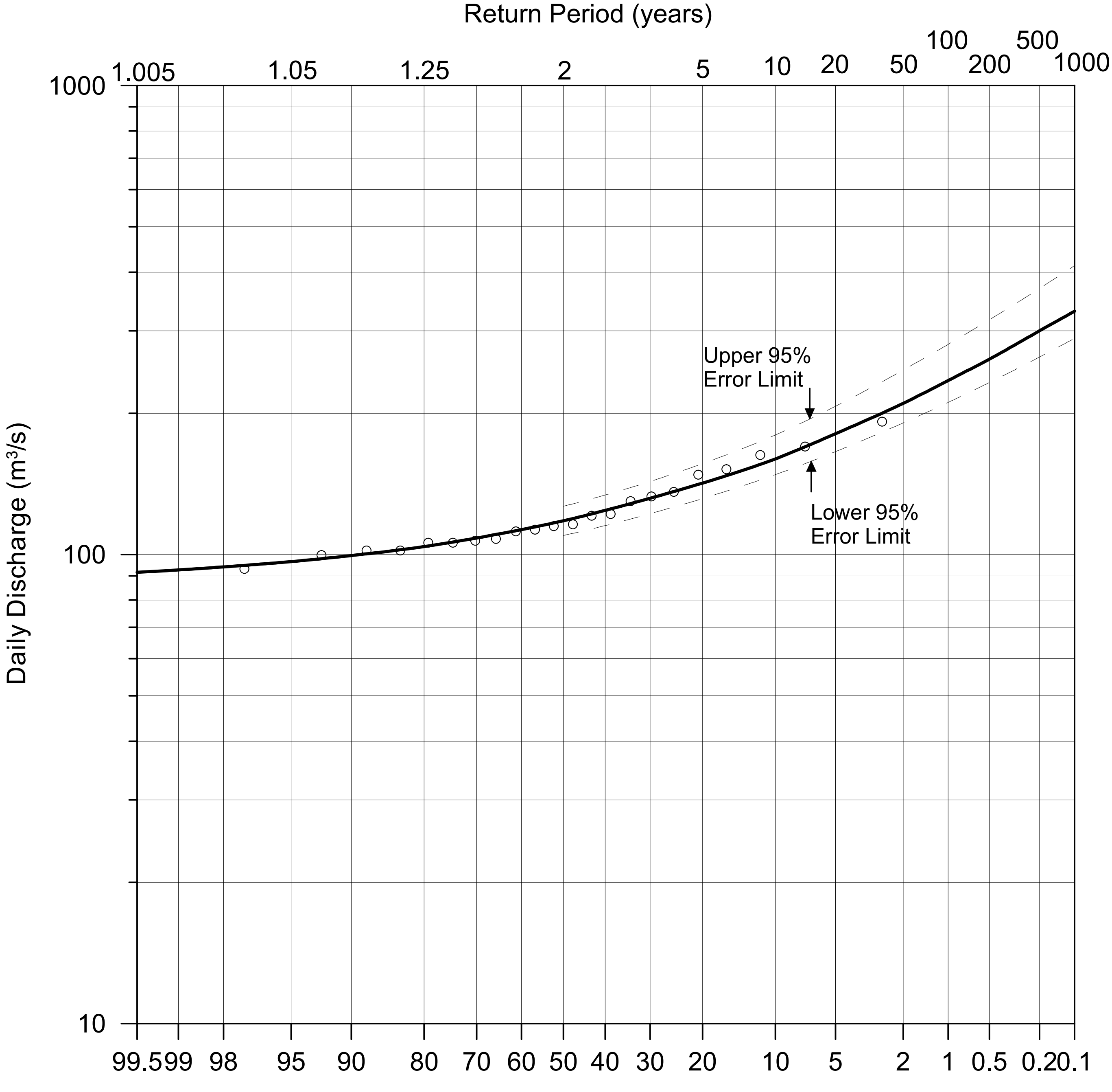


Figure A.3

Flood Frequency Plot
Big Creek near the mouth 09AH003 (1975-2002)
3 Parameter Lognormal Distribution with 95% Error Limits

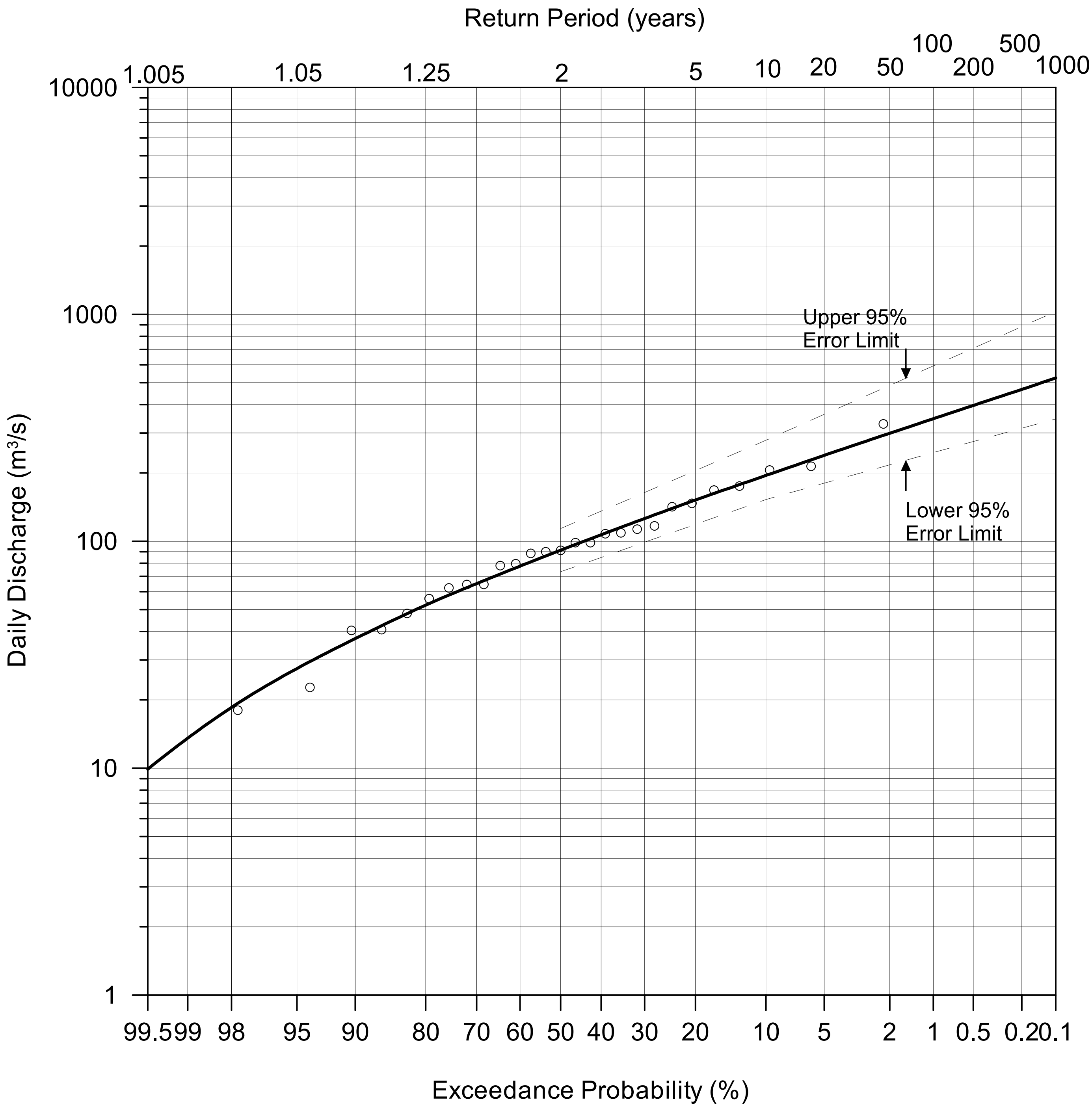
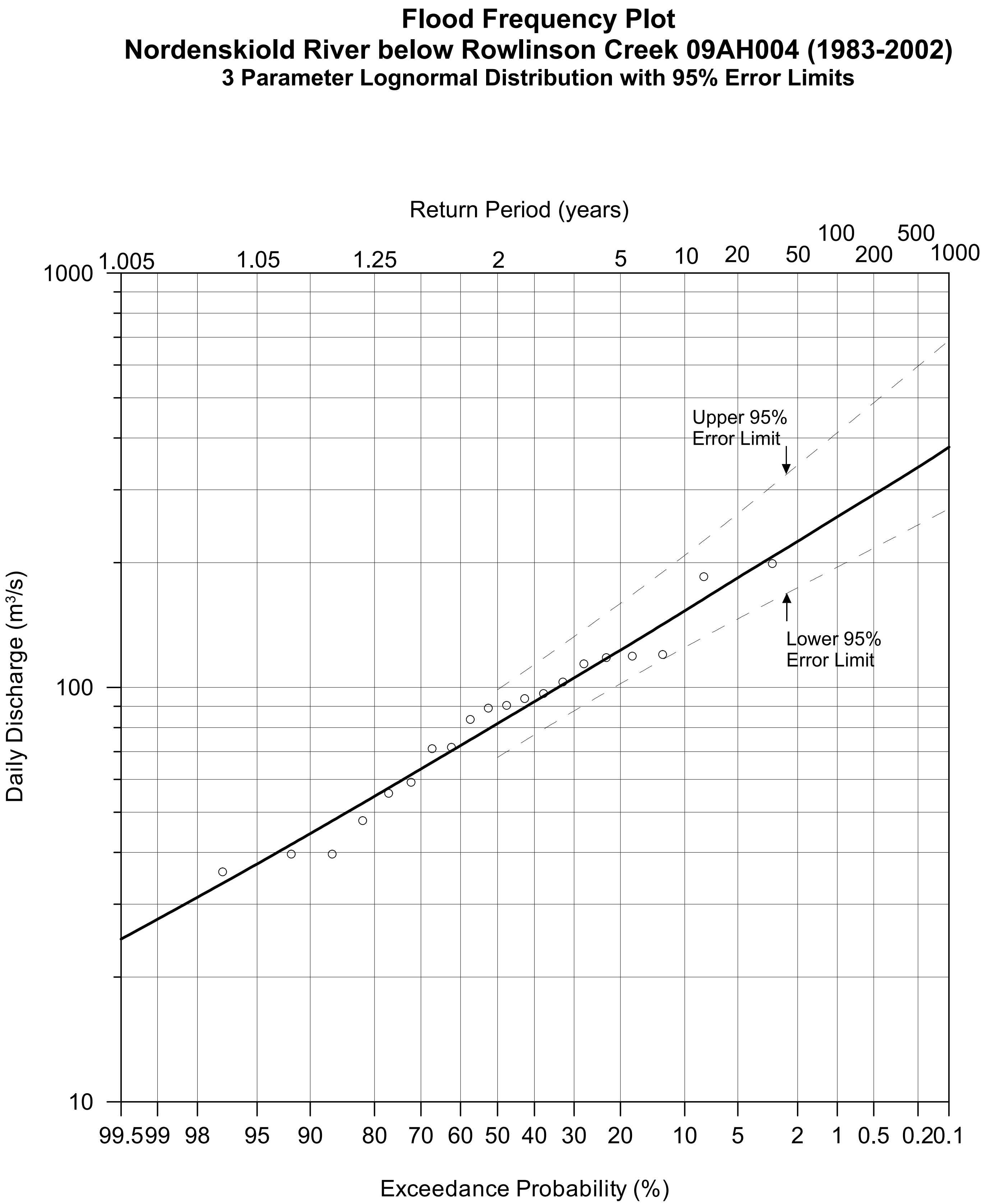


Figure A.4



Flood Frequency Plot
Nordenskiold River below Rowlinson Creek 09AH004 (1983-2002)
3 Parameter Lognormal Distribution with 95% Error Limits
Return Period (years)
1.005
1.05
1.25
2
5
10
20
50
100
200
500
1000
Daily Discharge (m³/s)
1000
100
10
Upper 95%
Error Limit
Lower 95%
Error Limit
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
Exceedance Probability (%)

Figure A.5

Flood Frequency Plot
South Big Salmon River near Carmacks 09AG001 (1953-1996)
3 Parameter Lognormal Distribution with 95% Error Limits

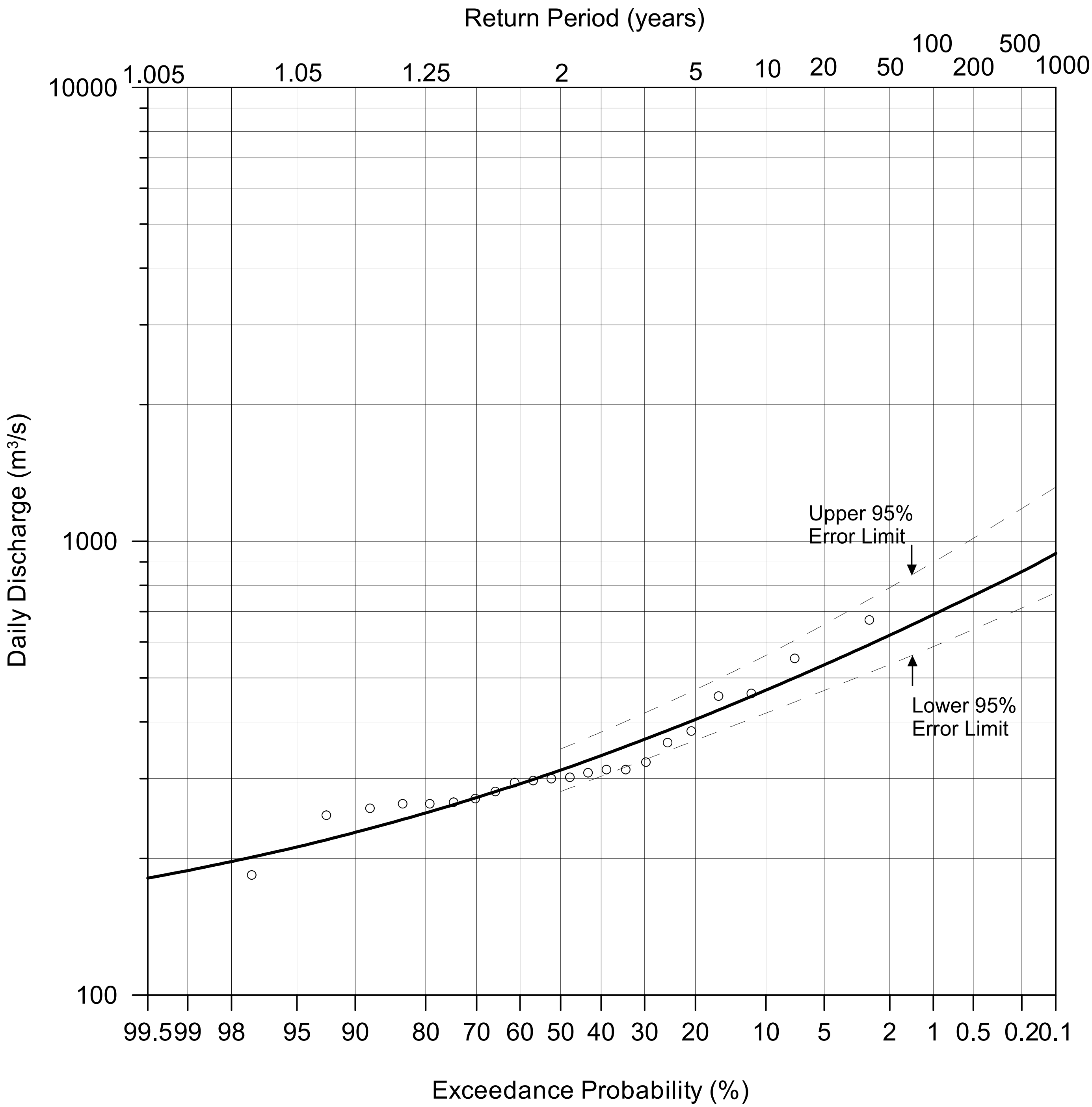


Figure A.6

Flood Frequency Plot
Ross River at Ross River 09BA001 (1962-2002)
Lognormal Distribution with 95% Error Limits

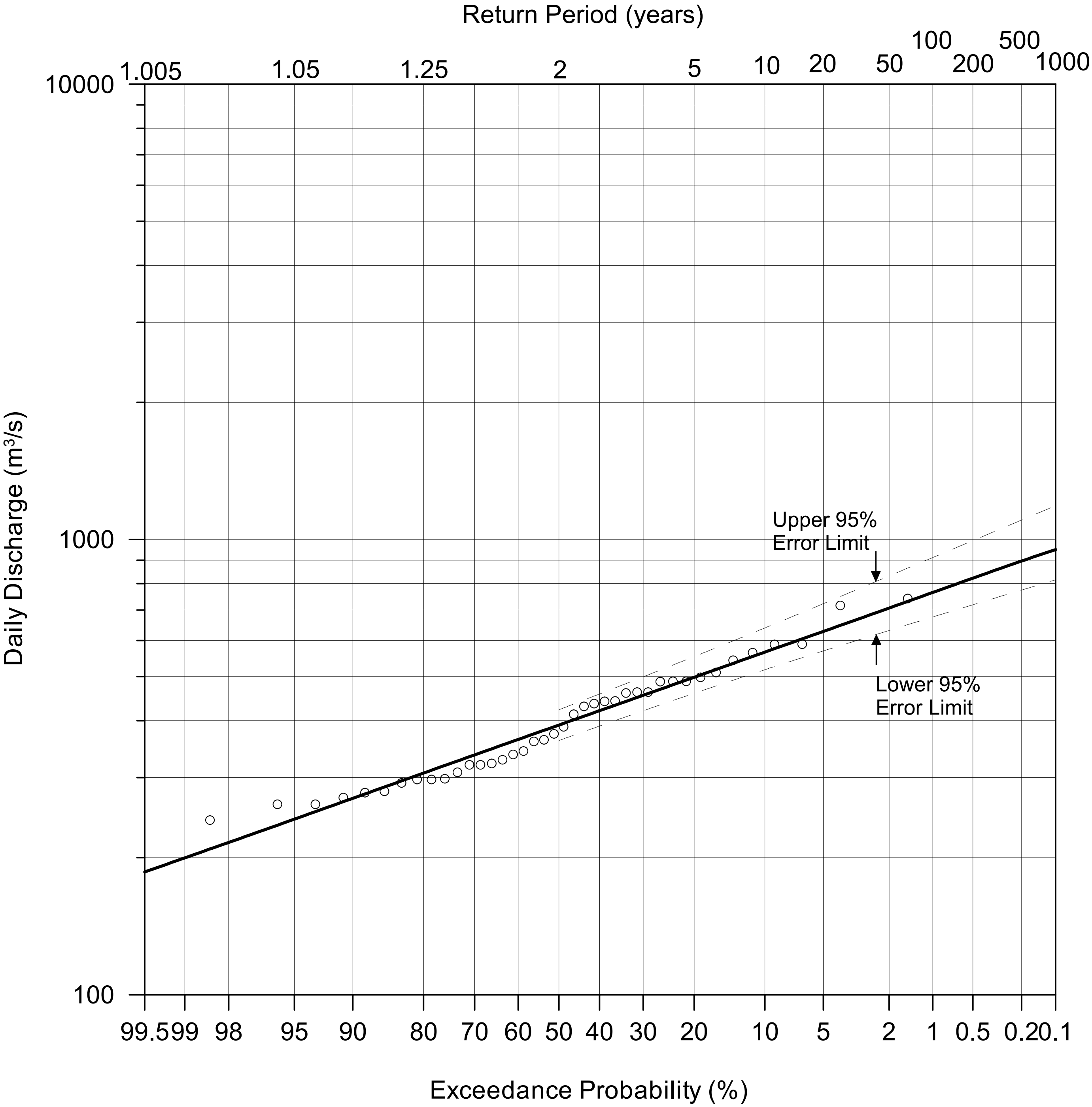


Figure A.7

Table A.1
Estimated flood discharges (daily) with 95 percent upper and lower error limits for streams in the Faro region

Return Period (years)	Vangorda Creek			South Big Salmon River:			South MacMillan River at km 407 Canol Rd.			Big Creek near the mouth			Nordenskiold River below Rowlinson Creek			Big Salmon River near Carmacks			Ross River at Ross River		
	95% Lower	Estimated Flood Discharge (m^3/s)	95% Upper	95% Lower	Estimated Flood Discharge (m^3/s)	95% Upper	95% Lower	Estimated Flood Discharge (m^3/s)	95% Upper	95% Lower	Estimated Flood Discharge (m^3/s)	95% Upper	95% Lower	Estimated Flood Discharge (m^3/s)	95% Upper	95% Lower	Estimated Flood Discharge (m^3/s)	95% Upper	95% Lower	Estimated Flood Discharge (m^3/s)	95% Upper
2	2.9	3.5	4.2	23	29	38	110	118	127	73	91	114	68	81.8	99	281	313	349	362	391	423
10	5.9	7.2	10	46	59	93	148	160	180	153	195	279	125	153	208	418	470	561	517	566	639
50	-	14	-	-	92	-	-	210	-	-	299	-	-	225	-	-	621	-	-	707	-
100	14	19	30	76	108	213	211	235	281	246	347	593	195	258	412	586	689	899	676	765	912
200	-	24	-	-	124	-	-	261	-	-	397	-	-	292	-	-	760	-	-	822	-
500	-	33	-	-	148	-	-	300	-	-	467	-	-	340	-	-	857	-	-	897	-
1000	29	43	75	110	170	405	289	330	413	346	530	1030	270	380	687	771	940	1317	815	950	1187

APPENDIX B

PROBABLE MAXIMUM PRECIPITATION FOR WAREHAM DAM, MAYO, YUKON

Probable Maximum Precipitation for Wareham Dam, Mayo, Yukon
Prepared by W.D. Hogg, November 2002

Introduction

This project involved estimating the Probable Maximum Precipitation (PMP) for the local drainage into the Wareham reservoir for the area below Mayo Lake. This is an area of about 900 km^2 above the town of Mayo, Yukon (63.6N, -135.8 W). Previous hydrologic studies have concluded that the limited storage capacity of the Wareham reservoir responds to intense rainstorm of one-day duration.

Climatologically, the basin is located on the northeastern slopes of the Yukon valley. Its high latitude location in the lee of the Saint Elias Mountains creates a cold, dry climate. Climate records from 1924 indicate average annual precipitation is slightly less than 300 mm; 187 mm of which falls as rain. Atmospheric moisture during large rain events arrives from the Gulf of Alaska. The basin terrain is rugged with an elevation range of about 1600 m. There are undoubtedly significant local orographic precipitation effects in the basin but the limited scope of this study did not consider them.

Because of the rugged terrain, steep precipitation gradients and very limited monitoring networks in the area of the basin, it was decided that the determination of depth-area relationships for historical storms for areas less than 1000 km^2 was not practical. Instead the PMP was based upon single station observations of precipitation and standard depth-area relationships for rain events. Efforts were made to verify that all events analysed were part of a larger area rain event and small area convective storms were excluded.

The PMP was determined based upon the following procedure:

1. Search daily precipitation records for the Yukon to identify the largest one-day rain events.
2. Using upper air records from Whitehorse, Fairbanks and Yakutat, determine the atmospheric precipitable water available to these events.
3. Using data from the same stations, determine the 100-year precipitable water for the area of the basin.
4. The ratio of maximum precipitable water to storm precipitable water forms the basis of the maximization factor, which was used to maximize the observed storm precipitation, transpose it to the basin and compute the point PMP estimate.
5. Standard depth-area relationships was used to calculate the PMP averaged over the area of the basin between Mayo Lake and the Wareham Dam.

Record Daily Rainfall

The Environment Canada archives of daily climate data to 1999 on CDROM were searched to identify the largest single day rainfall events in the Yukon. The five largest events are listed in Table 1. All were considered transposable to the project basin. The largest 1-day event at Mayo A was only 31.8 mm, a relatively small event considering the long record available but reasons to justify not transposing the larger storms observed elsewhere to Mayo could not be found.

Verification of the one-day record rainfall for the Yukon, observed at Quiet Lake on July 23, 1972, proved to be most troublesome. As detailed in Appendix 1, the observation for that day is almost certainly an accumulation of rain for the three previous days. Pacific and Yukon Region of Environment Canada agree as is detailed in Appendix 2. Therefore, percentage rainfall for the same days at Ross River, the nearest climate station, was used to distribute the Quiet Lake 3-day accumulation into daily values. The largest of these became the estimated 1-day record rainfall for Quiet Lake and is included in Table 1 as Quiet Lake (Adj.). For completeness, the archive value for Quiet Lake has also been maximized but it is recommended that this value be excluded from PMP consideration because of the likelihood that the rain did not fall on a single day. The adjusted value for Quiet Lake is based upon conservative assumptions and it is reassuring that the resulting values are similar to the other largest events observed in the Yukon.

Table 1

Event	1Day Rain	Storm Pw	Max Pw	Maxi-mization Factor	Maxi-mized Rain	1day Point PMP	24-hr Point PMP	900 km2 PMP
Quiet Lk 72-7 Unadj*	91.4	19	33	1.74	158.7 *			
Quiet Lk 72-7 Adj.*	66.8	19	33	1.74	116.0			
McMillan Pass 71-6	67.3	20	33	1.65	111.0			
Boundary 1971-6	61.5	16	33	2.06	126.8	**126.8**	**133.5**	**116.1**
Otter Falls 1990-8	55	28	33	1.18	64.8			
Blanchard 1992-6	56.0	25	33	1.32	73.9			

* The July 1972 value for Quiet Lake was rejected because it was believed to be a 3-day accumulation. The 1-day value was estimated to be 66.8 mm. See Appendix 1 for details.

Storm Moisture Maximization

Conventional PMP determination assumes that a storm of similar efficiency to observed events will generate proportionately more rainfall if more atmospheric moisture is available. Thus PMP determination requires knowledge about both the atmospheric moisture available to the observed storm and the maximum atmospheric moisture likely to be available to a similar storm in the area of the project basin. Traditional methods have used surface observations of dewpoint temperatures to estimate moisture in the total column of the atmosphere. This was based upon the assumption that surface dewpoint observations are more plentiful than upper air data and that surface conditions representative of the entire column of atmosphere for the storm moist air inflow can be found. The Yukon Valley is fortunate to have three upper air observing sites bracketing the areas of potential moist air inflow (Ykutat AL, Whitehorse YK and Fairbanks AL) while the meteorological effects related to the moisture inflow over the cold surface waters of the North Pacific and the basin location in an interior valley combine to produce conditions where surface measurements are likely to underestimate total atmospheric moisture. For these reasons, and because moisture determination from upper air data where available is more accurate anyway (Watt et al., 1989), Upper air data were used exclusively to determine both storm and maximum atmospheric moisture for this study. In all cases, estimates were based upon interpolation from all available soundings. Precipitable water estimates for each event examined are shown in Table 1.

As recommended by international and Canadian guidance for PMP determination, maximum precipitable water was based upon calculation of the 100-year return period estimates of that parameter for the three upper air stations. Temperature, pressure and humidity data were used to calculate total precipitable water for each sounding and maximum values for each month and the entire year were extracted. A Gumbel distribution fit using the method of moments was then used to estimate the 100-year precipitable water for each of the upper air sites. The 100-year precipitable water value for the year and for all three summer months (June, July, August) at the project basin location was estimated to be 33 mm.

With one exception, all three upper air stations estimated maximum atmospheric moisture available to storms consistently. The exception was the June 1971 Boundary storm. Both Whitehorse and Fairbanks measured peak atmospheric moisture near 14 mm near the storm date while on the coast, Yakutat measured 22 mm. A similarly large discrepancy was not observed for any other event and it was felt that 14 mm precipitable water was unrealistically dry for such a large rainfall. Since it has been common practice in other PMP studies to limit maximization factors to no higher than 2.0 in an effort to prevent unrealistic over maximization of events, it was decided to assume that some of the moisture observed at Yakutat during this event must have made it to the Boundary area and a precipitable water value of 16 mm, which generated a maximization factor near the standard limit of 2.0, was assumed.

Storm maximization must account for differences in atmospheric moisture availability between the storm location and the project basin. With one exception, observed storms occurred at locations with similar geography and at elevations within a few hundred metres of elevations in the project basin. At over 1400 m, the event at MacMillan Pass was significantly higher than other events but since moist air inflow during this event was from lower levels, which probably enhanced precipitation, a transposition adjustment to further increase precipitation was not considered appropriate even for this event. Therefore, no transposition adjustment was used for any of the events and the 100-year precipitable water estimate of 33 mm was used to determine the maximized rainfall estimate at the project basin for all storms. The maximized rainfall is included in Table 1.

<u>24-Hour PMP</u>

Climate day observations represent rainfall accumulations at fixed observing times but stream flow responds to the rainfall in a sliding 24-hour period regardless of how observers assign rain to different days. On average, 24-hour rain is 14% higher than climate-day rain. There is no way to determine how much rain fell during the 24-hours of the Boundary PMP event but we know it cannot exceed the total of 2 consecutive days. Since 3.3 mm was recorded on the day prior to the major event and only 0.8 mm on the day after, the most conservative assumption possible is that all of the rain on the day prior and the day of the recorded maximum actually fell within a 24-hour period. This is less than the average adjustment of 14% for conversion from 1-day to 24-hour values. Adding the maximized rain from the previous day results in the estimate of 24-hour point PMP of 133.5 mm.

Areal Reduction

As noted previously, there was insufficient data available to permit reliable determination of areal rainfall in the storms analyzed, particularly for an area as small as 900 km^2. Instead, standard depth-area relationships for storms similar to those occurring in the Yukon were used to estimate the average precipitation over the entire basin during the PMP event. An areal reduction factor of 0.87 as recommended by Watt et al. (1989) and consistent with the factor used in the original PMP estimate for Wareham Dam (ACRES 1987) was selected. The final estimate of 116.1 for average rainfall over the Wareham local drainage area was made by applying this reduction factor to the point PMP estimate as shown in Table 1.

Conclusions

Records of daily rainfall in the Yukon were searched to identify the largest one-day events. One of these, the record rain for the Yukon, was found to be a 3-day accumulation and was adjusted to account for this. The five largest events were maximized based upon atmospheric moisture available to the events and the 100-year maximum atmospheric moisture likely to occur in the vicinity of the project basin. The largest maximized 1-day value was increased to account for the possibility that the rain on an adjacent day and the day of maximum rain could have fallen in a single 24-hour period and a standard depth-area relationship was applied to determine the average PMP rainfall over the entire 900-km^2 basin. Based upon these procedures, the 24-hour PMP averaged over the Wareham local drainage area was estimated to be 116.1 mm.

References

ACRES 1987. The Yukon Development Corporation Water Management and Flood Studies for Mayo Hydroelectric Development. P8176.00, Dec. 1987, Calgary, 29 pp.

Watt, W.E., K.W. Lathem, C.R. Neill, T.L. Richards, editors, 1989. Hydrology of Floods in Canada: A Guide to Planning and Design. National Research Council of Canada NRCC No. 29734, Ottawa, 245 pp.

APPENDIX C

ROSE CREEK DIVERSION CHANNEL
CLOSURE SCENARIOS - GEOTECHNICAL

By: BGC Engineering Ltd.

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
1605, 840 – 7 Avenue S.W. , Calgary, Alberta, Canada. T2P 3G2
Phone (403) 250-5185 Fax (403) 250-5330

PROJECT MEMORANDUM

To:	**Northwest Hydraulics Consultants Ltd.**	**Fax No.:**	**Via e-mail**
Attention:	**Barry Evans, P.Eng., Senior Engineer**	**CC:**	
From:	**Holger Hartmaier (Ext. 113)**	**Date:**	**December 17, 2003**
Subject:	**Rose Creek Diversion Channel- Closure Scenarios- Geotechnical Considerations (Draft)**		
No. of Pages (including this page): 26 Pages		**Project No:**	**0257-020-01**

This memorandum summarizes geotechnical considerations and assumptions made with respect to various closure Scenarios for the Rose Creek Diversion Channel, located at the Faro Mine site, near Faro, Yukon.

1. BACKGROUND

As part of the long-term closure planning for the Anvil Range property, consideration was given to assessing the capacity of the Rose Creek Diversion Channel (RCDC) to handle extreme flood flows. The RCDC was originally designed to have a hydraulic capacity equivalent to a 50-year return period flood and contingency capacity for a 500-year return period flood, the latter assuming no freeboard. Northwest Hydraulic Consultants Ltd. (NHC) completed a hydrotechnical assessment of the RCDC (NHC, 2003), based on channel surveys done in 2003. Subsequently, NHC re-interpreted the site hydrology and estimated the magnitude of the Probable Maximum Flood (PMF). A PMF flood peak of 730 m^3/s was adopted for the RCDC.

Upgrading the RCDC to convey the PMF flows requires consideration of the existing Down Valley Tailings facility, the condition of the existing channel and dike and the geotechnical foundation conditions.

Three scenarios were discussed in a teleconference call on October 31, 2003 between NHC, SRK and BGC:

- Scenario 1: Increase the size of the Rose Creek Diversion channel along the south side of the tailings facility to convey the PMF.

- Scenario 2: Abandon the RCDC downstream of the plug dam. From the plug dam, convey the PMF over the tailings (assume tailings are covered with a soil cover) in a swale lined with rip rap to the Intermediate dam pond, then over a new spillway by-passing the Intermediate and Cross Valley Dams.
- Scenario 3: Involves removal of tailings from the Original, Second and Intermediate Impoundment to el. 1042 m amsl. The Rose Creek PMF is directed into the tailings area immediately downstream of the Pumphouse Pond. The attenuated PMF passes over the spillway located at the Intermediate Dam.

For each Scenario, passage of fish will be addressed.

2. EXISTING CONDITIONS

2.1 Rose Creek Diversion Channel

A contour plan generated by Yukon Engineering Services (YES) using survey data collected by YES during the summer of 2003 was used by NHC to create 39 cross-sections (NHC, 2003).

The RCDC can be subdivided into the following reaches, based on hydraulic aspects:

- The furthest downstream reach from cross-section 0.5 to 3 (for section numbering and location, see Figure 2 in NHC, 2003) is a mildly sloped section below the rock drop weir section where the diversion flow returns into the natural Rose Creek channel.
- The rock drop weir section from cross-sections 3 to 9 is a steeply sloped section consisting of numerous rock weirs. This section compensates for the difference in grades between the RCDC (0.2%) and the original Rose Creek valley (2%).
- A mildly sloped section above the rock drop weir section from cross-sections 9 to 30, which was constructed in 1980 to divert Rose Creek around the expansion of the tailings facilities. A fuse plug dam is located within the original Rose Creek channel between cross-sections 29-31.
- The upper end of the RCDC is a mildly sloped section that was in place prior to 1980 and is called the original diversion. This reach is located upstream of the fuse plug dam from cross-sections 30 to 39.

The RCDC extends for a total length of 3.6 km along the south valley wall of Rose Creek.

Assessment of the hydraulic capacity of the existing channel indicated the potential for overtopping of the right bank of the channel under the 500-year flood event. Additional sites where overtopping could occur were identified for the 500-year flood with 1.5 m of ice frozen to the channel. NHC (2003) estimated that raising of these low points would require 3200 m^3 of material.

Assessment of bed stability indicated that the rock drop weir section would be subject to full bed movement under 500-year flow conditions. The mildly sloped reaches upstream and downstream of the rock drop weir section would not be subject to bed movement. In the original diversion section, NHC recommended that field samples be obtained to confirm if the minimum bed D_{50} requirements are met.

Upgrading of rip rap was recommended in the rock drop weir section and in the mildly sloped section downstream to maintain bank stability under 500-year flow conditions. The mildly sloped section upstream of the rock drop weir was considered to have adequate bank protection for 500-year flow conditions, except for the original diversion, which likely needs upgrading (NHC, 2003).

No design information was available for the original diversion, upstream of the fuse plug dam. Design drawings and reports (Golder, 1980) are available for the reach downstream of the plug dam. Canal cross-sections shown on these drawings indicate that the following types of canal lining were utilized:

- Unlined channel in rock.
- Impervious lined channel to prevent seepage out of the RCDC.
- Thermal liner for protection of permafrost affected foundations.
- Canal outfall erosion protection downstream of the rock drop weir section.

In general, the channel profiles have a bottom width of 12.2 m, with side slopes of 2 Horizontal to 1 Vertical (2:1). The canal dike on the right bank is constructed of rockfill, with a crest width that varies from 7.8 m to 11 m. The drawings show the presence of a waste pile along the outer toe of the dike of unspecified dimensions.

No information was available as to the “as-built” extent of each of these sections. In 2003, BGC conducted a photo documentation of the entire RCDC. Based on a review of these photos, bedrock was noted in the channel in the following locations:

- Between cross-sections (CS) 11 and 12
- Between CS 17 and 18
- Between CS 23 and 25
- Between CS 27 and 28

Further investigations are required to delineate the “as-built” details of the RCDC.

2.2 Tailings Disposal Facility

From upstream to downstream, the tailings facility consists of the following components:

- Original tailings impoundment.
- Second impoundment.
- Intermediate impoundment.

- Intermediate Dam
- Polishing or Cross Valley Pond
- Cross Valley Dam

The RCDC was constructed to divert the flow of Rose Creek around the tailings facility. The original thalweg of Rose Creek lies under the tailings disposal area. The RCDC was constructed along the south valley slope of Rose Creek. Adjacent to the original channel upstream of the fuse plug dam (CS 31-39), the tailings level in the second impoundment is at about elevation 1061-1062 m amsl and covers the original valley floor to a depth of about 13 m. Further to the right, the tailings level in the original impoundment is at about elevation 1068.

The 2003 survey did not distinguish where the edge of the tailings disposal area was with respect to the outer toe of the RCDC dike. It was assumed that the tailings surface was located against the outer slope of the dike in the reach between CS 31-39.

Between CS 11 and CS 31, the dike lies adjacent to the Intermediate impoundment. The Intermediate pond is impounded against the Intermediate dam and lies adjacent to the RCDC dike in the reach between CS 11 and CS 14. Between CS 14 and CS 31, a narrow strip of ground separates the outer toe of the RCDC dike from the Intermediate tailings impoundment. Remnants of the former (pre-1980) Rose Creek channel are visible between CS 21 and CS 31.

The surface of the tailings in the Intermediate impoundment is at about elevation 1050 m amsl. The maximum depth of tailings is about 15 m. The tailings surface slopes down towards the Intermediate dam impoundment. Water levels in the Intermediate impoundment may fluctuate between elevation 1044 m amsl and 1048.8 m amsl. Recent surveys (YES, 2003) indicated a water level at 1047.26 m amsl. No information is available on the depth of water in the Intermediate pond. Water is siphoned from the Intermediate pond to the Cross Valley (polishing pond).

The Intermediate dam is a zoned earthfill embankment with a low permeability core excavated into the foundation. The dam has a crest width of 6-7 m and an overall length of 650 m (BGC, 2003). The lowest point on the crest, based on 2003 surveys is 1048.68 m amsl. A spillway is located on the north abutment, with a sill elevation of 1047.7 m amsl.

The Intermediate dam was constructed in stages. The first stage was built in 1981 to elevation 1035.7 m amsl. At that time, the spillway was located on the south abutment. In 1988, the dam was raised to elevation 1040.7 m amsl and the spillway moved to the north abutment. In 1989, the dam was raised again, to elevation 1045.7 m amsl. The final raise took place in 1991 to a design elevation of 1049.4.

The reach between CS 8 and CS 11 lies adjacent to the Cross Valley pond. CS 11 is close to the axis of the Intermediate dam. CS 8 is close to the axis of the Cross Valley dam. The Cross Valley pond, or polishing pond extends from the downstream toe of the Intermediate dam to the Cross Valley Dam. Water level (2003) was at elevation 1030.84. The outlet of the Intermediate dam spillway enters the pond on the north side. The Cross Valley dam spillway inlet is also located on the north abutment. In this reach of the RCDC, the top of the diversion channel dike is about 22 m above the polishing pond.

Pond levels are now lower than the originally designed operating levels. Normal pond levels are maintained between 1029.4 m amsl and 1031.7 ma masl (BGC, 2003). Water from the Intermediate dam is discharged into the polishing pond by siphons and then siphoned in the summer months directly into Rose Creek. Currently, water from the Faro pit is being treated through the mill water treatment system and is discharged indirectly to Rose Creek through the polishing pond.

The Cross Valley Dam retains the polishing pond for the tailings containment system. The dam was constructed to its final elevation of 1033.5 in 1981. It is a zoned earthfill embankment with a low permeability core and upstream blanket. The dam has a crest width of 6-7 m and an overall crest length of 500 m. The spillway, located on the north abutment has an inlet at elevation 1031.2 m amsl.

2.3 Geotechnical Considerations

2.3.1 Existing Canal Route

The RCDC traverses a wide variety of materials, ranging from bedrock to till and creek alluvium (Golder, 1980.). The channel is located on a north-facing slope, and much of the ground is permafrost affected, with sometimes ice-rich materials. The design of the channel included provisions for placement of insulating and filtering layers to prevent degradation of the permafrost areas. Winter construction was carried out to avoid thaw degradation during construction. The permafrost is considered "warm", with temperatures of 0^o to -1^o C.

The following geotechnical information was obtained from the design report for the "new" diversion below the plug dam (Golder, 1980). No information was available for the original diversion, but generally conditions are expected to be similar to those observed elsewhere along the RCDC.

- About 0.3 m of moss and organic debris generally blankets the ground along the canal route. This organic cover is instrumental in maintaining permafrost conditions, and freezes annually.
- Beneath the organic cover are black or brown organic silts and/or colluvium to a depth of about 1.2 m. This organic silt is often permafrost affected and may have water contents

as high as 120%. In the vicinity of creeks and groundwater flows, this material can remain unfrozen on an annual basis and may present trafficability problems.

- Beneath the organic silt is till. The upper surface of the till is often pebble rich, indicating that some erosion of the till surface may have occurred at one time. There is evidence of colluvial action in the upper till based on the presence of organic layers and/or white volcanic ash being interlayered with till.
- The till varies in condition and composition along the canal route. Frozen till may occur throughout the route, although continuously frozen, ice-rich, fine-grained till was found downstream of Station 0+900 on the canal baseline (approximately CS 25).
- Thermistors installed along the canal route confirmed that most of the ground was frozen except for localized areas that were affected by subsurface water flow.
- Till water contents ranged from about 7% to 39%, with average water contents of 18% found in test pit samples. Water content increased with proximity to the ground surface.
- Till composition is highly variable. In general the material is extremely broadly graded, with at least 9% by weight of silt and clay size particles. Most of the samples contained between 10% to 30% silt and clay sized particles. In general, the tills are non-plastic with occasional low plastic zones associated with higher fines contents.
- Coarse-grained materials were found in the plug dam area. The expected hydraulic conductivities ranged from 1 x 10^{-5} cm/s to 1 x 10^{-6} cm/s. Hydraulic conductivity measured on a sample of till compacted to 100% of Standard Proctor density was 3 x 10^{-7} cm/s. The hydraulic conductivity increased by 15 times after the sample was subjected to several freeze-thaw cycles.
- Shear strength parameters for the non-plastic till were estimated to be about 36^{o}, with higher plastic tills as low as 32.5^{o}.
- In situ densities of till ranged from 1780 to 2260 kg/m^3, with an average of 1989 kg/m^3.
- In the outfall portion of the canal (downstream of CS 9), alluvial deposits underlie the surficial organic silts. Some of these sand and gravel deposits are marginally frozen and required ripping during excavation. The alluvial deposits become sandier at depth.
- Bedrock was anticipated within the proposed canal depth in the following reaches (going downstream):
    - At the inlet
    - Sta. 0+50 to 0+230 (approximately CS 29 to CS 27)
    - Sta. 0+330 to 0+350 (CS 26)
    - Sta. 0+500 to 0+820 (approximately CS 25 to CS 23)
    - Sta. 1+490 to 1+600 (approximately CS 18 to CS 16)
    - Sta. 1+940 to 1+980 (approximately CS 14 to CS 13)
    - Sta. 2+200 to 2+410 (approximately CS 12 to CS 11)
- Bedrock consists of metamorphic units (phyllite and schists), dipping about 20 to 30 degrees to the south-southwest, into the hillside. A quarry was developed at Station 2+500 (near CS 10), south of the channel alignment.

- The specific gravity of the rock was found to be 2.80. The upper bedrock is disturbed and fractured. Hydraulic conductivities were not measured, but some till lining was anticipated to minimize seepage losses.
- In the south abutment of the Intermediate Dam, frozen till was expected, with the possibility of some colluvium and organic deposits. Shallow bedrock was also a possibility, with an irregular and possibly disturbed bedrock surface.
- Granular alluvium exists at the Cross Valley dam, which has infilled the valley to a depth of 50 m. At the south abutment of the dam, frozen tills, and possibly some fine grained colluvium were expected. The upper 1.5 m was expected to be ice rich.
- Downstream of the Cross Valley dam, the RCDC crosses an alluvial fan from a tributary of Rose Creek that flows down the south valley slope.

2.3.2 Tailings Impoundment Areas.

Mining commenced in 1969 at a production rate of 9200 tonnes per day. Tailings were initially placed in the Original Tailings pond and contained by a low dike constructed from valley alluvial materials (SRK, 1986). The dike was subsequently raised using both tailings and alluvial materials by upstream construction methods. Tailings were discharged into this impoundment by spigotting from the north slope of the impoundment The Original impoundment extends from the north valley wall to the mid-point of Rose Creek valley. It has a surface area of about 27 ha. and contains approximately 6 million m^3 of tailings (SRK, 1986).

The Second Tailings Impoundment was constructed in 1974 and extends across the original Rose Creek drainage. The dike is constructed from alluvial sands and gravels and was raised by the centreline construction method. Construction of this impoundment caused Rose Creek to be diverted onto a terrace along the south valley wall. Tailings were spigotted into this impoundment mainly from the north and south of the impoundment. The second Impoundment has a surface area of about 40 ha. And contains approximately 5 million m^3 of tailings to an average elevation of 1060 m amsl.

The Intermediate Dam was constructed in 1981 and has the capacity to retain 42 million m^3 of tailings. The Cross Valley Dam and polishing pond were constructed to achieve the required 60-day retention time for water treatment.

The oxidized and sulphide ores were milled and treated in the mill facilities to recover zinc and lead/silver concentrates, which were shipped off site. Major reagents added in the mill included soda ash, lime, copper sulphate, sodium sulphite, xanthate, sodium cyanide and iron from millrods and balls (SRK, 1986). During the period of mine shutdown, pit dewatering continued. Mine water was treated with lime and discharged to the Second Tailings Impoundment, from where it decanted into the Intermediate Dam Impoundment, then to the Cross Valley

Impoundment, from where it discharged into Rose Creek (SRK, 1986). The tailings are acid generating when exposed to oxygen and water.

In the upstream reach of the RCDC (CS 31 to CS 39), tailings have been assumed to reach the outer toe of the existing canal dike. From CS 31 to CS 14, the edge of the tailings is up to 90 m away from the toe of the dike. Between CS 14 and the Intermediate Dam (CS 11), the Intermediate pond covers the tailings to an undetermined depth along the south shoreline of the pond. The edge of the pond is close to the outer toe of the canal dike slope.

Information concerning the properties of the tailings was obtained from SRK (1991). This report provided an assessment of the decommissioning of the down valley tailings area and provided a good summary of tailings characteristics based on site investigations and laboratory tests, as noted in the following sections. Physical characteristics of the tailings considered in the SRK report include particle size distribution, solids specific gravities, in situ void ratio and density.

2.3.2.1 Particle Size Distribution

In general, the tailings consist of uniformly graded silt or fine sand, with considerable variation in the grain size distributions. Grain size variations occur as a result of pipeline discharge or sand/slime separation techniques, proximity to discharge point and milling process.

In 1981 and 1982, the milling process was changed producing tailings with finer particle size than those produced previously. The tailings in the Intermediate Dam Impoundment and the upper part of the Second Impoundment are on average finer than those in the original impoundment. Within in each impoundment, lateral sorting occurs as the tailings move away from the discharge points. The following spatial variations were noted in the Original and Second tailings impoundments:

In the Original Tailings impoundment:

- Coarse fractions are predominant in the northern and western part of the impoundment:
- The slimes fraction predominates in the southern and eastern part of the impoundment
- Coarse fractions are also found along the edges of the impoundment, largely where it was used as a construction material.

In the Second Impoundment:

- Pre- 1986 tailings were deposited by spigotting along the impoundment perimeter. The 1986 tailings were discharged only in the northwestern part of the impoundment. These cover the older tailings in the western part of the impoundment, but pinch out towards the east.
- Coarser fractions are found where overflow from the Original Impoundment occurred and where there was direct discharge- along the northwest part of the impoundment, along the Original Tailings Embankment and along the Second Tailings embankment.
- Slimes were found in the southern and central parts of the impoundment.

2.3.2.2 Specific Gravity

During the course of mining, various ore types were processed in the milling, resulting in variations in the specific gravity of the tailings. The tailings in the Original Impoundment have a higher solids specific gravity than those in the Second and Intermediate impoundments. This is due to the presence of barite with the massive sulphides in the early stages of mining of the Faro orebody. In the latter stages, the ore consisted of sulphide rich quartzites, with little or no barite. The range in specific gravity is also a reflection of varying sulphide content in the tailings samples.

Tailings samples taken during production in the Original Impoundment showed specific gravities ranging from 4.19 to 4.53, with a mean value of 4.5. Surface samples from the Original Impoundment appear to have lower specific gravity ranging from 3.26 to 3.83, with a mean value of 3.6

Tailings from the Second Impoundment range in specific gravity from 3.66 to 4.04, with a mean value of 3.8. No data was available regarding the changes in specific gravity with depth in the Second Impoundment.

Specific gravity of tailings from the Intermediate Impoundment ranged from 3.66 to 4.04, with a mean value of 3.86.

2.3.2.3 Void Ratio/Density/Porosity

Maximum and minimum tailings void ratios were determined to be 1.06 and 0.58 respectively. In situ densities were reported as relative densities, based on downhole geophysical techniques and investigations carried out by Golder Associates in 1977 (SRK, 1991). The corresponding in situ void ratios calculated from the relative density data ranged from 0.73 to 0.92. Void ratios tended to be higher in areas of predominantly fine-grained tailings than in areas of coarse-grained tailings. In the sands, void ratios were generally less than 0.85, whereas in the slimes, they were greater than 0.89.

Frozen tailings, with ice lenses up to 1.2 m thick were reported, which also reduces the overall average void ratio. Since these investigations were done, the tailings have undergone some degree of consolidation, and if there has been any melting, the overall in situ void ratio would be increased.

Dry solids densities of tailings depends on the method of deposition (spigotting), mill grind and ore source. Densities obtained in the laboratory ranged from 2.4 tonnes/ m^3 for above water deposition to 0.9 tonnes/m^3 for below water deposition. Within the down valley tailings area, in situ densities ranged from 1.7 to 1.9 tonnes/m^3 and the specific gravity of the solids is 3.8 (SRK, personal communication). This would correspond to an average void ratio of 1.18. Note that the high specific gravity of the solids can result in misleading interpretations of the in situ relative density based on unit weight determinations alone. Variations in specific gravity data may also yield somewhat different void ratios.

Porosities were determined for tailings and till samples. For tailings, the majority of values ranged from 32.4 to 49.1 %, with no apparent correlation between porosity and grain size fraction. Till porosities ranged from 28.4 to 42 % for materials placed during construction.
Golder recently (fall, 2003) completed a program of cone penetrometer testing of the tailings in the down valley to assess the potential for liquefaction. The results of this investigation were not available at the time of writing of this memorandum.

2.3.2.4 Hydraulic Properties

Hydraulic conductivity of the tailings is variable due to several factors:

- The degree of grinding in the mill at any given time.
- Segregation of the tailings discharge into coarse and fine fractions with distance from the discharge point.
- Zones of frozen tailings within the deposit

In the Original and Second Impoundments, various winter and summer discharge points were used, creating a complex, strongly anisotropic deposit of coarse and fine layers. Coarser grained material was used to raise the dike.

Tailings in the Intermediate Impoundment were deposited from a single winter and summer discharge point, likely resulting in a more uniformly graded deposit, as discussed in Section 2.3.2.1.

The hydraulic conductivity of unsegregated (saturated) tailings ranged from 1×10^{-5} cm/s to 5×10^{-5} cm/s, increasing to 1×10^{-3} cm/s at the surface, where coarse tailings have been leached and subjected to frost action. Tailings slimes have an average hydraulic conductivity of about 1×10^{-6} cm/s, which may be an order of magnitude less at higher levels of consolidation.

Frozen zones within the tailings have effectively zero hydraulic conductivity. The distribution of frozen zones within the tailings is not clear, but the recent grid of cone penetrometer holes (Golder, 2003) failed to encounter any frozen zones (SRK, personal communication). Previously (SRK, 1991) reported frozen zones at depths up to 18 m from the surface that may be associated with ice incorporated during winter tailings deposition. It is possible that some ice lenses may have disappeared over time. The presence of frozen tailings at depth that may melt-out over time has significant impacts on the long-term predictions of tailings behaviour.

2.3.2.5 Hydrogeology

Gartner Lee Limited assessed the hydrogeology of the down valley tailings disposal area in 2002 (GLL, 2002). The investigation program included drilling, installation of groundwater monitoring wells, test pitting and geophysics.

The tailings are largely exposed (beached), with a small portion flooded upstream of the Intermediate Dam. The saturated zone within the tailings extends to within 10 to 12 m of the surface. The water table within the tailings is largely controlled by the water level in the Intermediate Pond. A sand and gravel aquifer underlies the tailings, in the former Rose Creek valley, and ultimately discharges into Rose Creek. The aquifer overlies a denser basal till that rests on the valley bedrock. Seepage flow is predicted to be downward through the tailings into the aquifer. Regionally, the Rose Creek valley is a groundwater discharge area, with an upward flow gradient in the bedrock below the valley (SRK, 1986). This upward flow gradient prevents contaminated groundwater from entering the bedrock and confines the flow under the tailings within the valley alluvial aquifer.

In the Second Impoundment, water levels are close to the channel invert level (el. 1054 to 1055 m amsl) of the RCDC indicating that the water level in the channel may be controlling the water level in the tailings. Further downstream, water levels in the tailings drop below the level of the channel invert (el. 1049 ±), indicating a net potential seepage gradient from the channel towards the tailings. This gradient tends to increase as the relative elevation difference between the channel invert and the tailings surface increases in a downstream direction.

2.3.2.6 Thermal Conditions

The site is located within the discontinuous permafrost zone. In the north-facing slope along the alignment of the RCDC, almost continuous frozen ground was observed (Golder, 1980). Thermistor installations indicated ground temperatures in the range of 0 to -1^{o} C. No temperature measurements are available in the tailings. Based on the penetration resistances measured in the recent cone penetrometer testing, there appears to be no evidence for frozen tailings.

3. CLOSURE SCENARIOS

In the course of assessing the various closure Scenarios outlined in Section 1, several modifications were evolved based on the geotechnical and hydraulic considerations of the site. Details concerning the geotechnical aspects and considerations for each of these Scenarios are summarized in this section.

The following Scenarios were considered:

- Scenario 1- Increase the flood handling capacity of the existing channel by raising the height of the canal dike on the right bank.
- Scenario 1a- Increase the capacity of the existing channel by widening the channel by 5 m on the left bank side.
- Scenario 1b- Same as Scenario 1, except replace the drop weir section with a concrete stepped spillway and fish ladder to handle PMF flows.
- Scenario 2- Raise existing dike between CS 31 and CS 39 as in Scenario 1, but divert PMF flow across the tailings at the plug dam, using a dike constructed on tailings to direct the flow to a spillway at the Intermediate Dam.
- Scenario 3- Removal of tailings from Original, Second and Intermediate Impoundment to el. 1042 and divert Rose Creek PMF to enter impoundments immediately downstream of the Pumphouse Pond.

It should be noted that for each of these Scenarios there are various levels of unknowns and assumptions, which have implications on cost. Therefore the cost estimates presented should not be compared as if they were based on equal levels of uncertainty. A common cost element for all Scenarios considered is the need to undertake a program of seismic upgrading of the entire Down Valley tailings disposal area, which would be in addition to the earthworks costs presented here. This would involve upgrading the Intermediate and Cross Valley dams to withstand a Maximum Credible Earthquake (MCE) event. The level of upgrading required under each of the above scenarios may vary, depending on design details. For the purposes of this study, the seismic upgrading costs were assumed to be equal for each scenario, and therefore could be ignored in preparing the relative costs for each scenario.

3.1 Scenario 1

In this Scenario, the canal dike on the right bank is raised to provide sufficient freeboard for the PMF flow. All the PMF flow will be handled in the channel. The following assumptions were made with respect to this Scenario:

- The existing vegetation on the left bank would be left in place between CS39 and CS 31 to preserve the permafrost and provide some measure of erosion protection.

- Clearing and grubbing will be required over the entire footprint of the new dike on the right bank. Stripping of the ground surface, including the surface of the existing dike to a minimum depth of 0.5 m will be required.
- New construction on the right bank associated with the dike raising would not encroach or constrict the existing hydraulic channel.
- Nominal dike section is based on a crest width of 6 m and side slopes of 2 horizontal :1 vertical.
- In general, the entire existing channel required an upgrade of the rip rap to ensure channel stability under PMF flow conditions. Since the size of rip rap required will vary depending on local flow conditions and the thickness of the rip rap layer is a function of the maximum particle size a nominal thickness of 0.5 m rip rap lining was initially assumed for preliminary layout purposes. The quantities of rip rap included in the estimate however are based on the actual thicknesses assessed from hydraulic criteria by NHC. The rip rap zone extends from the crest of the new dike down to the toe of the existing channel. Rip rap is also required on the left (south) side slope of the RCDC between CS 10 and CS 31 up the hydraulic level (1 m above the water level).

The 39 channel cross-sections generated from the 2003 survey were the basis for the design of the conceptual dike sections. There is no “as-built” information regarding the construction details of the existing channel, so there was no opportunity to detail the interface between the new construction and the existing structure.

The dike raise was assumed to be made as a continuous extension of the existing dike slope in order to place the new dike within the existing dike footprint as much as possible. NHC provided water levels for PMF flow of 730 m^3/s at each of the 39 cross-sections. The top of the impervious core or water retention element of the new dike was set a nominal 1.0 m above the 730 m^3/s water level. The physical crest of the dike was set 1.0 m above the top of the impervious water retention element.

The dike section will utilize natural materials similar to those used to construct the existing structure. This includes compacted till for impervious liner, compacted sand and gravel or rockfill for dike shell, processed sand and gravel for filter and transition and crushed rock as rip rap base. The existing quarry located near the Intermediate Dam was assumed to be the source of the rip rap. This quarry was used to construct the existing diversion and the recent breach of the Fresh Water supply dam. Further investigations would be required to determine if the quarry has sufficient volumes of rock to satisfy the requirements of the PMF channel. If not, other quarry locations need to be identified.

Various reaches of the RCDC required specific design modifications in order to accommodate local site conditions. These are described in the following sections, from upstream to downstream.

3.1.1 Original Diversion Section (CS 39 to CS 30)

In the upstream section between CS 31 and CS 39 the proximity of the Second Impoundment tailings presents a concern regarding seepage flow between the channel and the tailings. The tailings surface is about 6 m above channel invert at CS 39. As discussed in Section 2.3.2.4, there is an overall seepage gradient from the diversion channel into the tailings. There is also a potential seepage gradient from the tailings impoundment towards the diversion channel during periods of runoff and infiltration from the tailings surface. Since the existing conditions have performed satisfactorily to date, it was assumed that no impervious liner should be placed on the existing dike slope, since it could prevent seepage from the tailings from draining during periods of infiltration, leading to a decrease in the overall stability of the slope.

The new dike extension incorporates an impervious liner on the canal side to prevent seepage loss through the dike during periods of flood flow. The base of the liner coincides with the elevation of the tailings in the Second Impoundment. The outer toe of the new dike extension is founded on the tailings. To account for varying tailings properties and potential for settlement, the outer dike slope was flattened to 3.5 H to 1 V. The dike shell is constructed of compacted sand and gravel. Maximum dike height above the tailings surface is about 5 m.

3.1.2 Fuse Plug Dam to Intermediate Pond (CS 30 to CS 14)

In this reach, the invert of the diversion channel begins to rise above the elevation of the adjacent tailings in the Intermediate Impoundment. There appears to be sufficient distance between the outer toe of the existing dike and the edge of the beached tailings to allow construction of the dike extension. The foundation conditions were inferred from previous investigation data and were assumed to comprise bedrock covered by sand and gravel alluvium associated with the former (pre-1981) Rose Creek diversion channel.

A fine compacted rockfill shell (200 mm minus) can be used in this section. The outer slope was shown at a conservative 2H:1V slope, however this could be steepened to 1.5H:1V when foundation conditions are confirmed in subsequent design phases.

The use of a rockfill shell requires the addition of a zone of compacted sand and gravel to act as a transition between the impervious liner and the rockfill. The fine rockfill was selected to maintain compatibility with the transition material. Detailed design of the dike zoning will be carried out during subsequent design phases when particle size gradations of actual construction materials are obtained.

On the canal side, the existing dike slope will be excavated to remove the existing rip rap and sub-excavated to place the impervious liner. To avoid encroachment of the new dike section into the hydraulic section of the PMF channel, a minor amount of additional material must be excavated. The crest of the new dike will be about 10 m above the existing ground surface at the upstream end and about 15 m high at the Intermediate Pond.

3.1.3 Intermediate Pond to Intermediate Dam (CS 14 to CS 11)

In this reach there is limited space between the outer toe of the existing dike and the edge of the Intermediate Pond. The invert of the diversion channel is about 3 m above the pond level of 1047.3 m amsl. Construction of the new dike will require unwatering a portion of the Intermediate Pond, as the outer toe will encroach on to the tailings. A rockfill cofferdam with a dumped till cofferdam seal on the outer slope will be constructed first to at least 1 m above the Intermediate Pond water level. Pumping can then unwater the enclosed portion of the Intermediate Pond.

There is no information available concerning the bathymetry in the Intermediate Pond, so the layout and volumes of materials are subject to change depending on site conditions. The shell of the dike will be constructed of compacted sand and gravel in order to be compatible with the tailings foundation. In subsequent design phases, further refinements could be made by incorporating rockfill and sand and gravel zones to accommodate actual site conditions and material characteristics.

On the canal side a new liner and rip rap erosion protection layer will be placed from the channel invert to the top of the dike. New rip rap will be placed on the left bank and channel of the RCDC as well.

3.1.4 Intermediate Dam to End of Existing Dike (CS 11 to CS 7)

This reach lies adjacent to the Cross Valley Pond, the Cross Valley Dam and the start of the drop weir section of the existing diversion channel. The drop weir section begins at CS 9. Beyond the outer toe of the existing dike, the original ground surface slopes down to the Cross Valley Pond. The elevation difference between the channel bottom of the existing dike and the surface of the Cross Valley Pond is about 18 m.

Foundation conditions vary from sandy gravely till in the upstream portion to alluvial sand and gravel downstream of the Cross Valley Dam. Due to the potentially pervious nature of these materials and seepage gradients into the slope above the Cross Valley dam, an impervious liner was included extending over the entire right bank channel slope. Further design work is required in this reach to confirm the design assumptions and properly assess the seepage conditions and the final design of the required liners. For the purposes of this study, it was assumed that the water in the channel was hydraulically confined by the groundwater table on the left bank, and that the impervious liner would only be required to prevent seepage through the right bank slope and dike.

The outer shell of the new dike can be constructed using compacted rockfill. The depth of stripping required may be up to 1 m deep to remove thicker accumulations of organic debris in this area. The overall height of dike raising required in this reach is about 5-6m to provide the required freeboard for PMF flows.

Rip rap will be required on both banks and the channel invert down to CS 10. Downstream of CS 10, the mean channel velocity increases to 10 m/s. and 2 m sized (D_{100}) stone size will be required for bank protection. This stone size may be impractical to obtain or place along the RCDC.

3.1.5 Downstream Section (CS 7 to CS 1)

This reach includes the downstream portion of the drop weir section and the outflow area where the diversion channel re-enters the natural Rose Creek channel. The drop weir section ends at CS 3. There is no existing dike along this section and a new dike must be constructed to provide the freeboard requirements for the PMF flows.

The existing channel is excavated into predominantly sand and gravel deposits associated with an alluvial fan of a tributary creek flowing down the south valley wall.

The proposed dike section is similar to that in the previous reach, consisting of a compacted rockfill shell and a full impervious liner on the right bank. There is no information available to determine if the existing channel is lined. The height of the new dike will be about 6-7 m above the surface of the existing channel bank.

Due to the high channel velocities in this reach, use of rock rip rap for channel bed and bank protection becomes impractical. The required rock sizes may be unobtainable within a reasonable haul distance of the site. As described in Section 3.3, Scenario 1b was developed to further address the need for a concrete spillway to handle the PMF flow downstream of CS 9.

3.1.6 Quantities and Cost Estimate

Table 1 summarizes the earthworks quantities and costs associated with Scenario 1 and does not include the additional costs for a spillway, outlet channel and fish by-pass, which are presented by NHC. The estimated cost for the earthworks portion associated with raising the dike along the entire channel was about $16.1 million.

Scenario 1b, which includes the costs associated with a concrete spillway downstream of CS 9 was subsequently developed as a preferred option over Scenario 1 and is discussed in more detail in Section 3.3.

3.2 Scenario 1a Widen Existing Channel by 5 m

This Scenario involves expanding the existing channel invert by 5 m into the left (south) bank. The side slopes are assumed to be inclined at 2H :1V. This Scenario was briefly considered to assess the relative merits of raising the dike versus increasing the width of the channel to increase existing channel capacity under Scenario 1. Based on 12 cross sections, the volume of excavation required to widen the channel is about 124,000 m^3. Assuming a unit cost for common excavation of \$8.00/m^3 the estimated cost for removing the material is about \$992,200. An additional \$2 million would be required to clear and grub this area as the channel expansion will involve cutting into the forested hillside pf the Rose Creek valley. Hydraulically, the 5 m expansion would reduce water levels in the channel, but still require construction of a new dike extension on the north bank of the channel, although not quite as high as in Scenario 1. A spillway, spillway channel and fish by-pass channel would also be required downstream of CS 9.

The estimated cost of the reduced dike raise on the right was estimated to be about \$12.1 million based on the cost estimate for a full dike raise provided in Scenario 1. The total estimated cost, including the additional clearing/grubbing and channel invert excavation is about \$15 million.

Environmentally, this Scenario increases the overall footprint of mine disturbance, for at most a marginal savings over Scenario 1. The main concern is the potential for long term degradation of permafrost-affected slopes in the left bank of the channel. The above cost assessment did not include the cost of thermal protection measures for the excavated slopes, which would require over-excavating beyond the nominal 5 m width and covering the slope with thaw-stable thermal protection materials. These costs would definitely drive the cost above the cost estimated for Scenarios 1 and 1b.

This Scenario was not considered to be a viable solution for handling the PMF flow.

3.3 Scenario 1b

This Scenario evolved from Scenarios 1 and 1a after it was determined that the channel bed in the rock drop weir section could not be made stable under PMF flow conditions with rip rap alone. Stabilizing Scenarios included obtaining larger size armour stone or embedding the riprap in concrete. Both of these Scenarios were not considered to be viable at the site for the following reasons:

- The required armour stone size (2-m, or larger) cannot be obtained from local sources and would be impractical to place along the RCDC.
- The concrete used to embed the rip rap would not stand up to the severe freeze-thaw action at the site over a long term period.

The only viable alternative is to replace the existing rock drop weir section with a concrete structure. In this Scenario, downstream of CS 10, the existing channel would be replaced by a concrete approach wall and floor leading to a concrete spillway and chute on the south abutment of the Cross Valley Dam. Between CS 10 and CS 11, the right side of the existing channel will be excavated to create the flow expansion leading into the approach channel at elevation 1048 m amsl.. At this point, the existing channel will continue below CS 10 as a fish by-pass channel The spillway chute discharges into a depressed stilling basin and an energy dissipating channel, containing scattered large boulders on top the channel rip rap. This leads to a rock lined channel, which returns the PMF flow into Rose Creek downstream of the Cross Valley Dam.

3.3.1 Quantities and Cost Estimate

Table 2 presents a summary of the quantities and capital cost estimate for the earthworks portion of Scenario 1b. The total capital cost of $13.4 million, excludes mobilization, demobilization, escalation and extra work allowances. The unit rates assumed for the cost estimate were in part derived from previous cost estimates used by BGC for construction work in the mine site area, but are nevertheless subject to revision depending on prevailing conditions at the time of construction. The associated cost estimates for the spillway, outlet channel and fish by-pass channel are presented by NHC.

The following points summarize the assumptions and comments regarding each of the major work items:

- Item 1, Clearing and grubbing includes the clearing and removal of all vegetation along the right bank footprint of the dike, the left bank between CS 10 and CS31 and the footprint of the spillway. Most of this work will be within the existing zone of clearing along the RCDC channel and is expected to involve removal of secondary shrub growth along the existing dike slopes and adjacent natural ground areas. No logging is expected to be required.
- Item 2, excavate and stockpile existing rip rap assumes that the rip rap being removed will not be re-used as the size of the rip rap must in general be increased to meet PMF flow requirements. The cost is based on the removal of the rip rap from the existing channel and hauling it to a stockpile area within 6 km of the site. The stockpile will be used for other mine site reclamation activities. The estimate of the volumes assumes rip rap will be removed along the entire length of the existing dike slope, from the crest

down to the channel invert. This volume is expected to be conservative as some sections of the channel are known to have no rip rap coverage.

- Item 3, excavate and stockpile existing liner assumes that the excavated liner will not be re-used. Although there is no information to indicate how much of the channel is lined, the volume estimate is based on removing a 1 m thick slice of soil from the existing dike slopes. This was assumed to be required in conjunction with the rip rap upgrading, which involves placing thicker layers of rip rap than currently exist. Due to the potential for contamination of this material by other materials, it will not be re-used as liner material. It is anticipated that a stockpile area will be used within 6 km of the site so that this material can be re-used for other mine site reclamation activities.
- Item 4, common excavation and stripping includes the removal of 0.5 to 1.0 m of material from within the footprint of the new dike and the left bank and channel sections where new rip rap is required and hauling it to a stockpile or waste disposal area within 6 km of the site. The volume includes removal of the impervious cofferdam seal for the portion of the dike constructed in the Intermediate Pond.
- Item 5, placing Zone 1 impervious liner assumes a minimum 1 m thick liner along the entire length of the channel, except as noted above for the upstream section adjacent to the Second Impoundment. This volume may be reduced when more information is available concerning the as-built conditions along the dike. The borrow source is assumed to be on the Vangorda Plateau, a one-way haul distance of 26 km. The price includes the cost for excavating, hauling, placing, spreading and compacting the liner.
- Item 6, placing Zone 3 Crushed rock transition/underlay assumes a borrow source within 6 km of the site. This item is used as rip rap bedding consisting of coarse crushed rock with a maximum particle size of 130 mm. The cost includes drilling and blasting, hauling to a stockpile, crushing and screening, loading and haul to site; placing, spreading and compacting.
- Item 7, placing Zone 3a Pit Run gravel transition/underlay assumes a borrow source within 6 km of the site. The cost includes excavating and loading the material from a granular borrow area, processing including screening and washing, loading and hauling to site, dumping spreading and compacting.
- Item 8, Zone 4 sand and gravel shell is pit run sand and gravel from a borrow source within 6 km of the site. The cost includes excavating, hauling, placing, spreading and compacting.
- Item 9, Zone 7 rockfill shell assumes rockfill will be obtained from a quarry located within 6 km of the site. The unit cost includes drilling, blasting, loading, hauling, spreading and compacting. Note that the rockfill may be required to meet a gradation specification (say 200 mm minus) and the rock must be proven to be non-acid generating. This excludes using waste rock from the mine dumps, unless a source of non-acid generating rock is identified. Rock from the existing quarry near the Intermediate Dam has been proven to meet these requirements. Other quarries may be required in order to meet the project requirements.

- Item 10, Zone 9, rip rap, this quantity is based on the actual thicknesses of rip rap liner required to ensure stability of the channel slopes during PMF flow conditions. For estimating purposes, the source was assumed to be in the existing quarry at the Intermediate dam, although there is no confirmation that this source will be able to satisfy all the size requirements.
- Item 11, Zone 1A, cofferdam seal assumes using till from the Vangorda Plateau, as for the Zone 1. The cost for this material assumes excavate, load, haul and dump, with no compaction. Removal of this material after completion of dike construction is included in the volumes for common excavation (Item 4). During construction, it may be possible to utilize some of the material removed from the existing channel excavation for cofferdam seal, but this will depend on material quality and scheduling.

3.4 Scenario 2

This Scenario assumes that an engineered cover material 2 m thick has capped the entire Intermediate Impoundment. It must be noted that the overall philosophy of this Scenario is at odds with the idea of placing a cap on the tailings. The objectives of the tailings cap are to minimize infiltration and to prevent oxidation of the tailings. Routing the PMF flood on top of the engineered cap will increase the potential for infiltration of oxygen rich water into the underlying tailings, thereby diminishing its effectiveness as a cover. Also, the portion of the PMF channel constructed on the tailings will require a liner over the entire channel width to replace the cap, as well as erosion protection.

The existing canal dike between CS 31 and CS 39, would be raised as in Scenario 1. At the plug dam, the existing dike would be removed and the PMF flow would be diverted over the Intermediate Impoundment via a channel contained by a dike constructed on the tailings cap along the right bank. The expanded channel would extend from CS 31 to CS 25. The existing diversion dike would be removed in this reach. The invert of the PMF channel at CS 28 would be 1054 m amsl, similar to the existing diversion channel at that point. Most of the channel would be excavated into native ground down to CS 25. The PMF channel dike and portions of the right channel bottom will be constructed on the tailings cap.

At CS 25, a headwall would be constructed across the existing diversion channel, with a 20-m long conduit sized to allow a maximum flow of 30 m^3/s down the existing diversion for fish passage. A second 450-m long fish passage conduit will be placed from just upstream of CS 11 to the left wingwall of the spillway at CS 8, where it will re-enter the existing diversion channel The existing dike on the right bank will act as a splitter wall to separate the fish channel from the PMF channel between CS 25 and CS 11. The invert of the expanded channel at CS 25 is 1052.75 m amsl.

Downstream of CS 25, the PMF channel has a bed width of 80 m with side slopes at 2H:1V. The left bank of the PMF channel will be a slope excavated into the existing diversion channel dike, maintaining a minimum crest width of about 7 m. Between CS 25 and CS 17, most of the channel section will be excavated in the natural ground between the existing channel and the edge of the tailings. Portions of the right side of the channel will require excavation be founded on tailings. The right bank slope of the PMF channel will be excavated through the tailings cap and protected with rip rap. The dike running along the north side of the PMF channel will be constructed on top of the cap. To minimize potential displacements of the cap due to the embankment surcharge, the interface between the cap and the base of the PMF channel dike will be covered by a layer of geogrid. The dike itself should be constructed of compacted sand and gravel to accommodate any long-term deformations or settlement that may occur due to consolidation of the underlying tailings and to be compatible with the gradation of the cap material.

From CS 17 to just upstream of CS 14, the proportion of the PMF channel on the tailings cap increases significantly. By section CS 15 and all the way to the Intermediate Dam, the channel will be entirely on the tailings cap, except for the left bank area. Since the channel invert is set by hydraulic requirements, portions of the channel will be excavated through the cap and into the underlying tailings. The current water level in the tailings in this area is controlled by the Intermediate Pond, at elevation 1047± m amsl. Channel invert level at CS 13 is 1045.4 m amsl, which would require excavating in tailings below the water table. In order to construct this excavation, water levels in the tailings must be substantially reduced in advance of construction.

It has been assumed that the Intermediate Pond will disappear when the tailings cap is constructed. As a minimum, the depression will be filled in at least to elevation 1047.3, the current water level, so that no water would accumulate on the surface of the cap. In that case, construction of the PMF channel in the vicinity of the Intermediate Pond area may require about 2.5 to 3.0 m of excavation through the cap material between CS 12 and CS 11.

At the right abutment of the Intermediate Dam (CS 11), the PMF channel becomes confined by a concrete wing wall on the right bank and a concrete lined bed across to the left bank toe. The invert elevation of the concrete lined bed at CS 11 is 1044 m amsl. The PMF channel merges with the existing diversion channel downstream of CS 11 to a concrete spillway headworks at the Cross Valley Dam. In the reach between the Intermediate Dam and the Cross Valley Dam, the PMF channel consists of a right bank concrete wing wall to contain the flow along the sidehill above the Cross Valley pond. The concrete lined channel bed has a width of 30 m and extends to the toe of the left bank slope. The left bank slope comprises a 5H:1V cut and fill section originating from the existing left bank of the diversion channel. This slope will be heavily armoured with rip rap all the way to the spillway. The fish conduit is located along the alignment of the present RCDC, which will be affected by the cut and fill sections.

At CS 8, at the left abutment of the Cross Valley dam, the PMF channel enters the spillway headworks. A concrete wing wall must be constructed along the left side of the channel to cut off flow down the existing diversion channel. The fish conduit will pass through the wing wall and discharge into the existing RCDC. The spillway has an overall length of 172 m and a width of 30 m.

3.4.1 Quantities and Cost Estimates

Table 3 summarizes the preliminary estimates of quantities and capital costs for the earthworks portion of Scenario 2. The estimated capital cost is about $29.5 million excluding mobilization and demobilization, escalation and extra work allowances. The estimated capital cost of the spillway, outlet channel and fish by-pass channel is presented separately by NHC.

No costs have been included for dewatering tailings in the Intermediate Impoundment during construction. Further information would be required regarding the long-term water table situation in the tailings if a tailings cap is in place. The preliminary quantities were estimated from cross-sections provided by NHC along the channel showing the required hydraulic profile of the channel.

The following sections describe the assumptions made for each of the major work items listed in Table 3:

- Item 1, Clearing and grubbing includes the clearing and removal of all vegetation along the new PMF channel and spillway footprint. Under Scenario 2 it was assumed that the tailings area has been capped by 2 m of cover material by others and that no clearing or grubbing would be required. Most of the work is expected to involve removal of secondary shrub growth along the existing RCDC and the area between the RCDC and the tailings impoundments. No logging is expected to be required.
- Item 2,Excavate and stockpile existing rip rap includes removing the existing rip rap from the RCDC channels that will be modified (downstream of CS 31) and replacing the existing rip rap between CS 31 to CS 39. It was assumed that the rip rap could not be re-used as the new channel will require a larger size. The existing rip rap will be removed and hauled to a stockpile within 6 km of the site, for re-use in other mine site reclamation work.
- Item 3 Excavate and stockpile tailings cap, includes removal of portions of the tailings cap along the PMF channel in order to establish the required channel grades. Since this material was specifically selected as a cover material based on its properties, it requires segregation from the other materials so it can be re-used for other mine reclamation applications. A stockpile area for this material was assumed to be located within 6 km of the site.

- Item 4, Common excavation and stripping includes the removal of 0.5 to 1m of material from within the footprint of the dike where it is constructed on natural ground or on top of the existing dike (CS 31-39). It also includes stripping of the ground surface under the areas where new rip rap will be placed in the channel bed and left bank area between CS 10 to CS 31 and under the footprint of the spillway. Common excavation does not include tailings, which are covered under Item 5. The cost assumes that the material will be hauled to a stockpile or disposal area within 6 km of the site.
- Item 5, Excavate and dispose of tailings includes the excavation, hauling and disposal of tailings in a designated disposal area. Tailings excavation will be required in some sections along the PMF channel in the Intermediate Impoundment (CS12-15). Since the tailings contain sulphides and metals and are potentially acid generating, they must be disposed of in an environmentally secure area. It was assumed that the excavated tailings would be disposed of in the flooded Faro Pit, which already contains tailings.
- Item 6, Place geogrid under dike foundation. This includes the section of dike constructed on the tailings cap between CS 28 to CS 11. The geogrid will be placed on the surface of the tailings cap. Depending on the type of cap material used, some surface preparation or stripping or both may be required to install the geogrid. A 1 m zone of stripping was included in the cost estimate under Item 3 for this purpose.
- Item 7 Zone 1, Impervious liner, assumes a 1 m thick liner along the entire length of the channel between CS 31 to CS11.1 (start of concrete lined section). The extent of the liner needs to be verified based on as-built conditions in the reach between CS 31-39 adjacent to the Secondary Impoundment. Additional liner may be required in this reach to prevent infiltration of water into the tailings area, which have been capped at great expense to keep water out. Nevertheless the capping will not prevent groundwater seepage entering the tailings from the valley sides. No liner was allowed for on the left bank along the concrete lined section downstream of CS 11.1. Final details of seepage control requirements must be based on details to be obtained in future investigations.
- Item 8, Zone 3 Crushed rock transition/underlay. This material is required as underlay for all rip rap areas between CS 9-31. The crushed material is specified as having a maximum particle size of 130 mm and a D_{50} of 40 mm. It was assumed that this material would be produced from quarry rock by drilling and blasting. The rock would be hauled to a stockpile for crushing, screening and washing. The cost also includes loading from the processed rock stockpile and hauling, spreading and compacting the material on the dike and channel.
- Item 9, Zone 3a Pit run gravel transition/underlay. includes the cost of excavating, hauling, processing (screening and washing), loading from the processed stockpile, hauling, spreading and compacting this material, which is used as rip rap underlay between CS 31-CS 39. The borrow source is assumed to be within 6 km of the site.
- Item 10, Zone 4 sand and gravel shell for the dike assumes suitable pit run sand and gravel can be obtained from a borrow source located within 6 km of the site. The cost includes excavating, hauling, spreading and compacting.

- Item 11, Zone 9, rip rap includes all the size classes specified by NHC on the basis of channel and bank velocities along the PMF channel. The D_{50} stone size for bank and channel bed protection ranges from 25 mm to 600 mm, with layer thicknesses ranging from 300 mm to 1.0 m.

3.5 SCENARIO 3- PASS PMF FLOWS OVER TAILINGS IMPOUNDMENTS

The final Scenario involves removal of all the tailings (to be done by others) in the Original, Second and Intermediate Impoundments down to elevation 1042 m amsl. The Rose Creek PMF will enter the impoundments immediately downstream of the Pumphouse Pond. The attenuated PMF will pass over a new spillway located in the north abutment of the Intermediate Dam.

Under this scenario, the existing Rose Creek Diversion channel will remain as is. A new headwall will be constructed across the existing channel in the vicinity of CS 39. A conduit through the headwall will allow a maximum flow of 30 m^3/s for fish passage in the RCDC.

The excavated tailings will be relocated into the abandoned Faro pit. The remaining tailings will be flooded to elevation 1045 m amsl with a water cover. The existing spillway on the north side of the Intermediate Dam has a sill elevation at 1045 m amsl and discharges into the Cross Valley Pond. The lowest point on the Intermediate Dam, as surveyed in 2003, was 1048.68 m amsl. The top of the impervious core is at elevation 1049.2 m amsl. Assuming an initial water cover at elevation 1045 m amsl, the PMF water level behind the Intermediate Dam is predicted to be at 1048.1 m amsl, leaving 1.1 m of freeboard on the impervious core.

A new concrete spillway will be constructed on the north bank, replacing the present unlined emergency spillway channel that discharges into the polishing pond. The new spillway will have a 55 m wide sill at elevation at 1045.0 m amsl and pass the PMF flood over the Intermediate Dam and around the Cross Valley Dam and Polishing Pond. The upstream portion, between the Intermediate Dam and the Cross Valley Dam will be stepped, leading to a chute and stilling basin downstream of the Cross Valley Dam.

The north bank location for the spillway was chosen over the south bank because of the width of spillway required. Topographically there is more room along the north side to construct this spillway. Foundation conditions are expected to comprise a mixture of till, sand and gravel and colluvium. Bedrock in the north abutment of the Intermediate Dam, under the existing spillway is at about elevation 1040 m amsl. At the Cross Valley Dam, the bedrock under the spillway channel is at about elevation 1015 m amsl.

There are no additional earthwork costs associated with this scenario as all associated removal of tailings will be done by others. The estimated capital cost of the new spillway, outlet channel and fish by-pass channel are presented separately by NHC. The additional earthworks required for this scenario are associated with the seismic upgrading program of the Down Valley tailings impoundment. Estimating these requirements is beyond the scope of the present study, but these are described in an introductory fashion in the next section.

3.5.1 Seismic Design Considerations

In 2002, Klohn Crippen Consultants Ltd undertook an independent dam safety review of the dams on the Anvil Range property, including the Intermediate and Cross Valley dams (KC, 2002). A "Very High" consequence category was assigned to both the Intermediate and Cross Valley dams based on the environmental consequences of a major uncontrolled release of tailings from the Down Valley tailings area. This classification then requires the following:

- The safety of these structures must be ensured against extreme events such as the Probable Maximum Flood (PMF) and Probable Maximum Earthquake (MCE).
- Independent safety reviews should be conducted every five years, and
- The dam owner is required to prepare and Emergency Preparedness Plan (EPP).

Under Scenario 3, the tailings area will be designed to accommodate the PMF. BGC (2003) has recently completed an EPP as well as an Operation, Maintenance and Surveillance (OMS) Manual for dams on the Anvil Range property, including the Intermediate and Cross Valley dams, in their current configuration. The major outstanding requirement is an overall assessment of the seismic stability of the Down Valley tailings area, including the Intermediate and Cross Valley dams. Elements of this assessment are understood to be underway, which includes the cone penetrometer field program recently completed by Golder (Golder, 2003). As part of the 2002 dam safety review, Klohn Crippen undertook a preliminary seismic stability assessment of the Intermediate Dam. The analysis used peak ground accelerations of 0.15 g for the 1:10,000 year return period earthquake and the assumption that the tailings did not liquefy. The analysis indicated potential for serious damage to the dam, but would not likely result in the release of water or tailings. The assessment noted that the potential for fluid release would be further reduced if the Intermediate Pond was lowered. The stability of the Intermediate Dam is also dependent on the water level in the polishing pond. Rapid drawdown of the water level in the Polishing Pond could lead to instability of the downstream shell of the Intermediate Dam.

With the proposed water cap set at elevation 1045 in Scenario 3, the overall stability of the Intermediate Dam must be assessed under both static and MCE seismic loading conditions. The seismic assessment should include the overall assessment of liquefaction potential in the down valley area and effects on the stability of the Intermediate and Cross Valley dam. No cost allowance has been included for the structural upgrades required for the two dams to meet MCE requirements. Note that some type of seismic upgrade will be required for all scenarios considered. Therefore the embedded cost for this aspect should be considered to be common to all of the options, although the details may vary depending on the actual water levels to be retained behind the Intermediate dam.

4. CONCLUSIONS AND RECOMMENDATIONS

Table 4 is a summary of the preliminary capital cost estimates for the earthworks components for each of the scenarios considered

Table 4 - Preliminary Cost Estimate Summary

Scenario	Earthworks	Comments
1	$16,100,000	Concrete spillway required in drop weir section
1a	$15,000,000	Does not include costs for permafrost protection.
1b	$13,404,527	Same as Option 1 to CS 9, new spillway adjacent to Cross Valley Dam.
2	$29,460,297	Spillway on south abutment of Intermediate and Cross Valley Dams
3	See comments*.	New concrete spillway on north abutment of Intermediate and Cross Valley Dams. Cost for seismic upgrading of Intermediate and Cross Valley Dams not included.

Scenario 1b is the favoured option for increasing the capacity of the existing channel. This option requires a spillway to be constructed in the left abutment of the Cross Valley Dam.

The most expensive scenario is 2, which involves raising the existing dike upstream of CS 31 and constructing a new dike on tailings between CS 31 and CS 11.1. The widened PMF channel will be partially constructed across the tailings impoundment. The approach channel and spillway from the Intermediate Dam to the Cross Valley Dam involves construction of a concrete wall and floor.

Scenario 3 requires only a new spillway to be constructed around the north abutments of the Intermediate and Cross Valley Dams. The additional hidden cost associated with this option is the cost for removal and relocation of the tailings down to elevation 1042 m amsl. The seismic upgrading costs, common to all the other scenarios considered represents the only significant civil earthworks cost component.

Assuming the embedded costs for seismic upgrading of the Down Valley tailings are approximately equal for each Scenario and the costs associated with the tailings relocation are justified, then Scenario 3 is also expected to be the most favoured and recommended option for handling the PMF flows in the RCDC during closure. This Scenario also offers several environmental advantages with respect to closure of the tailings impoundments:

- A significant volume of tailings will be removed from the tailings impoundments and placed into more secure storage in the Faro Pit.
- No new construction or exploitation of natural resources would be necessary to construct a tailings cap, assuming the lowered tailings would be flooded with a water cap.
- Seismic upgrading of the Down Valley tailings impoundment would ensure that the existing facility is stable in the post closure period at a cost equivalent to the other options.
- There would be no significant increase of the current mine disturbed footprint, as the new spillway and seismic upgrading would take place within the existing footprint.
- There is more information available to proceed with the design of Scenario 3 compared with the information gaps leading to design and construction of the other options.

5. CLOSURE

This memorandum summarized the geotechnical considerations and associated cost estimates for earthworks for various closure options designed to increase the capacity of the RCDC to pass PMF flows.

We trust that this information meets with your requirements at this time. Should you require any additional information, or have any questions, please do not hesitate to contact the undersigned at your convenience.

Yours truly,
Per BGC Engineering Inc.

Holger Hartmaier, M.Eng., P.Eng.
Senior Geotechnical Engineer

Reviewed by:

James W. Cassie, M.Sc., P.Eng.
Specialist Geotechnical Engineer

HHH/sf

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TABLE 1: Preliminary Cost Estmate						
Rose Creek Diversion Channel Closure Options						
Scenario 1- Raise Existing Diversion Dike for PMF 800 m^3/s flow:						
ITEM	**DESCRIPTION**	**ESTIMATED QUANTITY**	**UNIT**	**UNIT PRICE**	**EXTENSION**	**COMMENTS**
1	Clearing and grubbing	272,961	m^2	$8.00	$2,183,688.00	Assumes right bank dike footprint only.
2	Excavate and stockpile existing rip rap	17,742	m^3	$8.00	$141,936.00	Assumes existing rip rap will not be re-used.
3	Excavate and stockpile existing liner	35,483	m^3	$8.00	$283,864.00	Assumes existing liner will not be re-used.
4	Common excavation and stripping	187,361	m^3	$8.00	$1,498,888.00	Material will be hauled to stockpile or waste area within 6 km. (includes removal of cofferdam seal).
5	Place Zone 1 impervious liner	94,885	m^3	$19.00	$1,802,815.00	Assumes new material to be obtained from Vangorda Plateau- 26 km one-way haul distance.
6	Place Zone 3 Crushed rock transition/underlay	7,503	m^3	$12.00	$90,036.00	Blasted rock used for crushing obtained from quarry within 6 km haul distance.
7	Place Zone 3a Pit run gravel transition/underlay	11,560	m^3	$14.00	$161,840.00	Assumes borrow area within 6 km, includes washing and screening and double handling.
8	Place Zone 4 sand and gravel shell	207,668	m^3	$7.00	$1,453,676.00	Assumes processing and hauling from borrow area within 6 km.
9	Place Zone 7 rockfill shell	408,099	m^3	$7.00	$2,856,693.00	Assumes suitable rockfill from waste dumps within 6 km. Note- must be non-PAG.
10	Place Zone 9 rip rap	146,730	m^3	$37.00	$5,429,010.00	Based on FWS bid- rip rap quarry site adjacent to RCDC.
11	Place Zone 1A cofferdam seal	5,475	m^3	$18.00	$98,550.00	Based on new Zone 1 from Vangorda Plateau borrow sources. Dumped only.
	TOTAL				$16,000,996.00	Excludes cost of stepped concrete spillway in drop weir section
	(Excluding mob/demob., escalation					
	and extra work allowances)					

TABLE 2: Preliminary Cost Estmate						
Rose Creek Diversion Channel Closure Options						
Scenario 1b: Same as Option 1 with Spillway at CS 9						
ITEM	**DESCRIPTION**	**ESTIMATED QUANTITY**	**UNIT**	**UNIT PRICE**	**EXTENSION**	**COMMENTS**
1	Clearing and grubbing	265,310	m^2	$8.00	$2,122,480.00	Assumes right bank dike footprint only. Includes spillway area.
2	Excavate and stockpile existing rip rap	14,691	m^3	$8.00	$117,528.00	Assumes existing rip rap will not be re-used.
3	Excavate and stockpile existing liner	29,382	m^3	$8.00	$235,056.00	Assumes existing liner will not be re-used.
4	Common excavation and stripping	161,836	m^3	$8.00	$1,294,684.00	Material will be hauled to stockpile or waste area within 6 km. (includes removal of cofferdam seal and spillway excavation).
5	Place Zone 1 impervious liner	88,345	m^3	$19.00	$1,678,555.00	Assumes new material to be obtained from Vangorda Plateau- 26 km one-way haul distance.
6	Place Zone 3 Crushed rock Transition Underlay	5,984	m^3	$32.00	$191,488.00	Assumes processing and hauling from borrow area within 6 km. Crushing will be of blasted rock.
7	Place Zone 3a Pit run gravel	3,091	m^3	$14.00	$43,267.00	Assumes processing and hauling from borrow area within 6 km.
8	Place Zone 4 sand and gravel shell	200,958	m^3	$7.00	$1,406,706.00	Assumes processing and hauling from borrow area within 6 km.
9	Place Zone 7 rockfill shell	344,604	m^3	$7.00	$2,412,228.00	Assumes suitable rockfill from waste dumps within 6 km. Note- must be non-PAG.
10	Place Zone 9 rip rap	103,074	m^3	$37.00	$3,813,738.00	Based on FWS bid- rip rap quarry site adjacent to RCDC.
11	Place Zone 1A cofferdam seal	5,475	m^3	$18.00	$98,550.00	Based on new Zone 1 from Vangorda Plateau borrow sources. Dumped only.
	TOTAL				$13,414,280.00	
	(Excluding mob/demob., escalation					
	and extra work allowances)					

TABLE 3: Preliminary Cost Estimate						
Rose Creek Diversion Channel Closure Options						
Scenario 2: PMF Channel on tailings:						
ITEM	**DESCRIPTION**	**ESTIMATED QUANTITY**	**UNIT**	**UNIT PRICE**	**EXTENSION**	**COMMENTS**
1	Clearing and grubbing	322,917	m^2	$8.00	$2,583,336.00	Assumes requirements do not pertain to tailings cap area.
2	Excavate and stockpile existing rip rap	15,191	m^3	$8.00	$121,528.00	Assumes existing rip rap will not be re-used.
3	Excavate and stockpile tailings cap	271,845	m^3	$8.00	$2,174,760.00	Assumes minimum 1 m stripping under dike, material hauled to stockpile within 6km for re-use by others
4	Common excavation and stripping	817,950	m^3	$8.00	$6,543,600.00	Material will be hauled to stockpile or waste area within 6 km.
5	Excavate and dispose of tailings	100,865	m^3	$8.00	$806,920.00	Assumes material hauled into Faro Pit for disposal. Say 6 km haul distance.
6	Place geogrid under dike foundation	54,725	m^2	$15.00	$820,875.00	Upstream of CS 11
7	Place Zone 1 impervious liner	230,683	m^3	$19.00	$4,382,967.50	Assumes new material to be obtained from Vangorda Plateau- 26 km one-way haul distance.
8	Place Zone 3 Crushed rock Transition/underlay	126,443	m^3	$32.00	$4,046,160.00	Assumes processing and hauling from borrow area within 6 km using blasted rock for crushing.
9	Place Zone 3a Pit run transition/underlay	11,560	m^3	$12.00	$138,720.00	Assumes processing and hauling from borrow area within 6 km.
10	Place Zone 4 sand and gravel shell	176,533	m^3	$7.00	$1,235,727.50	Assumes processing and hauling from borrow area within 6 km.
11	Place Zone 9 rip rap	178,640	m^3	$37.00	$6,609,680.00	Unit rate based on FWS bid- rip rap quarry site adjacent to RCDC.
	TOTAL				$29,464,274.00	
	(Excluding mob/demob., escalation					
	and extra work allowances)					