SRK CONSULTING INC.

HYDROTECHNICAL STUDY
FOR CLOSURE PLANNING
FARO MINE SITE AREA, YUKON
FINAL REPORT

JUNE 2004

nhc northwest hydraulic consultants

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

HYDROTECHNICAL STUDY FOR CLOSURE PLANNING FARO MINE SITE AREA, YUKON

Submitted to:

SRK Consulting Inc.
Vancouver, BC

Submitted by:

Northwest Hydraulic Consultants Ltd.
Edmonton, AB
&
BGC Engineering Inc.
Calgary, AB

June 2004

6399/4685

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

1.1 Background

This report provides summarizes **nhc**'s hydrotechnical assessment in relation to closure planning for the Faro Mine Area, Anvil Range Mining Complex in Yukon. This updates a preliminary assessment by Northwest Hydraulic Consultants Ltd. (**nhc**, 2001). BGC Engineering Inc. (**BGC**) provided geotechnical input as part of developing closure scenarios involving 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 is described within 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 provide better knowledge of runoff characteristics through correlation with Rose Creek flow data. The assessment was 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.

[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 additional years of flow data are 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 also been followed in the present study. This procedure is referred to as **Method 2**, and includes error band estimates.

---

[4] Data provided by Gartner Lee Ltd., Yellowknife.

[5] Verbal communication with Eric Denholm of Gartner Lee Ltd.

Initially, a homogeneity test was performed on the annual flood peak series of eight gauging stations (Table 3) within about 150 km of Faro and with nine or more years of record 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 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 6. The log-log regression fitting lines for the plots are:

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

For mean annual floods

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

$(R^2 = 0.84)$

For 100-year floods

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

$(R^2 = 0.87)$

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. These ratios were determined from the Vangorda Creek record: for the mean annual flood; that is:

Design Inst/Daily Ratio = mean ratio

and for the 100-year flood, approximating the high 95 percentile; that is:

Design Inst/Daily Ratio (95%) = mean ratio + 2 3 standard deviation

Figures 7a through 7f show the synthesized flood frequency plots for the six sub-basin sites.

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.

Extreme Flood Curves. Figure 9 shows extreme flood peak curves for the mean annual to 1000-year flood for mine site drainage areas from 4 to 300 km^2. The flood curves were generated following the procedure used to produce the adopted flood estimates for the six sites listed in Table 5, and enable flood peaks to be estimated for any drainage within the Faro mine site area.

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]

It has been suggested that establishing a Faro Creek streamflow gauge would allow correlation of Faro Creek flow data with the records of Vangorda and/or Blind Creeks, thereby allowing generation of an extended historical Faro Creek flow record.[8] A strong correlation between Faro and Vangorda Creeks records is unlikely to be established, given that local hydrologic events produce different effects on either side of the Faro/Vangorda watershed divide. This is amply illustrated by Table 6, which lists the annual maximum and second highest peak flows in 2000 and 2002 of North Fork Rose Creek and Vangorda Creek; for example, in 2000, the annual peak flow in North Fork Rose Creek occurred in June while in Vangorda Creek the annual peak occurred in August.

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 10 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 periods when data were collected at both stations - June to July

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

[8] From review comments (dated June 15, 2004) by J.R. Janowicz, Manager, Hydrology, Yukon Environment, of the Hydrotechnical Study for Closure Planning draft report (of December 2003) to M. Crombie, Director, Abandoned Mines and Assessment.

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.

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[9].

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 investigated 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.

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

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.

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 Probable Maximum Flood

2.2.1 Background

The routing of extreme floods up to the probable maximum flood (PMF) down Rose Creek diversion is a component of closure planning for the Faro mine area. This section revises the earlier PMF estimates for Rose Creek and provides an opinion as to the confidence level of flood predictions.

The revised PMF estimates are utilized in Section 3 to prepare nominal designs to convey extreme floods up to the PMF through a modified Rose Creek Diversion Channel (RCDC).

2.2.2 Definitions

The PMF is the flood that results from the probable maximum precipitation (PMP). The World Meteorological Organization (1986) defines the PMP as "theoretically the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location at a certain time of year" (from Hansen et al. 1982).

Rainfall is the primary cause of the PMF in the lower latitudes. At the higher latitudes in the Yukon - and especially with large drainage areas - snowmelt, or a combination of rainfall and snowmelt may be the cause of the PMF.

The PMF estimates presented in the following section are based on PMP estimates of maximized rainfall only as drainage areas are small - less than 250 km^2.

2.2.3 Probable Maximum Flood (PMF) Estimates

Inputs

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) and was applied to all streams irrespective of stream length and other characteristics.

Revised Probable Maximum Precipitation (PMP)

Mayo PMP. The 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. The drainage into the Wareham reservoir has an area of about 900 km^2 and lies between El. 600 and 2060 m.

A 24-hour point PMP of 133.5 mm was estimated for the Mayo area (Hogg, W.D, November 2002; see Appendix B) by maximization of the five largest single-day rainfall events in the Yukon[10] using the upper air records from Whitehorse YK, Fairbanks AL and Yakutat AL. A transposition adjustment was not considered necessary in the storm maximization process as the geography and elevation of the observed rainfall events and Mayo are not significantly different. The largest maximized event resulted from the 61.5 mm of rain that fell at Boundary[11] 200 km northwest from Mayo (and 420 km northwest from Faro) on June 31, 1971. The Boundary Met station is located 50 km west of Dawson at an elevation of 1036 m.

A 24-hour average PMP of 116 mm was estimated for the 900 km^2 Mayo drainage by multiplying the 133.5 mm point PMP by an area reduction factor of 0.87.

Faro PMP. This was based on the Mayo PMP 24-hour point PMP estimate of 133.5 mm. The Mayo analysis utilized the largest maximized single-day rainfall event recorded in the Yukon, and therefore is applicable to areas of the Yukon of similar geography and elevation as that of the observed rainfall event[12].

---

[10] The five Met stations surround Faro: 190 km NE, 130 km SSE, 240 & 310 km SW, and 420 km NW.

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

[12] Comment from Wim Veldman (Hydroconsult) at the Anvil Range Mining Corporation 3rd Technical Workshop, Vancouver, February 17, 2004.

The Faro Mine Site and Mayo watersheds lie in rugged terrain at somewhat similar elevations; mine site watersheds range between El.1040 and 1800 m, compared to El. 600 to 2060 m for the Mayo watershed. However the size of the Faro and Mayo watersheds are significantly different. Faro mine site watersheds are relatively small ranging from 16 km^2 for Faro Creek at the confluence with North Fork Rose Creek to 230 km^2 for Rose Creek at Stn. X14 downstream of the tailings complex. By comparison, the Mayo watershed is large covering about 900 km^2.

The Mayo 24-hour point PMP estimate of 133.5 mm was computed from maximization of large area rain events - small area convective storms were excluded (Hogg, November 2002). For the small watersheds of the Faro mine site, localized convective storms imbedded in large area events could produce extremely intense rainfall, particularly at higher elevations. To allow for this possibility, the Mayo PMP estimate was arbitrarily increased by 50 percent for Faro.

A 24-hour point PMP of 200 mm was adopted for the Faro mine site area.

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 7 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)[13]. Times were varied according to drainage area raised to the power of 0.6.

PMF Estimates

The PMF was computed for four locations on Rose Creek (3, 4, 5 and X14 - see Figure 2). The procedure followed was:

- An effective precipitation, that is the proportion of the precipitation that runs off, of 160 mm (80 percent of the 200 mm point PMP) was adopted.

[13] It has been assumed that the flow-through rock drain through the Haul road will be removed and so will not impede flow.

- An effective T_p-hour point PMP was adopted by multiplying the effective 24-hour precipitation by the T_p/24-hour 100-year rainfall ratio for the site area computed from the Rainfall Frequency Atlas of Canada (Hogg and Carr, 1985)[14].

  where T_p = time to peak of 3, 4 or 6 hours (see Table 7).

- The average PMP over each catchment was computed from the point PMP using the World Meteorological Organization (1974) area-reduction curves (see **nhc**, 2001; Figure 11).

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

The estimated PMF peak discharges are listed in Table 8. 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 PMF hydrographs for the four locations are presented in Figure 11.

Table 9 compares the revised and previous PMF estimates (**nhc**, 2001). PMF peaks have been reduced to between 47% and 64% of the earlier estimates.

Sensitivity of PMF Estimates to Time to Peak

The effect on the PMF estimates of varying the adopted time to peak values in Table 7 by 625% is shown below.

Variation in Time to Peak	Effect on PMF Estimate
25% reduction	20% increase
25% increase	10% decrease

Effect of the Haul Road/Flow-through Rock Drain on PMF Estimates

North Fork Rose Creek flow is conveyed through the haul road via the flow-through rock drain (see Figure 2). Currently, the drain retards flow causing water to pond on the upstream side of the road (Photo 7).

The PMF peak estimates for locations downstream of the haul road listed in Table 8 assume that flow is not attenuated by the drain. This requires that either the drain is replaced by a suitably sized conduit to convey peak flows without upstream ponding, or the haul road is removed.

[14] The adopted Tp/24-hour rainfall ratios were 50% for Tp = 3 hours, 56% for 4 hours and 68% for 6 hours.

Upgrading the haul road to a PMF water retention structure would significantly reduce down-valley flood peaks and the cost of conveying the Rose Creek PMF around the tailings ponds. Retaining the total North Fork Rose Creek PMF hydrograph flow volume of 15310^6 m^3 (see Figure 11 hydrograph for Loc. 3) behind an upgraded haul road would create a reservoir with the following dimensions:

- length 2500 m;
- maximum width at haul road dam face 700 m; and
- maximum depth at haul road dam face 30 m, corresponding to El. 1120 m, or 20 m below the haul road crest of El. 1140 m.

The effect of a reservoir that temporarily retains the total PMF entering the reservoir on the downstream Rose Creek PMF peak discharges listed in Table 8 would be to reduce peaks by about:

- 40% (690 m^3/s down to 410 m^3/s) above the RCDC (Loc. 5); and
- 35% (783 m^3/s down to 500 m^3/s) downstream of the RCDC at Sta. X14.

Comparison of Faro PMF Estimates to Recorded Yukon & Canadian Extreme Floods

Here the Faro PMF estimates of Table 8 are compared to recorded extreme Yukon and Canadian floods utilizing the Creager diagram.

The Creager diagram plots peak discharge per unit area against drainage area and is a practical tool for comparing flood data and estimates (Watt et al. 1989). The original diagram plotted "unusual flood discharges" for approximately 730 rivers in the USA and 30 in other countries (Creager et al. 1945). Curves for selected values of Creager's C were drawn through the data; the higher the value of C, the more extreme the flood.

Figure 12 shows the Creager diagram (less the original data points) with the following data superimposed:

- PMF peak estimates for the Faro mine site (Table 8).
- Extreme floods in central Yukon catchments that drain into the Tintina Trench[15] (Table 10). [The Yukon data were limited to floods with Creager C values greater than 10.]
- Selected extreme floods in Canada - none in the Yukon (Watt et al. 1989; Table 3.1).

[15] The Tintina Trench runs in a southeast direction from the Alaska/Yukon border near Dawson across the interior Yukon plateau to about 40 km SE of Ross River. Faro townsite is 60 km NW of Ross River.

The PMF peak estimates for Faro plot just above the C=20 curve in Figure 12. Creager C values range from 22 to 24 for the four mine site locations (Table 8).

Focusing on the portion of the Creager diagram with drainage areas in the 20 to 400 km^2 range - Faro mine site size - two of the four extreme Canadian flood points lie above Faro PMF peak estimates. These four Canadian flood peaks are for B.C. streams draining the Coastal Mountains or the west coast mountains of Vancouver Island, areas with significantly heavier short-duration rainfalls than the relatively dry Yukon interior[16].

The Yukon extreme flood data plotted in Figure 12 are for drainages in excess of 30,000 km^2 or 100 times the Faro area (Table 10)[17]. Creager C values are less than 18 for all data except for the Yukon River flood of June 1964 at Stewart River [C=27] and Dawson [C =30]. The excessive flow of the Yukon River at these locations originates from the White River which drains the high elevation Wrangell and St. Elias Mountains in southwest Yukon. These mountains are in the precipitation shadow of the Coastal Mountains and separate the Yukon interior from moist Pacific air. The extreme White River flow probably resulted from a combination of rainfall and snowmelt. Yukon River data that includes White River flow is therefore not representative of conditions in the relatively dry area of central Yukon where Faro is situated.

In summary, the Creager diagram indicates that the Faro PMF estimates [C = 22 to 24] are reasonable as they exceed the recorded extreme floods of central Yukon [maximum C = 18].

Confidence Level of PMF Estimates

The two most important inputs in the PMF estimation procedure are the PMP and time to peak. The new PMP estimate is based on the 2002 PMP study for Mayo, an area of similar hydrologic characteristics. For Faro, the point PMF has been increased to 150 percent of the Mayo estimate to account for the smaller Faro watersheds and higher elevations. The new Faro PMP estimate has been criticized somewhat with some parties saying the estimate is too low while others say it is too high. Expert review of the Faro PMP is recommended to finalize the estimation.

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.

[16] The estimated 6-hour, 100-year return period rainfalls for the drainages of the four BC streams are 160% to 270% of the corresponding value for Faro. Rainfall estimates computed using the Frequency Atlas of Canada (Hogg & Carr 1985).

[17] Creager C values were less than 10 for all Yukon extreme flood data for catchments smaller than 30,000 km^2, and included the Yukon Water Resource small catchments data set.

Recommendations to Check and Refine PMF Estimates

The following is recommended:

1. **PMP Estimate**. Have a recognized hydrometeorologist with extensive experience in PMP estimation review the Faro PMP. The author of the Mayo PMP study, W.D. Hogg, is recommended.
2. **Time to Peak Estimates**. The establishment by DIAND in December 2003 of remote weather stations at Faro and Vangorda mine sites will enable computation of time to peak in future rainfall/flood events.[18] This will require:

   i) reduction of short duration rainfall data from the DIAND weather station data loggers; and
   ii) computation of discharge hydrographs from the streamflow data logger records of the existing gauging stations, for the rainfall flood periods.

   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.

3. **PMF Estimation using a Numerical Model**. Development of a watershed runoff model to compute PMF values from the finalized PMP. The U.S. Army Corps of Engineers HEC HMS (Hydrologic Modeling System) program is recommended. The preliminary cost estimate for the modelling is $13,000 (not including GST) and would take about four weeks to complete.

[18] The location of the weather stations are shown in Figures 1 and 2.

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 left side of 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 13 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 developed 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 13 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 14 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 13) 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 28-31.
- 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 natural fishway should be constructed to ensure unrestricted fish access upstream and downstream in Rose Creek. A natural 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 weirs 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 it 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 11 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 15 shows the typical design of the upgraded channel along the mildly sloped reaches upstream of cross-section 9 where velocities of up to 3.6 m/s were computed for PMF conditions. 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 and is an impracticable size to use for channel protection on steep slopes. Imbedding large-sized riprap in concrete was considered, but is not suitable for the long term as the protection would not withstand the severe freeze-thaw action.

The practical means of conveying flow down the steeply sloped rock weir section was put on hold at this time, and other variations of Scenario 1 were reviewed.

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 increases the overall footprint of mine disturbance for, at most, a marginal savings in earthworks costs over Scenario 1.

Velocities were still excessively high down the rock drop weir section of the channel, requiring something other than riprap to protect the steeply sloped channel from erosion.

The main geotechnical concern is the potential for long term degradation of permafrost-affected slopes in the left bank of the channel. The earthworks cost assessment in Appendix C of $15,000,000 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 earthwork costs above the $16,100,000 estimate for Scenario 1 and possibly above the $13,400,000 estimate for Scenario 1B (see Appendix C; Table 4).

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 3.3) leading to a concrete spillway down the rock drop weir section 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 Figure 16 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 17 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 on a 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[19]. 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 18 illustrates a suitable generic fish ladder design.

3.2.2 Scenario 1B Costs

Table 12 summarizes preliminary Scenario 1 costs for earthworks, spillway structural concrete, downstream outlet channel and fish by-pass channel.

Preliminary cost estimate: **$32,100,000**.

19 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.

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 13). 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 19a, b and c).

The Figure 19a 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 19b plan:

- CS 14 to 12. Figure 20 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 21 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 19c and 22).

Table 13 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 20, for example) refer Section 3.4 of Appendix C (from **BGC**).

3.3.2 Scenario 2 Costs

Table 14 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 13). 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 14) 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 15.
- 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 19a.

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 23 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 24 spillway profile shows that bedrock is close to the surface at the spillway headworks.

3.4.3 Scenario 3 Costs

Table 16 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 1B	**$32,100,000.**
Scenario 2	$59,900,000.
Scenario 3	**$32,600,000.**

Scenarios 1B and 3 have comparable costs, and are the least expensive options for conveying the PMF down the Rose Creek valley.

4. CONCLUDING REMARKS

The study's two main components were to:

1. Update and re-assess the Faro Mine Site area hydrology, and specifically extreme flood estimates up to the probable maximum flood (PMF).
2. Assess three possible scenarios for routing extreme floods up to the PMF through a modified Rose Creek Diversion channel to down valley of the mine site tailings facilities.

4.1 Extreme Flood Hydrology

The two most important inputs in the PMF estimation procedure are the probable maximum precipitation (PMP) and time to peak - the time it takes for the whole watershed to contribute flow and runoff to reach a peak at the downstream location.

The Faro PMP estimate was based on the 2002 PMP study for Mayo (Hogg 2002). The Mayo analysis utilized the largest maximized single-day rainfall event recorded in the Yukon (at Boundary Met station; El. 1036 m) and therefore is applicable to areas of the Yukon of similar geography and elevation as that of the observed event. The Mayo PMP estimate was somewhat arbitrarily increased by 50 percent to account for small, localized convective storms imbedded in large area events, and for the small-sized drainages of the Faro mine site.

It is recommended that an experienced hydrometeorologist assesses the Faro PMP.

Time to peak values used were based partially on a semi-empirical procedure tempered by observations of mine site watershed conditions.

In December 2003, DIAND established remote weather stations at Faro and Vangorda mine sites. Analysis and comparison of short duration rainfall data from these weather stations with local streamflow station hydrographs, will enable computation of time to peak for the gauged streams and the re-estimation PMF values.

4.2 Closure Scenarios

Three scenarios were assessed for conveying extreme flood flows up to the PMF down a modified Rose Creek Diversion Channel.

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 a swale lined with riprap to the south abutment of the Intermediate Dam where a new spillway conveys flow to downstream of the Cross Valley Dam.

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 down a spillway sited at the south abutment of the Intermediate Dam.

During the development of conceptual designs for the scenarios, variations evolved and the location of components changed for practical reasons:

- Scenario 1 evolved to three designs:
    - Scenario 1 - raising the right dike height only;
    - Scenario 1A - widening the existing RCDC invert by 5 m into the south bank in combination with raising the right dike to a lesser height; and
    - Scenario 1B (the preferred design) - similar to Scenario 1 except that a concrete spillway is utilized to convey flow down the steeply sloped rock drop weir section adjacent to the Cross Valley Dam.
- For Scenario 3, the spillway location was changed from the south of the Intermediate Dam to the north abutment where foundation conditions are more attractive.

Preliminary capital cost estimates for earthworks, concrete structures and riprap protection were developed for the three scenarios. The costing procedure involved many assumptions and unknowns, which may not be consistent from one scenario to another, and yet affect costs. Also, the scenarios have differing additional initial costs, including the cost of:

- placing and maintaining a soil cover over the Intermediate Pond tailings surface - Scenario 2; and
- excavation and relocation of pond tailings - Scenario 3.

Section 4 of Appendix C expands upon the costing procedure followed, assumptions, and unknowns.

In conclusion, Scenarios 1B and 3 have comparable capital cost and are less expensive than Scenario 2.

B.J. Evans, P.Eng.
Senior Engineer

E.K. Yaremko, P.Eng.
Review Principal

PERMIT TO PRACTICE
Signature
Date 30 Aug 04
PERMIT NUMBER: PP043
Association of Professional Engineers of Yukon

5. REFERENCES

Beard, L.R. 1962. Statistical methods in hydrology. Civil works investigation, U.S. Army Corps of Engineers, Sacramento District.

Creager, W.P., Justin, J.D. and Hinds, J. 1945. Engineering for dams: V.1, general design. John Wiley and Sons, New York.

Hansen, E.M., Schreiner, L.C. and Miller, J.F. 1982. Application of probable maximum precipitation estimates - United States east of the 105th Meridian. Hydrometeorological Report No. 52, National Weather Service. Washington D.C.

Hogg, W.D. Nov. 2002. Probable maximum precipitation for Wareham Dam, Mayo, Yukon. Prepared for Yukon Energy Corporation.

Jungwirth, Schmutz and Weiss, eds. 1998. Fish Migration and Fish Bypasses. Fishing New Books. Blackwell Science Ltd. England.

Mutreja, K.N. 1986. Applied hydrology. Tata McGraw Hill, New Delhi.

Northwest Hydraulic Consultants Ltd. (**nhc**) 2001. Hydrotechnical assessment for the Faro Mine Site. Prepared for BGC Engineering Inc./ Deloitte & Touche Inc.

nhc, 2002. Wilsey Dam Fish Passage Concepts. Prepared for Fisheries and Oceans Canada, Kamloops.

nhc, October 2003. Rose Creek Diversion Channel at Faro Mine, Yukon - Hydrotechnical investigation. Prepared for BGC Engineering Inc./ Deloitte & Touche Inc.

Viessman, W. et al. 1977. Introduction to hydrology. IEP-A Dun-Donnelley Publisher, New York.

Watt, W.E. et al. (editors) 1989. Hydrology of floods in Canada: A guide to planning and design. Associate Committee on Hydrology, National Research Council Canada, Ottawa.

World Meteorological Organization 1986. Manual for the estimation of probable maximum precipitation. Operational Hydrology Report No. 1. WMO - No. 332. Geneva, Switzerland.

TABLES

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

Month	Monthly Discharge in m^3/s							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 Volume 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

Notes:
1. Data from Gartner Lee Ltd. Whitehorse
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 Volume 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

Notes:
1. Data from Gartner Lee Ltd. Whitehorse
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) in m^3/s						
		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	43.0
South Big Salmon River below Livingstone Creek	09AG003	33.7	59.3	92.0	108	124	148	170
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	700	760	857	940
Ross River at Ross River	09BA001	408	566	707	765	822	897	950

Table 5
Estimated mean annual 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	11.0	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
Comparison of North Fork Rose Creek and Vangorda Creek peak flow data for 2000 and 2002

North Fork Rose Creek, Stn. R7			Vangorda Creek, Stn. 29BC003		
Date		Daily Flow (m^3/s)	Date		Daily Flow (m^3/s)
2000 June 8	(annual max.)	4.55	2000 June 7	(2nd highest)	2.71
August 27	(2nd highest)	4.31	August 27	(annual max.)	3.45
2002 June 9	(annual max.)	7.67	2002 June 9	unknown as incomplete record	1.93
August 28	(2nd highest)	7.35			

Note:

Incomplete data for Vangorda Creek in 2002. The peak flow of the incomplete record was 3.69 m^3/s on May 29.

Table 7
Adopted times to peak 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 8
Estimated Probable Maximum Floods for the Faro Mine site

Mine Site Sub-basins	Drainage Area (km^2)	PMF Peak Discharge (m^3/s)	Creager C
North Fork Rose Creek at Flow-through Rock Drain (Loc.3)	118	504	23
Fresh Water Supply Dam (FWSD) catchment (Loc.4)	67	354	23
Rose Creek above Tailings Diversion Channel (Loc.5)	203	690	22
Rose Creek downstream of Tailings Diversion Channel (Stn. X 14)	230	783	24

Table 9
Comparison of Probable Maximum Flood estimates

Mine Site Sub-basins	Drainage Area (km^2)	PMF Peak Discharge		Ratio of Discharges
		nhc 2001 (m^3/s)	**nhc** 2004 (this study) (m^3/s)	**nhc** 2004/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 10
Extreme flood values in the Yukon

WSC Station No.	Station Name	Gross Area (km^2)	Max. Disch. (m^3/s)	Date	Creager C
9BC003	Pelly River at Pelly Crossing	49,000	4300 [a]	28-May-57	14
9CD001	Yukon River above White River	150,000	7700 [a]	25-Jun-62	18
9DC002	Stewart River at Mayo	31,600	4110 [a]	10-Jun-64	15
9DC003	Stewart River above Fraser Falls	30,600	3500 [b]	18-Jun-92	13
9DD002	Stewart River at Stewart Crossing	35,000	4330 [b]	11-Jun-64	15
9DD003	Stewart River at the mouth	51,000	5640 [b]	13-Jun-64	18
9EB001	Yukon River at Dawson	264,000	14900 [a]	11-Jun-64	30
9EB002	Yukon River at Stewart R.	251,000	13300 [a]	12-Jun-64	27

Note: Flood values are for central Yukon catchments that drain into the Tintina Trench.

[a] maximum daily
[b] instantaneous peak

Table 11
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)	Channel Velocity (m/s)	Channel Froude No.
39	1054.00	8.91	1062.92	1.0	0.11
38	1054.50	8.39	1062.90	1.1	0.12
37	1055.00	7.70	1062.71	2.3	0.28
36	1054.25	8.34	1062.59	2.5	0.29
35	1054.00	8.56	1062.56	2.2	0.26
34	1053.75	8.78	1062.53	2.1	0.23
33	1053.50	9.00	1062.50	1.8	0.21
32	1053.25	9.28	1062.53	1.3	0.15
31	1053.00	9.52	1062.52	1.2	0.13
28	1053.50	8.33	1061.83	3.6	0.48
27	1053.00	8.55	1061.55	3.6	0.48
26	1052.75	8.54	1061.29	3.5	0.46
25	1052.25	8.84	1061.09	3.4	0.42
24	1052.50	8.26	1060.76	3.3	0.44
23	1052.00	8.22	1060.22	3.5	0.47
22	1051.75	8.09	1059.84	3.7	0.51
21	1051.50	8.33	1059.83	2.8	0.35
20	1051.25	8.31	1059.56	3.2	0.42
19	1050.75	8.61	1059.36	3.1	0.38
18	1050.75	8.43	1059.18	3.4	0.42
17	1050.50	8.35	1058.85	3.7	0.50
16	1050.25	8.26	1058.51	3.6	0.48
15	1050.00	8.04	1058.04	3.8	0.52
14	1049.75	7.99	1057.74	3.4	0.46
13	1049.50	8.05	1057.55	3.6	0.48
12	1049.25	7.93	1057.19	3.6	0.48
11	1049.00	7.12	1056.12	4.7	0.69
10	1048.25	7.50	1055.75	3.7	0.49
9	1048.00	5.86	1053.86	6.1	0.97
8	1042.75	3.77	1046.52	9.6	1.81
7	1035.75	5.07	1040.82	6.8	1.13
6	1030.50	4.56	1035.06	8.5	1.49
5	1026.50	4.09	1030.59	8.0	1.48
4	1022.50	4.57	1027.07	7.5	1.41
3	1021.00	6.28	1027.28	4.3	0.61
2	1020.25	7.13	1027.38	3.2	0.43
1	1020.25	7.09	1027.34	2.8	0.38

Table 12
Preliminary costs estimate for Scenario 1B
with concrete spillway from CS 9 to 6

Earthworks	Spillway	Outlet Channel	Fish By-Pass	Total Cost
$13,400,000	$17,400,000	$400,000	$900,000	$32,100,000

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 2).
2. Spillway costs are for structural concrete only.
3. Units costs:

Structural concrete	$1,200	per cubic metre
Fish by-pass	$1,500	per linear metre

Table 13
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 14
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 15
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 16
Preliminary costs estimate for Scenario 3

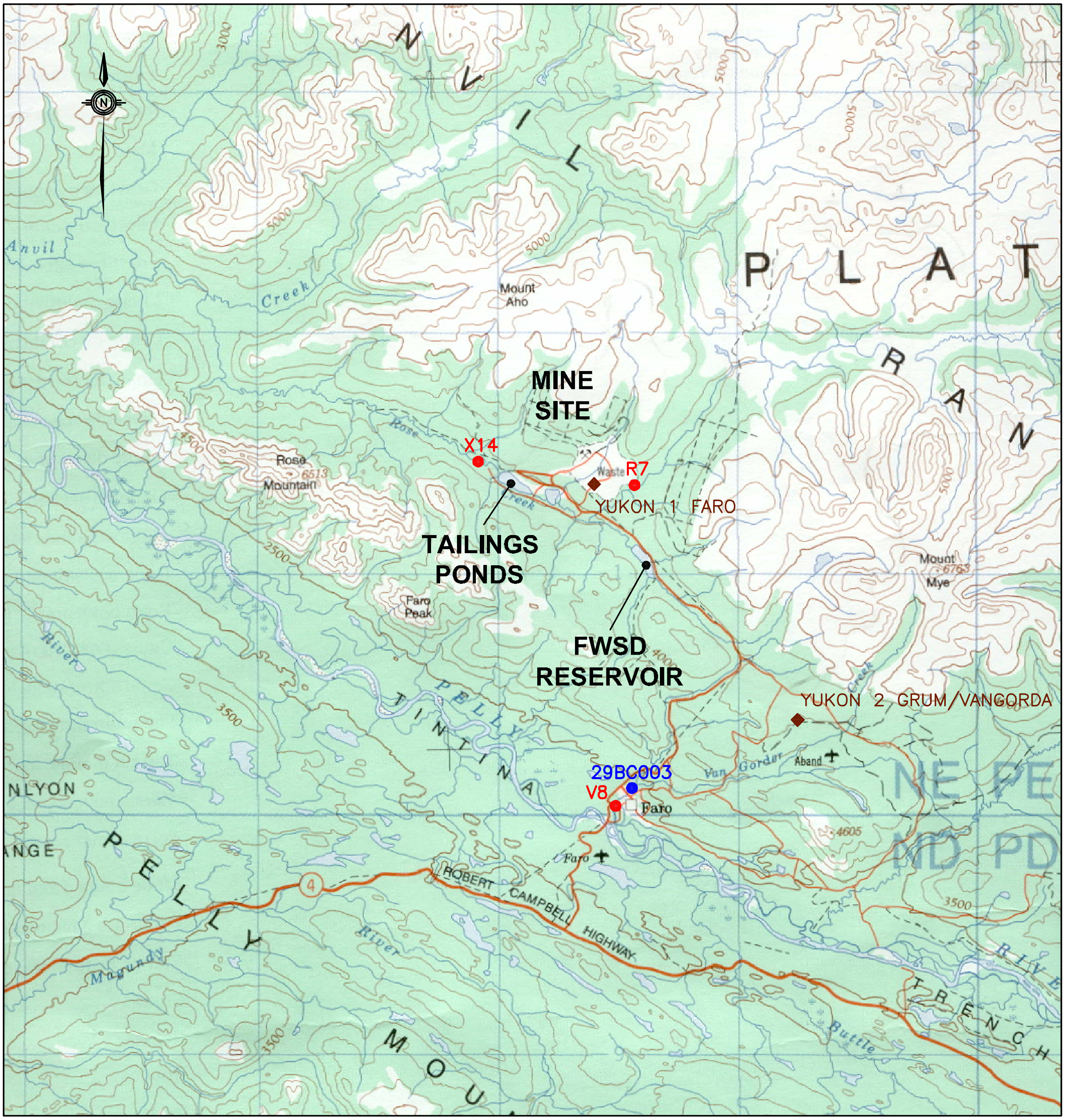
CS 39 Headwall	Spillway	Outlet Channel	Fish By-Pass	Total Cost
$200,000	$31,100,000	$400,000	$900,000	$32,600,000

Notes:

1. Spillway costs include headworks, stepped & chute spillways, & stilling basin and are for structural concrete only.
2. Units costs:

Units costs:	Structural concrete	$1,200	per cubic metre
	Fish by-pass	$1,500	per linear metre

FIGURES



5 km 0 5 10 km

SCALE 1:250,000

LEGEND:

Symbol	Description
R7 (red dot)	GAUGING STATION – MINE SITE STAFF
29BC003 (blue dot)	GAUGING STATION – DIAND
YUKON 1 (brown diamond)	WEATHER STATION – DIAND

NOTES:

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

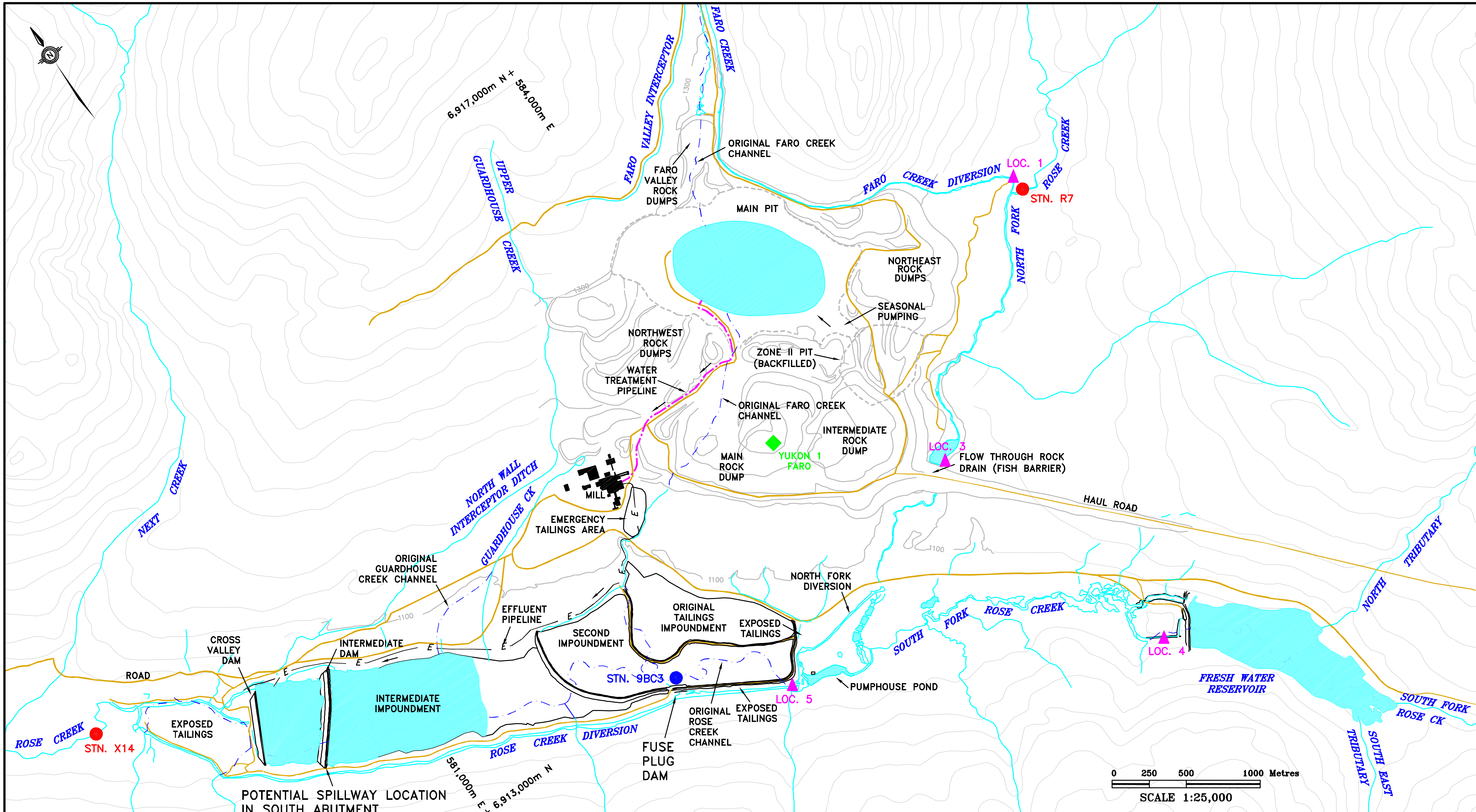
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

LOCATION PLAN

Dwg. 6399–001	29 Oct 2003	**Figure 1**

northwest hydraulic consultants ltd.



LEGEND:

- ROADS
- EXISTING SURFACE DRAINAGE
- PRE–MINE DRAINAGE
- E EFFLUENT PIPELINE
- WATER TREATMENT PIPELINE
- SURFACE WATER
- STN. R7 STREAMFLOW GAUGING STATION
- LOC. 1 STREAM CATCHMENT LOCATION
- STN. 9BC3 DISCONTINUED WATER SURVEY OF CANADA STREAMFLOW GAUGING STATION 09BC003
- YUKON 1 FARO METEOROLOGICAL STATION (ESTABLISHED BY DIAND IN DEC. 2003)

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.

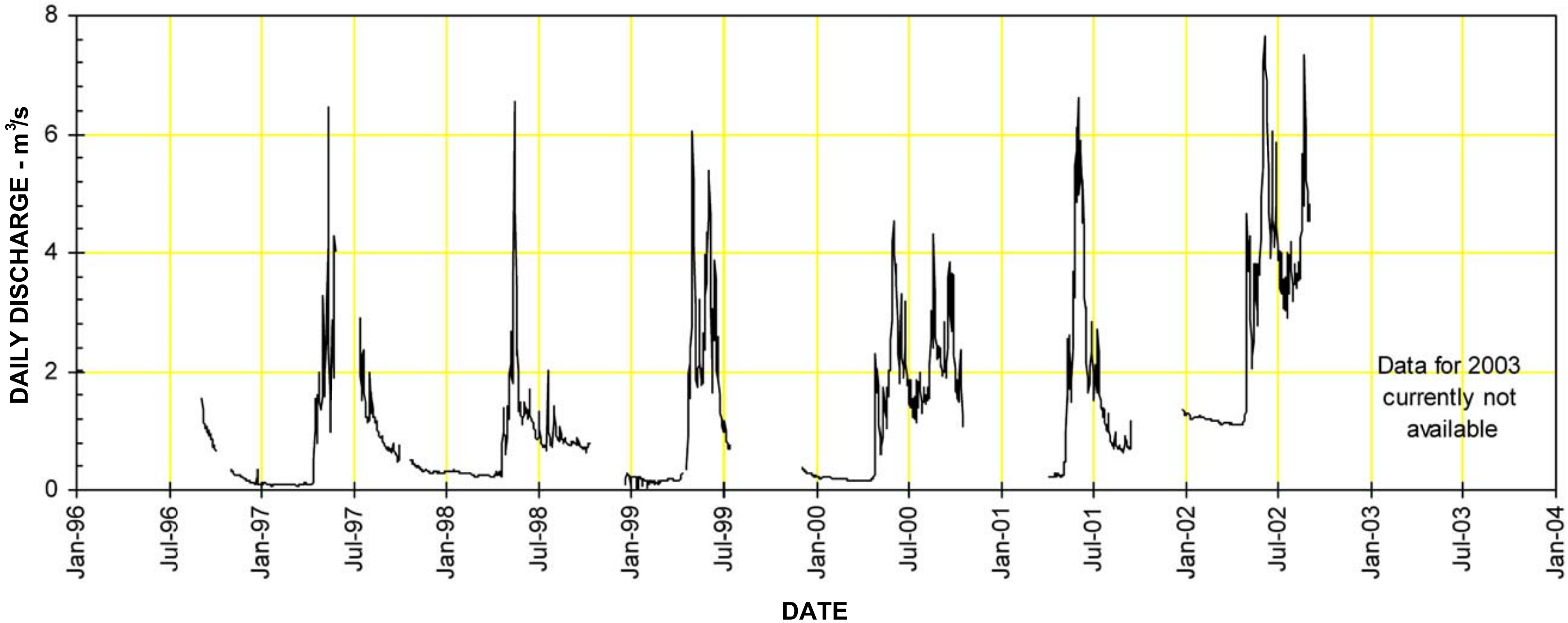
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

OVERVIEW OF FARO MINE SITE

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

northwest hydraulic consultants ltd.



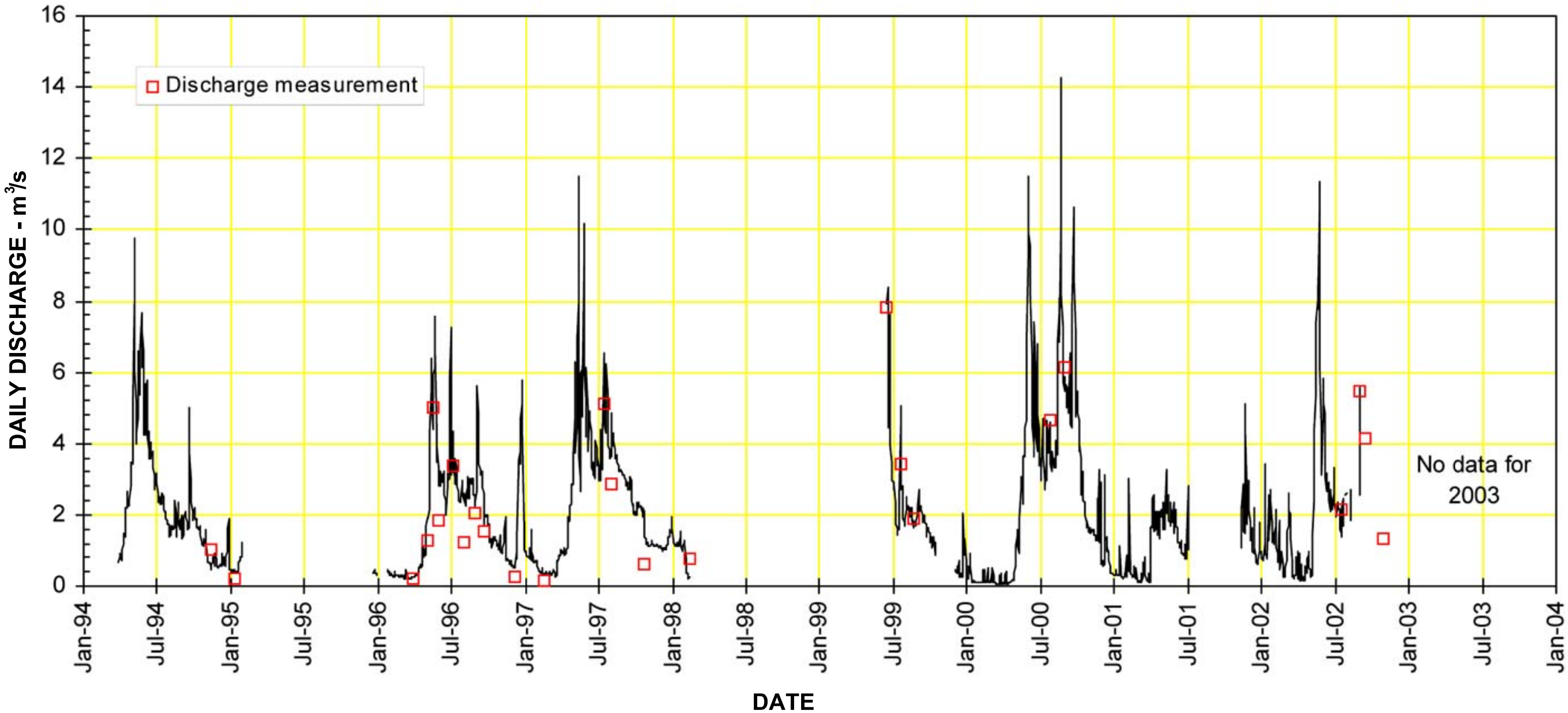
SRK CONSULTING/DELOITTE & TOUCHE

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

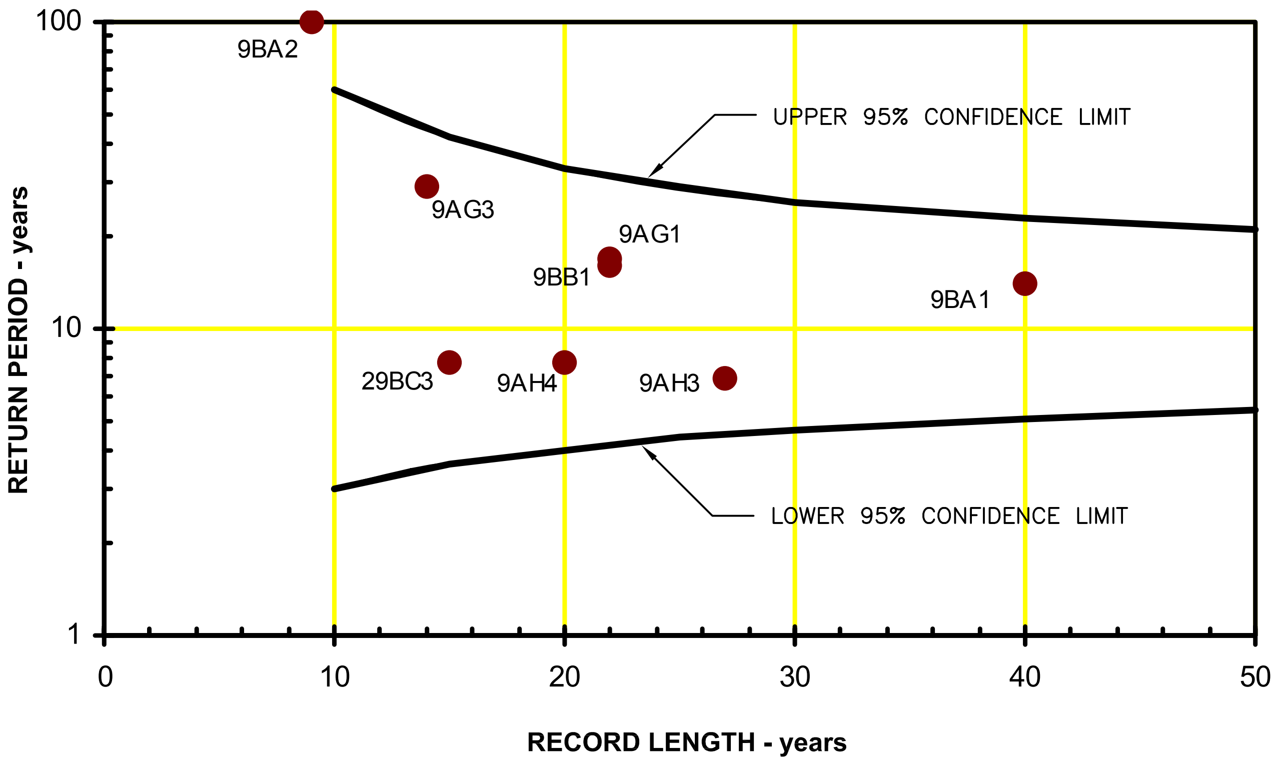
FARO MINE SITE HYDROTECHNICAL STUDY

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)



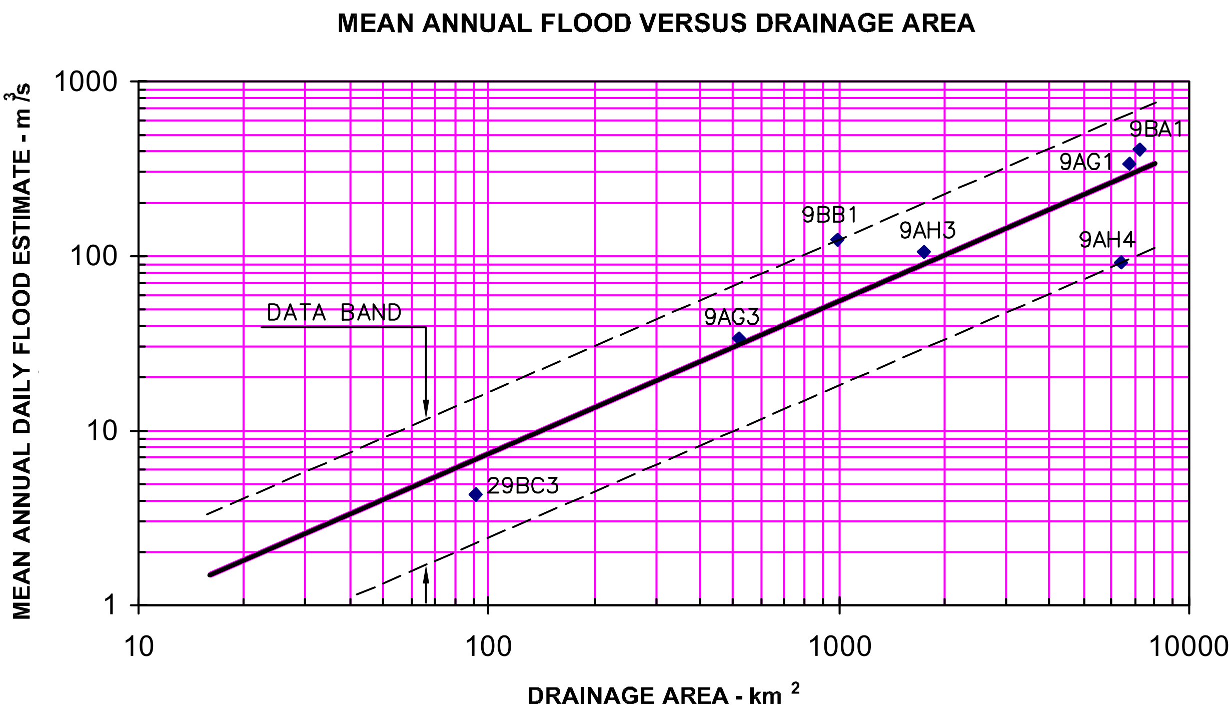
SRK CONSULTING/DELOITTE & TOUCHE

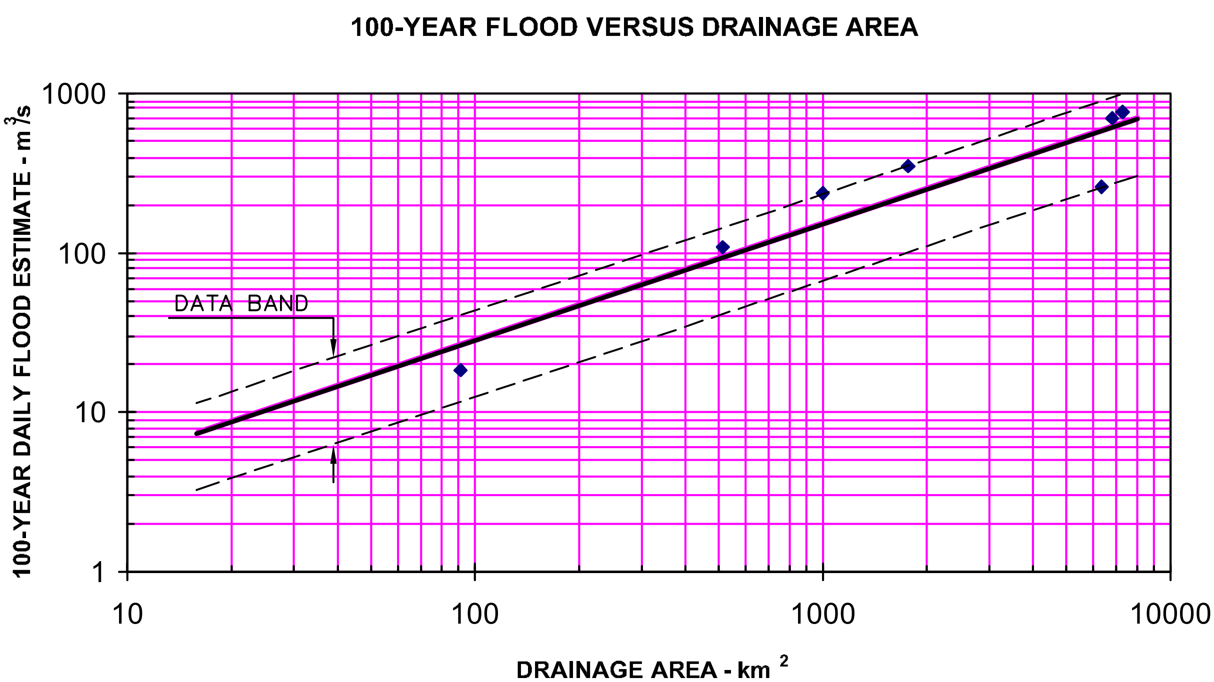
FARO MINE SITE HYDROTECHNICAL STUDY

HOMOGENEITY TEST
FOR GAUGING STATIONS NEAR FARO

Dwg. 6399-005	29 Oct 2003	**Figure 5**

northwest hydraulic consultants ltd.





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.		

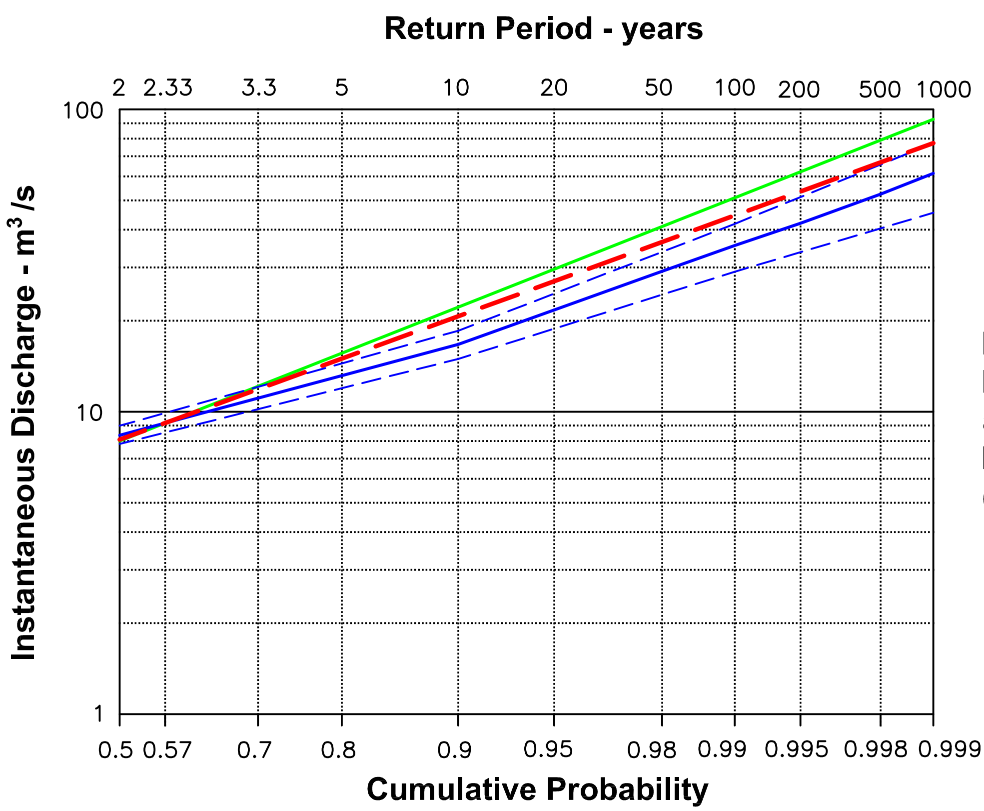


Fig. 7a
North Fork Rose Creek above Faro Creek Diversion Channel (Stn. R7)

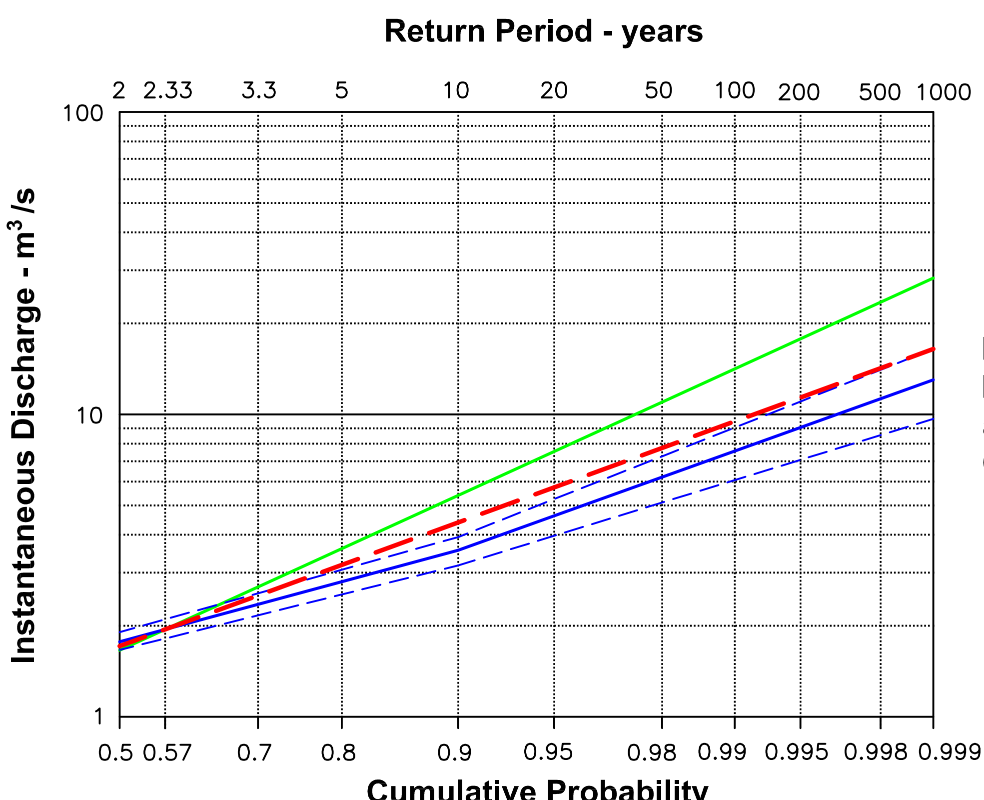


Fig. 7b
Faro Creek & Diversion above North Fork Rose Creek (Loc. 1)

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.

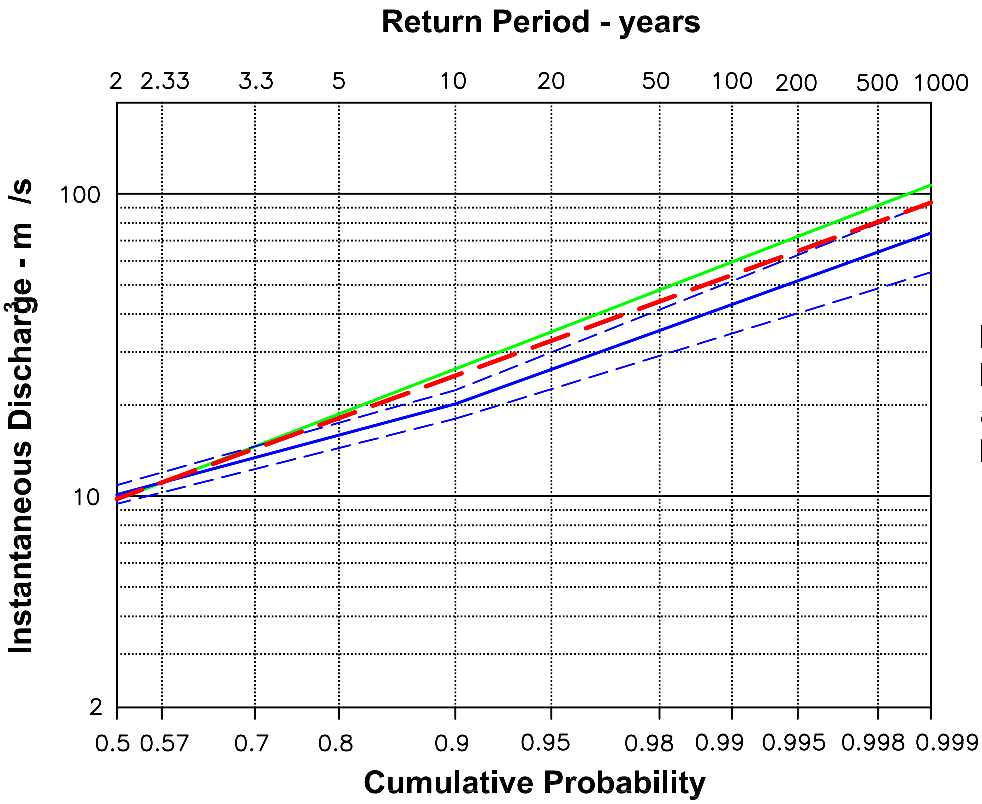


Fig. 7c
North Fork Rose Creek at Flow-through Rock Drain (Loc.3)

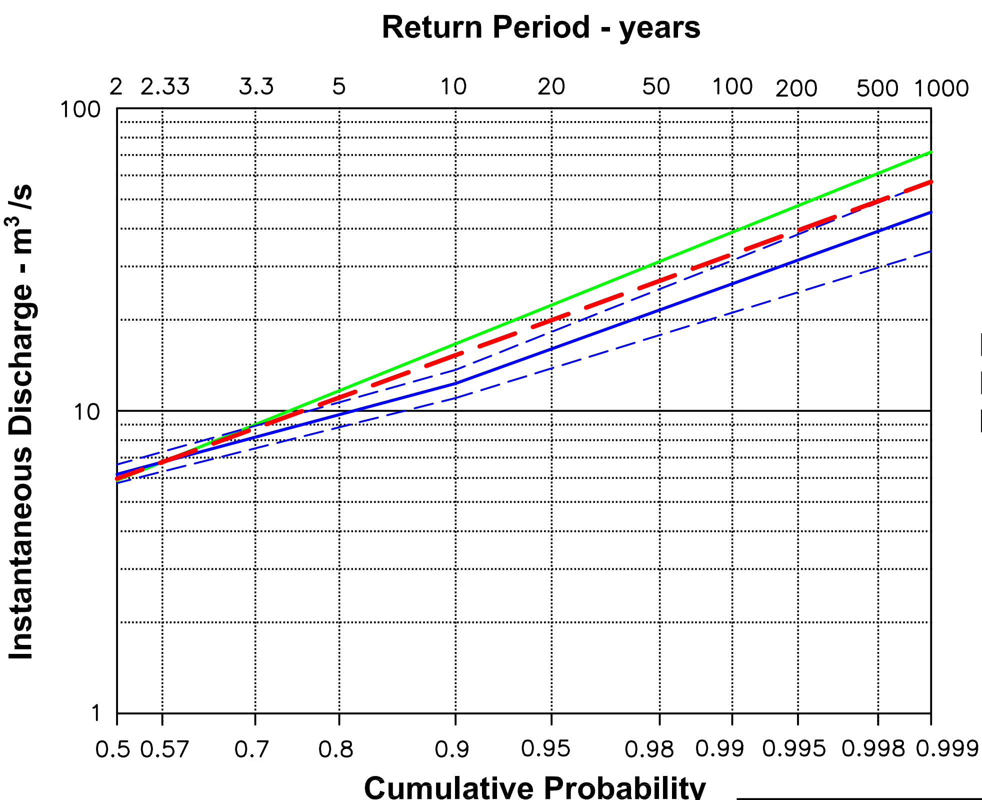


Fig. 7d
Fresh Water Supply Dam (FWSD) Catchment (Loc. 4)

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.		

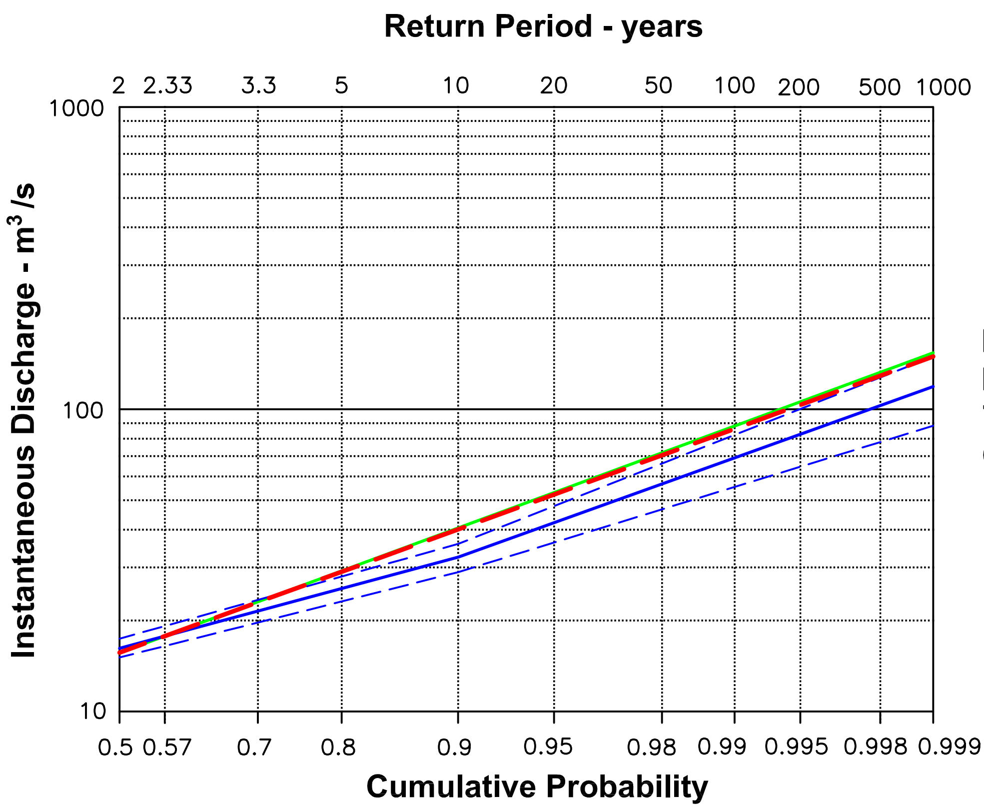


Fig. 7e
Rose Creek above Tailings Diversion Channel (Loc.5)

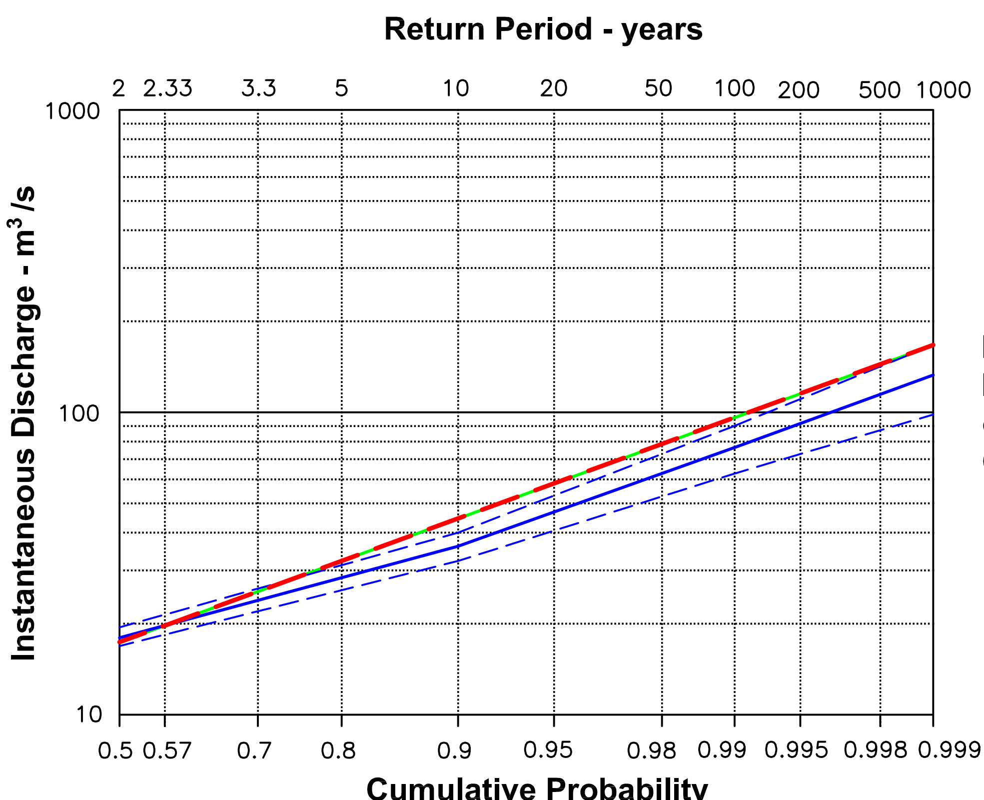


Fig. 7f
Rose Creek downstream of Tailings Diversion Channel (Stn. X14)

LEGEND:

RECOMMENDED FLOOD FREQUENCY CURVE

METHOD 1 FLOOD FREQUENCY CURVE

METHOD 2 FLOOD FREQUENCY CURVE

UPPER AND LOWER 95% ERROR LIMITS (METHOD 2)

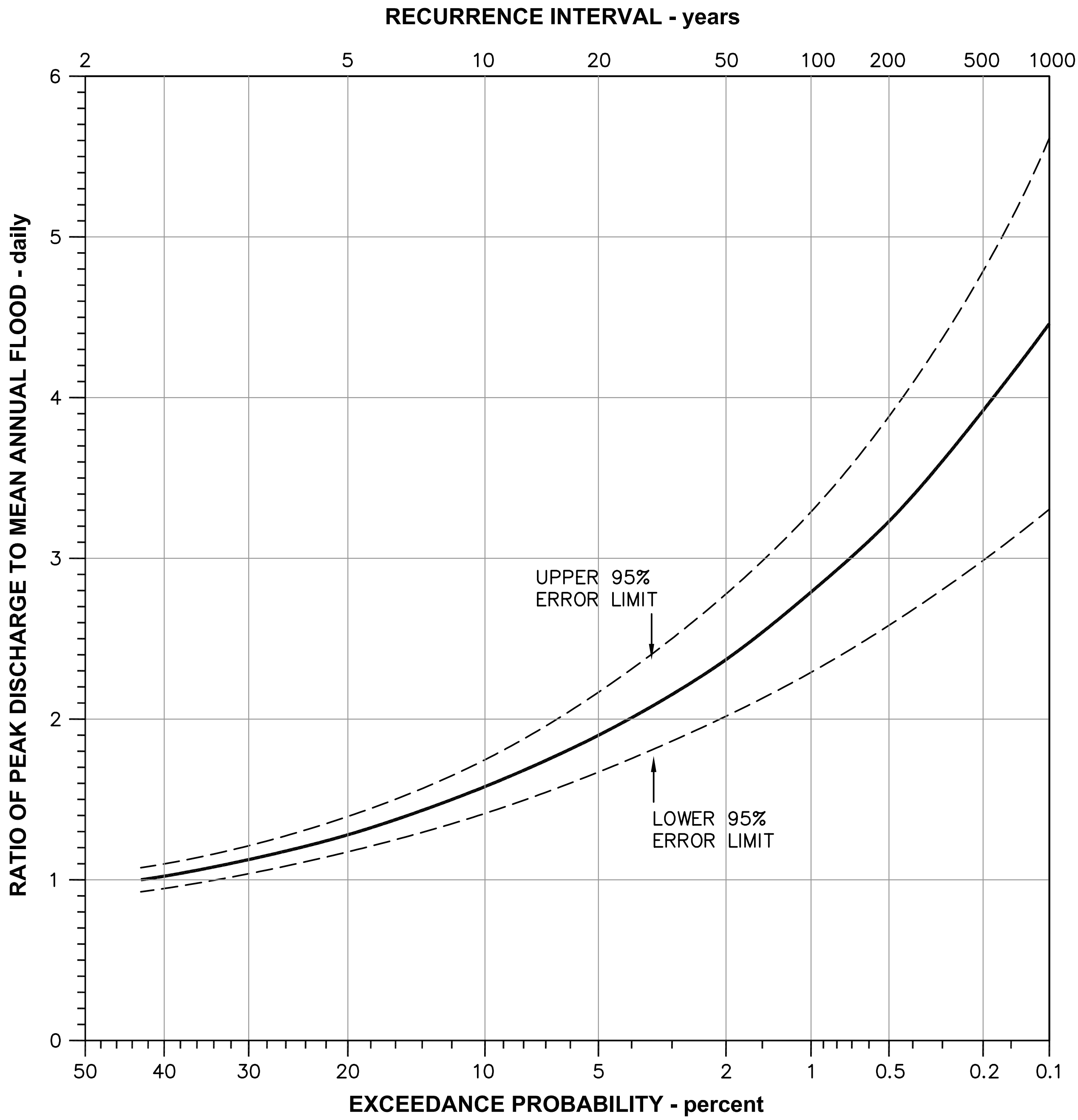
BGC ENGINEERING INC.

FARO MINE HYDROLOGY

SYNTHESIZED FLOOD FREQUENCY PLOTS FOR MINE SITE

Dwg. 6399–010	5 Nov 2003	**Figure 7 e,f**

northwest hydraulic consultants ltd.



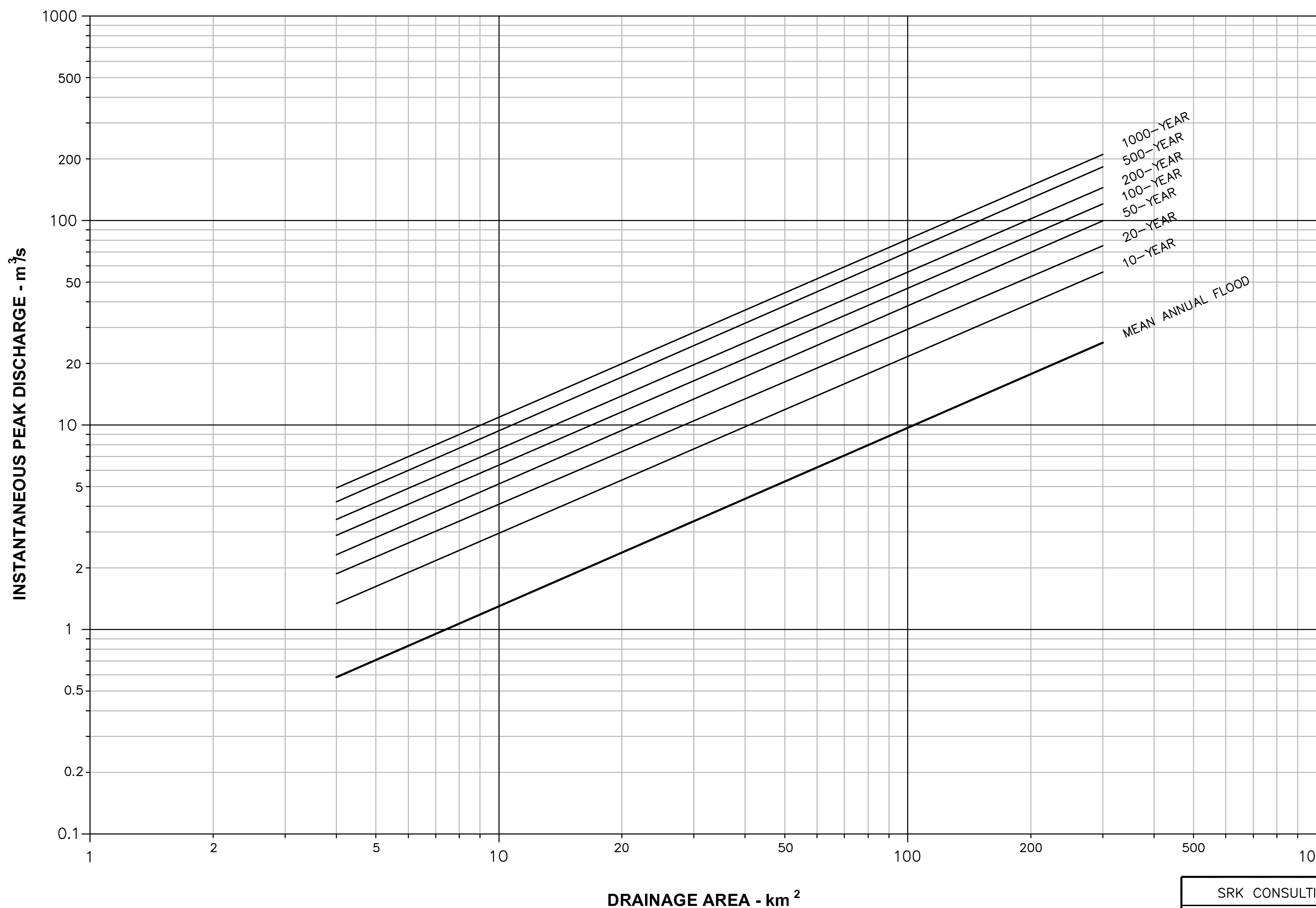
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

DIMENSIONLESS FLOOD FREQUENCY CURVE FOR FARO REGION

Dwg. 6399-011	5 Nov 2003	**Figure 8**

northwest hydraulic consultants ltd.



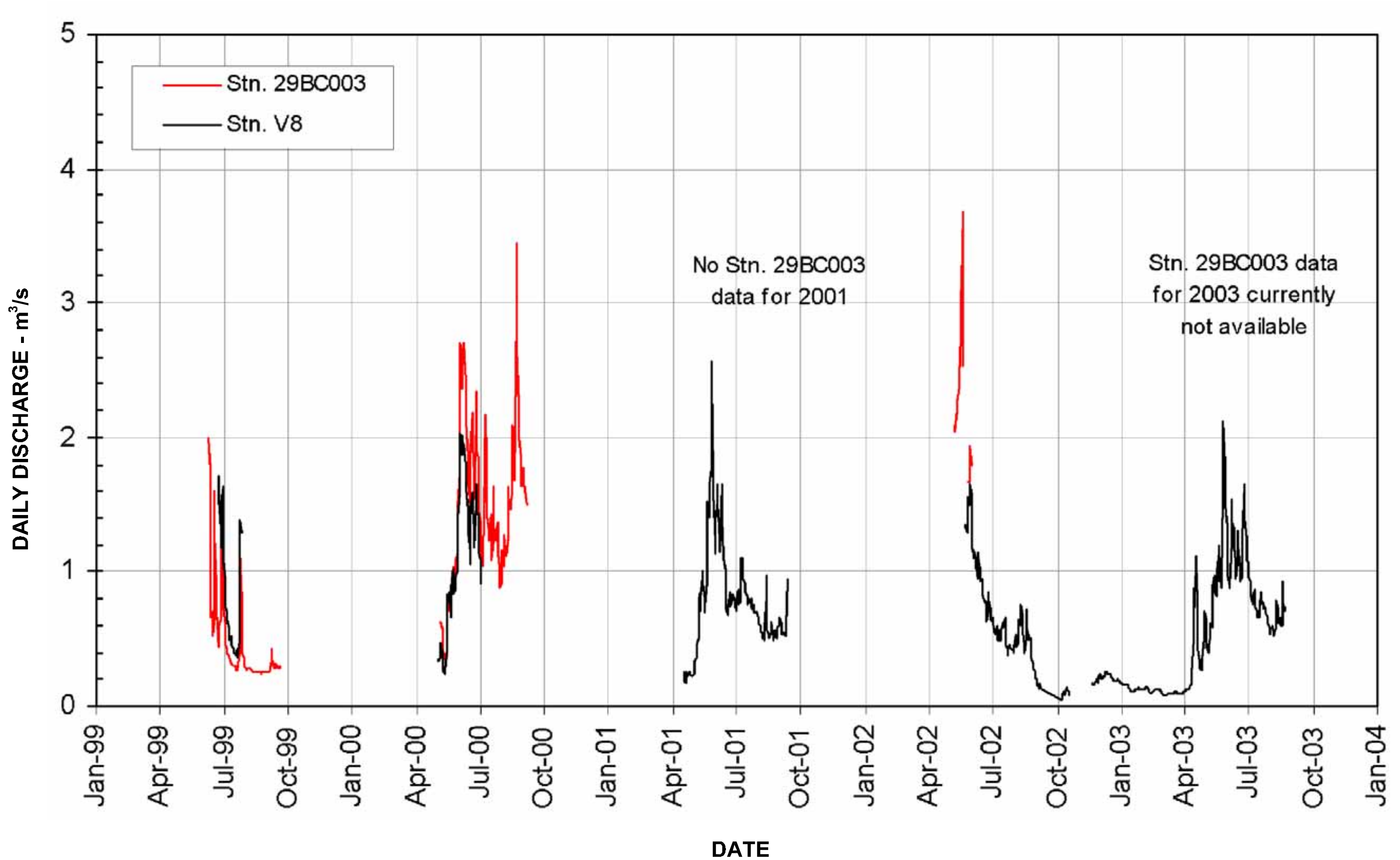
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

EXTREME FLOOD CURVES
FOR FARO MINE SITE AREA

Dwg. 6399–023	7 May 2004	**Figure 9**

northwest hydraulic consultants ltd.



SRK CONSULTING/DELOITTE & TOUCHE

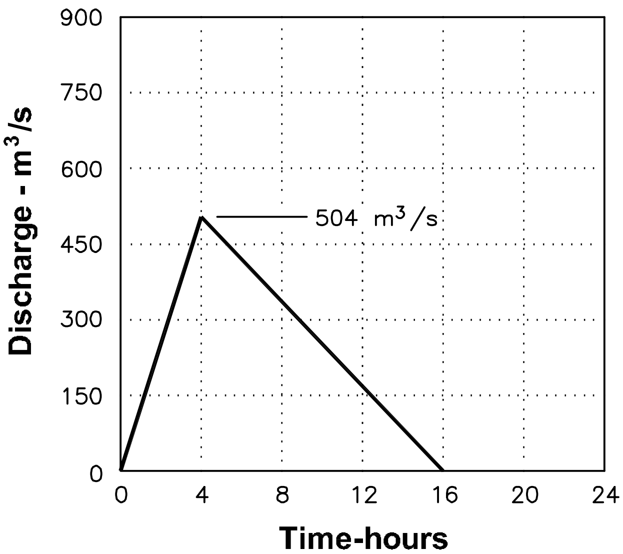
FARO MINE SITE HYDROTECHNICAL STUDY

COMPARISON OF VANGORDA CREEK HYDROGRAPHS 1999-2003

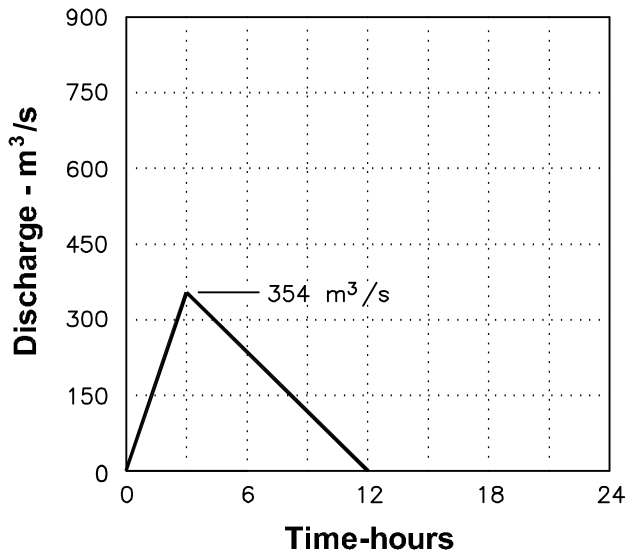
Dwg. 6399-009	29 Oct 2003	**Figure 10**

northwest hydraulic consultants ltd.

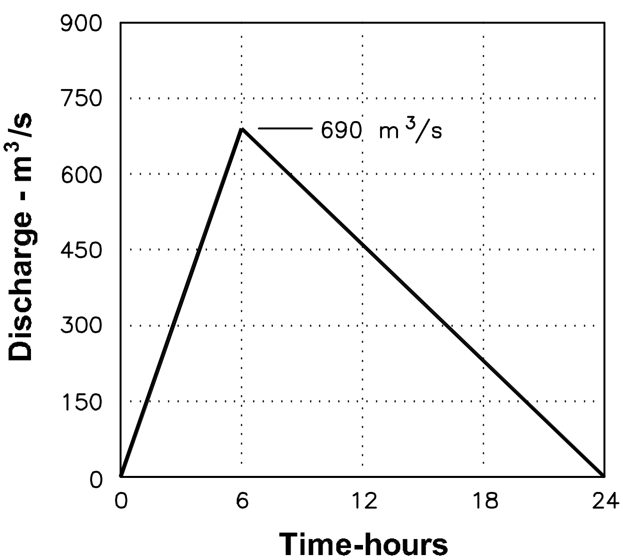
North Fork Rose Creek at Flow-through Rock Drain (Loc. 3)



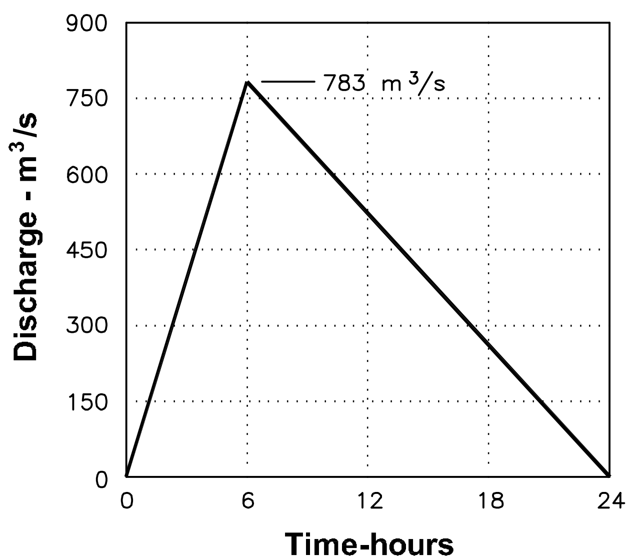
Fresh Water Supply Dam (FWSD) catchment (Loc. 4)



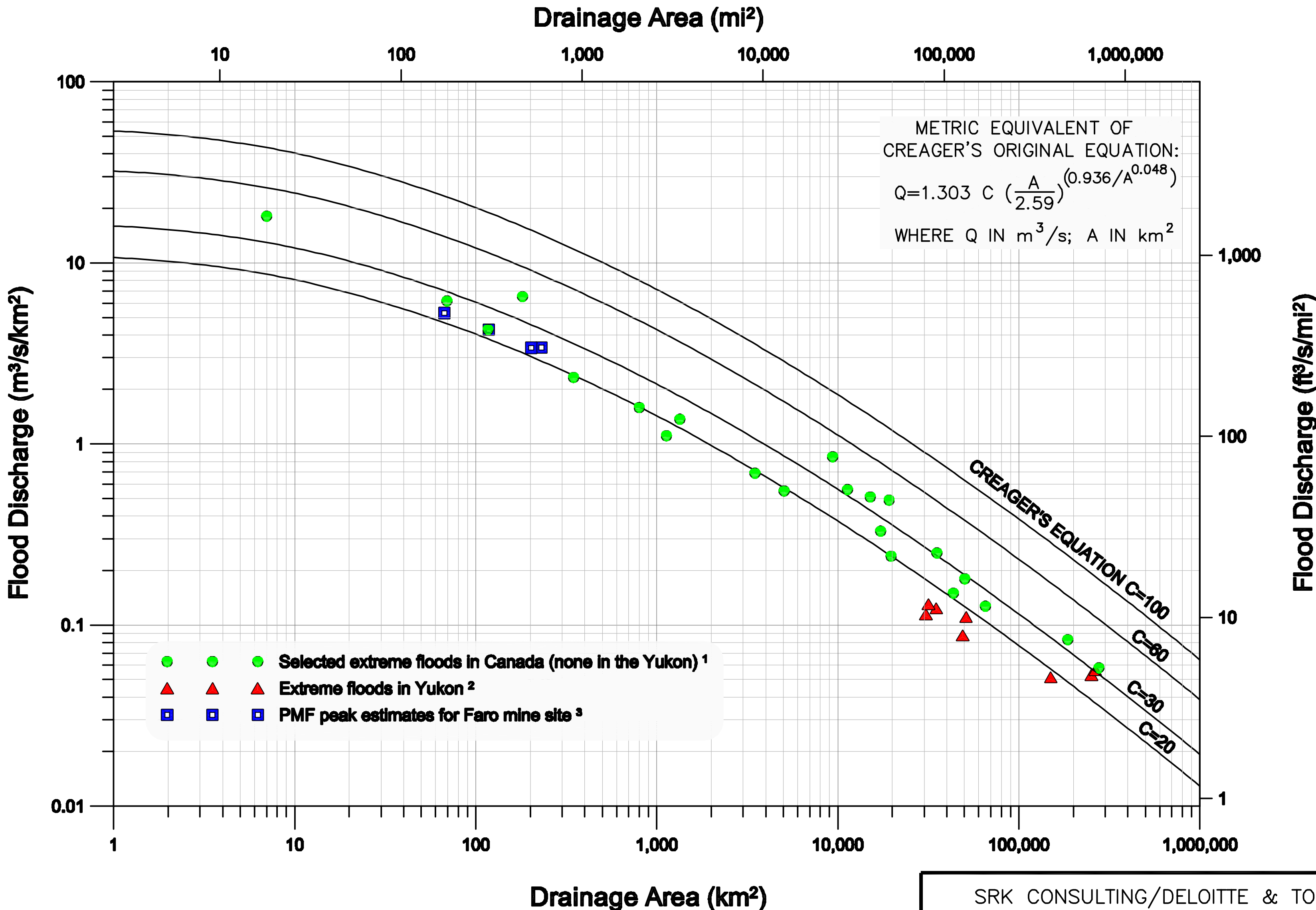
Rose Creek above Tailings Diversion Channel (Loc. 5)



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



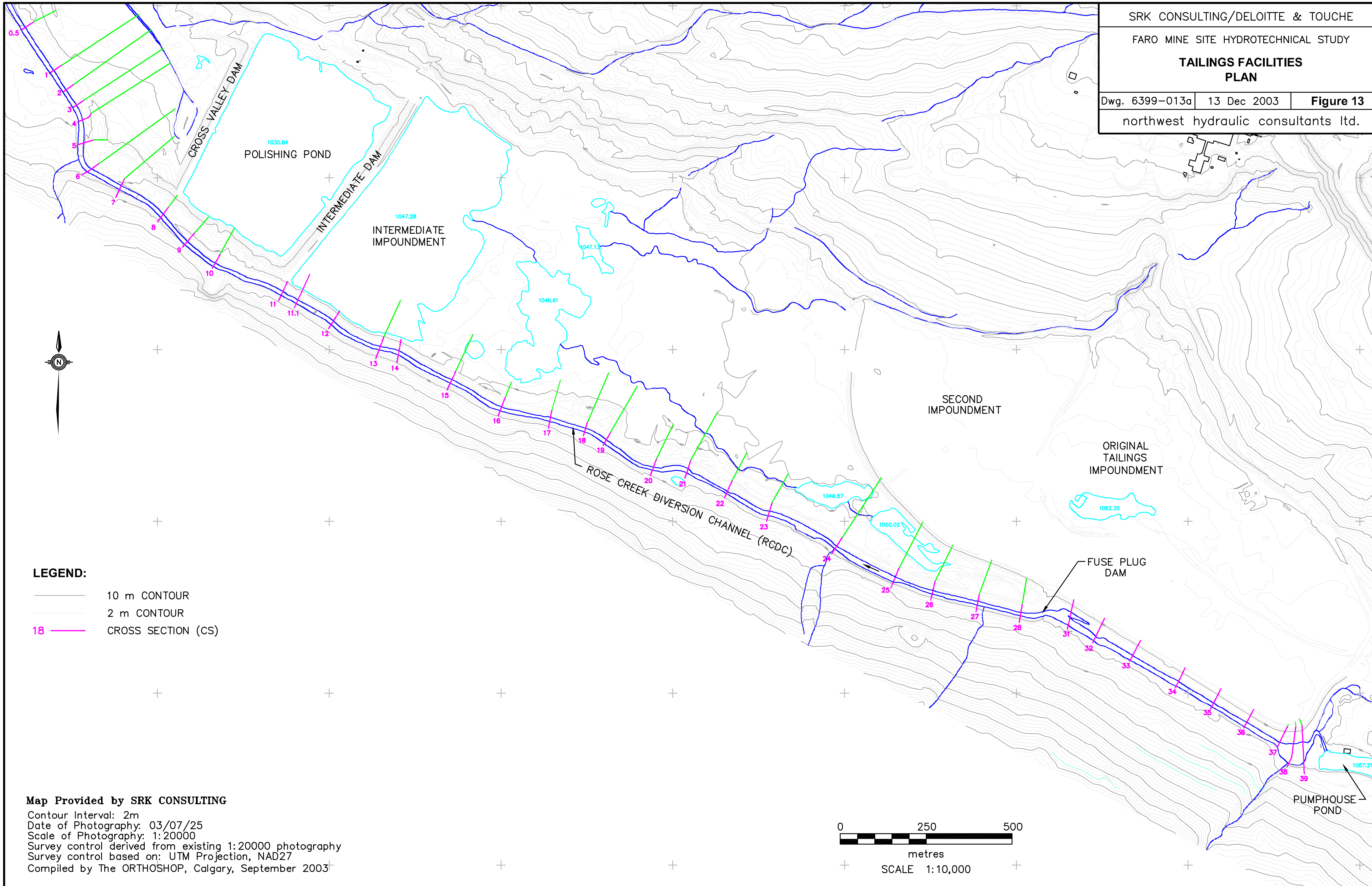
BGC ENGINEERING INC.		
FARO MINE HYDROLOGY **PMF HYDROGRAPHS FOR MINE SITE**		
Dwg. 6399–015	19 Nov 2003	**Figure 11**
northwest hydraulic consultants ltd.		

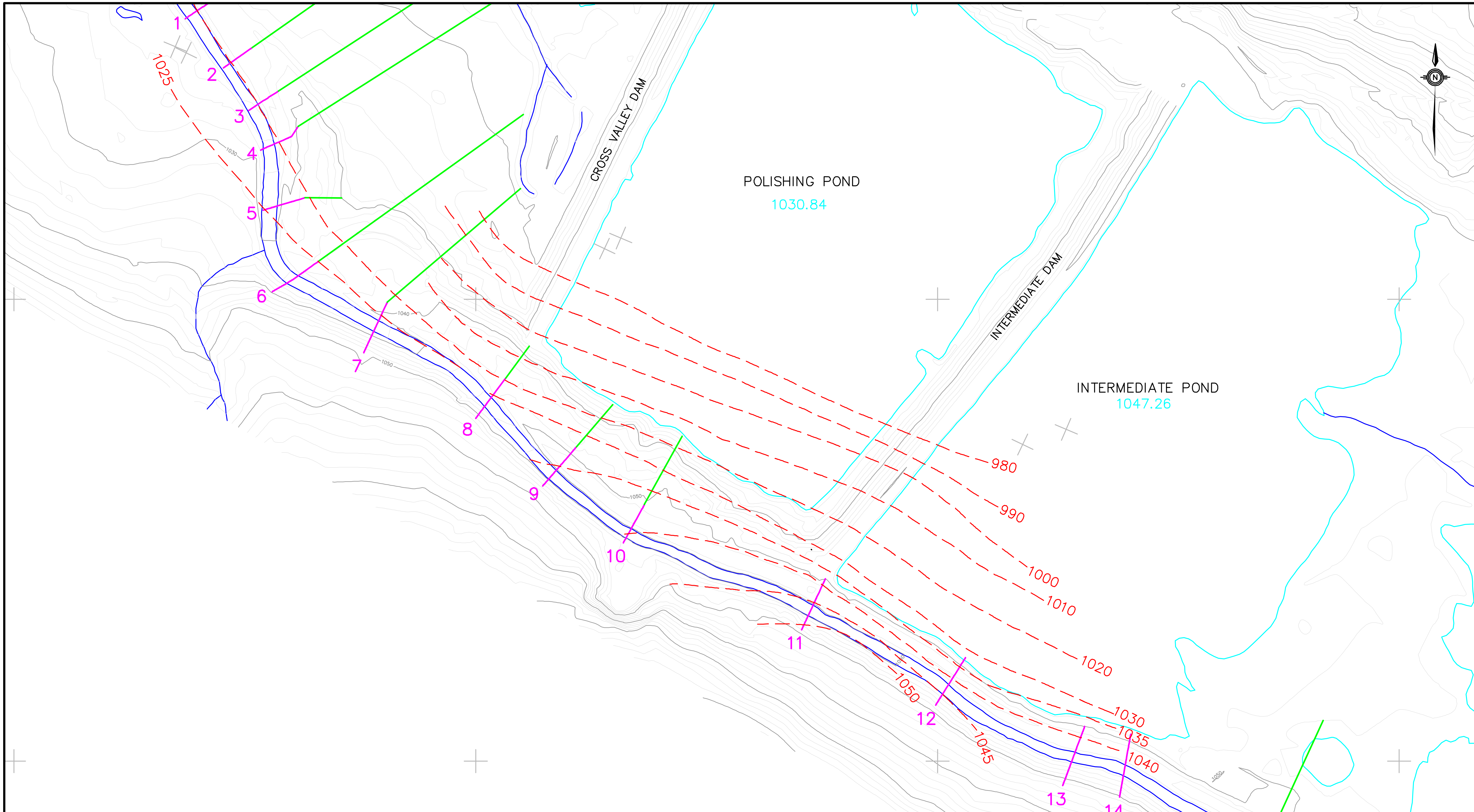


NOTES:

1 – FROM HYDROLOGY OF FLOODS IN CANADA, TABLE 3.1 (WATT et al. 1989)
2 – FROM TABLE 10
3 – FROM TABLE 8

SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
CREAGER DIAGRAM WITH CANADIAN DATA SUPERIMPOSED		
Dwg. 6399-024	27 May 2004	**Figure 12**
northwest hydraulic consultants ltd.		


SRK CONSULTING/DELOITTE & TOUCHE
FARO MINE SITE HYDROTECHNICAL STUDY
TAILINGS FACILITIES PLAN
Dwg. 6399-013a
13 Dec 2003
Figure 13
northwest hydraulic consultants ltd.
CROSS VALLEY DAM
POLISHING POND
INTERMEDIATE DAM
INTERMEDIATE IMPOUNDMENT
SECOND IMPOUNDMENT
ORIGINAL TAILINGS IMPOUNDMENT
ROSE CREEK DIVERSION CHANNEL (RCDC)
FUSE PLUG DAM
PUMPHOUSE POND
LEGEND:
10 m CONTOUR
2 m CONTOUR
18 CROSS SECTION (CS)
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

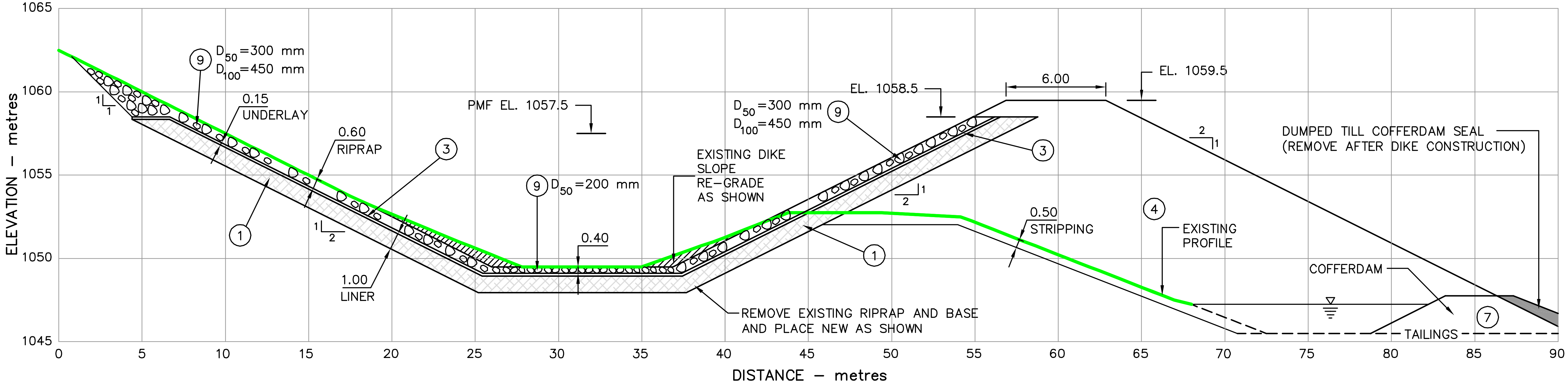
LEGEND:

1030 — — — GEODETIC ELEVATION OF BEDROCK SURFACE CONTOUR
SOURCE: BGC ENGINEERING

NOTE:

1. BEDROCK CONTOURS FROM BGC ENGINEERING INC.

SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
BEDROCK SURFACE CONTOUR MAP		
Dwg. 6399-013	12 Dec 2003	**Figure 14**
northwest hydraulic consultants ltd.		



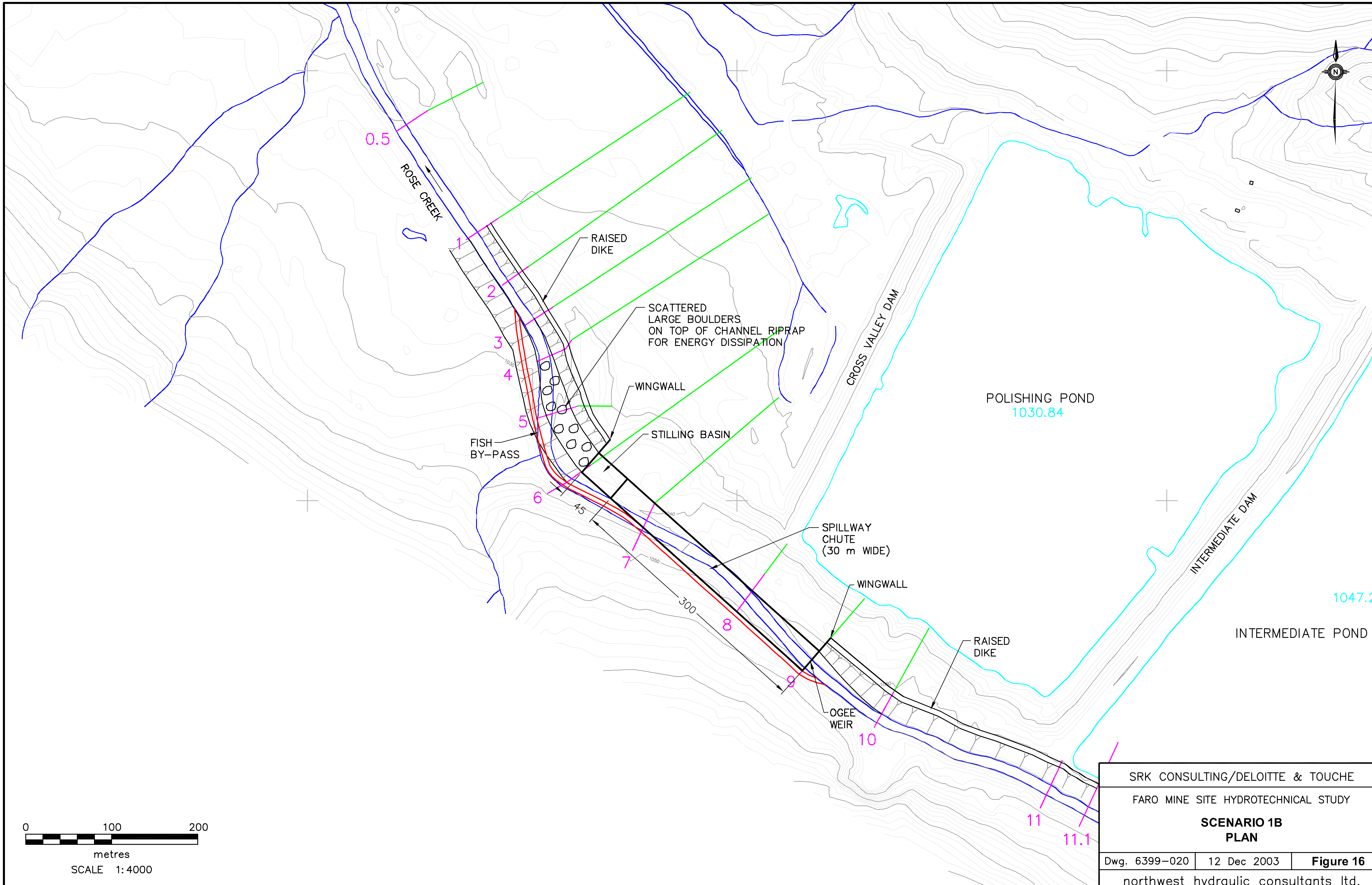
LEGEND:

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)

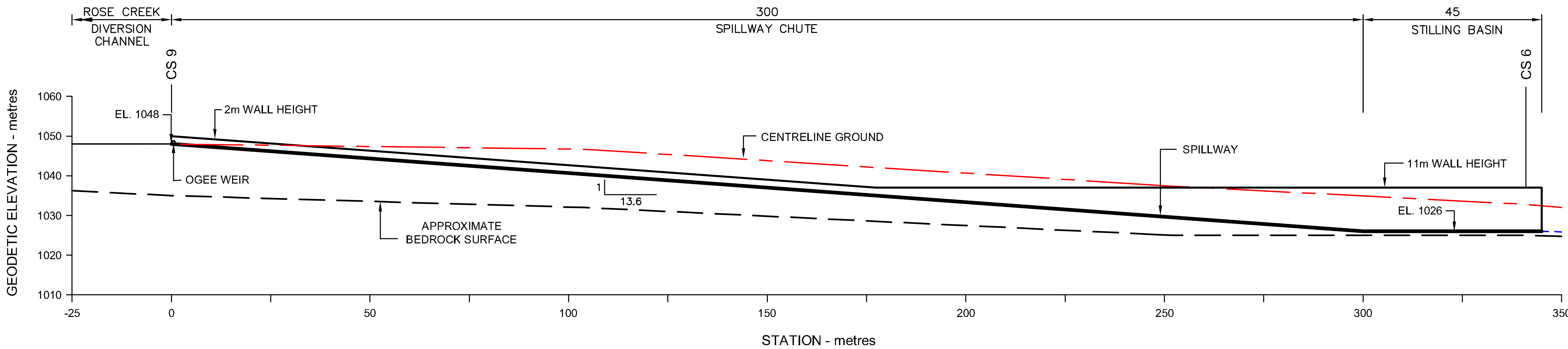
NOTES:

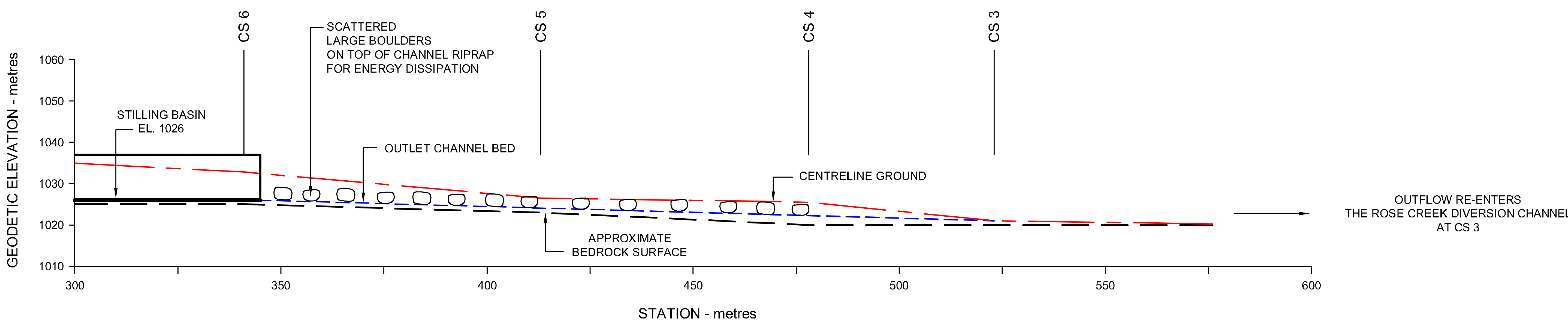
1. ALL DIMENSIONS ARE IN METRES UNLESS STATED OTHERWISE
2. ELEVATIONS FOR CS 13 ONLY
3. SCALE 1:250

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 15**
northwest hydraulic consultants ltd.		



SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
SCENARIO 1B PLAN		
Dwg. 6399–020	12 Dec 2003	**Figure 16**
northwest hydraulic consultants ltd.		





SCALE 1:1000

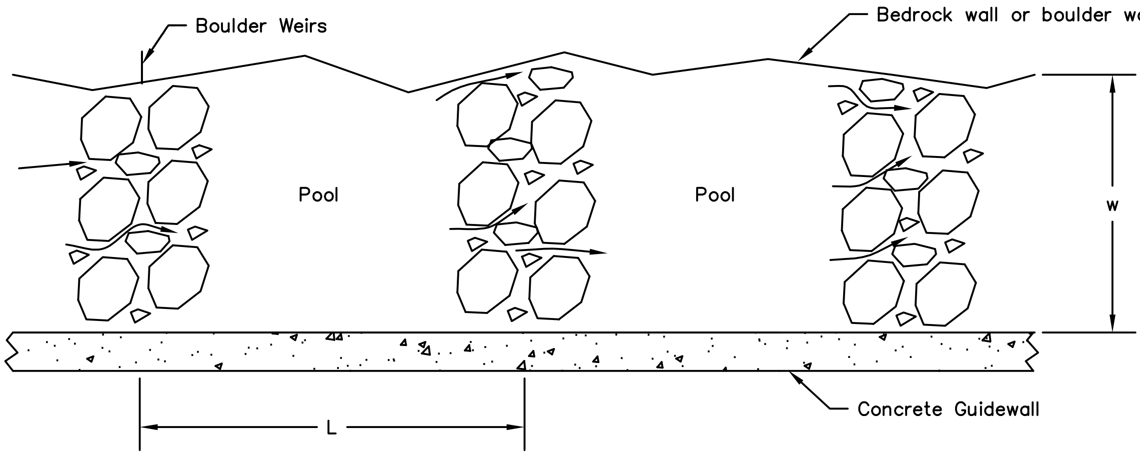
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

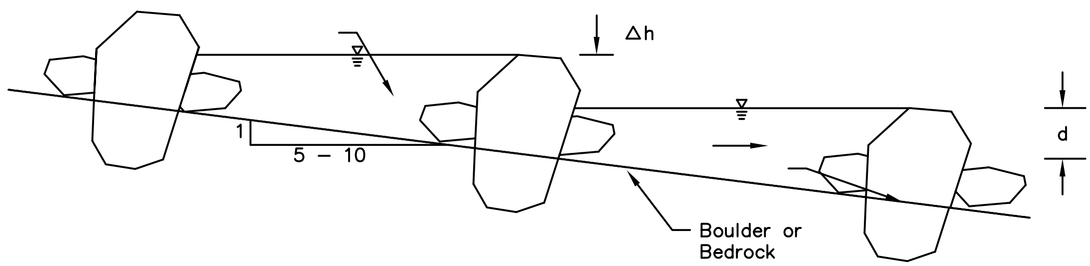
SCENARIO 1B
SPILLWAY CENTRELINE PROFILE

Dwg. 6399–021	12 Dec 2003	**Figure 17**

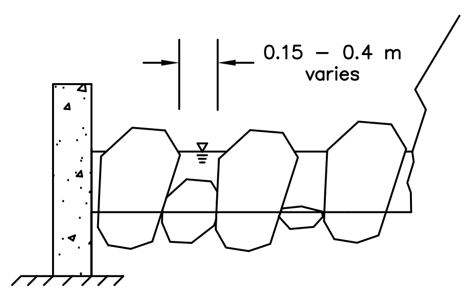
northwest hydraulic consultants ltd.



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

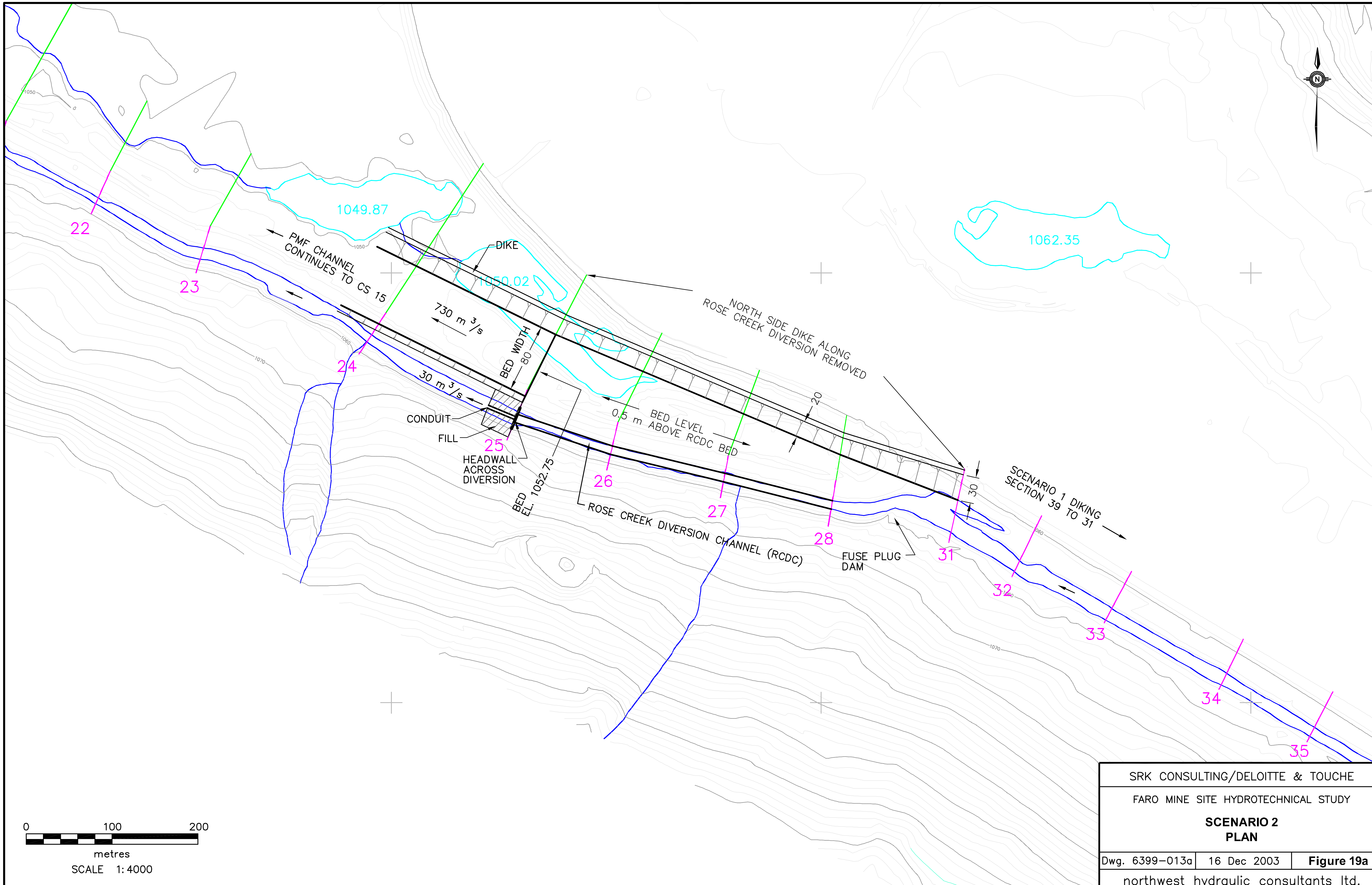
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

NATURAL CHANNEL FISHWAY
STEP POOL DESIGN

Dwg. 6399-022	15 Dec 2003	**Figure 18**

northwest hydraulic consultants ltd.



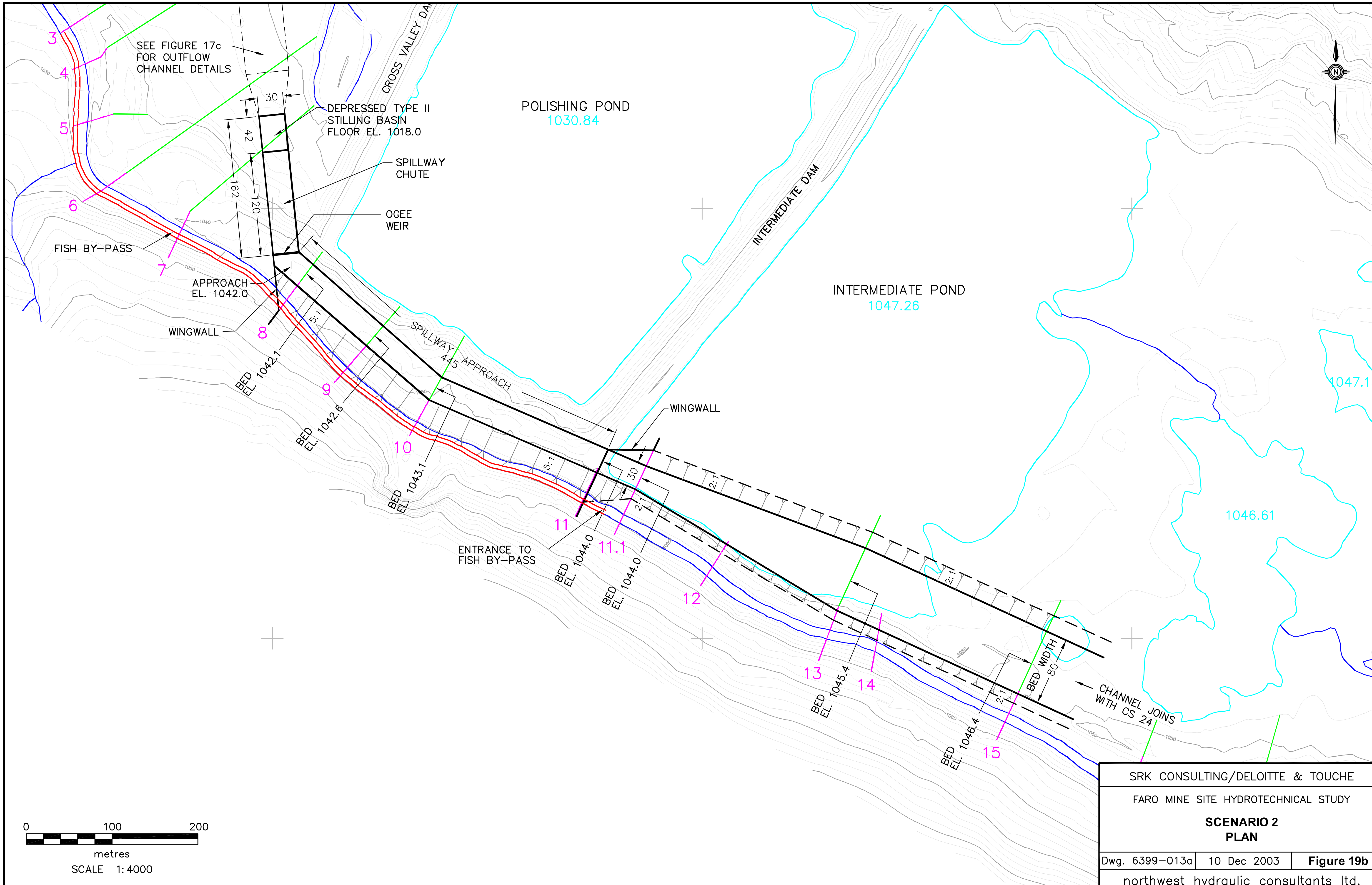
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

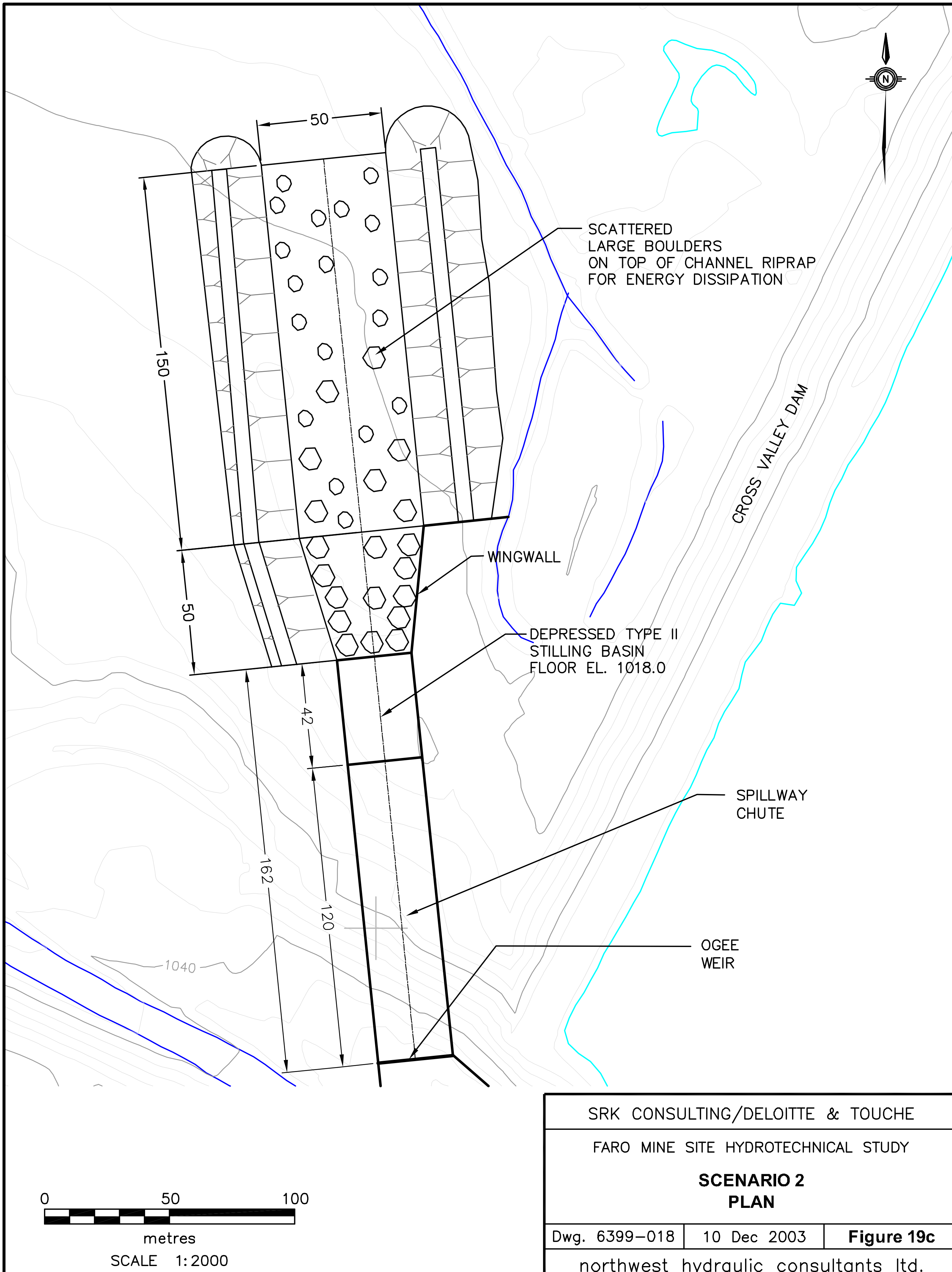
SCENARIO 2
PLAN

Dwg. 6399-013a	16 Dec 2003	**Figure 19a**

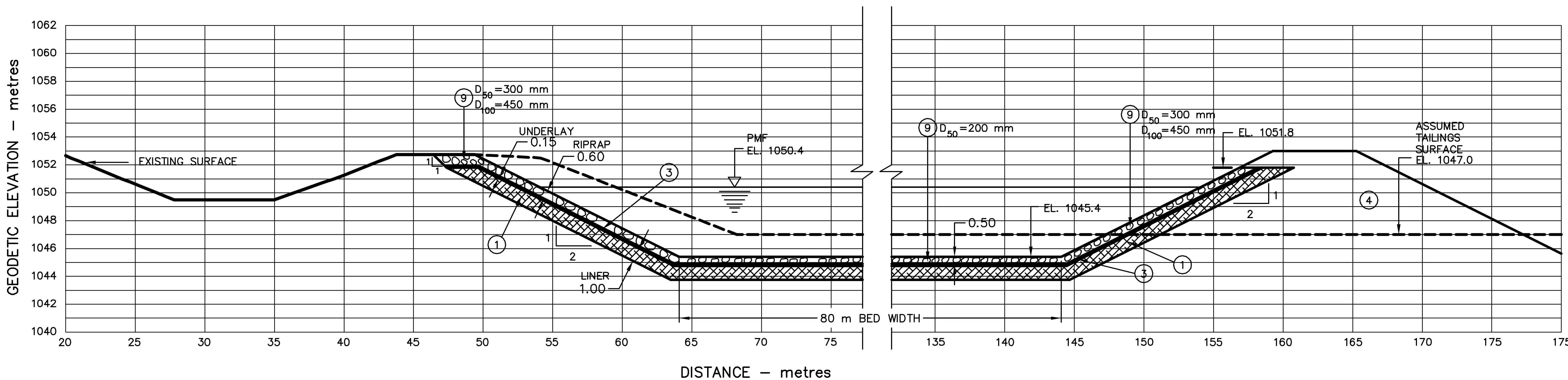
northwest hydraulic consultants ltd.



SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
SCENARIO 2 PLAN		
Dwg. 6399-013a	10 Dec 2003	**Figure 19b**
northwest hydraulic consultants ltd.		



SRK CONSULTING/DELOITTE & TOUCHE		
FARO MINE SITE HYDROTECHNICAL STUDY		
SCENARIO 2 PLAN		
Dwg. 6399–018	10 Dec 2003	**Figure 19c**
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

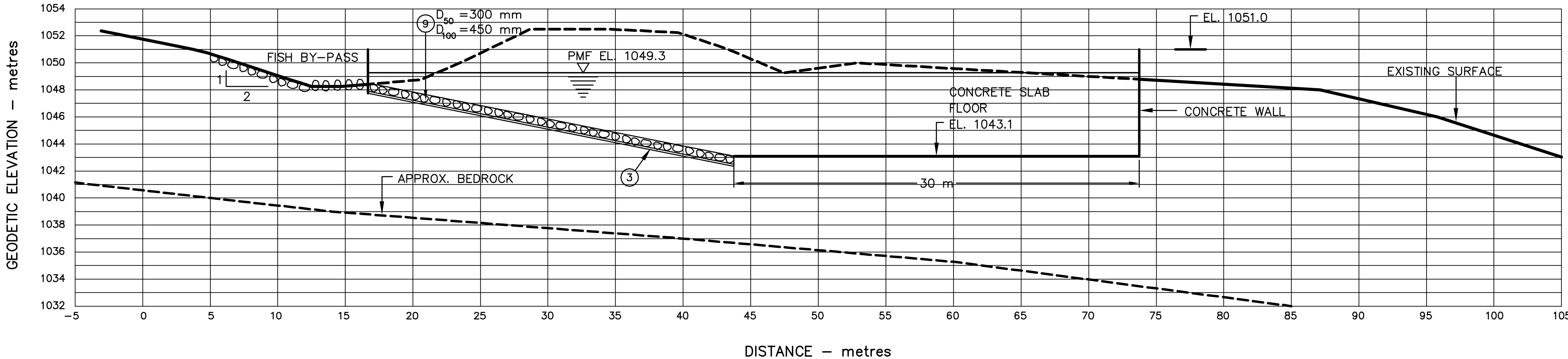
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

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

Dwg. 6399–017	15 Dec 2003	**Figure 20**

northwest hydraulic consultants ltd.



LEGEND:

(1) IMPERVIOUS CORE – COMPACTED TILL

(3) RIPRAP BASE

(4) COMPACTED SAND AND GRAVEL

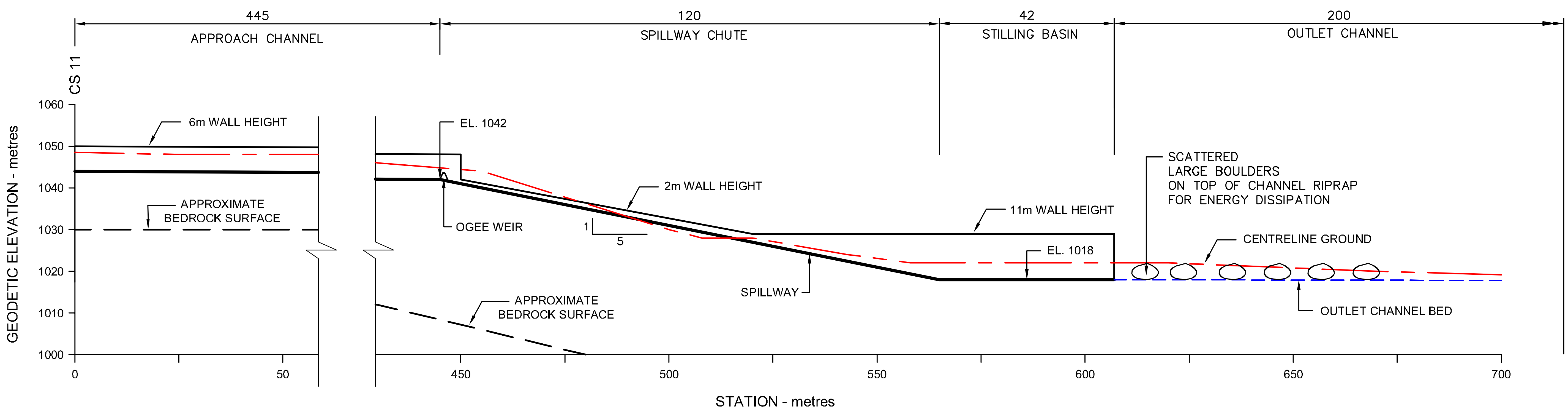
(7) FINE COMPACTED ROCK FILL (200 mm MIN.)

(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 21**
northwest hydraulic consultants ltd.		



SCALE 1:1000

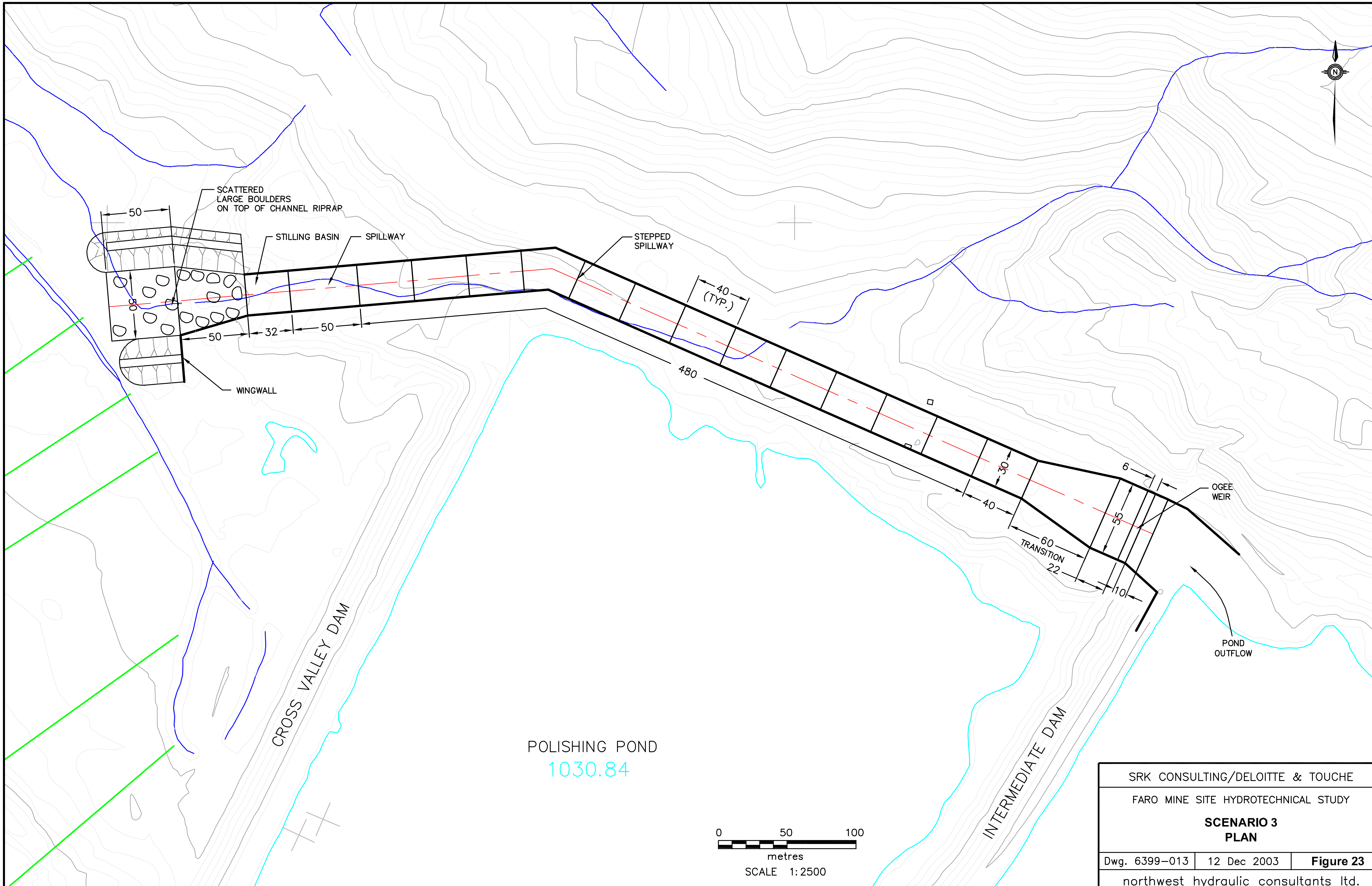
SRK CONSULTING/DELOITTE & TOUCHE

FAROMINE SITE HYDROTECHNICAL STUDY

SCENARIO 2
SPILLWAY CENTRELINE PROFILE

Dwg. 6399–019	16 Dec 2003	**Figure 22**

northwest hydraulic consultants ltd.



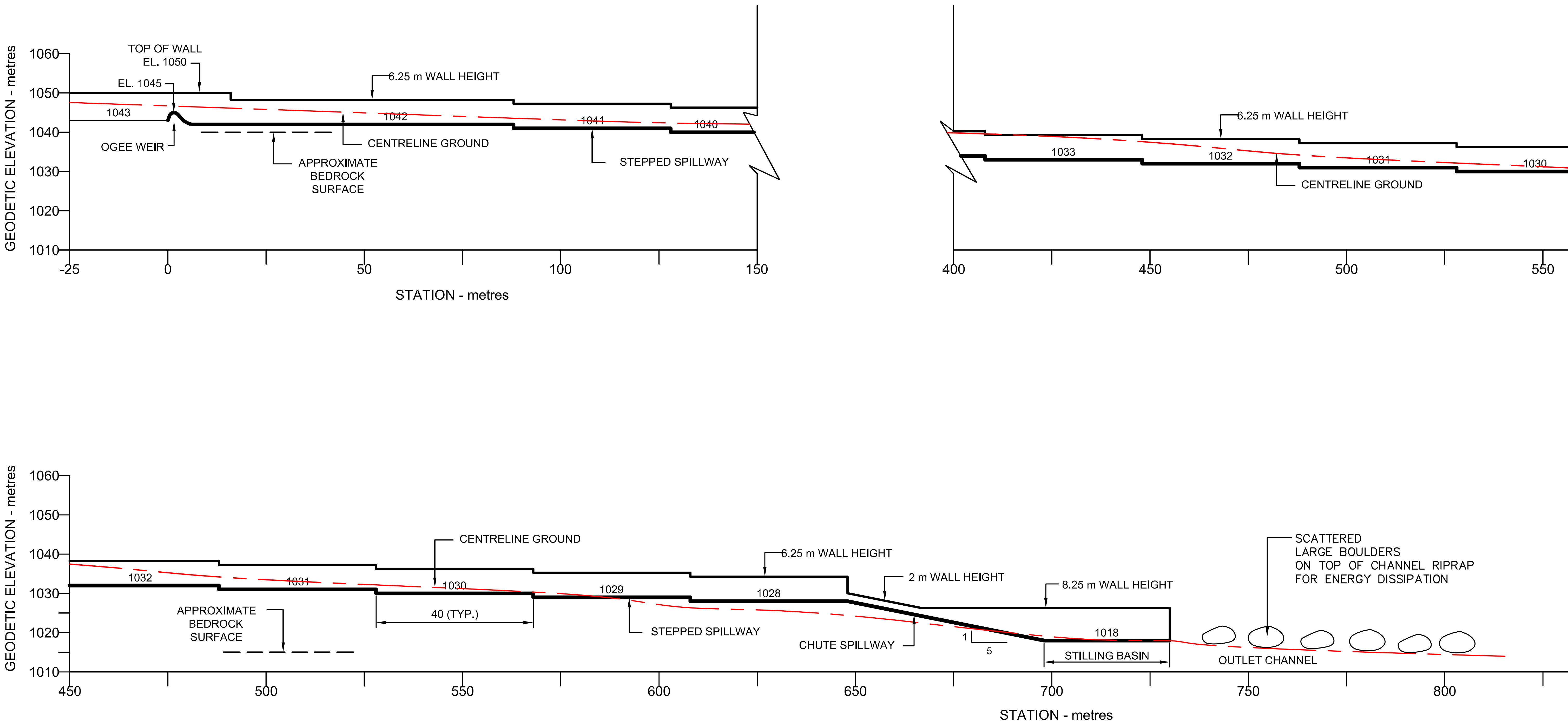
SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

SCENARIO 3
PLAN

Dwg. 6399–013	12 Dec 2003	**Figure 23**

northwest hydraulic consultants ltd.



SCALE 1:1000

SRK CONSULTING/DELOITTE & TOUCHE

FARO MINE SITE HYDROTECHNICAL STUDY

SCENARIO 3
SPILLWAY CENTRELINE PROFILE

Dwg. 6399–701	15 Dec 2003	**Figure 24**

northwest hydraulic consultants ltd.

PHOTOGRAPHS

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

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

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

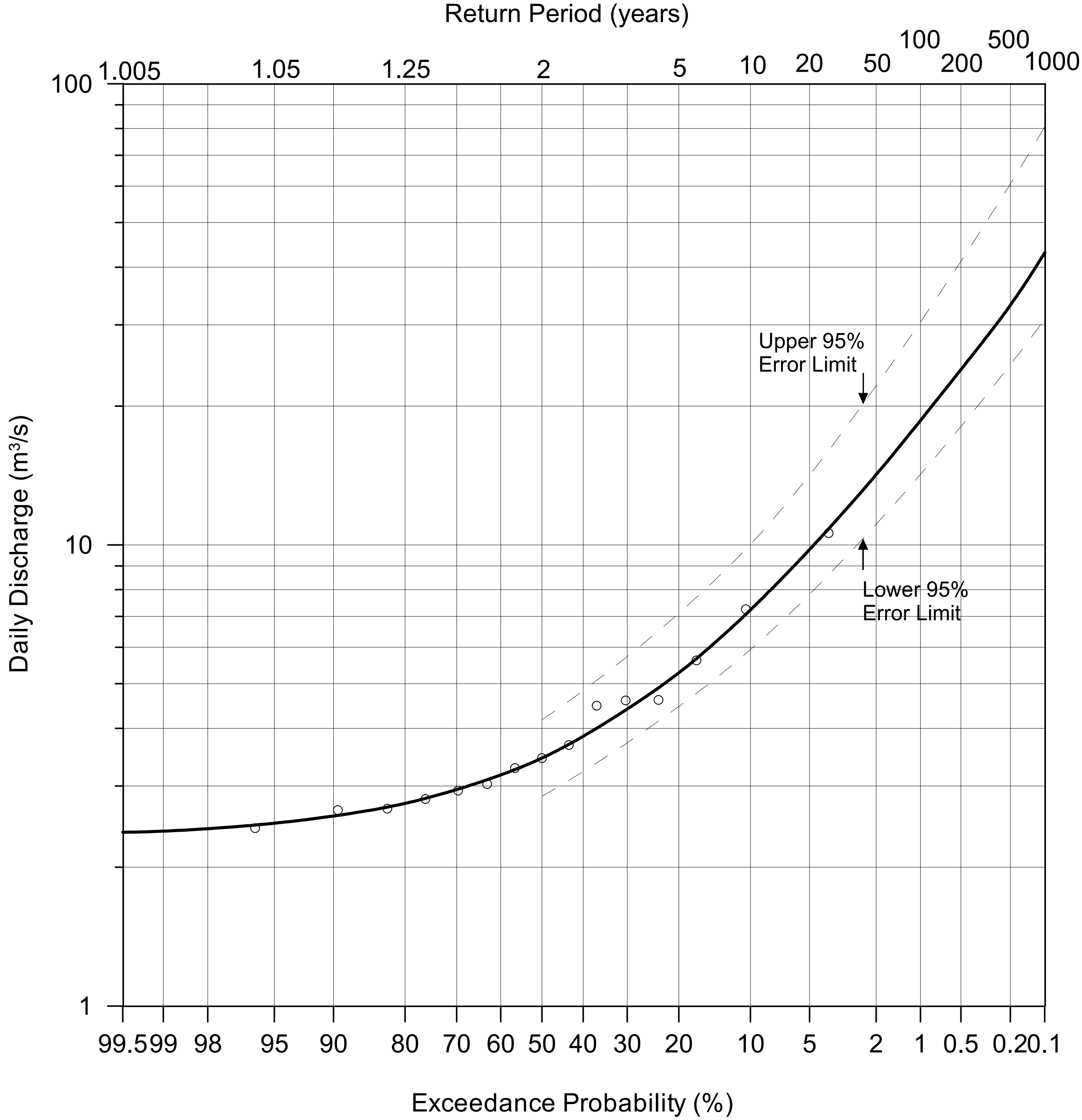


Figure A.1

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

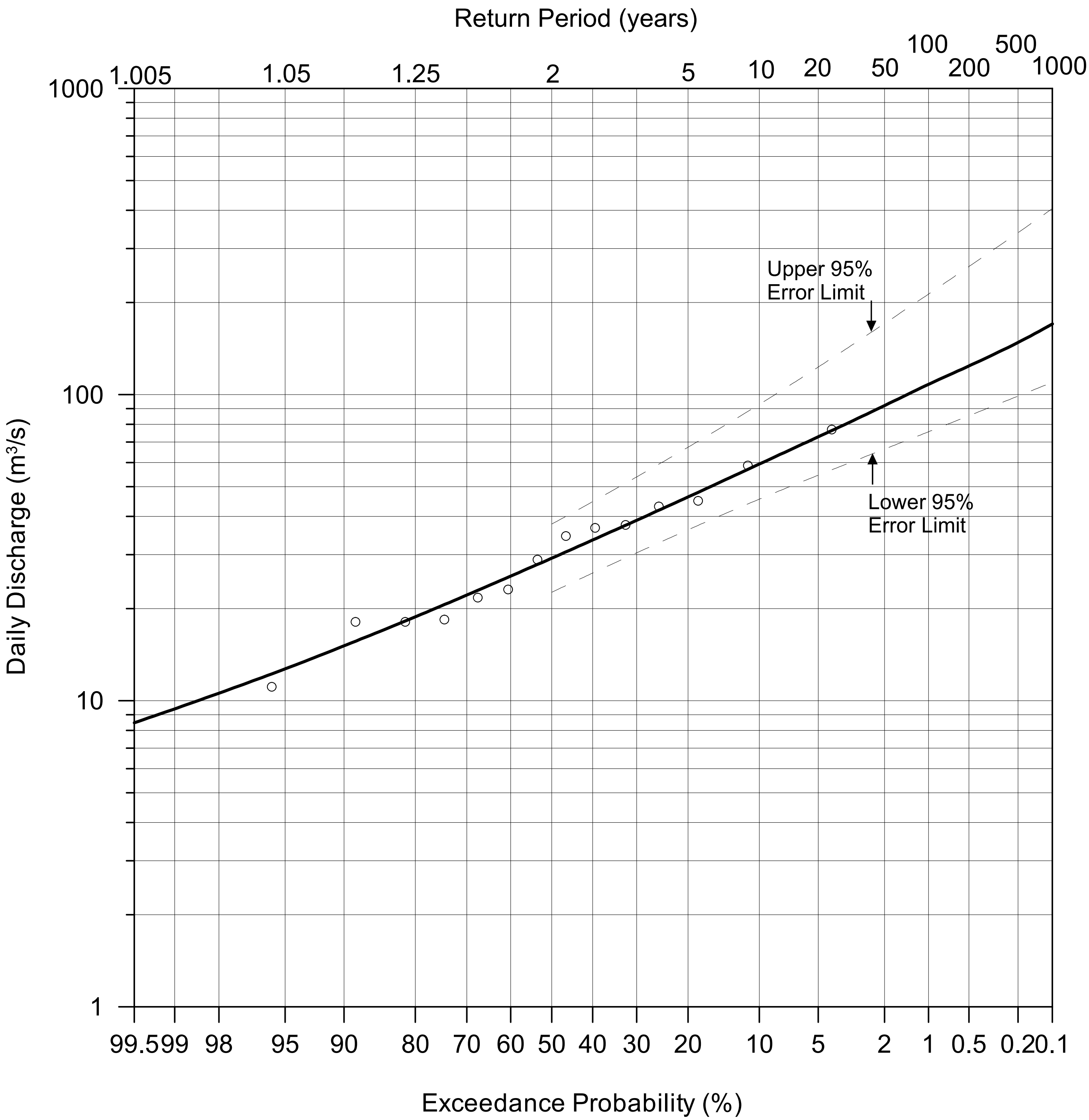


Figure A.2

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

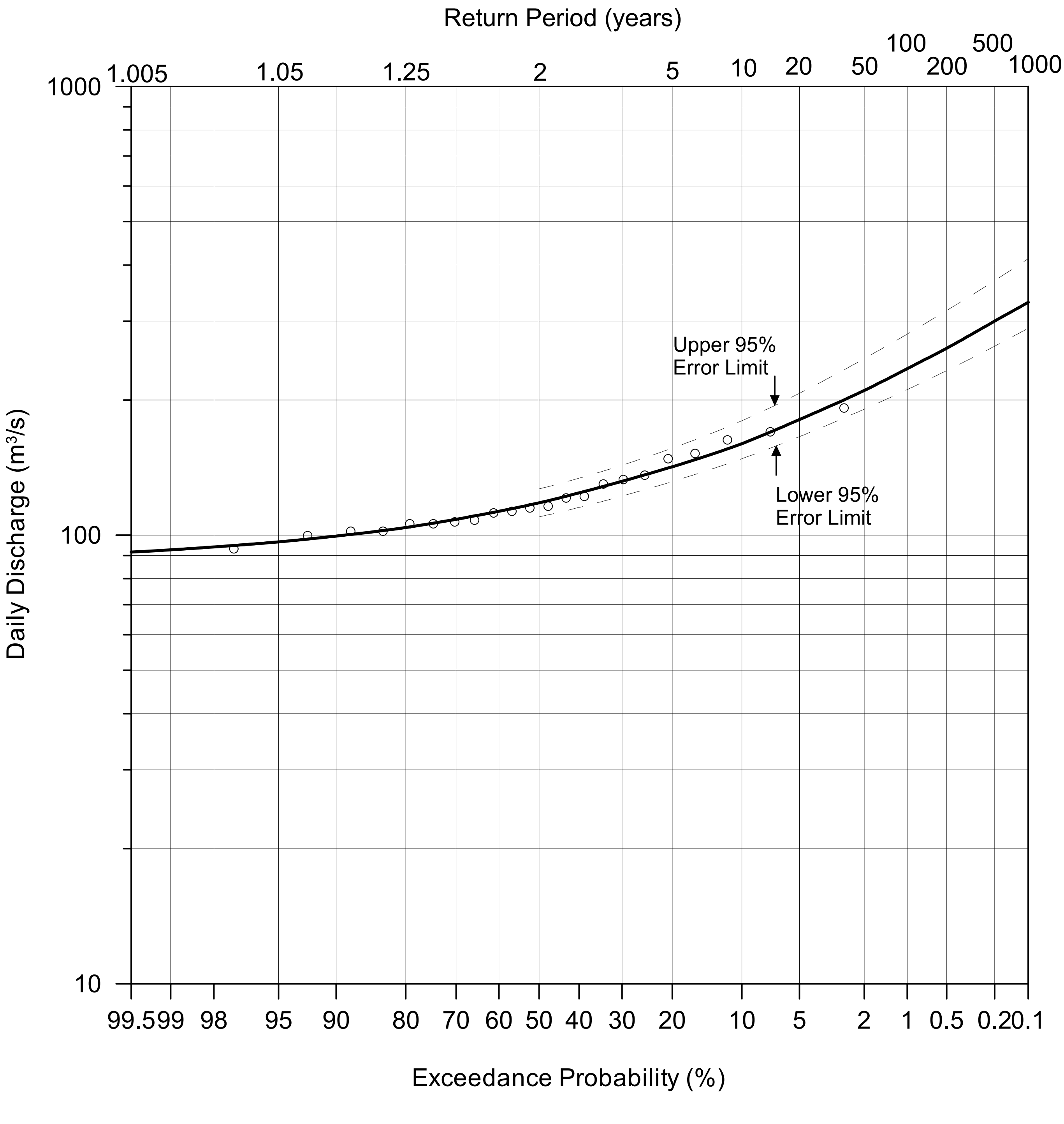


Figure A.3

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

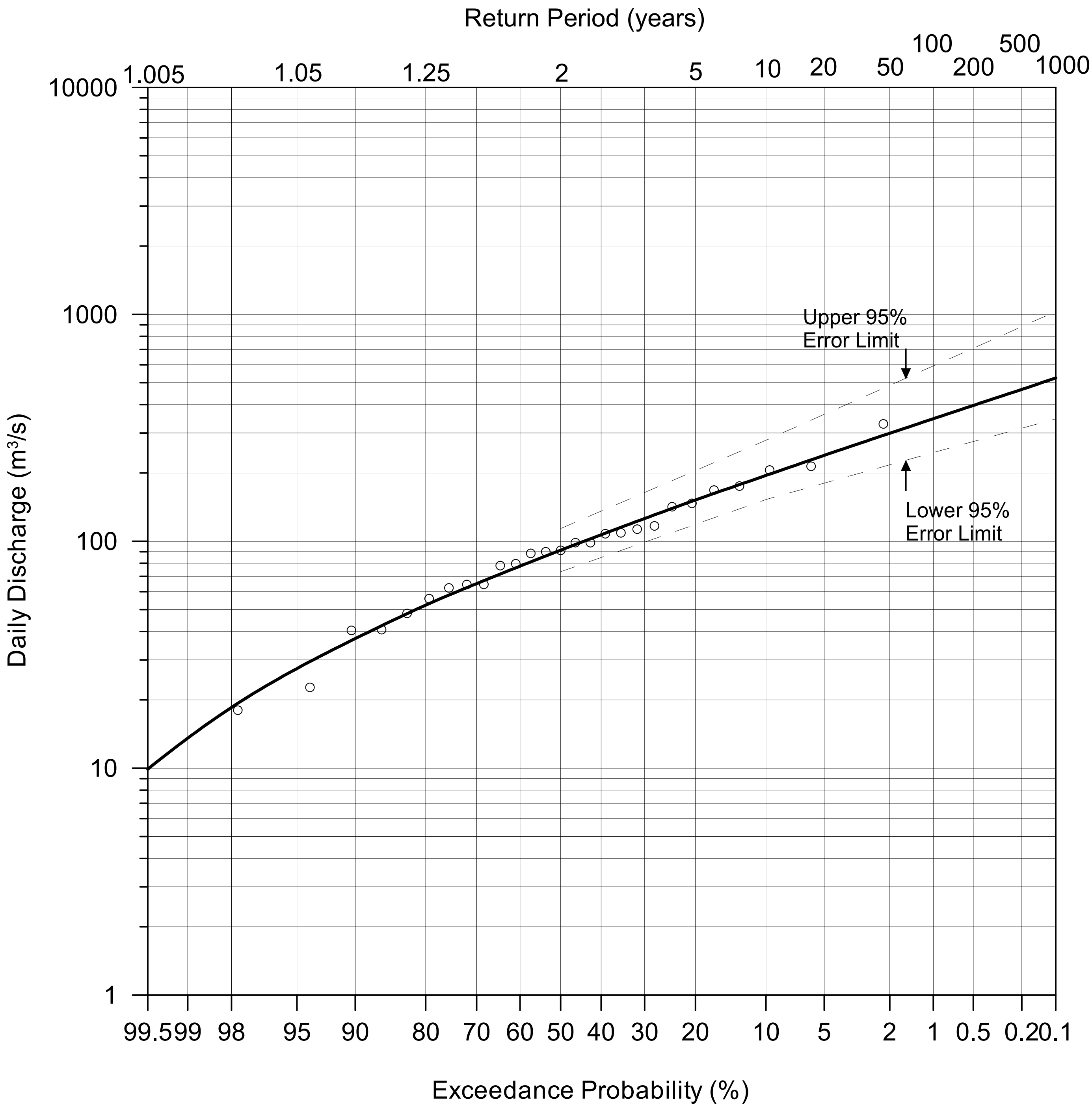


Figure A.4

Flood Frequency Plot
Nordenskiold River below Rowlinson Creek 09AH004 (1983-2002)
3 Parameter Lognormal Distribution with 95% Error Limits

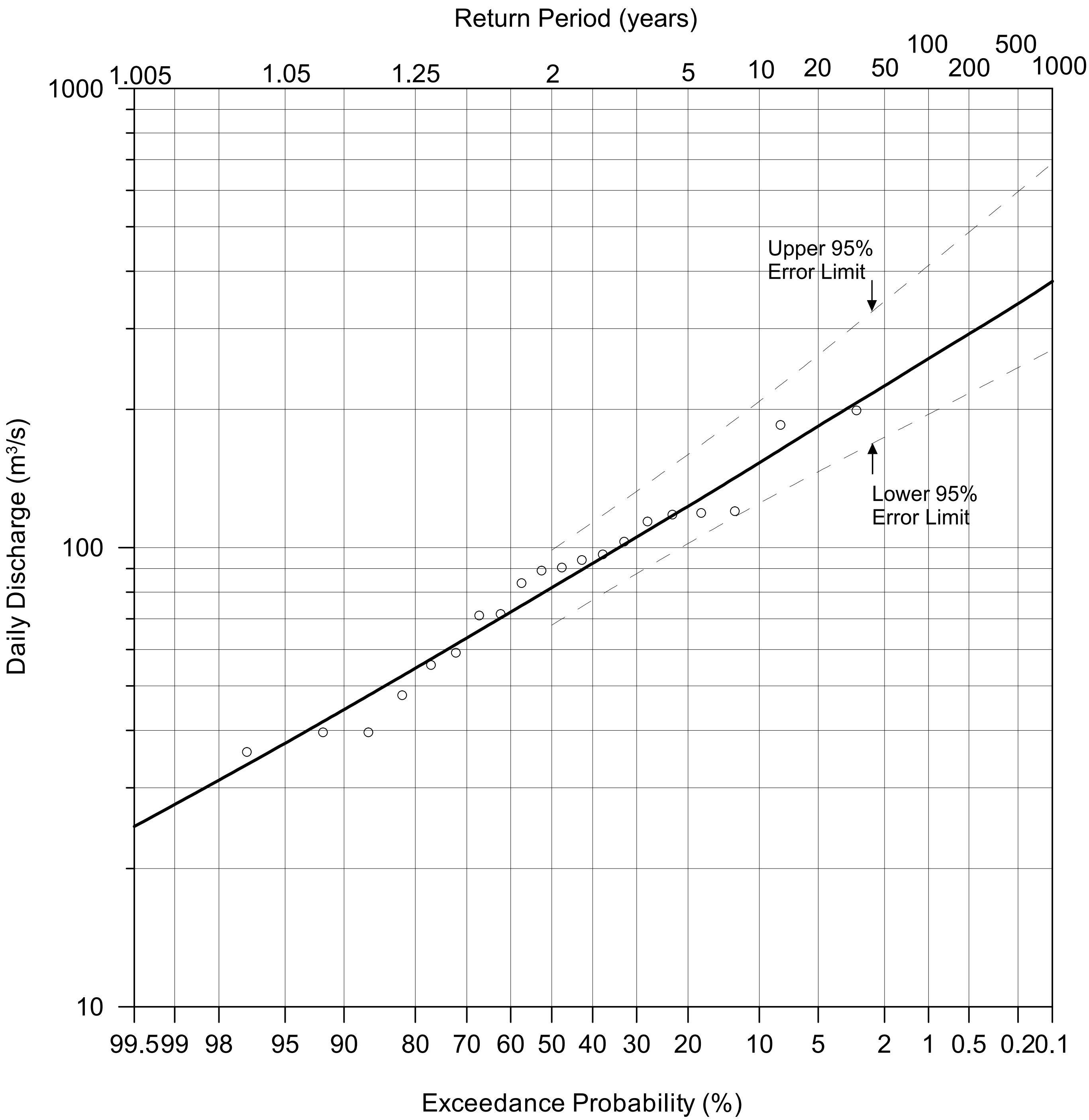


Figure A.5

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

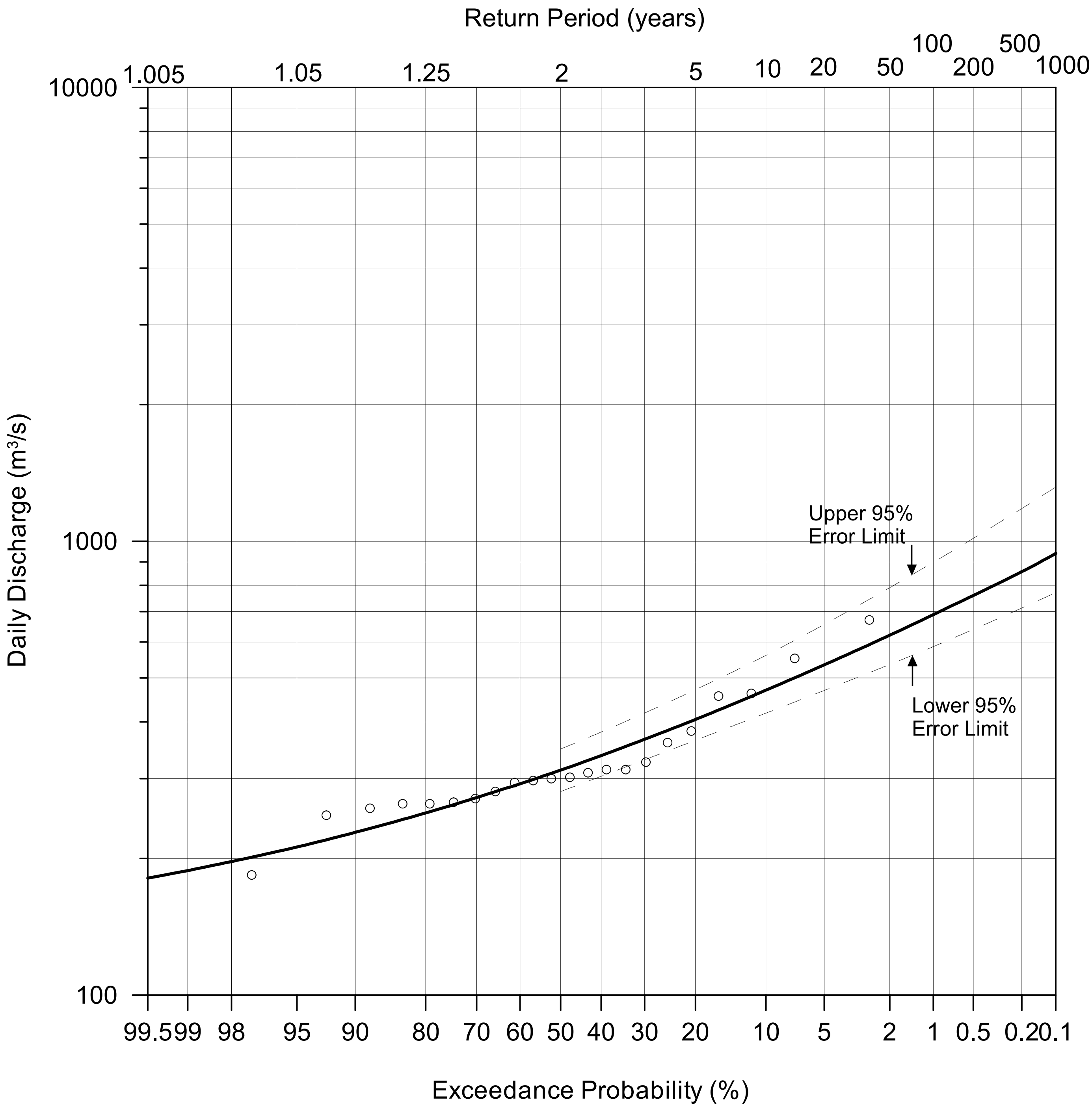


Figure A.6

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

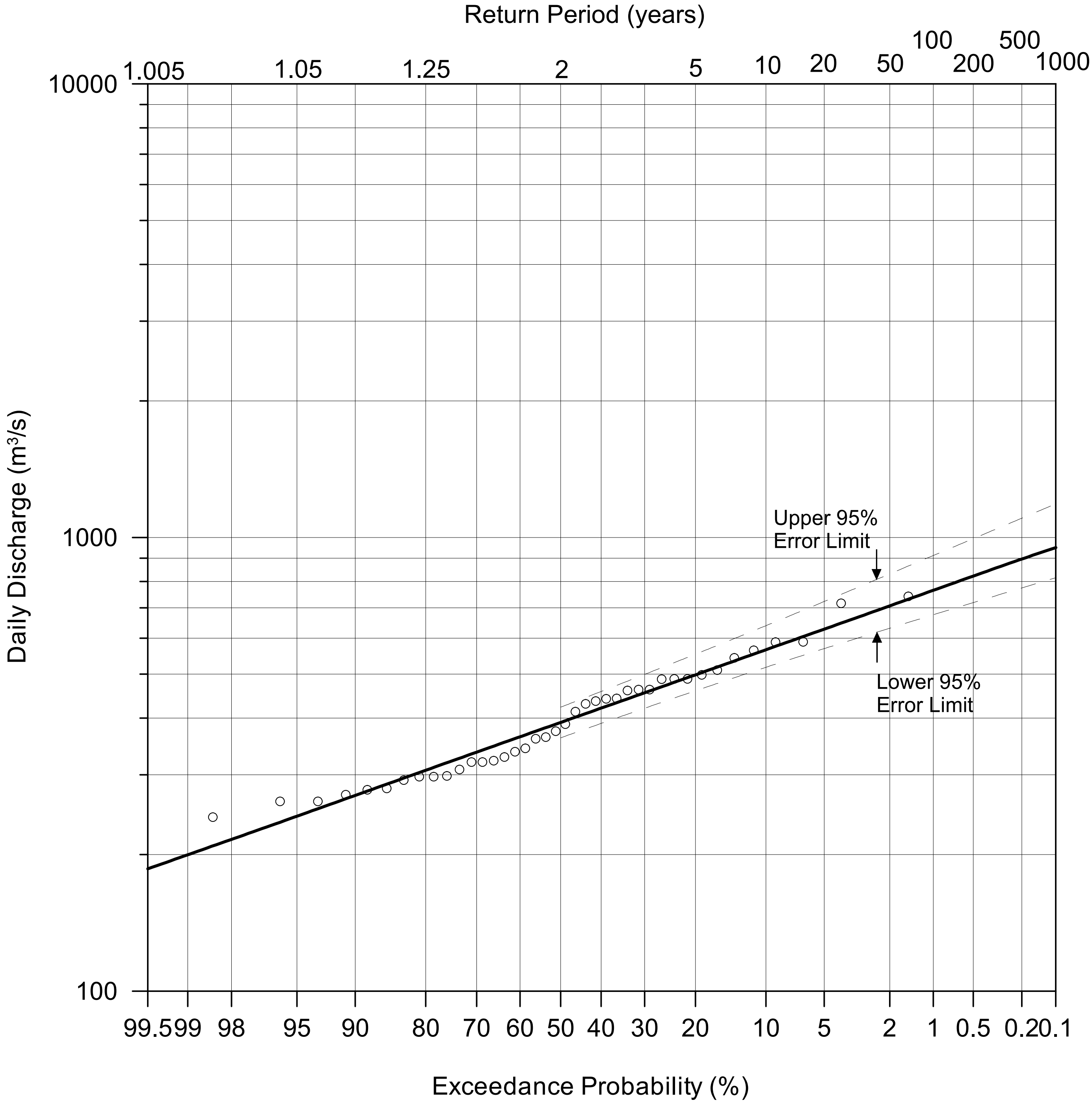


Figure A.7

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

<u>Introduction</u>

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.

<u>Record Daily Rainfall</u>

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 CONSIDERATIONS

By: BGC Engineering Inc.

APPENDIX C

See separate PDF File on CD

cover photo credit - Mike Bryson

nhc

northwest hydraulic consultants