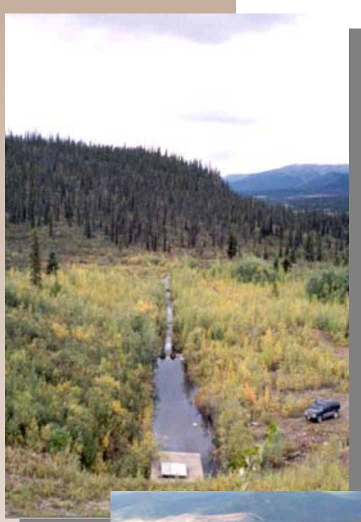
Deloitte & Touche

Preliminary Breach Design Fresh Water Supply Dam, Faro Mine



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PRELIMINARY BREACH DESIGN
FRESH WATER SUPPLY DAM, FARO MINE

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PRELIMINARY BREACH DESIGN
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1 INTRODUCTION

1.1 General

The Interim Receiver for Anvil Range Mining Corporation is planning to breach the Fresh Water Supply Dam (FWSD) prior to the 2004 spring freshet. The FWSD is no longer needed and, in the absence of further work, constitutes a risk to the tailings infrastructure and receiving environment located downstream.

The work presented herein was completed in response to a directive from Fisheries and Oceans Canada (DFO) addressed to both the Interim Receiver and Indian and Northern Affairs Canada (DIAND). After receiving the directive and discussing it with DIAND, the Interim Receiver retained a team of engineers and scientists to develop a preliminary design for breaching of FWSD. The team included BGC Engineering Inc.(BGC) and Gartner Lee Ltd. (GLL) working under the direction of SRK Consulting Inc.(SRK).

At a meeting of the engineering team with representatives of DIAND and DFO, options for meeting the objectives outlined in the DFO directive were reviewed and a preferred design concept was selected. The selected design concept consists of an engineered breach through the body of the dam along the alignment of the original channel and re-establishment of the pre-construction creek. The engineering team has subsequently prepared a preliminary design based on that concept.

This report describes the current situation at and around the FWSD, develops design criteria for the breach, and presents analyses and preliminary drawings that will form the basis of the breach design. This report and the preliminary design presented herein are intended to fulfill the requirements of the DFO directive. Additional investigations and assessments are ongoing, and the intention is that a detailed design will be completed by April 1, 2003.

1.2 DFO Directive

DFO issued its directive to the Interim Receiver and DIAND in a letter dated November 15, 2002. The letter requires, pursuant to Subsection 37(1) of the Fisheries Act, that the Interim Receiver and DIAND provide DFO with "plans and specifications pertaining to the total breach of the Faro FWSD and the interim lowering of its reservoir". Specifically, the following plans and specifications are to be submitted on or before February 3, 2003:

1. Practical engineering plans and costs to conduct a full breach of the Faro FWSD and low-level outlet pipe removal over fall-spring 2003-2004.
2. Engineering plans as requirement #1 and costs to accommodate retrofitting of flood attenuation options such as French drains within the outlet configuration.
3. Engineering plans as in requirement #1 and costs to accommodate fish passage through the dam under most flow conditions.
4. Fish and fish habitat mitigation and compensation plans and their cost to restore that portion of the original Rose Creek within the reservoir and the related tributaries as per information request included in correspondence from DIAND in November 2002."

The letter further indicates that construction work at the FWSD will require Fisheries Act authorizations, under Section 32 and 35(2), which will in turn require a positive review under the Canadian Environmental Assessment Act. The letter also states that "through a full breach DFO would consider reduction in downstream risks by return to a stable system as sufficient offsetting justification for loss of the reservoir".

1.3 Clarification Letters

DIAND submitted a letter, dated December 16, 2002, to DFO requesting clarification of two points in the DFO directive. DFO responded, in a letter dated January 7, 2003, with the following clarifications:

- DFO is prepared to consider plans that do not elaborate on subsequent retrofitting of flood peak attenuation if clear technical justification is provided.

- DFO is prepared to consider alternatives that satisfactorily mitigate the risk imposed on the downstream environment without excavation of the low level pipe, if clear technical justification is provided.

1.4 Project Schedule

Representatives of DFO, DIAND, SRK, BGC, GLL, Geo-Engineering Inc. and Hydroconsult attended a project planning meeting in Vancouver on December 2 and 3, 2002. The meeting began with a discussion of the risks associated with the FWSD, and the options for reducing those risks. The breach design concept discussed above was selected as the preferred approach. A summary of the meeting is included in Appendix A.

A preliminary project schedule was then developed and the timing of design, permitting and construction activities was estimated. A copy of that schedule, slightly modified to include the timing of tender calls, is provided as Figure 1.1. The key project requirements and schedule constraints are as follows:

- The DFO directive specifies requirements for February 3, 2003;
- Requirements under the Canadian Environmental Assessment Act (CEAA) are likely to include a Comprehensive Study;
- A DFO Authorization will be required;
- A Water Licence will be required;
- Construction and revegetation in dry areas could be initiated in summer 2003;
- Fish will need to be removed from the FWSD reservoir before dewatering can be completed;
- Breach construction should be completed before the freshet in 2004; and
- Upstream construction and revegetation could be completed in summer 2004.

The meeting attendees concluded that the project schedule will be very challenging, particularly in light of the devolution of federal powers to the Yukon Territorial Government, and the potential complications arising from the Yukon Environmental Assessment Act. It was agreed that meeting the schedule would require regular communication amongst all parties. Representatives of DFO, DIAND, Environment Canada, the Interim Receiver and the engineering team were subsequently nominated to participate in regular conference calls, and to coordinate the related activities of their organizations.

2 SITE DESCRIPTION AND CURRENT CONDITIONS

2.1 Location

Faro Mine is located in the central Yukon, approximately 200 km north-northeast of Whitehorse. The Faro mine site is approximately 22 km north of the Town of Faro. The FWSD and reservoir are located south of the main access road to the Faro Mine site, approximately 5 km from the mine gatehouse. The location of the Faro Mine and the FWSD within the mine site are shown in Figure 2.1 and Drawing 1.

The report contains two versions of design drawings, a set of full size (D-sized) design drawings and set of reduced (50% reduction) design drawings. The reduced drawings are identical, except for scale, to the D-sized drawings.

2.2 Fresh Water Supply Dam

2.2.1 Dam Construction and Components

The FWSD was constructed as part of the original mine development and was used to supply water for the milling process. A subsequent Water Licence requirement specified a minimum flow of 75 L/s in the Rose Creek Diversion, and the FWSD was used to sustain that flow during winter. A recycle water system replaced the reservoir as the primary supply of water to the processing plant in 1997. Fresh water supply from the FWSD is not required for the current care and maintenance activities.

The FWSD was constructed in 1968, on the South Fork of Rose Creek. Section, profile and plan views of the FWSD are shown in Drawing 2.

The dam is a zoned earthfill dam consisting of a low permeability compacted core and upstream blanket material (Zone 1), a broadly graded granular shell (Zone 2) and a compacted random fill zone (Zone 3). The Zone 3 material consists of material that did not meet the specifications for either Zone 1 or Zone 2 (Drawing 2). Zone 1 includes an upstream cutoff trench, but it does not extend to bedrock and is therefore referred to as a partial cut-off. The base elevation of the partial cut-off was limited to elevation 1079.9 m amsl (approximately coincident with the creek thalweg), to ensure that the excavation was completed in the dry. The Zone 1 material was extended approximately 80 m into the reservoir, forming an upstream blanket to reduce seepage under the partial cut-off. The seepage blanket is 1.5 m thick and is covered by a 0.6 m thick layer of gravel.

At the downstream toe of the dam, a seepage collection trench was installed. It extends to bedrock or a maximum depth of 4.6 m (15 feet). The material within the seepage collection trench was specified as filter gravel with 1% silt sized particles allowed. This trench was installed at the toe of the dam in the deepest section of the valley and extends about 114 m east of the valve house along the toe of the dam. The seepage collection trench exits near the valve house and feeds water into the fresh water channel (Drawing 2).

A cofferdam was constructed upstream of the seepage blanket (Drawing 2) to ensure that construction of the FWSD was completed in the dry. The cofferdam was constructed from compacted Zone 1 soil (Drawing 3) and had a crest elevation of 1083.5 m amsl. A 1.07 m (42 inch) diameter diversion pipe was used to direct the inflows around the cofferdam, under the FWSD construction site and into the downstream environment. After construction of the FWSD was completed, the central portion of the cofferdam was breached and the portion of the diversion pipe under the dam was plugged (Drawing 2). The cofferdam was located and the breaching of the cofferdam was viewed as part of the diving inspection performed in December 2002 (Territorial Diving Technologies, 2002).

The general arrangement of the FWSD is shown in Figure 2.2. According to a recent (2001/2002) survey of the dam (BGC, 2002), it is approximately 410 m long, 20.5 m high at its highest point (from original ground level) and 6 m to 7 m wide at the crest. A view of the crest of the FWSD is shown in Photo 1. The slope of the upstream face of the dam is approximately 2.6H:1V and the slope of the downstream face of the dam is 2H:1V. Table 2.1 contains a summary of the key elevations for various features of the dam. The elevations contained in Table 2.1 have been converted from imperial units to metric and have been corrected for the variation between the original mine datum and mean sea level (subtraction of 109.2 feet is required to convert original data to msl).

Table 2.1
Key Elevations of the FWSD (1968 as built drawings)

Feature	Elevation (m amsl)
FWSD crest	1099.0
Top of Zone 1 (core)	1097.5
Spillway crest	1096.3
Top of trash rack (inlet to low-level outlet pipe)	1083.8
Inlet elevation of low-level outlet pipe	1082.0
Spring line of low-level outlet pipe at the inlet location	1077.8
Natural elevation of ground in the center of the FWSD	1078.4
Natural ground elevation at the u/s toe to the FWSD	1080.8
Cofferdam crest	1083.5

Water is released from the reservoir by two means: an overflow spillway located on the crest of the dam near the north abutment (Photo 2) and a low-level outlet pipe, which runs through the base of the dam near the south abutment (Photo 3). The location of the two outlets from the reservoir is shown on Figure 2.2 and Drawing 2.

The low-level outlet (LLO) consists of:

- A 1.07 m (42-inch) diameter open orifice inlet covered with a trash rack (Drawing 3).
- A 1.07 m concrete-encased pipe, which was placed in a trench excavated into bedrock with seepage collars, extending beneath the dam footprint. The total length of the LLO pipe, measured from the inlet to the valve house is 127 m (based on the as-built drawings).
- A valve house and outlet located on at the downstream toe of the dam (Drawing 3). In the valve house, the 1.07 m pipe is reduced to 0.61 m (24-inch) diameter. Following the reducer fitting, two valves are located in close succession. The outflow downstream from the valves releases into a chamber of the valve house (Photo 3), hitting the downstream wall of the structure for energy dissipation. The water in this chamber then flows over a weir in the side wall of the valve house and into the fresh water channel. The fresh water channel carries the flow downstream (Photo 4 and Figure 2.2) until it reaches the confluence with the natural channel.

An overflow spillway is located near the northern abutment of the dam (Figure 2.2). The spillway consists of a 30-m wide concrete sill, with vertical concrete wing walls (Photo 2). The top of the core (Zone 1) is located 1.5 m above the spillway crest. The calculated spillway discharge capacity is 94 m^3/sec (NHC, 2001). The calculation was based on treating the spillway as a broad crest weir, with a reservoir elevation of 1097.8 m asl (the top of the core). The spillway discharges water into the South Fork of Rose Creek via a discharge channel excavated through rock and overburden materials. The discharge passes through two culverts under the access road (photo 5). Downstream from the culverts the water rejoins the natural channel, as shown in Figure 2.2.

2.2.2 Dam Performance

Seepage was noted at the downstream toe of the FWSD immediately following first filling of the reservoir, which occurred during the 1969 freshet. Based on the near-surface groundwater table and anticipated increase of pore pressure from the construction of the dam lead designers to expect seepage to occur at the downstream toe of the dam (Ripley, Klohn & Leonoff, 1969). Shortly after construction was completed, the appearance of seepage and accompanying sand boils lead to the design and construction of a downstream toe drain/berm in 1969. The location of the berm is shown in plan on Drawing 2 and the details of the berm construction are shown on Drawing 3. The designers noted that the total amount of seepage at the toe was less than expected.

Following a stability assessment by Golder Associates in 1988, another toe berm, including toe drainage measures, was placed for enhancement of the downstream slope stability. The berm addition contains a preferentially pervious lower section (0.5 m thickness) that provides discharge capacity for artesian seepage. The 1989 toe berm is 55 m long, approximately 7 m high at the original toe of the dam, and slopes downstream at 7.5H:1V to a seepage collection trench at the toe. The proposed construction details for the 1989 berm are shown on Drawing 3. The extent of the 1989 berm construction is reflected in the topographic contour lines included on Drawing 2. As-built records related to the construction of either the 1969 or 1989 berms have not been located.

Significant longitudinal cracking has been encountered on the upstream side of the dam crest for nearly 20 years (Photo 6). The cracking has been professionally investigated on more than occasion, most recently in 1994 (Golder 1994). The cracking was attributed to frost action on the upstream side and crest of the dam. The investigation traced the cracks to a depth of almost 2 m. The results from thermistor monitoring at the crest of the dam indicates that frost penetrates to a depth of approximately 4 m, during normal reservoir operations. Results from the winter of 2001/2002 indicated that the depth of frost penetration was 6m (BGC, 2003), the reservoir level was elevation of 1090 m amsl during 2001/2002.

2.2.3 LLO Performance

The LLO operated between 1968 and 1997 in order to provide a consistent minimum flow of 0.475 m^3/s (0.4 m^3/s during initial operation). No available deficiencies or repairs to the LLO are documented. During a site review in support of a potential dam raise (Dome/Acres, 1984), the valves of the LLO were opened fully. Although there are some discrepancies between the reported flow (Klohn Crippen, personnel communication, 2003) and expected flow for this condition, the LLO was successfully operated (19 years ago) for a short time at full flow conditions. During the 1984 flow test, significant vibration and noise was reported.

A diving inspection of the LLO was performed in 2002 (Diving Dynamics, 2002). The inspection revealed four key results:

1. Sonic testing indicated that the wall thickness of the pipe was 44% to 54% of its original thickness in three general locations. The thinnest wall thickness measured was 0.165 inches (compared to the original 0.375 inch) approximately 5 m downstream from the inlet.
2. The inside pipe walls were coated with a “growth material” with an average thickness of 50 mm.
3. The measured length of the LLO was about 15 m shorter than shown on the as-built drawings. The divers measured a total pipe length of 112 m and the as-built drawings indicate a total length of 127 m.
4. A bend, not shown on the as-built drawings, was encountered under the dam.

An assessment of the valves of the LLO is currently underway, to be finalized early February 2003. Preliminary conclusions (Klohn Crippen, personal communication, 2003) indicate the following:

1. Under the current configuration, the LLO, the outflow should be limited to 0.82 m^3/s (5.
2. A minor variation between the existing piping arrangement and the as-built drawings was noted. A 0.37-m (14.5-inch) length of pipe was located between the two valves.
3. The piping within the valve house is not supported. The support pedestals were cast prior to placement of the piping and a gap exists between the pipe and the supports.
4. Forty pipe wall measurement were made. The minimal wall thickness encountered was 0.300 inch, which is 80% of the original thickness.

Recent calculations by Klohn Crippen (2003) indicate the maximum flow through the LLO in its current configuration is 0.82 m^3/s (10,650 Igpm). The current maximum flow rate is limited by the pressure drop across the valves and the resulting risk of cavitation. Preliminary calculations indicate that, if the pressure drop could be limited through the use of an orifice, the maximum total flow rate could be increased to 2.3 m^3/s (30,000 Igpm). That rate assumes that the reservoir would be full (water elevation of 1096 m amsl), and maximum flow rates would be lower at lower water levels.

2.2.4 Reservoir Operations

The fresh water supply (FWS) reservoir occupies part of the base of the Rose Creek valley. Figures 2.3 and 2.4 are air photos of the reservoir area. Figure 2.5 presents a new topographic base plan of the reservoir area that was prepared for this project. The new base plan required the combination of results from a number of previous topographic surveys. No survey of the entire project area has been performed. Drawing 4 shows topography and bathymetry of the reservoir and immediate downstream areas.

The reservoir is approximately 1,454 m long with an average width of 315 m and an average depth of 8 m. Three tributaries, shown on Figures 2.3 and 2.4 and Drawing 4, flow into the reservoir. The largest of the three is the main channel of the South Fork of Rose Creek. Two smaller tributaries, North Tributary and the Southeast Tributary, are located near the east end of the reservoir.

Construction of the reservoir has shortened the lengths of the tributaries that formerly existed in the reservoir area. The length of South Fork of Rose Creek that has been either submerged or modified as part of the dam construction is 2,420 m. About 190 m of the Southeast Tributary and 440 m of the North Tributary are submerged by the reservoir.

Prior to 1997, mine operations required approximately 0.4 m^3/s of fresh water to operate the concentrator plant. The current Faro Water License (QZ95-003) requires that a minimum flow of 4.5 m^3/min (0.075 m^3/s) be maintained in the Rose Creek diversion canal for fisheries and conservation purposes. The reservoir would typically (during mine operations) fill completely (to 1096.3 m amsl) and water would flow through the spillway from June through late fall. In 1976, steel I-beams were placed within the spillway to allow for the addition of stop logs to raise the retained reservoir elevation. Stop-logs were typically placed across the spillway in the fall to provide increased water storage capacity. In 1999, DIAND instructed that the stop-log system be removed due to concerns that the excess water pressure caused by a higher reservoir elevation could lead to increased seepage at the downstream toe and exacerbate cracking of the crest.

No specific reservoir elevation data was kept regarding historical normal and extreme year operating practices. However, site staff noted that spring reservoir levels could be very low (a drawdown of 9 to 10 m below the spillway invert occurred on at least one occasion), with a typical winter drawdown of about 6 m. During mine operations the outflow from the LLO was a minimum of 0.475 m^3/s.

Since the beginning care and maintenance activities at the mine, the reservoir has fluctuated naturally with flows through the spillway for 8 to 9 months of the year and the continual release of water through the LLO. The water released through the LLO runs year-round and the discharge is estimated to be between 0.2 to 0.5 m^3/s.

The operation of the reservoir will be modified in 2003 to minimize the risk of uncontrolled breaching. Following modifications to the valve arrangement of the LLO (Section 5.1.1), it will be used, potentially in combination with siphons and pumps, to nominally maintain the reservoir elevation between 1090 and 1091 m amsl. Reservoir levels greater than 1091 m amsl may occur on a short-term basis but are expected to be limited to periods of several days.

2.2.5 Storage Capacity Curve

The total height of the FWSD at its centerline is 20.5 m, which is the difference between the dam crest and the natural bed of the creek (see Table 2.1). The normal full supply level of the reservoir is 1096.3 m amsl (elevation of the spillway crest). The total storage of the reservoir is 4.2 million m^3 at the full supply level. The height-capacity curve for the reservoir is shown in Figure 2.6 (based on the bathymetric survey by GLL, 2002).

The recent (GLL 2002) height-capacity curve was compared to previous versions (Parsons-Jurden, 1968, Kilborn, 1986 and Harder, 1991). The Parsons-Jurden and Kilborn curves agree very well with the GLL (2002) results. The 2002 curve was therefore selected for use in the breach design. (The height-capacity curve developed by Harder was very different from the other two. It suggests that the storage volume of the reservoir is 5.7 million m^3 at elevation 1096.3 m amsl, whereas the other curves all indicate about 4.2 million m^3 of storage at that elevation. No method description or raw data from the Harder curve could be found. It is noteworthy here primarily because it appears to be the source of confusion in some of the reports prepared in the 1990's.)

2.2.6 Previous FWSD Investigations

A number of investigations have been performed at the FWSD, and provide additional information on site conditions. The following provides a summary of the main investigations that were reviewed as part of the breach design:

- Parsons (1967) performed borehole drilling along the original alignment of the dam. The investigation consisted of wash bore drilling to determine the depth of bedrock. The soil was described as sand, gravel and boulders with some silt.
- Ripley, Klohn and Leonoff Ltd. (1968) performed boreholes drilling along a revised alignment for the dam, the dam was constructed on the revised alignment. The investigation included characterization of the depth to bedrock, characterization of the soil conditions (including laboratory testing) and determination of the ground water elevation. The pre-construction creek was described as being incised in a shallow meandering channel about 6.1 m (20 feet) and about 1.5 m (5 feet) deep. The creek bed dropped from about elevation 1080.7 m amsl to 1074.6 m amsl as the stream moved in a relatively straight channel as it passed a terrace where the dam was constructed.
- Dome/Acres (1985) performed drilling and testing pitting as part of an investigation for dam raising. The investigation included the installation of a thermistor and a number of piezometers both at the toe and in the crest of the dam.
- Golder Associates (1988) performed drilling to determine the piezometric levels in the dam, the soil properties of the dam and the foundation material properties. This investigation was performed as part of a stability assessment of the dam. This lead to the installation of a toe berm in 1989 to improve downstream stability for earthquake loading.
- Diving Dynamics (2001) completed a diving inspection of the low level outlet. The four main results of the inspection were: The wall thickness was reduced to 44 to 54% of its original thickness in three locations. The LLO was measured to be approximately 15 m shorter than shown on the as-built drawings. A bend of the LLO was encountered beneath the main footprint of the dam. A wall coating about 50 mm thick was encountered.
- BGC (2001) performed drilling on the downstream side of the dam to characterize the depth to bedrock and soil properties. This investigation included installation of piezometers downstream of the toe of the dam.
- BGC (2002) performed drilling in preparation for lowering of the existing spillway through the excavation of a bedrock notch. Drilling concentrated on determining depth to bedrock, soil and bedrock properties. Piezometers were installed within the dam to replace a non-functioning piezometer.
- Territorial Diving Technologies (2002) replaced the trash rack that was removed during a previous dive inspection. ROW inspection of the cofferdam location to confirm presence and to determine if breaching of the cofferdam had been performed. Depth of sediment near the LLO inlet was measured and two sediment samples were collected.
- Klohn Crippen (2003) performed a dam safety review of the dams at the Faro Mine, including the FWSD, in accordance with CDA guidelines. The review included a review

of existing information available and a site inspection. The dam was given a preliminary classification and dam safety issues (both actual deficiencies and data gaps) were identified and discussed.

- Klohn Crippen (2003) performed an inspection of the low level outlet within the valve house and evaluated the condition of the valves. Calculations were made to determine safe flow rates for the LLO.

2.3 Site Climate

2.3.1 Temperature

The Anvil (operated by Environment Canada) climate station was located at the mine site at an elevation of 1158 m amsl. The station no longer operates, but temperatures were recorded from 1967 to 1980 (RGC, 1996). The mean monthly temperatures are listed in Table 2.2.

Table 2.2
Mean Monthly Temperatures (°C) at Anvil Climate Station (1967-1980)

Parameter	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec	Yr.
Daily Max. Temp. (°C)	-15.1	-8.3	-5.3	2.2	9.3	16.0	17.5	15.2	9.6	1.6	-7.0	-12.6	1.9
Daily Min. Temp (°C)	-24.9	-18.8	-17.3	-8.7	-1.8	3.0	5.0	3.3	-0.9	-8.1	-16.7	-22.4	-9.0
Daily Temp. (°C)	-19.8	-13.9	-11.2	-3.2	4.0	9.9	11.5	9.5	4.6	-3.1	-11.6	0	-3.4

The temperature data from 1967 to 1980 indicate a mean annual temperature of -3.4°C. July is the warmest month, with a mean daily temperature of 11.5°C, and January is the coldest month, with a mean daily temperature of -19.8°C. Over the period of record, temperature extremes of 29.4 and -46.1°C have been measured.

2.3.2 Precipitation

The mean annual precipitation (MAP) at the Faro airport station is 304.7 mm, based on precipitation data from 1978-2001 (BGC 2002). This total comprises roughly equal proportions of rainfall and snowfall as water equivalent. The mean monthly distribution of precipitation is listed in Table 2.3. For the period of record the driest and wettest months are typically April and July, respectively. The greatest monthly precipitation measured over the period of record was 116.2 mm in August 2000.

Table 2.3
Monthly Mean Precipitation at Faro Airport, Yukon (1978-2001)

Month	**Mean Precipitation (mm)**
January	14.3
February	12.1
March	10.5
April	7.2
May	24.3
June	35.8
July	58.9
August	46.8
September	38.2
October	24.9
November	17.2
December	14.6
Annual Total	**304.7**

2.3.3 Snowpack

The Rose Creek snow course at the site was operated by DIAND from 1975 to 1985. The snow course was located near and at a similar elevation (1080 m) as the tailings impoundment area. The accumulation of snow at the tailings impoundment typically begins in October, and the snow has generally melted by the end of April, although in 1985 it persisted into May. At maximum snowpack in March or April the density of the snowpack is about 200 kg/m^3.

2.3.4 Wind

Wind data from the Faro airport indicates that the prevailing wind direction is from the southeast, following the alignment of the Tintina Trench. The long-term monthly mean wind speed data collected at the Faro airport are summarized in Table 2.4 (RGC, 1996). The wind data was collected through the use of an anemometer on a 10m tower near the airport terminal. The data is measured at each hour of the day, 365 days of the year.

Table 2.4
Long-term Monthly Mean of Wind Speed at the Faro Airport

Month	Mean Wind Speed (m/s)
January	1.4
February	1.7
March	2.2
April	2.6
May	2.7
June	2.7
July	2.6
August	2.1
September	2.1
October	2.2
November	1.7
December	1.5

2.4 Site Hydrology

The Rose Creek watershed covers an area of approximately 340 km^2 and is a significant part of the 980 km^2 Anvil Creek watershed, which drains the Southeast slopes of the Anvil Range Mountains. All of the Faro Mine site facilities are within the Rose Creek watershed.

Mine site creek and diversion canal flow measurement are made using automatic water level recorders and manually at specific flow measurement stations. The streamflow monitoring network was upgraded in about 1990, with further updates made in 1996.

Two local stream gauging stations have been operated by site staff since 1997: Stn. R7 (drainage area of 95 km^2) which is located on the North Fork of Rose Creek upstream from the mine site and Stn. X14 (drainage area 230 km^2) which is located on Rose Creek downstream from the mine site. Flow records from station R7 are shown in Figure 2.7. The flow records for R7 are representative of non-disturbed stream flow in the area of the Faro Mine. Although the station monitors a greater drainage area (95 km^2 versus 67 km^2) than exists for the FWSD the general shape measured inflows should be representative of the inflows that could be expected.

The FWSD reservoir drains an area of 67 km^2. A summary of the estimated monthly in flow (NHC, 2001) to the reservoir is included in Table 2.5. The monthly inflow to the reservoir was estimated based on gauged stream flow downstream from the reservoir (Station X14) and from the North Fork Rose Creek (Station R7). The data from station R7 was scaled based on relative drainage areas to estimate flow into the reservoir.

The data is presented as minimum, average and maximum flow values encountered between the years of 1996 and 2000 (NHC, 2001). Also included in Table 2.5, for comparison, are the estimated average inflows to the reservoir made prior to the construction of the reservoir (Parsons, 1968). The estimated average inflow hydrograph for the FWSD (2001 estimate) is shown in Figure 2.8.

A regional analysis (NHC, 2001), based on seven gauging stations, was performed to estimate the peak inflow events at the FWSD, the results of the analysis is included in Table 2.6.

Table 2.5
Estimated Monthly Inflow to the FWS Dam Reservoir in m^3/s

Month	**Estimated Monthly Flow (m^3/s), Year of Estimate**			
	Average (1968)	**Minimum (2001)**	**Average (2001)**	**Maximum (2001)**
January	0.113	0.164	0.228	0.295
February	0.085	0.145	0.207	0.281
March	0.085	0.149	0.202	0.254
April	0.113	0.212	0.351	0.679
May	1.42	0.967	1.542	2.009
June	2.92	0.957	2.025	2.654
July	1.49	0.676	1.281	2.442
August	0.906	0.556	0.926	1.523
September	0.736	0.548	0.617	0.741
October	0.595	0.385	0.523	0.616
November	0.269	0.347	1.026	1.725
December	0.198	0.287	0.459	0.661

Table 2.6
Estimated Peak Inflow Events at the FWSD

Return Period	Flood (m^3/s)
2 year	5.6
10 year	17
100 year	39
500 year	63
PMF	550

The regional analysis found that spring freshets (snowmelt events) defined the peak flood events at all stations. Therefore the events listed with a specific return period are spring freshet events. The PMF flood event was developed based on probable maximum precipitation (NHC 2001).

The reservoir drawdown planned for this project will not commence until mid August based on permitting considerations. An analysis, details provide in Appendix B, of the potential inflows during the drawdown period (August to November) was performed. The analysis considered the inflows that could be expected in this timeframe, and since the dominant spring freshet data was removed, the floods expected were significantly smaller. The 2-year inflows provided (Table 2.7) can be thought of as the average inflows that could be expected in this period. Note that this data included in Table 2.8 estimates smaller average inflows than Table 2.5. The data provided in Table 2.5 was used in design, since it is based on measured flows from the site, rather than regional analysis. The data provided in Table 2.7 provides an estimate of the peak inflows due to storm events.

Table 2.7
Estimated Floods during Proposed Drawdown Period

Return Period (years)	Estimated peak discharge (m^3/s) during the following number of days									
	1	2	3	4	7	10	30	60	90	122
2	1.14	1.10	1.07	1.04	0.97	0.92	0.77	0.67	0.59	0.50
10	2.50	2.26	2.12	2.03	1.85	1.72	1.37	1.13	0.98	0.85
25	3.28	2.94	2.76	2.59	2.38	2.18	1.63	1.33	1.17	1.01
50	3.93	3.51	3.30	3.10	2.85	2.59	1.82	1.47	1.33	1.13
100	4.66	4.16	3.89	3.64	3.35	3.04	2.05	1.64	1.49	1.26

The regional analysis was extended to the period between December to March to estimate the winter base flow conditions (Appendix B) that will have to be managed throughout the construction period. This analysis determined that no significant rainstorms or early snowmelt events was recorded in this timeframe. Based on this the pumping system would be required to handle the winter baseflow. The baseflow can be expected to be highest at the beginning of the construction period and then follow a recession curve through the remainder of the year, a situation shown for Station R7 (Figure 2.7). In a typical year the peak inflow of 0.2 m^3/s could be expected with an average inflow of 0.11 m^3/s s. The corresponding flows for a 100 year wet year would be 0.56 m^3/s and 0.28 m^3/s.

The potential low flow conditions in the summer were determined based on a regional analysis of the period, June through September (Appendix B). The analysis determined that the 7-day low flow would be 0.47 m^3/s and 0.29 m^3/s, respectively for the 2-year return period (average) and 10-year return period events.

2.5 Site Seismicity

A range of peak ground accelerations (PGA) have been used for the design of various facilities at the Faro Mine. The PGA is used as part of slope stability analysis to estimate the effects of earthquake loading. A summary of the various PGA valves determined for the Faro Mine is included in Table 2.8. When not specifically estimated as part of the assessment, the maximum credible earthquake (MCE) was estimated by doubling the magnitude of the 475-year event.

Table 2.8
Summary of PGA Earthquake Loading

Study	**Return Period Event**				
	475	1,000	10,000	Estimated MCE (twice the 475-year event)	MCE (deterministic)
Klohn Leonoff (1981)	0.07g	0.10g	0.32g	0.14g	0.40g
Dome (1984)	0.063g	0.08g		0.126g	
Golder (1989)	0.08g			0.16g	
Robertson (1996)	0.05g		0.13g	0.10g	
BGC (2001)	0.063g	0.080g		0.126g	
Klohn Crippen (2003)	0.06g		0.16g	0.12g	

The studies sited above note that the source of the PGA estimation was the Pacific Geoscience Centre (PGC) of the Geological Survey of Canada. Until recently, the PGC used historical earthquake data to determine the PGA. As part of the most recent assessment of the earthquake loading, the PGC estimated the PGA based on both historical data and regional active fault data (methodology included in 2003 estimation of PGA).

Only the 1981 earthquake assessment (Klohn Leonoff, 1981) included a deterministic calculation of the PGA based on activity of local faults. Based on the historical movement rates within the Tintina Trench, it was estimated that an earthquake with a magnitude of M6.5 could occur. Using an empirical formula that included the distance from the site to the location of the earthquake, a peak ground acceleration of 0.40g was estimated. Using the same methodology and an estimated M6.0 earthquake within the local faults of the Rose Creek valley, a PGA of 0.36g was estimated for the site.

2.6 Aquatic Environment

2.6.1 Reservoir Water Quality

The FWS Reservoir develops thermal stratification and is likely a typical dymictic lake with a spring and fall turn-over period. Temperature and dissolved oxygen data collected in July and August of 2002 (Figure 2.11) clearly show thermal stratification occurring around 5 - 6 m. Harder (1991) reported temperature data from the Reservoir, which showed only a weak thermal cline in the lake. Surface temperatures as high as 16° C were recorded and the bottom temperatures between 5^{o} and 11^{o} C have been recorded (Figure 2.11). Dissolved oxygen profiles are also included in Figure 2.11 and show that the Reservoir waters are for the most part well oxygenated. The August 2002 deep water dissolved oxygen level was below 6.5 mg/L, the lower limit considered by the CCME as optimal for cold water fish.

Water quality data collected in August 2002, indicates the lake is oligotrophic. Water samples were collected at surface and at 10 m depth on August 9, 2002 for analyses of physical tests, nutrients and total metals. Results are tabulated in Table 2.9. The secchi depth reading (on both August 9 and July 26, 2002) was 5 m indicating a relatively clear waterbody. Nutrient concentrations (nitrogen and phosphorus) also support the conclusion that the Reservoir is oligotrophic.

Total metals are at low levels in the Reservoir, with many below detection and only one metal, lead, exceeding the corresponding CCME guideline for freshwater aquatic life. Lead was 0.0014 mg/L at surface and 0.0012 mg/L at depth; slightly exceeding the 0.001 mg/L guideline. Much of the surrounding geology contains high levels of metals. However, the drainage flowing to the Reservoir is not within a major deposit area. As there is not available data on water quality in the South Fork of Rose Creek upstream of the Reservoir, it is not known if there are metals bound to sediment that drop out in the Reservoir or if the creek generally has the same low concentration of metals as the Reservoir.

2.6.2 Rose Creek Water Quality

Considerable water quality data is available for several locations along Rose Creek. The water flowing into the Reservoir was monitored at two stations (SMC and SRC). Prior to 1990, station SRC was located just upstream of the FWS Reservoir. The second station (SMC) was located on Small Creek, which is a tributary of the South Fork of Rose Creek. Both of these stations were sampled by mine staff in the early 1970's and in 1989 and 1990. However, there is some question about these data as there appear to be some errors in the data set, specifically, the values for the dissolved metal concentrations exceed the values for the total metal concentrations. Generally, water at these sampling stations is alkaline (~ 8 pH units). The average alkalinity was 33.9 mg/L for SRC (which drains granitic rocks) and 215 mg/L for SMC. Total sulphate and zinc concentrations were low at both sampling stations ranging from 4 to 47 mg/L and 0.002 to 0.029 mg/L, respectively.

There are also two sample stations on the Rose Creek Diversion Channel. The first, referred to as station X3, is located at the upstream end of the channel and the second, X10, is located at the downstream end of the diversion channel (Figure 2.12). Station X3 includes all flow from the North and South Forks of Rose Creek except for some partial North Fork flow at times when the North Fork Diversion has been in use. Station X10 includes the influences of two tributary inflows from the south side of the Rose Creek valley and possible lateral seepage from the Second tailings impoundment. Water pH at stations X3 and X10 are similar and have been steady over time, with average values of 7.6 and 7.9, respectively. Sulphate concentrations are also similar and have generally been less than 60 mg/L with several isolated spikes. Total zinc concentrations have generally been less than 0.10 mg/L with occasional higher spikes. The concentration of total zinc at location X10 has generally been slightly greater than location X3 since 1995. The record of total zinc concentrations for station X3 displays seasonally (winter) elevated concentrations up to 1.85 mg/L form 1987 to 1991 that is attributed to the capture in pumping wells of groundwater containing elevated zinc concentrations. The elevated zinc concentrations were not observed at downstream location X10. The practice of augmenting the winter water supply from those pumping wells adjacent to the tailings impoundment was subsequently discontinued.

A summary of the water quality data from these stations can be found in Table 2.9.

2.6.3 Reservoir Sediments and Sediment Quality

A preliminary estimate indicated that between 24,000 and 49,000 m^3 of sediment could be expected within the reservoir. These estimated volumes were made by assuming that the reservoir had an average sediment thickness of 50 mm or 102 mm spread evenly through the reservoir surface area of 486,600 m^2. Measurement of the thickness of sediment was recently performed near the upstream toe of the FWS dam. This location would typically be expected to be one of the locations that would have the thickest deposition of sediment and the finest grained material. The thickness of the sediment was measured (Territorial Diving Technologies, 2002) at eight points near the upstream side of the dam, and ranged between 50 mm and 175 mm (2 and 7 inches), with an average of 102 mm (4.1 inches). Additional sediment thickness testing will be performed to confirm the total sediment thickness to be expected in the reservoir due to the limited aerial extend of the current testing.

Two samples, from near the location of the LLO inlet, were collected during the recent diving inspection, the samples were tested to determine their grain size and metals content. Two photos of the exposed surface of the reservoir base, Photos 11 and 12, show that the soils consist of mostly course grained soil. The results of the grain size testing (Figure 2.10) indicated that the samples were sand with some silt. A copy of the results of the metal testing is included in Table 2.10. Note that based on its angularity, the gravel component is thought to be the result of contamination from the gravel cover overlying the upstream seepage blanket, and not part of the natural sediment. During the summer of 2002 the reservoir was temporary drawn down by approximately one metre and Photos 13 and 14 were taken near the inlet to the FWSD reservoir. These photos indicate that the sediment at this location is finer grained than the samples that were collected at the FWSD. During the planned testing program for sediment thickness additional samples will be collected and their grain size determined.

2.6.4 Stream Substrates

The South Fork of Rose Creek above the reservoir has a gradient ranging from 2% near the reservoir increasing to 5% above the haul road. The dominant substrate in this section is boulder with cobbles and gravels, as shown in Photo 7. The north tributary to the reservoir is somewhat incised and has a dominant substrate of cobble and boulders, Photo 8. The southeast tributary has a somewhat different character with a substrate dominated by fine material interspersed with boulders, Photo 9. The South Fork of Rose Creek, downstream of reservoir (Figure 2.9), is a low gradient, sinuous channel altered by beaver activity, Photo 10. The substrate in this area is primarily fine sediment. The current position of the South Fork of Rose Creek is shown in Figure 2.9 as being slightly different than the original position.

Table 2.9
Water Chemistry of the Faro Minesite Freshwater Supply Reservoir

Parameter	Detection Limit	CCME Guideline	Surface 02/08/09	10 m Depth 02/08/09
Physical Tests				
Secchi depth	field		5 m	
Conductivity	field		80 uS	91 uS
Temperature	field		12.3 C	5.1 C
Dissolved oxygen	field	>9.5 early life stages, >6.5 others	9.7	6.2
Hardness ($CaCO_3$)	0.6		48.9	59.1
pH	0.01	6.5 - 9.0	7.69	7.34
Turbidity	field		1 NTU	1 NTU
Total Suspended Solids	3		<3	<3
Nutrients				
Nitrate Nitrogen	0.005		<0.005	<0.005
Nitrite Nitrogen	0.001	0.06	<0.001	0.002
Total Dissolved Phosphate	0.002		0.002	0.003
Total Phosphate	0.002		0.003	0.002
Total Metals				
Aluminum	0.005	0.1[a]	0.047	0.073
Antimony	0.0005		<0.0005	<0.0005
Arsenic	0.0005	0.005	<0.0005	<0.0005
Barium	0.02		0.02	0.03
Beryllium	0.001		<0.001	<0.001
Boron	0.1		<0.1	<0.1
Cadmium	0.00005	0.0011[a]	<0.00005	<0.00005
Calcium	0.05		15.1	17.8
Chromium	0.001		<0.001	0.001
Cobalt	0.0003		<0.0003	<0.0003
Copper	0.001	0.002[a]	0.001	0.001
Iron	0.03	0.3	0.07	0.11
Lead	0.0005	0.001[a]	**0.0014**	**0.0012**
Lithium	0.005		<0.005	<0.005
Magnesium	0.1		2.7	3.6
Manganese	0.0003		0.0077	0.0118
Mercury	0.00005		<0.00005	<0.00005
Molybdenum	0.001	0.073	<0.001	<0.001
Nickel	0.001	0.025[a]	<0.001	<0.001
Potassium	2		<2	<2
Selenium	0.001	0.001	<0.001	<0.001
Silver	0.00002	0.0001	<0.00002	<0.00002
Sodium	2		<2	<2
Thallium	0.0002	0.0008	<0.0002	<0.0002
Tin	0.0005		<0.0005	<0.0005
Titanium	0.01		<0.01	<0.01
Uranium	0.0002		0.0006	0.0008
Vanadium	0.03		<0.03	<0.03
Zinc	0.005	0.03	<0.005	<0.005

Notes to Table 2.9:
Results are expressed as milligrams per litre except where noted.
< = Less than the detection limit indicated.
a = criteria has been calculated based on hardness and pH of samples
shaded values indicated exceedences of the CCME guidelines for freshwater aquatic life

Table 2.10
Metal Concentrations in Sediment (by ICP method)

		FWSD Sediment Data		Regional Sediments (Reference Sites) (Godin and Davidge, 2002)						Sediment Criteria (CCME 1999)	
		15109	15110	Vangorda Creek Sediments (1989)			Pelly River Sediments (1991)				
Sample:		Sample #1	Sample #2	Site Location 222			Site Location 247			ISQG	PEL
				Max (n=9)	Min (n=9)	Average (n=9)	Max (n =10)	Min (n=10)	Average (n=10)		
Element											
Ag	ppm	<0.2	<0.2	5	<2	<1.2	<2	<2	<2		
Al	%	1.83	1.9	2.9	0.79	1.9	1.3	0.85	1.0		
As	ppm	5	5	**27**	**10**	**17**	**19**	-8	0.8	5.9	17
Ba	ppm	160	210	326	124	270.4	0.106	0.0787	0.08706		
Be	ppm	0.5	0.5	2.0	0.4	1.5	0.4	0.3	0.36		
Bi	ppm	<5	<5	-	-	-	-	-	-		
Ca	%	0.70	0.74	1.0	0.5	0.7	2.6	2.2	2.5		
Cd	ppm	**<1**	**<1**	**3.1**	**0.8**	0.3	**2.7**	**1.0**	**2.1**	0.6	3.5
Co	ppm	9	16	-20	-20	-20	-20	-20	-20		
Cr	ppm	**123**	**127**	**45**	29	**38**	32	26	29	37	90
Cu	ppm	**168**	**135**	**39**	17	30	35	22	27	36	197
Fe	%	3.1	4.45	5.0	17	34	3.0	2.3	2.6		
K	%	0.25	0.29	0.42	0.06	0.24	0.29	0.17	0.23		
Mg	%	0.73	1.22	1.1	0.45	0.69	1.2	1.1	1.1		
Mn	ppm	865	900	1260	334	1000	668	359	486		
Mo	ppm	2	2	<2	<2	<2	6	5	5		
Na	%	0.06	0.04	0.068	0.01	0.035	0.01	0.008	0.0094		
Ni	ppm	33	52	56	30	40	46	32	39		
P	ppm	590	750	1400	610	1000	1600	1200	1380		
Pb	ppm	**66**	**52**	**120**	**38**	**91**	16	9	11	35	91
Sb	ppm	5	5	-	-	-	-	-	-		
Sc	ppm	3	5	-	-	-	-	-	-		
Sn	ppm	<10	<10	<8	<8	<8	<8	<8	<8		
Sr	ppm	60	42	51	23	43	87	80	84		
Ti	%	0.04	0.06	0.11	0.33	0.68	0.027	0.017	0.021		
V	ppm	33	56	50	24	40	73	54	64		
W	ppm	<10	<10	-	-	-	-	-	-		
Y	ppm	7	10	-	-	-	-	-	-		
Zn	ppm	97	107	**662**	59	**312**	**251**	**165**	**205**	123	315
Zr	ppm	7	12	-	-	-	-	-	-		

Notes: Bold indicates concentrations exceed interim sediment quality guidelines (ISQG).
Underlined values indicate concentrations exceed probable effects levels (PEL)

2.6.5 Fish in Reservoir and Streams

Fish species present in the upper Pelly River watershed include chinook and chum salmon, lake trout, lake, broad, humpback and round whitefish, least cisco, inconnu, arctic grayling, northern pike, burbot, longnose sucker and slimy sculpin. Various studies regarding fish presence and habitat use have been conducted at the Anvil Range Mine Complex between 1974 and 2002. During these studies, arctic grayling (*Thymallus arcticus*), burbot (*Lota lota*), chinook salmon (*Oncorohynchus tshawytscha*), slimy sculpin (*Cottus cognatus*), longnose sucker (*Catastomas catostomus*) and round whitefish (*Prosopium cylindraceum)* have been captured in the Anvil watershed for presence, population, distribution and metal analysis purposes. Figure 2.13 and Table 2.11 indicate fish presence by stream reach. Chinook have been noted spawning within lower Anvil Creek during some years surveyed (in relatively low numbers when compared regionally) and juveniles have been noted in the lower 23 km of Anvil Creek in moderate numbers (based on regional comparisons, RGC, 1996) and in the lower end of Rose Creek.

Arctic grayling are the dominant species in Rose Creek and have been captured throughout the Rose Creek mainstem, and in the North and South Forks, including headwater areas of both. Harder (1991) reported that on a regional basis the South Fork of Rose Creek supports the greatest densities of arctic grayling. Once arctic grayling reach maturity (age three to four in the study area), they spawn annually between early May and early June in the Rose Creek drainage (Weagle, 1981; Harder, 1988).

Fish populations have been isolated by several barriers in the watershed (Figure 2.13). The population in the mainstem of Rose Creek can access reaches 1 and 2 of the South Fork, up to the spillway culverts, and a portion of reach 1 of the North Fork, to a culvert above the ponds. A diversion channel on the North Fork flows intermittently when there is flow there is fish can access to the upper section of reach 1 of the North Fork.

Arctic grayling are present in the FWS Reservoir and can access the reaches immediately upstream of the Reservoir of two small tributaries and reach 4 of the South Fork of Rose Creek. Culverts at the mine access road, approximately 4.5 km upstream of the Reservoir are the first of several barriers in the south fork of Rose Creek. There is also a separate population of grayling that exists above at the haul road rock drain and are distributed up to the headwaters and Dixon Lake. Similarly the section of the North Fork above the haul road rock drain barrier supports a population of grayling that also extends up to a series of ponds in the headwaters of this fork.

Several studies of fish in the FWS Reservoir have been conducted between 1981 and 2002 for presence, relative population and metals analysis purposes. Arctic grayling, slimy sculpin and burbot have been documented in the Reservoir and the South Fork of Rose Creek upstream. The arctic grayling caught in the Reservoir have ranged in size from 15 to 31 cm. Catch results of gill net sampling conducted in 2002 were directly compared to gill net sampling conducted in 1981 and 1989, as adequate data was available from these historical studies to derive an estimate of the number of fish caught per unit of effort. The 1981 studies (Weagle, 1981) indicate a catch ranging from 2 to 10 fish or 0.02 to 0.11 arctic grayling per 100 m^2 net area per hour for three 48 hour sets during July, August and October. The August 1989 study (Harder 1991) captured 24 fish or 8.33 arctic grayling per 100 m^2 net area per hour during a day set. During the 2002 study, sinking and floating nets set during the day resulted in a catch of only 3 grayling (0.27 arctic grayling per 100 m^2 net area per hour), while an overnight set resulted in a catch of 61 fish (3.26 arctic grayling per 100 m^2 net area per hour). These results do not necessarily indicate a particular long-term trend in fish populations in the Reservoir. Of the three sampling events, catch per unit effort was lowest in 1981 and highest in 1989.

The general conclusion of studies between 1981 and 2002 indicate that the arctic grayling population in the Reservoir and in reach 4 of the South Fork of Rose Creek area are in good biological condition and sufficient habitat exists to support all life phases of arctic grayling.

2.6.6 Stream Habitat

The Faro Mine is wholly within the Rose Creek watershed and as such there have been several alterations to the creek associated with the Anvil Range Mine Complex. These alterations include diverting Faro Creek around the Faro pit to enter the North Fork, rather than flowing directly into the mainstem of Rose Creek. The Faro pit and associated dumps are located north of the mainstem, just west of the North Fork. In addition to the diversion of Faro Creek, additional alterations to the Rose Creek watershed include:

- Diversion of the mainstream around the tailings impoundment facilities.
- Creation of the pumphouse pond on the mainstream at the upstream end of the Rose Creek diversion.
- Diversion of the lower 500 m of the North Fork.
- Construction of the FWSD converting approximately 1500 m of stream habitat into lake habitat.
- Construction of the haul road over the North Fork, all tributaries to the east and the upper south fork.

Table 2.11
Rose Creek Watershed Fish Habitat by Stream Reach

Watercourse	Reach	Arctic Grayling Habitat Rating				Channel Width	Gradient (%)	Channel Type	Bed Material[a]	Cover	Barriers	Other
		Spawning	Rearing	Summer	Winter							
Rose Cr.	1	high	high	moderate	moderate	25	1.5	riffle (pool, run)	gravel (cobble, fine)			side channels, flows in winter
Rose Cr.	2	high	high	moderate	moderate	13	1	run (riffle, pool)	cobble, gravel (boulder, fines)	20% - pools, cutbanks, boulder, little wood debris		side channels, flows in winter
Rose Cr.	3	moderate	low	moderate	moderate	20	1.5	run (riffle, step-pool sections)	gravel, cobble (boulders in steps)	5% - boulders	step sections a potential juvenile barrier	flows in winter
Rose South Fork	1	moderate	high	high	high	10	1.2	riffle (pool, run) and pond	cobble (boulder, gravel)			pumphouse pond
Rose South Fork	2	low	high	high	high	6	0.5	pool (run)	fines			beaver dams, multiple channels
reservoir	3	low	high	high	high						spillway culverts	
North tributary	1	low	moderate	moderate	low	3.5	10	riffle (step, pool)	boulder (cobble)	boulder	mine road culvert	
Southeast tributary	1	low	moderate	low	low	2	4	glide	fines (boulder at mouth)	vegetation	low flow	
Rose South Fork	4	moderate	moderate	moderate	moderate	7	2.2	riffle (pool, run)	boulder (cobble, gravel)	20% - boulder, pools, vegetation, cutbanks		
Rose South Fork	5	low	low	moderate	low	7	4.3	riffle (pool, run)	boulder		mine road culvert, haul road and steep section	
Rose South Fork	6	moderate	low	low	low	5	5	riffle (run, step, pool)	boulder, cobble	20% - boulder, vegetation, cutbank		
Rose South Fork	7	low	high	moderate	moderate			Dixon Lake	fines			Dixon Lake
Rose North Fork	1	moderate	moderate	moderate	moderate	7	1.4	pool (riffle, run)	boulder (cobble, gravel, fine)	ponds, pools	mine road culvert (on one of the two lower channel options)	two options for water flow below mine road - through boulder channel or series of ponds
Rose North Fork	2	moderate	moderate	moderate	low	10	2	riffle (pool, run)	cobble (gravel, fines)	20% - pools, cutbanks, boulder	haul road	side channels
Rose North Fork	3	moderate	high	moderate	moderate	9	2	run (riffle, pool)	cobble (fines, gravel)			side channels, ponds
Rose North Fork	4	low	low	low	low			ponds	fines			ponds and small lake but very low pH (3) noted

Notes:
a. fines = <2mm, gravel = 2-64mm, cobble = 64-256mm, boulder = 256-4,000mm; sub-dominant substrate in brackets
See Figure 6 for reach location

A classification of fish habitat in the Anvil watershed was conducted in 1989 and 1990 (Harder & Associates 1991) and other reports summarized in RGC (1996). These reports provide details of fish habitat in Anvil Creek. The most notable feature of Anvil Creek is the availability and use of Chinook spawning habitat primarily in the lower reaches of Anvil Creek.

The following details of fish habitat in Rose Creek are based on Harder's reports (Harder, 1988 and 1992) and fieldwork by Gartner Lee in July and August of 2002. A habitat summary of Rose Creek, by stream reach, is outlined in Table 2.11 and the reach breaks and main habitat features are shown on Figure 2.11. Habitat descriptions focus on arctic grayling. Lower Rose Creek (reaches 1 and 2 on Figure 2.11) contain good quality habitat for spawning and rearing arctic grayling and moderate value habitat for adults and sub-adults during both summer and winter. Rose Creek meanders through this section and contains diverse habitat including gravels for spawning as well as deep pools and side channels. Based on Harder (1988), arctic grayling spawn in this reach. Flow is expected here in the winter. Next Creek flows into Rose Creek from the north at the upstream end of reach 2. This creek is narrow with little flow over a relatively steep gradient (>10%) of step-pools resulting in low value for all life stages of arctic grayling.

The diversion channel around the tailings (reach 3) is considered to have low rearing habitat and moderate value for spawning, winter and summer habitat. The upper 2/3 of the diversion is a wide (20 m) channel with predominantly gravel and cobble substrate. The lower 1/3 contains steps of boulders and pools. Velocities in the lower section may make it difficult for juvenile grayling passage. Based on Harder (1988), arctic grayling spawn in this reach. Flow is expected here in the winter and is augmented with release from the Reservoir.

Reach 1 of the South Fork of Rose Creek includes the pumphouse pond and a natural channel that is predominantly riffle over cobble. Due to this combination and augmented winter flows, the habitat value is moderate for spawning and high for rearing, winter and summer habitat. Reach 2 is a meandering section with side channels created by beaver dams. The deep water and augmented slow flow over a substrate of fines result in high value habitat for rearing, winter and summer habitat but low value for spawning. Reach 3 is the Reservoir (Section 2.6.6), which provides high value habitat for rearing, winter and summer habitat but low value for spawning arctic grayling. There are two culverts under an access road that cross the lower end of the FWSD Spillway and form an impassable barrier for fish movement from Rose Creek into the FWS Reservoir.

The unnamed tributary that flows into the Reservoir from the north is a relatively steep gradient from the current Reservoir shore to the mine access road 40 m upstream with cobble substrate (Photo 8). This channel section provides moderate rearing and summer habitat and low spawning and winter habitat. The unnamed tributary that flows to the Reservoir from the southeast is an unconfined low-flow channel through the willow-sedge-spruce valley with a substrate of fines (Photo 9). The channel at the mouth is defined with boulder substrate. This channel offers moderate rearing and low value spawning, winter and summer habitat. Reach 4 of the South Fork, is predominantly riffle channel over boulder and cobble from the Reservoir (Photo 10 and Figure 2.10) with some beaver dams in the upper end. Habitat is considered moderate for all grayling life stages. Reach 5 contains three barriers at the lower end: a culvert under the mine access road, the rock drain under the haul road and a steep gradient section (>20%). Fish cannot move in either direction across this section. This section offers moderate summer and low spawning, rearing and over-wintering habitat. Reach 6 is predominantly riffle over boulders at a 5% slope, with habitat considered moderate for spawning (grayling have been observed spawning at the upper end) and low for all other life stages. Dixon Lake is a shallow basin in reach 7, which offers low spawning habitat but high value habitat for rearing, and moderate value for over-wintering summer and winter habitat.

Within the South Fork, arctic grayling populations both upstream and downstream of the FWSD appear to have sufficient habitat exists to support all life phases. Within the study area, the best spawning habitat for arctic grayling is found at the upper end of the Rose Creek diversion channel and within the South Fork just downstream of the Reservoir. Patches of spawning habitat are also present in the North Fork and in the south fork upstream of the Reservoir.

The best quality rearing summer habitat for fry, juvenile fish and adults is located within the pumphouse pond, the South Fork just downstream of the Reservoir, and possibly within the Reservoir itself. Arctic grayling fry normally spend at least their first summer in stream habitat however, Harder (1988) only captured grayling young-of-the-year in the FWS Reservoir. Summer habitat also exists in pools within the South Fork upstream of the Reservoir and the North Fork.

A large area for over-wintering has been created by the Reservoir. Fish located downstream of the dam likely over-winter in the pumphouse pond or other deep areas of Rose Creek, where water flows continue below the ice.

2.6.7 Reservoir Habitat

Data to update the bathymetric map of the Reservoir was collected on August 6 and 7, 2002. Laberge Environmental Services collected the data. Fifteen transects were established and geo-referenced using a Garmin 12XL GPS unit. A Raytheon Survey Fathometer was used to collect the depth data. The elevation of the water surface at the time of the survey was determined relative to established survey points on the dam crest and was established to be 1095.5 m geodetic. The transect locations and depth data were entered into Auto CAD and a contour map generated and provided in Figure 2.14.

Habitat data in the form of substrate composition in the littoral area of the Reservoir was also collected in July 2002 (Gartner Lee, 2002). The substrate along the shore measured to determine its type and size by using an Aquaview underwater camera and a metre stick for reference. This information was overlaid on the bathymetric map to determine the habitat polygons from 0 to 6 metres depth to characterize fish habitat under current full pool conditions of the Reservoir.

As a general rule the near shore area of a lake to a depth of 6 m is defined as the littoral zone (RIC, 1999) and is the most productive area of the lake as light penetrates to the bottom sediments and aquatic plants can grow. Descriptions of each habitat polygon were developed (Gartner Lee, 2002). In addition, shoreline substrate exposed by wave action and vegetation was noted for the shoreline perimeter polygons, as shown in Photos 13 though 16. In general, the north side of the reservoir is more gradually sloped than the south side and has a greater abundance of aquatic vegetation. At the east side, two shallow bays are present. Aquatic vegetation identified within the Reservoir includes milfoil (*Myriophyllum sibiricum*), bur-reed (*Sparganium angustifolium*), pondweed (*Potamogeton alpinus*) and mare's tail (*Hippuris vulgaris*). The near shore substrate is predominantly fines with boulders, with cobble and gravel visible in some areas. Angular material is located along the south side of the Reservoir at the shale bedrock bluffs. Boulders are visible lining the upstream end (to 4 m depth) of the flooded South Fork Rose Creek channel (Photo 15). Branches from flooded vegetation (willow) are located throughout the Reservoir and are, generally denser at the 6 to 12 m water depth. In addition, stumps from cut trees (white spruce) are present throughout the Reservoir.

Prior to reservoir formation, reach 3 of the south fork of Rose Creek meandered along the valley bottom. There are no pre-impoundment studies of the habitat conditions of this section of Rose Creek. The following information has been inferred from pre-impoundment mapping of the stream channel and the surrounding stream habitat conditions. The total length of the stream channel lost to reservoir formation, from east end of reservoir to end of the freshwater supply channel below the valve house, is approximately 2,400 m. The average channel width in this section was 10 m providing an estimated 24,000 m^2 of stream habitat. This section of stream has an average slope of 1% with sections ranging from 0.2 to 3.2 % (Drawing 4). Given the range of gradients, it is anticipated that the very low gradient stream section provided the meandering type habitat conditions similar to Reach 2 of the south fork of Rose Creek while the steeper gradients would provide more pool-riffle habitat conditions. Prior to construction of the dam, stream based fish populations were able to migrate freely up the south fork of Rose Creek to the culverts under the mine access road.

2.7 Wildlife

Wildlife studies have been completed in the project area most of which focus on big game animals. Fannin sheep reside in the Faro area with a lambing area identified in the headwaters of the south fork of Rose Creek approximately 4.5 km to the south of the FWS reservoir. Moose are also common in the area but no specific habitat issues have been reported in the project area.

Caribou are also in the area with the Pelly drainage area identified as winter range while alpine and sub-alpine zones of the Anvil Range are known summer range. Grizzly and Black bears have been frequently observed around the mine sites. A review of background reports including the RGC (1996) and Anvil Range Mining Corp (2002) do not provide any specific details on small mammals, or birds that are found in the study area.

2.8 Vegetation

The Anvil Range Mining Complex is located within the Yukon Plateau (North) Ecoregion, in the Boreal Cordillera Ecozone (Yukon Conservation Society, 1995). The region lies within the zone of discontinuous, widespread permafrost. Depressional areas consist of peat bogs, fens and local palsas. Lowlands frequently contain hummocks and sedge tussocks. Upland areas commonly include scree slopes and steep south-facing slopes with vegetation dominated by grasses. Treeline occurs at 1350 to 1500 m ASL.

Six vegetation zones were mapped within the study area, based on the field studies and mapping undertaken by Montreal Engineering in 1975. The vegetation zones include flood plain forest, upland forest, bog forest, alpine tundra, subalpine transition, and alluvial plain shrub. The FWS Reservoir is in the alluvial plain shrub zone.

The south fork of Rose Creek and its tributaries are included in the alluvial plain shrub vegetation zone. Shrub birch, shrubby cinquefoil *(Potentilla fruiticosa*), Scouler's willow and other willow species dominate the vegetation communities in the alluvial plain shrub zone. Scattered stands of white spruce and alpine fir also occur. Dwarf shrubs consist of crowberry, Labrador tea, low-bush cranberry, dwarf dogwood, dwarf blueberry (*Vaccinium caespitosum*) and arctic willow. Herbs species include arrow-leafed senecio (*Senecio triangularis*), tall Jacob's ladder (*Polemonium acutiflorum*), sweet coltsfoot (*Petasites hyperboreus*), alpine harebell, wormwood, arctic lupine, clubmoss, common horsetail (*Equisetum arvense*), grass (*Arctagrostis* sp.) and sedges. Feathermoss may form extensive mats in the alluvial plain shrub zone. Lichens, not well represented in this zone, include *Cladonia alpina* and other *Cladonia* species.

2.9 Resource Use

Other than mining, resource use in the Rose Creek watershed appears to be somewhat limited. Arctic grayling are the third most popular sport fish in the Yukon and sport fishing is known to occur in the accessible areas of Rose Creek, in particular the lower end of the South Fork and within the Reservoir (Harder, 1991). While there are no specific records, big game hunting and fur trapping are also resource uses in the area. Caribou, moose and mountain sheep are all known to frequent the Rose Creek watershed.

First Nations traditionally used the Rose and Anvil Creek areas for hunting and trapping and they fished Chinook salmon near the mouth of Anvil Creek. Since the establishment of the Faro Mine, the hunting and trapping activities in the area have been discontinued (Anvil Range, 2002).

3 DESIGN CRITERIA

3.1 Review of Project Alternatives

As noted in Section 1.4, representatives of DFO, DIAND, SRK, BGC, GLL, Geo-Engineering Inc. and Hydroconsult attended a project planning meeting in early December. During the meeting, the attendees identified and evaluated seven alternatives for reducing the risks associated with the FWSD. A summary of this evaluation is provided below. Further details are provided in Appendix A.

The objectives for the FWSD breach were clarified as follows:

- Keep water levels within the range of natural fluctuations.
- Avoid long duration inundation.
- Avoid fish stranding in higher ponds.
- Respect dam stability constraints on dewatering rates.
- Provide fish passage for up to 10-year flood.
- Minimize risk of ice blockages downstream of the pumphouse, specifically by avoiding increases in flow rates after ice formation.
- Control sediment releases.
- Avoid creation of small volume pond that would result in winter fish kill.
- Consider construction, maintenance, future construction, and appurtenant costs.

The following alternatives were then considered and evaluated against the objectives:

- French drain;
- Existing 42-inch culvert, open full;
- Continual pumping;
- Broad crested weir;
- Complete removal of the FWSD;
- Embankment notch; and
- Bedrock notch.

Four of the alternatives ("French drain", "42-inch culvert", "broad-crested weir", and "continual pumping") were consequently rejected. The remaining three, all of which are variations of a dam breach, were judged to be capable of meeting the listed objectives. The main difference between these three is cost. The "complete removal" alternative would be significantly more costly than the either of the "notch" alternatives and was therefore rejected. The "bedrock notch," which was contingent on the removal of the low-level pipe, was judged to be more expensive than the "embankment notch." However, attendees concluded that both alternatives should be considered until the issue of the low-level pipe removal was clarified (see Section 1.2).

DFO subsequently confirmed that it is prepared to consider pipe decommissioning alternatives without excavation of the pipe (see Section 1.3). Based on this clarification, the "embankment notch" was selected as the preferred design alternative.

3.2 Dam Classification

The Canadian Dam Association (CDA) has published dam safety guidelines that are intended for use by dam owners and engineers so that the safety of existing dams can be evaluated in a consistent and adequate manner across Canada. The evaluation of a dam's safety commences with its classification in accordance with the consequences of failure. The dam classification constitutes the basis for analysing its safety and establishes the appropriate design loading events that the structure must safely withstand.

The dam classification system recommended in the CDA guidelines (1999) is shown in Table 3.1.

Table 3.1
CDA Dam Classification in Terms of Consequences of Failure

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

Notes to Table 3.1

a) Incremental to the impacts which would occur under the same natural conditions (flood, earthquake or other event) but without the failure of the dam. The consequence (i.e. loss of life or economic loses) with the higher rating determines which category is assigned to the structure. In the case of tailings dams, consequence categories should be assigned for each stage in the life cycle of the dam.
b) The criteria which define the Consequence Categories should be established between the Owner and the regulatory authorities, consistent with societal expectations. Where regulatory authorities do not exist, or do not provide guidance, the criteria should be set by the owner to be consistent with societal expectations. The criteria may be based on levels of risk which are acceptable or tolerable to society.
c) The owner may with to establish separate corporate financial criteria which reflect their ability to absorb or otherwise manage the direct financial loss to their business and their ability for damages to others.

No previous classification (according to CDA guidelines) of the FWSD has been made. However, relevant information is available from a Qualitative Risk Assessment of the Down Valley Tailings area (BGC, 2001), a Dam Safety Review for the Anvil Range Property (Klohn Crippen, 2002) and a Risk Assessment of the Fresh Water Supply Dam (SRK, 2003).

The objective of the BGC report was to identify potential failure modes for the dams, diversion canals and associated structures within the Down Valley area and to estimate the probability of these failures occurring. This report assigned that the consequences of failure of the FWSD would result in repair, fines and clean-up costs in the range of U$10 million to U$100 million.

The report summarizing the recent dam safety review (Klohn Crippen, 2002) noted that the detailed analysis of the incremental consequences of dam failure was beyond the scope of that review, but the FWSD was likely to be a very high consequence structure.

The risk assessment performed by SRK for the FWSD included detailed dam break analyses for dam failure with (1) the existing FWSD in place and (2) following removal of the FWSD:

- With the current dam in place, the consequences of a probable maximum flood (PMF) event include overtopping of the FWSD, overtopping of the intermediate tailings dam and the large-scale release of tailings solids.
- With the current dam in place, the consequences of a "sunny day" failure (i.e. failure caused by an event other than a flood, such as piping) are the same as those of a PMF event except that the volume of released tailings solids associated with the "sunny day" failure might be smaller due to the smaller volume of water.
- When the FWSD is breached, the consequences of the PMF event are essentially the same as with the FWSD in place, although the volume of released tailings solids might be smaller due to the smaller volume of water, i.e. in the absence of a FWS reservoir.

Failure of the FWSD is not expected to cause any fatalities. However, given the financial and environmental consequences projected by the BGC and SRK reports, a "very high" classification is concluded to be appropriate for the existing FWSD.

Following the breach of the dam, it will no longer retain water and, will be exempt from this classification system. Therefore, the design criteria for the breached dam are open to interpretation based on the general requirements from the DFO directive (Section 1.2) and the objectives identified in the December planning meeting (Section 3.1 and Appendix A). The design criteria used to develop the preliminary design of the breach are discussed in the following sections.

3.3 Design Flood

The design flood affects the design of erosion protection to prevent undercutting of the slopes within the dam breach and to maintain the position of the stream channel within a stationary corridor (referred to as a floodplain in this report).

As noted above, typical guidelines, such as those by CDA (1999), do not provide suitable criteria for selecting the design flood. In the absence of firm guidance, the 1 in 500-year flood was proposed in discussions involving DFO, DIAND, Environment Canada and the Interim Receiver and accepted as the design flood. The 1 in 500-year flood event corresponds to a snowmelt event and has an instantaneous flood peak of 63 m^3/s.

The rationale for this selection was that the Rose Creek diversion canal, which is situated downstream of the FWSD, is sized to pass the 1 in 500-year flood. Selection of the 1 in 500-year flood provides consistency with the current capacity of the Rose Creek diversion canal. It is noteworthy, however, that the Rose Creek diversion canal will likely change as the final closure plan is developed and subsequently implemented. Furthermore, given the infrequency and relatively minor consequences associated with exceedance of the 1 in 500-year flood, it could be argued that a smaller flood, such as the 1 in 200-year flood, would be acceptable.

3.4 Flood Peak Attenuation

The initial directive from DFO stated that the design of the breach should accommodate retrofitting of flood peak attenuation. The subsequent DFO clarification confirmed that attenuation was not a requirement, provided there is clear technical justification for its exclusion (Section 1.3).

Previous investigations of flood peak attenuation have reached the following conclusions:

- The current FWSD provides little attenuation of multi-day floods. For example, NHC (2001) found that the current FWSD only reduced peak flows associated the 500-year flood from 63 m^3/s to 60 m^3/s. The reason is that the 500-year flood is a snowmelt event that would have a duration of about 20 days and would bring over 5.5 million m^3 of water into the reservoir, which greatly exceeds the storage capacity.

- There are configurations that could provide some attenuation of the probable maximum flood predicted by NHC (2001). The attenuation is realized, in part, since that flood is assumed to arise from a single intense precipitation event that lasts only a few hours. The FWSD reservoir is then able to store a significant part of the flood, thereby reducing peak outflows by a significant fraction (roughly 25%, SRK, 2003). Nonetheless, even the attenuated PMF is sufficient to cause a breach of the downstream Intermediate Dam and Cross-Valley Dam (see Drawing 1).

Clearly, the current configuration of the FWSD offers little significant attenuation of peak flood events. Any other configuration of the FWSD, including anything that could be designed as part of the breach would suffer from the same limitations. There is simply not enough capacity in the FWSD reservoir to allow any significant attenuation of extreme snowmelt events. There is a possibility that the breach design could incorporate some attenuation of the PMF, but that attenuation would not be sufficient to protect the downstream structures in their current configuration.

We conclude that there are no design modifications of the dam breach that could provide significant flood peak attenuation.

3.5 Beneficial Use Habitat Requirements

There are three components to the beneficial use habitat requirements:

- The breach design should produce habitat for arctic grayling.
- Fish passage should be possible up to a specified flood level. Beyond this level, the water velocity will exceed the capability of the fish to swim upstream.
- To the extent possible, there should be sufficient water to allow fish passage under normal low river flow conditions.

Stream resident grayling rely on riffles for food production and pool habitat for resting and escape/cover habitat to avoid predators and for over-wintering habitat. Based on these habitat needs the general habitat design will be a riffle and pool configuration.

In regards to the low level flood for fish passage, the 1 in 10-year flood event was proposed and accepted at meetings involving DFO, DIAND and the Interim Receiver. The rationale for this selection was that this level is a commonly selected flood for projects involving channel restoration. It was agreed that another suitable flood could be selected if the 1 in 10-year flood is too restrictive.

The 10-year return period flood has an instantaneous peak flow of 17 m^3/s (Table 2.6). Arctic grayling are judged to be the species of interest in the system. The swimming speed for Arctic grayling is provided in Table 3.2, which has been obtained from DFO land development guidelines for the protection of aquatic habitat (1993). Based on these guidelines and the breach channel length of about 270 m, the burst speed (4.3 m/s) is selected as the upper velocity limit at the riffles and the prolonged speed (2.1 m/s) is selected as the upper velocity limit in the pools during the 10-year flood.

Table 3.2
Sustained, Prolonged and Burst Swimming Speed for Adult Artic Grayling

Sustained Speed (m/s)	Prolonged Speed (m/s)	Burst Speed (m/s)
0 – 0.8	0.8 – 2.1	2.1 – 4.3

Notes to Table 3.2:

Sustained speed – maintained indefinitely

Prolonged speed – maintained for up to 200 minutes

Burst speed – maintained for up to 165 seconds.

In addition to the requirement for passing the upper flood, another part of the development is that fish must be able to pass the newly constructed channel during certain flow conditions. The interpreted normal low flow condition within the South Fork of Rose Creek is 0.46 and 0.12 m^3/s respectively for summer and winter conditions. The design will consider this low flow such that fish habitat is maintained within the breach channel. It should be recognized that this requirement is somewhat opposed to the requirements of erosion protection. The final design, therefore, needs to consider a compromise between the minimum flow and erosion protection requirements.

3.6 Low-level Outlet Pipe

The initial directive from DFO called for the removal of the low-level pipe. A subsequent clarification confirmed that the removal of this pipe was not a requirement, provided clear technical justification for leaving it in place is provided.

It is understood that the basis of the original pipe removal requirement was a concern that that a significant portion of the flow in the creek would be captured by the low-level pipe, thereby reducing the volume of flow available for fish movement during periods of low flow. Another concern is the human safety aspects posed by an open 42-inch pipe.

In regards to the first concern, most of the low-level pipe is situated in bedrock, significantly offset from the original creek channel and the proposed location of the breach (Drawings 2 and 5). There is a possibility that, during high flow periods, some water could seep around the low level pipe. During the excavation of the bedrock trench into which the low-level pipe was installed, the permeability of the bedrock immediately adjacent to the trench would have increased due to the increased fracturing associated with the drilling and blasting. However, the area of this potentially fractured bedrock zone will be much less than the area of the alluvial sediments. As a result, the vast majority of the groundwater flow is expected to remain within the alluvium.

The second concern remains relevant. The design must prevent the public from accessing the interior of the pipe. However, that can clearly be achieved without complete removal, for example by plugging the openings at both ends.

Another factor to be considered is the difficulty of removing the low level pipe. Pneumatic hammers and/or drilling and blasting would likely be required to remove the pipe at a cost similar to the cost of bedrock excavation. In addition, removal of the pipe, given its embedment in concrete, could further increase the permeability of the local bedrock. A reduction of the permeability of the bedrock adjacent to the pipe could be achieved by grouting, but it would be a costly exercise.

We conclude that it would be prudent to leave the low level pipe in place, and plug all openings.

3.7 Stability

The breach of the FWSD will, as noted previously, change the classification of this structure. The stability criteria that will remain important during and following the breach activities include:

- The upstream stability during reservoir drawdown;
- The upstream and downstream stability of the dam at the end of drawdown and in the long term; and,
- The stability of the sides of the breach, particularly where the existing dam height is significant.

3.7.1 Reservoir Drawdown

Generally, a decrease in retained pond levels increases the factor of safety for embankment dams, although not in all cases. Terzaghi and Peck (1967) note that for dams with a sloping core, such as the FWSD, the stability of the upstream slope “may be more critical at an intermediate level, known as partial pool, than with the reservoir full”. Furthermore, if the reservoir is lowered too quickly, there is a risk that the upstream slope can fail due to what is known as rapid drawdown.

The CDA (1999) recommendations for the factors of safety applicable to static stability analyses of embankment dams, including rapid drawdown, are summarised in Table 3.3. These guidelines are applicable prior to the breaching.

Table 3.3
Factors of Safety for Static Assessment (CDA, 1999)

Loading Conditions	Minimum Factor of Safety	Slope
Steady state seepage with maximum pond height	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3[a]	Upstream
End of construction before reservoir filling	1.3	Downstream and upstream

[a] Higher values may be required if drawdown occurs frequently during operations.

Mitchell (1983) provides typical safety factors for impoundment dams, as summarized in Table 3.4. Within the context of Table 3.4, the FWSD would be a high risk dam during drawdown.

Table 3.4
Typical Safety Factors for Impoundment Dams (after Mitchell, 1983)

Case	High Risk Dam	Low Risk Dam
[a] end of construction	1.3	1.3*
[b] normal operation	1.5	1.3
[c] rapid drawdown	1.3*	1.1
[d] earthquake loadings	1.2	1.1
[e] earthquake loadings in combination with [a], [b] or [c]	1.1	1.0

* Applicable to the FWSD

In consideration of Tables 3.3 and 3.4, the appropriate factor of safety against failure of the upstream face of the dam during reservoir lowering is 1.3.

3.7.2 Upstream and Downstream Slopes

Given that the FWSD will no longer be a dam following the proposed breach and the FWSD is part of a mining project, it is reasonable to base the stability criteria for the upstream and downstream slopes on factors of safety applicable to waste rock dumps. A typical example of minimum factors of safety for waste rock dumps is provided in Table 3.5, the source of which is the guidelines published by the BC Mine Waste Rock Pile Research Committee in 1991. The breached FWSD would fit with Case B, as the consequences of a failure of either the upstream or downstream slope are not severe.

Table 3.5
Interim Guidelines for Minimum Design Factor of Safety – Waste Rock Dumps

Stability Condition	Case A – more severe	Case B – less severe
Stability of Dump Surface		
Short term (active)	1.0	1.0
Long term (closure)	1.2	1.1*
Overall stability (deep-seated)		
Short term (active)	1.3 - 1.5	1.1 - 1.3
Long term (closure)	1.5	1.3*
Pseudo-static	1.1 - 1.3	1.0*

* Applicable to the FWSD

In consideration of Table 3.5, the appropriate factors of safety against failure of the upstream and downstream face of the dam in the long term are 1.1 at the dam face (shallow failure), 1.3 for a deep-seated failure and 1.0 under pseudo-static loading conditions, i.e. in response to the design earthquake.

3.7.3 Side Slopes in the Breach

During the excavation of the breach, the appropriate factor of safety for the side slopes corresponds to what is known as the end of construction case. This case is noted in Table 3.4, which indicates the minimum factor of safety is 1.3. This value is higher than the short-term value (1.1) for the stability of the dump face because of safety concerns for men and equipment working at the toe of the slope and because of the potential consequences to the works at the toe of the slope.

In the long term, it is reasonable to base the stability criteria for the side slopes in the breach on minimum factors of safety provided in Table 3.5. Therefore, the appropriate factors of safety against failure of the side slopes in the breach in the long term are 1.1 for at the slope face (shallow failure), 1.3 for a deep-seated failure and 1.0 under pseudo-static loading conditions.

3.8 Design Earthquake

Well built embankments that are sited on firm non-liquefiable foundations and that do not incorporate large bodies of materials which, if saturated, might lose most of their strength during earthquakes, can be designed and evaluated using seismic coefficient methods (pseudo-static analysis).

The pseudo-static analysis requires selection of a peak ground acceleration, which in turns requires selection of the maximum design earthquake (MDE). Selection of the MDE should be based on the consequences of dam failure. The consequences of a failure following the breach are judged to be low.

The MDE selected for the related stability analyses has an estimated return period of 475 years. This corresponds to a 10% probability of exceedance in 50 years and is consistent with the MDE recommended by the National Building Code of Canada for structures built in Canada. The 475-year return period event has been determined during various assessments to be consistently between of 0.05g and 0.08g. The MDE selected for use in the designing the side slopes of the FWSD breach is 0.06g, which is the value estimated in the most recent analysis, by the Pacific Geoscience Centre.

4 DESIGN

4.1 Breach Location and Width

As indicated in Section 3.6 the position of the breach was evaluated as part of the option analysis. Placement of the breach as close as possible to the pre construction location of the channel offered several advantages, such as:

- The channel will be placed within its original alluvium.
- Maximization of the use of the pre-FWSD channel locations.
- No exposure of bedrock.
- Remove the need to construct inlet and outlet structures.

The breach width, referred to as the floodplain, was chosen to be 20 m. This was done in accordance with the guideline that the breach provide minimal flood retention. The 20 m width was selected based on flood routing previously performed as part of the FWSD risk assessment (SRK, 2003). Based on that previous flood routing the 20 m width of breach provides minimal reduction of peak floods valves. Analysis (1-D) of the 500-year return period flood indicated the depth of water would be 0.92 m above the floodplain elevation, and that the average water velocity during such a flood would be 2.7 m^3/s. Detailed 2-D flood routing was not performed as part of the preliminary design, but will be performed as part of the later detailed design.

4.2 Reservoir Drawdown and Schedule

The schedule for initiating the reservoir drawdown will be determined by the approval process. Reservoir drawdown, beyond the new reservoir operating range of between 1090 and 1091 m amsl, will not commence until the initial live fish transfer from the reservoir has been completed. The live fish transfer cannot commence until DFO approval of this project has been obtained. Initial discussions with DFO have indicated that the appropriate approval for the project will not be obtained until 15 August 2003.

The existing configuration of the valves should be limited to passing a flow of 0.82 m^3/s, this being the flow quantity at the onset of damaging cavitation and a portion of the pipes being unsupported (Klohn Crippen, personnel communication, 2003). The final design calculations have not been prepared, but it is expected that following the installation of an orifice plate the safe outflow could be increased to 2.3 m^3/s.

Operating the reservoir in the new operating range through the use of the LLO will require passing up to 2.3 m^3/s through the fresh water channel. A preliminary evaluation of the channel (4 m wide base, 10 m wide at bankfull with a base slope of 1%) indicates that a discharge of 2.3 m^3/s would result in a water depth of less than 0.4 m. Further evaluation of this aspect of will be performed as part of final design.

The dam breaching must be completed prior to the onset of the spring 2004 freshet. Recorded and expected dates for the onset of the spring freshet are summarized in Table 4.1 (from NHC, 2001). The actual timing of spring freshet will, of course, be dependent on the climatic conditions in 2004. For the purposes of planning the dewatering (and construction) the onset of the spring freshet was assumed be April 14. That date defines the last possible construction day.

Table 4.1
Onset of the Spring Freshet

	Start Date for spring freshet
Earliest anticipated:	April 14
Earliest recorded:	April 21
Latest recorded:	May 20
Latest anticipated	June 1

The rate of reservoir lowering will be limited by the need to maintain stability of the upstream face of the dam. The rate of reservoir lowering during the 2001/2002 winter period was limited to about 7 cm per day (BGC 2002) with no adverse effects on the dam performance. (Although additional cracking was observed on the crest of the dam in the spring of 2002 it was not considered significant.) The BGC (2002) memo also discusses that the reservoir was lowered at a rate of 16 cm per day over a time period of nine days with no apparent problems.

At a rate of 7 cm/day, it would take 128 days to lower the reservoir from an elevation of 1091 m amsl (upper end of the operation zone), to the 1082 m amsl elevation of the inlet to the LLO. Assuming that reservoir drawdown commences on 15 August 2003, this would enable construction to start about 15 January 2003. That would leave approximately 89 days to complete the breach and channel rehabilitation.

If initiation of the drawdown were to be further delayed, faster drawdown rates could be considered. A detailed analysis of the time required to complete the construction and estimated flow rates versus applied head on the LLO will be completed as part of detailed design. The required drawdown rate will be determined and, if necessary, the potential for inducing failure of the upstream face of the dam will be further assessed.

Both the stored reservoir water and the water flowing into the reservoir will need to be removed in order to complete the drawdown. The volume of water stored between elevation 1091 and 1082 m amsl is approximately 1.9 million m^3 (Figure 2.6). Based on a simple linear assumption of the storage curve, in order to reduce the stored water level by 7 cm/day, approximately 14,850 cubic meters of stored water will need to be removed each day (~0.18 m^3/s). Taking into account the average monthly inflow estimates from Table 2.5, the discharge rates required to complete the drawdown will range between values of 0.4 to 1.2 m^3/s (not allowing for flood events). Those outflow rates can be achieved using the LLO, once modifications to the valve are completed (see Sections 2.2.3 and 5.1.1). The effects of storm inflows on the drawdown have not been evaluated in detail, but should be managed relatively easily considering the outflow capacity of the LLO following modification. Evaluation of the capacity versus elevation of the LLO and the potential effects of storm events will be managed during drawdown.

Once the water level in the reservoir reaches a depth of 1082 m, the LLO can no longer be used to either lower the water further or maintain the reservoir level. As shown in Table 2.5, the average inflows in the period of January through April (prior to the freshet) will be between 0.12 and 0.2 m^3/s. The volume of water remaining in the reservoir at elevation of 1082 is about 50,000 m^3. To remove the residual water and to control inflows, it was assumed that a Caterpillar diesel-powered pump with a capacity of 5,000 USgpm (0.315 m^3/s) will be installed. One of these pumps working full time could remove the remaining stored water in a maximum of 5 days. One pump working part time will be able to maintain the water level upstream of the cofferdam.

4.3 Breach Side Slopes

The design criteria for stability analyses used to determine the side slopes of the breach were outlined in Section 3.7. Preliminary stability analyses were performed to assess the stability of slide slopes of 2.5H:1V and 3H:1V (Appendix C). The analyses considered the critical section, through the exposed core of the dam, and used conservative strength properties for the soils within that section. The analysis was performed for the following conditions:

- Static analysis - immediately post construction, with high piezometric conditions within the dam.
- Static analysis – long-term conditions, with the expected long-term piezometric conditions.
- Pseudo-static analysis of earthquake loading and long-term piezometric conditions.

The results are summarized in Table 4.2. Based on this preliminary analysis the side slopes of the overall breach were chosen as 3H:1V. The analysis shows that the 2.5H:1V slope does note meet the required criteria for the immediate post construction and for the full value of the most extreme earthquake considered possible at this site (historically derived earthquakes). Further analysis will be performed as part of the final design. The final design slope stability analysis will include a sensitivity analysis of the soil properties.

Table 4.2
Results of Preliminary Stability Analysis

Condition Analyzed	Calculated Factor of Safety		Required Factor of Safety
	Side Slope of Breach		
	2.5H:1V	3H:1V	
Static, immediately post-construction	1.12	1.29	1.3
Static, long term	1.40	1.59	1.3
Pseudo-static (g = 0.06), long term	1.17	1.30	1.0
Pseudo-static (g = 0.16), long term	0.90	0.98	1.0

Two benches, having a width of 2 m, are planned at elevations 1087 and 1092 m amsl. The benches were included to limit erosion, by breaking up the constant slope, and limiting the velocity run-off due to direct rainfall. Erosion potential on the breach side slopes, in the area of the exposed core will be checked during detailed design (effects of revegetation will be considered).

The breach location and general arrangement are shown on Drawing 5.

4.4 Channel Width and Depth

The design objective of relevance to the selection of channel dimensions is the requirement that the channel provide for fish passage under most flow conditions. The initial design channel width was selected to mimic the natural channel. The original investigation for the dam construction (Ripley, Klohn & Leonoff, 1968) reported that the channel at the location of the dam was 6.1 m wide and 1.5 m deep under bankfull flow conditions.

The overall grade of the channel through the dam construction footprint is 1%. The side slopes of the channel were selected to be 2H:1V. Using the overall grade of the expected channel base combined with the dominant discharge of 5.6 m^3/s (2-year return period) general hydraulic geometry guidelines were used to estimate the channel characteristics (Kellerhals and Church, 1989). Based on the general guidelines, a 7.7 m wide channel would be expected to be stable for the selected dominate flood. Given this general guideline the bankfull width of the channel was selected to be 8 m.

Using the 2-year return period flood as defining the bankfull flow, a 1-D flood analysis was used to determine the depth of flow. This analysis indicated that the channel should have a depth of 0.56 m. The general plan for the channel base is to construct the channel in a series of riffle-pool elements. Under such an arrangement, the crest of the riffles define the hydraulic control for the channel, and act as a series of broad-crested weirs. Based on the results of the analysis, the channel depth was selected as 0.7 m (defined as the difference between the crest of the riffles and the floodplain).

Given that the 2-year flood was used to determine the bankfull conditions and the 10-year flood, the 10-year flood overtops the channel and flows in the channel and floodplain. Preliminary 1-D analysis of the water velocities under the 10-year flood indicated that the water in the channel would have a velocity of 2.5 m^3/s and the water flowing in the floodplain would have a velocity of 0.9 m^3/s. The depth of water on the floodplain would be about 0.2 m.

The details of the channel layout are included on Drawings 6 through 8.

4.5 Channel Elements

Preliminary channel design was based on empirical relationships (Appendix B) developed largely in western Canada. The empirical relationships provided the following guidelines for the design of the channel:

1) the slope of the channel (approx. 1%) is consistent with a riffle-pool morphology;
2) the pool-riffle-bar unit has a characteristic spacing of five to seven channel widths;
3) for a stable channel, pools comprise about 50% to 75% of the channel length and have a wide range of depths;
4) the average width of a stable channel with a bankfull discharge of 5.6 m^3/s should fall in the range of 8 to 12 m, based on hydraulic geometry relationships for gravel rivers in Canada; and,
5) the soils investigation report referred to in the previous section suggests the actual channel width was about 6 m.

The channel details are included on Drawings 6 through 8.

The preliminary design is expected to produce pool sections that have a residual total depth of 1.5 m of water. This is anticipated to provide sufficient depth that over-winter ice conditions will not freeze the pool to the bottom. The average winter base flow of 0.12 m^3/s is expected to provide over-winter refuge with sufficient oxygen for fish survival.

4.6 Erosion Protection

Two different zones of erosion protection are planned: protection of the edge of the floodplain (the residual dam structure) and protection within the main channel and floodplain surface. The two zones have different erosion protection requirements.

The erosion protection within the floodplain is designed to be somewhat mobile. The velocities present within the main channel will exceed the resistive capacity of the riprap and erosion of the channel will occur. This will allow the channel to migrate naturally. The size of the riprap used was selected based on the average velocity (for a 500-year flood), of 2.8 m/s, assuming that the channel section was not present.

The erosion protection at the edge of the flood plain has been designed to be stable during the 500-year return period flood. The average velocity during the 500-year flood period event was increased by 4/3 to determine the design velocity. The riprap was sized based on the design velocity, constructed having a 2H:1V side slope and a factor of safety of one. The width of the erosion protection is twice the D_{50} of the riprap. The erosion protection extends to a depth of 2 m below the surface of the flood plain. Detailed scour analysis will be performed as part of final design to confirm the appropriate depth for this erosion protection element.

Due to the size difference between the Type I riprap and the base soil, geotextile separation layer will be required.

4.7 Sediment Control Elements

Two components of sediment control are required in the design: control of sediments generated during construction; and determination of the potential post-construction effects of sediments stored within the reservoir.

Control of the sediments generated during construction will be minimized by performing all construction work in the dry. Following installation of an upstream cofferdam, pumping will be initiated to direct water around the construction area and into the downstream channel. Section 5 provides more details on the construction plan.

As discussed in Section 2.6.3, the preliminary estimate of total volume of sediment within the reservoir is between 24,300 and 49,800 m^3. Testing of two samples collected from the base of the reservoir near the location of the inlet of the LLO indicated that approximately 18% to 35% of the sediment are fine-grained portion (silt and clay sizes). The fine-grained portion of the sediment would likely be of concern with respect to the development of turbid water, the coarser grained sediment will either not be mobilized or will be captured in pools within the channel. The mass of the sediment in the reservoir is estimated to be between 28,800 to 58,800 t (using a density of 1.2 t/m^3 and not considering the gravel component as sediment), with between 9,790 and 29,400 t of the sediment being fine-grained.

To assess the likely maximum potential effects of sediment mobilization, it was assumed that all of the fine-grained sediments would be mobilized in the first post-construction freshet. The total flow associated with a 2-year freshet was estimated at 5,500,000 m^3 (developed using an instantaneous inflow of 5.6 m^3/s and the spring inflow hydrograph from NHC, 2001). These assumptions result in estimated suspended sediment concentrations ranging from 1,800 to 5,300 mg/L. The actual suspended sediment load will depend on the amount of sediment exposed to erosion, which in turn will depend on the timing of storm events, magnitude of flood events and the vegetation within the reservoir. Sediment loads greater than 25 mg/L are to be expected during the first freshet (natural sediment loads have not been assessed as part of this preliminary design).

These calculations suggest that post-construction sediment mobilization from the reservoir area will be similar to that experienced in typical freshet flows over undisturbed areas. Additional calculations could be done at this time, but were not considered to be justifiable given the very limited information about sediment volumes and composition. A field program is planned for February 2003 that will include additional sediment sampling to confirm both the estimated volume of sediment in the reservoir and the grain-size characteristics.

The best available technology for sediment control is vegetation of surfaces as soon as possible after they become exposed. Pre-construction and post-construction vegetation plans for the base of the reservoir are described further in Section 5.

4.8 Requirements for Final Design

Final design is expected to be initiated following the submission and review of the preliminary design. The final design is not expected to change substantially the general design intent, but may change some of the specific details. Some of the issues to be further evaluated in final design include:

- Finalization of the side slope of the breach;
- Finalization of the breach width;
- Finalization of the riffle and pool elements;
- Finalization of the erosion protection at the edges of the floodplain; and,
- Finalization of the rip-rap sizing within the base of the breach.

Additional field data will be collected in support of the final design. The planned field program includes:

- Sediment sampling to determine depth of sediment at a variety of locations through the reservoir;
- Collection of sediment samples;
- Physical and chemical testing of the sediment samples;
- A detailed survey of the base of the reservoir base to obtain information near the inlet of the LLO, the cofferdam location and the pre-construction channel location upstream from the cofferdam;
- A detailed survey of the existing downstream channel; and
- A detailed survey of the existing upstream channels, South Fork of Rose Creek, Southeast Tributary and North Tributary.

During the summer of 2003 a second field program will be undertaken to collect additional information that cannot be collected in winter (given snow cover and ice conditions on the reservoir). The information collected in the second program will not affect the final design. The anticipated summer 2003 field program includes:

- Accurately estimating the number of fish that are living in the FWSD reservoir and the accessible reaches from the reservoir;
- Accurately estimating the number of fish living between the FWSD and the weir section of the Rose Creek diversion; and
- Determining the carrying capacity of the post FWSD breach and determine the amount of live transfer of fish required.

5 CONSTRUCTION PLANS

This section summarizes the construction plans as they are currently envisioned. Drawing 9 illustrates the current footprint of the dam and outlines the general location of the proposed construction activities. The location of the existing riprap stockpile, the spoil zone and the re-vegetation scheme are also shown.

5.1 Preparatory Activities

In order to prepare for construction of the breach, the following activities will take place:

1. Repair the valves and outlet of the LLO.
2. Lower the reservoir.
3. Live capture and transfer of fish.
4. Preliminary revegetation.
5. Prepare construction and spoil areas.
6. Stockpile required materials.

5.1.1 Repair low level outlet valves

The piping within the LLO valve house will be repaired in order to pass an outflow volume of 2.3 m^3/s. Although details of the modifications are still under discussion, it is thought that the problems with the current configuration can be solved by installing an orifice plate and upgrading the support for the LLO pipe where it passes through the valve house.

Installation of an orifice plate and support for the pipes will require:

- Dewatering of the outlet chamber.
- Cutting of the projecting 24-inch diameter outlet pipe.
- Welding of a slip flange onto the pipe.
- Fabricating a suitably dimensioned and machined orifice plate and bolting it to the slip flange on the pipe.
- Welding a thrust collar onto the 24-inch diameter pipe immediately against the upstream side of the downstream wall of the valve chamber.
- Shortening the length of the pipe projecting into the chamber.
- Removal of the weir plate located at the end of the fresh water channel.

This task is likely to be performed in March 2003.

5.1.2 Lower the Reservoir

As noted in Section 2.2.2, the water in the reservoir level will be maintained at 1090 to 1091 m amsl throughout 2003. This will be accomplished through the use of the upgraded LLO, potentially in combination with siphons and pumps.

The reservoir will be lowered to the inlet elevation of the LLO, 1082 m amsl, by passing water through the LLO. Approval from DFO is not expected before 15 August 2003. Prior to this date, the reservoir will be operated as outlined in Section 2.2.2. Following approval DFO approval, live fish transfer will be initiated. Completion of the live fish transfer will require approximately 15 days following receipt of the DFO approval. Reservoir lowering will commence following this fish transfer, starting approximately 1 September 2003.

Approval of this project by the Water Board is not anticipated prior to 1 November 2003. Prior to this approval, the construction activities required to complete the breach excavation cannot commence. The limits of these two distinct portions of the permitting process define a period of approximately 60 days in which to lower the reservoir.

Once the piping upgrades discussed in Section 5.1.1 are completed, i.e. in March 2003, the capacity of the LLO will exceed the average inflows to the reservoir, allowing for rapid dewatering of the reservoir, if required. It is expected that the rate of reservoir dewatering will be limited by acceptable rates to ensure that rapid drawdown failures on the upstream face are not initiated, as discussed in Section 4.3. Using a constant drawdown rate of 7 cm/day (the typical drawdown rate used in winter dewatering in 2001/2 and 2002/3), it would take 123 days to draw the reservoir to the inlet elevation, or until 29 December 2003. The reservoir drawdown rate will be re-evaluated upon finalization of the design, schedule and receipt of the permits for the project.

5.1.3 Live Capture and Transfer of Fish from Reservoir

The live fish capture and transfer will be performed prior to the initiation of the reservoir drawdown program. The capture will be initiated with the reservoir at an elevation of 1090 m amsl, the planned lower limit of the 2003 operating elevation. The transfer will be initiated following receipt of DFO authorization for the project and will be completed by approximately 1 September 2003. The fish will be transferred to an area that has sufficient capacity to accept the number transferred fish.

The detailed fish transfer plan will be developed occur during the summer of 2003, following a field program to determine the current population within the reservoir and the accessible reaches of South Fork of Rose Creek upstream of the reservoir. The fish population that can be supported in assessable reaches (post breaching) will be determined during the summer field program. Although the detailed transfer plan has not been developed, it is anticipated that the fish will be transferred to Rose Creek (downstream of the tailings impoundment) or Anvil Creek. The transfer will be accomplished using trucks with the appropriate tanks.

The live fish transfer component of the project is time-sensitive. Successful completion of the live transfer is not thought possible beyond about October 15 to November 1, 2003. The temperature of the water will affect the fish activities and will limit the success of the capture program. Following the development of ice on the surface of the reservoir, the fish transfer program will be severely limited. The date of successful fish transfer program is weather dependent and, therefore, extremely difficult to predict.

5.1.4 Install Sediment Control Measures

Prior to commencing any construction activities at the dam, adequate sediment and erosion control measures will be implemented. The sediment control plan consists of two main elements: revegetation of the reservoir and isolation of the construction site.

Given the planned reservoir operating elevation of between 1090 to 1091 m amsl for 2003, this upper portion of the exposed base of the reservoir will be seeded/planted during the spring/summer of 2003 (Phase 1 revegetation on Drawing 9). During dewatering of the reservoir, additional seeding/planting will be performed to revegetate the exposed base as quickly as possible, thus limiting the sediment mobilization the following spring. The specific timing of revegetation work in the fall of 2003 will be dependent on the weather conditions and the dewatering schedule.

The reservoir is planned to be operated at an elevation of approximately 1090 m amsl throughout 2003. The new operating level, reduced from the typical 1096.3 m will expose a portion of the reservoir (see Drawing 9). Planting of that portion of the reservoir will commence following stabilization of the reservoir elevation near 1090. The reservoir revegetation performed during Phase 1 will allow optimization of the larger Phase 2 revegetation program to be completed in the summer of 2004.

The seed mixture used for the re-vegetation is likely to consist of a native mix of grasses and contain a large grass seed variety (i.e. some form of rye) that will germinate rapidly and provide immediate erosion control. This planting will also provide root structure to reduce erosion during spring snowmelt and rains. Seed mixture will be spread using a "quad." While some disturbance of surface materials is acceptable, if the soil is too wet for quad access, aerial application such as fixed wing aircraft, will be used. Hand seeding is also a possibility, particularly on steep slopes along the south shore and the west end along the north shore. The exposed riparian zone will include re-vegetation with grass and willow plantings. The willows will be planted in a 10 m wide zone (extending 10 m from the edge of the bank) adjacent to the South Fork of Rose Creek and a 5 m wide zone surrounding the smaller tributaries, as shown in Drawing 9.

Seed will be spread on any areas exposed between Phase 1 seeding and dewatering before permanent snow cover, so that seed is in place under the snow and can germinate early in the spring immediately following snow melt.

Willow staking will be carried out 10 m on either side of the South Fork of Rose Creek channel and 5 m for the smaller tributaries to promote the re-establishment of the riparian vegetation. Using the estimated alignment of the original channel (Drawing 4), this will involve an area approximately 2400 m long and 20 m wide, for a total area of 48,000 m^2. Willow stakes should be placed on 1 m centres; therefore, 48,000 stakes will be required. Stakes should be collected while the plants are still dormant (i.e. late winter or early spring before leaves start to form). Prior to reservoir drawdown, only the exposed area above elevation 1090 m amsl will be staked. The remainder will be undertaken in the spring of 2004.

Throughout the remainder of the construction period, the reservoir will be frozen and snow covered with little likelihood of generating sediment. Further details concerning revegetation following construction are provided in Section 5.2.9.

5.1.5 Prepare Construction and Spoil Areas

The stockpile of riprap and the spoil (contractor's work) areas are shown on Drawing 9. Both spoil areas are relatively flat with sporadic grass cover. The spoil area downstream of the FWSD has previously been used for construction activities at the dam and was cleared previously. The spoil area upstream of the dam will be within the former reservoir base. No clearing will be required as part of the preparation of the spoil areas. Construction of perimeter containment dikes will be performed as part of the placement of spoil.

A portion of the spoil material (coarse-grained material) will be placed to fill the existing fresh water channel. The expected position of the breach section and rehabilitated section of the channel are shown on Drawing 8. The portions outside of the completed channel on this drawing will be filled, shaped to match the existing and planned contours, and revegetated.

It is expected that a portion of the spoil area downstream of the dam (alternatively, a portion of the spillway could be used) will be used as the contractor's work area. This area will require only minor preparation for the construction offices, storage facilities and fuelling area. The fuelling area is to be constructed in accordance with the appropriate regulations for fuelling near water sources. No fuel tanks are proposed for the construction area. The existing fuel tanks at the mine shall be used or the contractor's fuel supply tanks will be stored near the mine office. Transportation of fuel from the mine will be performed using trucks as it is needed. Spill contingency plans will be prepared as part of the final design and will be the contractor's responsibility.

5.1.6 Stockpile Required Materials

The fill materials required to complete the construction of the breach and erosion control elements include:

- Type I riprap: material is required for protection of the edges of the floodplain to limit erosion of the exposed breach side slopes. This material will also be used to form the riffle sections. The median size of this material is 480 mm. This material is currently stockpiled at the FWSD (Drawing 9).
- Type II riprap: material is required to line the base of the channel (outside of the riffle) and to line the surface of the floodplain. The median size of this material is 90 mm.
- Bedding Gravel: material required to line the upstream portion of the riffle and required for placement within the pool section of the channel. The material will consist of gravel sized particles with some sand and less than 5% silt and clay.
- Geotextile: required to act as a separation layer between Type I riprap and the adjacent soil (dam core, dam shell and in situ foundation soil).

5.2 Construction Activities

Excavation of the breach through the dam will require the disposal of approximately 75,000 m^3 of material. The construction activities will result in the completion of the breach and a section of rehabilitated section of channel to replace the existing fresh water channel (Figure 5.1). Figure 5.2 shows schematically the general arrangement of the FWSD area and creeks following completion of the project.

The construction related activities involved in breaching the dam will include the following:

1. Construct cofferdam and perform water management.
2. Excavate the breach.
3. Excavate the channel.
4. Perform channel rehabilitation
5. Place riprap for erosion protection.
6. Construct the riffles.
7. Remove the cofferdam.
8. Abandon the low level outlet.
9. Clean up and revegetate the remainder of the site.

5.2.1 Construct cofferdam and perform water management

A description of the cofferdam used as part of the FWSD construction is included in Section 2.2.2. As shown in Drawing 4, a 12 to 13 m wide portion of the cofferdam was breached in 1968 and some general lowering of the crest was performed. One section of the LLO (42-inch pipe) was left in place under the cofferdam and the ends of the pipe were reportedly sealed.

When (or before) the reservoir level has been lowered to 1082 m asml, the cofferdam will be reconstructed with a crest elevation of 1083.5 m amsl. This will require the rebuilding of only that portion of the dam that was breached in 1968, completed upgrading to the upper portion of the dam and reconstruction that portion of the cofferdam surface that was lowered. An estimated 1,600 m^3 of fine-grained fill placement will be required to rebuild the existing cofferdam. Some additional work maybe required to ensure the water does not escape through (or under) the cofferdam and breach the structure given its unknown condition (especially around the 42 inch diameter pipe) following submergence since 1968.

The average inflows during this period are estimated to be approximately 0.12 m^3/s. This water will be pumped around the construction site to maintain a water level of 1082 m behind the cofferdam. Preliminary details were provided in Section 5.1.4. A 1.5 m freeboard will be maintained to allow storage of approximately 100,000 m^3 of water. Although the time frame of the construction is such that no storm events are expected, the storage volume in the freeboard provides a contingency for failure of the pumping system. The 1.5 m of freeboard allows the storage of the average inflow for about 9 days, should the pumps fail.

The construction site will be isolated from the upstream and downstream portions of the creek. Water flowing in the creek will be directed around the active construction area. Details of this water management process will be finalized during detailed construction planning but will depend on the sequencing of the construction activities and detailed planning performed by the contractor. A preliminary dewatering plan is outlined as follows.

During reservoir drawdown, water will be transferred via the LLO to the downstream channel. Following dewatering to the inlet of the LLO, the reservoir water elevation will be at 1082 m amsl and the cofferdam reconstruction work will commence across the base of the reservoir. Water levels will be maintained behind the cofferdam through the use of a pump(s). The water which collects upstream of the cofferdam will be pumped into a flume or open pipe located to the south of the south abutment of the dam. The flume/pipe will discharge into the natural channel, downstream of the construction zone. The volume of water expected from pumping is much less than that expected to be discharged during dewatering. It is expected that ice levels on the creek formed during drawdown will allow the discharged water to effectively pass through the creek section. The pump(s) used will have appropriate screens on the inlets.

Pumping of water retained behind the cofferdam will continue until the cofferdam removal and the tie-in of the newly constructed channel with the existing channel have been completed.

Cofferdam removal and completion of the channel construction within the footprint of the cofferdam will require that the water be conveyed around the former location of the cofferdam. In order to achieve this, a temporary sump will be excavated in the base of the natural channel upstream of the cofferdam. The water will be pumped from the sump while the cofferdam is excavated and the upstream section of the channel is completed. The sump will be subsequently backfilled with material removed during its excavation and the water allowed to flow through the breach.

5.2.2 Excavation of the Breach

The breach excavation consists of the bulk excavation that will extend the breach section to the floodplain level, Drawing 5. The excavation will start following the construction of the cofferdam. The expected construction start date is 5 January 2004. The breach excavation will require the removal of about 72,400 m^3 and local haul to spoil. The excavation will extend through the dam section and the two downstream berms, the downstream seepage collection trench and the upstream seepage blanket (Drawing 5). Drawing 5 shows the location and general arrangement of the breach.

Drawings 2 and 3 show the material from which the dam is constructed. The different soils used during the construction of the dam will be separated as they area excavated (to the extent practical) and kept separate in the spoil areas. The materials to be excavated are expected to include:

- Existing phyllite riprap.
- Shell material.
- Core material.
- Random material.
- In situ foundation soils.

The phyllite riprap that exists on the face of the dam was determined to be potentially acid generating (PAG) in a preliminary study (BGC, 2002). Therefore, riprap excavated as part of the breach construction and all the riprap located to the south of the breach section will be separated and hauled to the Faro Pit (or other approved disposal area) for disposal.

Excavation of the breach will extend to the approximate level of the original South Fork of Rose Creek, returning the creek to its original alluvium. The excavation extending to the floodplain level is expected to be above the post-construction groundwater table, although some water can be expected in the working area as the fill soils of the dam drain. Further evaluation of the proposed floodplain base will be performed in final design and the need for water management evaluated.

The possibility of beginning the breach excavation prior to installation of the cofferdam will be evaluated as part of detailed design.

5.2.3 Excavation of the Channel

The base of the excavated channel will link the thalweg of the natural channel upstream of the cofferdam, with the rehabilitated channel downstream. The excavated channel section will contain the majority of the flows following completion of the project. The excavation of the channel (Drawing 7) is expected to extend below the base of the pre-1968 channel location, and therefore below the natural water table. Assessment of the effects of the groundwater table on the excavation and, later, fill placement will be performed as part of final design. The need for temporary dewatering will also be evaluated at that time.

Excavation of the channel will require removal of 2,100 m^3 below the proposed base of the floodplain. Over-excavation of the floodplain surface (for placement of erosion protection) will require 1,800 m^3 of excavation.

5.2.4 Channel Rehabilitation

Channel rehabilitation will be performed to replace the straight section of the fresh water channel with a channel and floodplain more closely matching the creek prior to the construction of the FWSD. The rehabilitated channel will match the elevation of the base of channel in the breach section and the base of the natural section of creek, requiring a length of 275 m to be rehabilitated. Drawing 7 and Figure 5.1 indicate the approximate location of the channel rehabilitation portion of the project, in contrast to the preparation of the breach section. This portion of the work will be performed on a field-fit basis, attempting to place the rehabilitated channel back within its pre-1968 channel location to the extent possible.

In rehabilitating this section of the channel, the floodplain and channel will use the pre-existing topography to define the floodplain and channel, to the extent possible. Sections of the channel rehabilitation will require the complete rebuilding of the channel and floodplain. Those portions of the construction will include adding sinuosity to the channel (matching with to the natural channel location) and potentially the addition of riffle-pool or other habitat elements.

5.2.5 Place riprap for erosion protection

Type I riprap will be placed at the edges of the floodplain to protect the residual dam structure from erosion (Drawing 7). A geotextile separation layer will be required between the Type II riprap and the foundation (base) soils, given the variation between the grain size of the riprap and base soils. Installation of this erosion protection element will likely extend below the groundwater table, and therefore may become extremely difficult to install. The possibility of placing the riprap in a launching apron will be evaluated as part of final design and used as necessary. Installation of this erosion protection element will require placement of approximately 1,300 m^3 of material.

Type II riprap, 0.3 m thick, will be placed in the base of the channel and on the surface of the floodplain. This will require the placement of 1,800 m^3 of material.

5.2.6 Construct riffles

Riffles will be constructed from Type I riprap, to the dimension shown on Drawing 8. Installation of the riffles may require excavation below the water table. The effects of this will be evaluated in final design and water management instituted, as needed.

5.2.7 Remove cofferdam

Following the completion of the breach and channel rehabilitation sections, the water stored by the cofferdam will be pumped out. As outlined in section 5.2.1, a sump will be required to complete the pumping. Excavation of roughly 3,500 m^3 of material will be required to remove the cofferdam and to the final portion of the floodplain and channel sections.

Following removal of the cofferdam, erosion protection for the channel and floodplain portion of this section will be installed. The sump will be backfilled with the material removed during its creation and the pumping program will be discontinued, with water flowing through the completed section of the channel. The flow will be evaluated as it reaches the downstream portion of the construction zone to ensure that the flow remains within the channel, flowing under the ice.

5.2.8 Abandon the low-level outlet and valve house

Abandonment of the inlet of the LLO will consist of removal of the existing trash rack and covering (plugging) the inlet with a steel plate. The remaining structure will be buried and the ground surface shaped to match pre-construction slopes.

Abandonment of the valve house will consist of removal of the structure, removal of a portion of the piping that was previously located within the valve house, plugging the end of the LLO and burial of the exposed footprint. The resulting ground surface will be shaped to ensure drainage and will be revegetated in the spring/summer of 2004.

5.2.9 Clean up

After all construction activities are completed the construction site will be cleaned up. During the spring of 2004 the remainder of the site will be revegetated, as required.

5.3 Post-Construction Activities

5.3.1 Complete channel upgrades

The completed breach section of the channel will be evaluated following the spring 2004 freshet and through the summer to ensure that the elements are performing as expected. Due to the winter construction period, some frozen material may be incorporated in the construction zone. The effects of any settlement due to the winter construction, build up of sediment in the channel and the performance of the riffle elements in the spring flood will be evaluated and necessary repairs made during fall 2004. The habitat elements included in the rehabilitated channel section will be evaluated and repaired, if necessary.

5.3.2 Phase 2 revegetation of reservoir area

The spring/summer 2004 revegetation will continue the planting in both the general reservoir and the riparian zone (Section 5.1.4). The area of the reservoir to be revegetated is shown as Phase 2 revegetation (Drawing 9), but will also include the filled portions of the fresh water channel, the spoil areas and the breach section of the FWSD.

5.3.3 Perform environmental monitoring

Environmental monitoring is described in detail in Section 8.

5.4 Construction QA/QC

A quality assurance / quality control program will be developed as part of the final design.

5.5 Construction Schedule

The preliminary schedule for the entire project is presented in Figure 1.1. This schedule includes the main tasks, including the design and permitting required to complete the project. Figure 1.1 further indicates that the main elements of the construction will occur between 24 November 2003 and 31 March 2004. A more detailed construction schedule will be developed as part of the detailed design.

6 ENVIRONMENTAL ASSESSMENT

This section to be completed on or before 7 February 2003.

7 FISH HABITAT MITIGATION AND COMPENSATION PLAN

This section to be completed on or before 7 February 2003.

8 ENVIRONMENTAL MONITORING PLAN

This section to be completed on or before 7 February 2003.

9 COST ESTIMATE

9.1 Overview

The DFO directive, discussed in Section 1.2, specifically requests that cost estimates be provided for the breach and any associated work. A cost estimate has been prepared on the basis of the preliminary design and preliminary construction plan.

The estimate attempts to cover all activities associated with the breach project. It does not take into account the fact that some costs, such as those for monitoring, could be reduced by coordination with other projects or that, depending on valve modifications, pumping may not be required.

9.2 Basic Assumptions

Completing the preliminary cost estimate required several assumptions regarding further steps in the project design and permitting, the construction process, the home base for the contractor and the nature of how each component of the construction (and engineering) tasks will be completed. The following list summarizes the critical assumptions underlying the preliminary cost estimate:

- Further field data collection and detailed design will be required;
- Production of tender documents for an open tender of major works will be required;
- Review and permitting will proceed as shown in the project schedule (Figure 1.1);
- The low level pipe will be able to pass most of the flows necessary to maintain the reservoir at the desired level and during the spring 2003 freshet and dewater the reservoir thereafter, but a provisional cost of $200,000 for additional pumping has been included;
- All major equipment and workers would be mobilized from Yukon locations;
- Re-vegetation activities within the reservoir will be performed manually;
- Riprap stock-piled below the FWSD will be suitable for the breach side slope erosion protection;
- The fleet of major equipment for the project will include two tracked excavators, one rubber tired loader, one crawler tractor and between 3 and 6 trucks.

9.3 Quantities

The following quantities were used as a basis for the preliminary cost estimate:

- Quantities for excavation uses the in-place volume, generally referred to as bank cubic meters (bank m^3).

- Quantities for haulage and wasting used the bank cubic meters based on the neat volumes taken from the drawings.
- Lump sum values and unit rates were based on estimated production and haulage rates for appropriately sized equipment. Some of these estimated unit rates were vetted against known costs for other projects.

Cost estimating for civil construction can be rather complicated due to the required compatibility of the equipment sizing (*e.g.* certain loader for certain haul trucks) and the equipment access and mobility constraints. Those complications have not been assessed in the preliminary design. In addition, local contractors will have particular equipment available. Actual construction costs will reflect the capabilities or limitations of that equipment and the bidding strategy of the contractor. Therefore, significant variations in bid prices are possible.

9.4 Cost Estimate and Contingencies

The preliminary estimate of costs to complete the FWSD breaching is summarized in Table 9.1. A detailed breakdown of the capital construction cost estimate is included in Appendix D.

As noted in Table 9.1 a contingency of 30% was applied to the subtotal cost estimate. That level of contingency is appropriate to cover the substantial estimating error associated with preliminary design. Individual contingencies were not included on the task or phase estimates.

Table 9.1
Estimated Total Cost to Complete FWSD Breach

Item	Project Phase			Total
	Preliminary Design to Feb 3, 2003	Final Design to Mar 30, 2003	Tendering & Construction 2003/2004	
Define Final Configuration Options	$35,000			**$35,000**
Field Investigation	$6,000	$25,000	$50,000	**$81,000**
Fisheries Studies	$10,000	$30,000	$50,000	**$90,000**
Engineering Design, incl. valve assessment	$100,000	$95,000	$25,000	**$220,000**
Pumping (provisional)			$200,000	**$200,000**
Tendering			$50,000	**$50,000**
Capital Construction Cost			$1,557,271	**$1,557,271**
Construction Indirects (20% of capital)			$311,454	**$311,454**
Monitoring (as defined in Section 8)			$50,000	**$50,000**
Permitting	$30,500	$30,500	$100,000	**$161,000**
Subtotal	$181,500	$180,500	$2,393,725	**$2,755,725**
Contingency (30%)				$826,718
Total				**$3,582,443**

10 CLOSURE

In response to a DFO directive dated November 15, 2002, an engineering team, under the direction of SRK Consulting, has completed this report describing the preliminary design of a breach of the FWSD at the Faro Mine, Yukon Territory. Following the submission of this report, and with input from DFO, DIAND, Environment Canada and others, the engineering team will commence detailed design of the FWSD breach. The current project schedule calls for the completion of the detailed design by April 1, 2003.

Figure 1.1 provides a list of the tasks and the current schedule required to complete the breach in time for the spring freshet of 2004. As noted above, it is a challenging schedule, particularly in relation to the permitting. In order to expedite the completion of individual tasks in a timely manner, on-going project communications will be required. In particular, a planning meeting is tentatively scheduled for early February to clarify design goals and task responsibilities. In addition, the regular bi-weekly teleconference meetings will continue.

This report "**Preliminary Breach Design, Fresh Water Supply Dam, Faro Mine**", has been prepared by:

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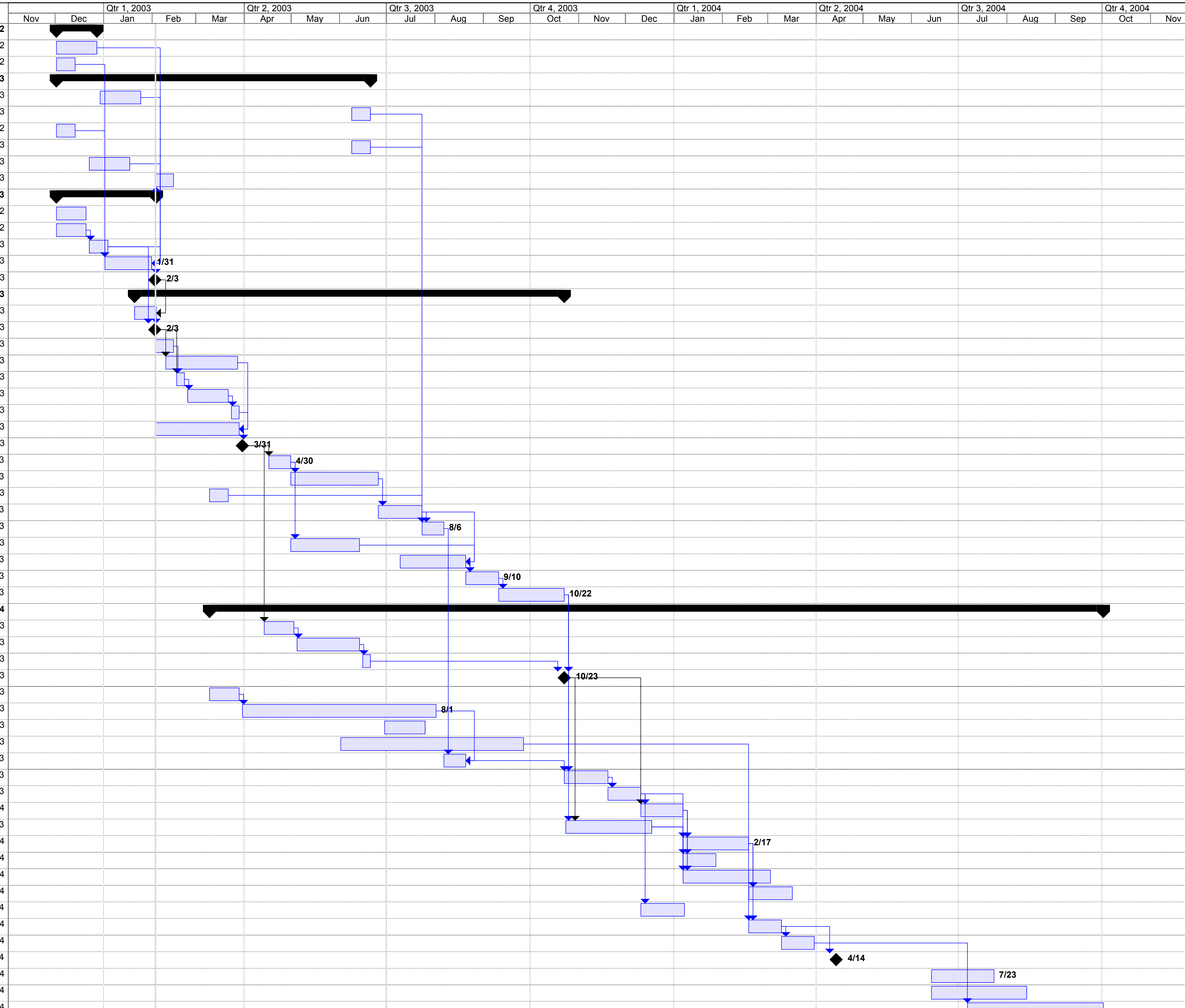
Figures

FWSD BREACH PROJECT SCHEDULE

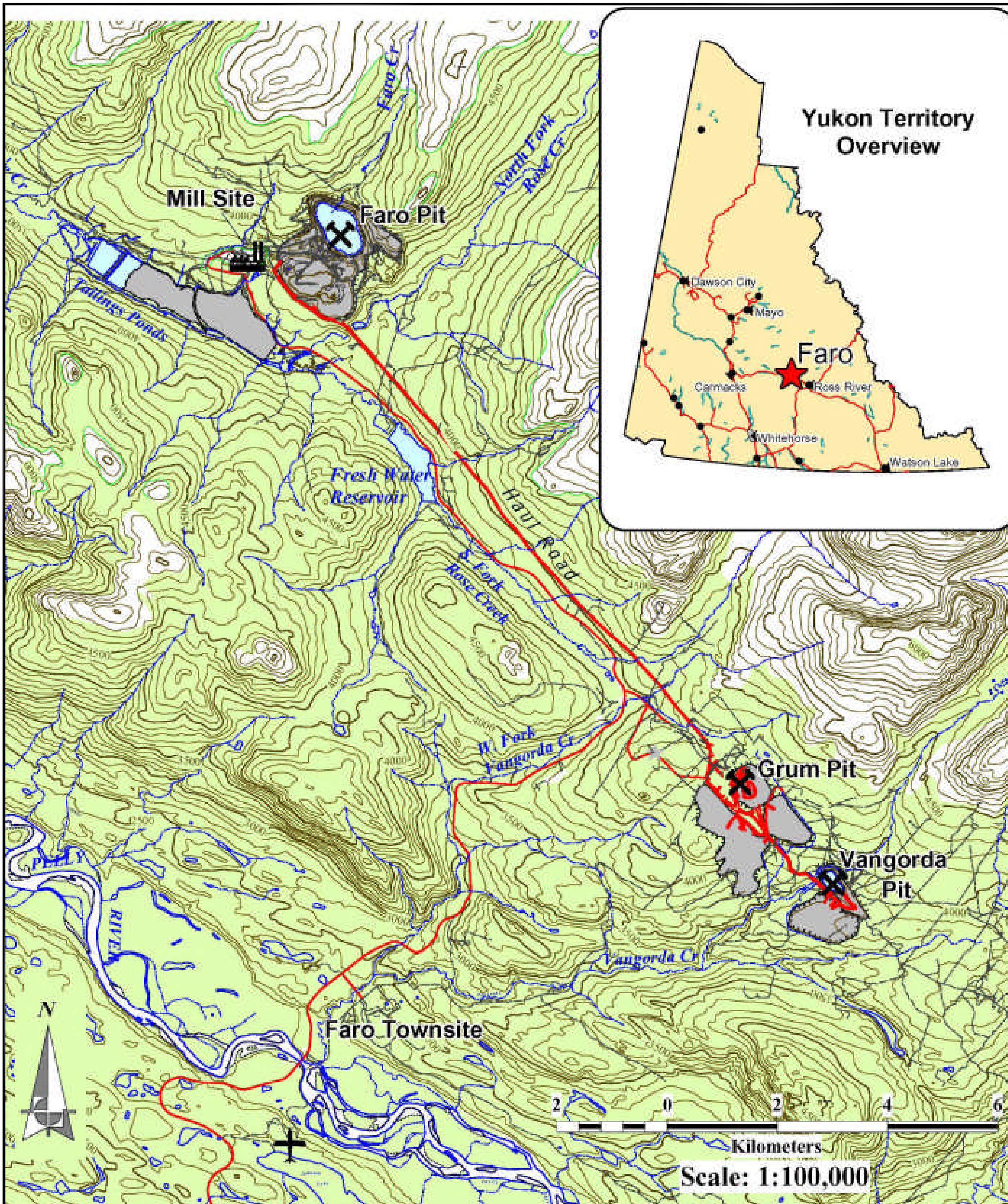
ID	Task Name	Duration	Start	Finish
1	**Issue Clarification**	**20 days**	**Mon 12/2/02**	**Fri 12/27/02**
2	Determine attenuation needs	20 days	Mon 12/2/02	Fri 12/27/02
3	Determine low level pipe status	10 days	Mon 12/2/02	Fri 12/13/02
4	**Data Collection**	**145 days**	**Mon 12/2/02**	**Fri 6/20/03**
5	Determine cofferdam status	20 days	Mon 12/30/02	Fri 1/24/03
6	Evaluate Revegetation options	10 days	Mon 6/9/03	Fri 6/20/03
7	Evaluate Pipe valves	10 days	Mon 12/2/02	Fri 12/13/02
8	Survey Fish population	10 days	Mon 6/9/03	Fri 6/20/03
9	Delinate sediment type	20 days	Mon 12/23/02	Fri 1/17/03
10	February Field Program	10 days	Mon 2/3/03	Fri 2/14/03
11	**Design**	**46 days?**	**Mon 12/2/02**	**Mon 2/3/03**
12	Assess Riprap Need	15 days	Mon 12/2/02	Fri 12/20/02
13	Select Design Flood	15 days	Mon 12/2/02	Fri 12/20/02
14	Complete Fisheries Habitat Mitigation Plan	10 days	Mon 12/23/02	Fri 1/3/03
15	Prepare Preliminary design	22 days	Thu 1/2/03	Fri 1/31/03
16	Submit Preliminary Design	1 day?	Mon 2/3/03	Mon 2/3/03
17	**Permitting**	**197 days**	**Tue 1/21/03**	**Wed 10/22/03**
18	Prepare Project Description	10 days	Tue 1/21/03	Mon 2/3/03
19	Submit Plans (project description)	5 days	Mon 2/3/03	Fri 2/7/03
20	Application for DFO Authorization	10 days	Mon 2/3/03	Fri 2/14/03
21	Continue EA Public Consultation	34 days	Mon 2/10/03	Thu 3/27/03
22	Terms of Reference	5 days	Mon 2/17/03	Fri 2/21/03
23	Develop EIS	20 days	Mon 2/24/03	Fri 3/21/03
24	Review EIS	5 days	Mon 3/24/03	Fri 3/28/03
25	Prepare final design	40 days	Mon 2/3/03	Fri 3/28/03
26	Complete final design	1 day	Mon 3/31/03	Mon 3/31/03
27	Prepare Comprehensive Study Report	10 days	Thu 4/17/03	Wed 4/30/03
28	Complete CEAA Public review	40 days	Thu 5/1/03	Wed 6/25/03
29	NWPA Public notification period	10 days	Mon 3/10/03	Fri 3/21/03
30	Receive ministerial decision	20 days	Thu 6/26/03	Wed 7/23/03
31	Complete DFO authorization	10 days	Thu 7/24/03	Wed 8/6/03
32	Submit Water Licence Application	32 days	Thu 5/1/03	Fri 6/13/03
33	Water Licence Public Review Period	30 days	Thu 7/10/03	Wed 8/20/03
34	Complete Water Licence Hearing	15 days	Thu 8/21/03	Wed 9/10/03
35	Issue Water Licence	30 days	Thu 9/11/03	Wed 10/22/03
36	**Construction**	**410 days?**	**Mon 3/10/03**	**Fri 10/1/04**
37	Prepare and issue tender documents	15 days	Mon 4/14/03	Fri 5/2/03
38	Tender period	30 days	Mon 5/5/03	Fri 6/13/03
39	Review and evaluate bids	5 days	Mon 6/16/03	Fri 6/20/03
40	Issue contract	1 day?	Thu 10/23/03	Thu 10/23/03
41	Install orifice plate	15 days	Mon 3/10/03	Fri 3/28/03
42	Maintain reservoir levels	90 days	Mon 3/31/03	Fri 8/1/03
43	Phase 1 revegetation	20 days	Mon 6/30/03	Fri 7/25/03
44	Produce Riprap	85 days	Mon 6/2/03	Fri 9/26/03
45	Salvage Fish	10 days	Thu 8/7/03	Wed 8/20/03
46	Complete reservoir dewatering	20 days	Thu 10/23/03	Wed 11/19/03
47	Initiate short term pumping	15 days	Thu 11/20/03	Wed 12/10/03
48	Install Cofferdam	19 days	Thu 12/11/03	Tue 1/6/04
49	Initiate Notch Excavation	39 days	Fri 10/24/03	Wed 12/17/03
50	Complete Notch Excavation	30 days	Wed 1/7/04	Tue 2/17/04
51	Decommission low level pipe	15 days	Wed 1/7/04	Tue 1/27/04
52	Grade upstream of notch	40 days	Wed 1/7/04	Tue 3/2/04
53	Complete Downstream works	20 days	Wed 2/18/04	Tue 3/16/04
54	Construct Upstream Mitigation	20 days	Thu 12/11/03	Wed 1/7/04
55	Place riprap	15 days	Wed 2/18/04	Tue 3/9/04
56	Remove cofferdam	15 days	Wed 3/10/04	Tue 3/30/04
57	Freshet 2004 (early start date)	1 day?	Wed 4/14/04	Wed 4/14/04
58	Evaluate upstream/downstream works	30 days	Mon 6/14/04	Fri 7/23/04
59	Complete Revegetation	45 days	Mon 6/14/04	Fri 8/13/04
60	Post Construction Monitoring	63 days	Wed 7/7/04	Fri 10/1/04



Project: FWSD Breach Schedule.feb_3
Date: Mon 2/3/03

Task		Progress		Summary		External Tasks		Deadline	
Split		Milestone		Project Summary		External Milestone			

Figure 1.1



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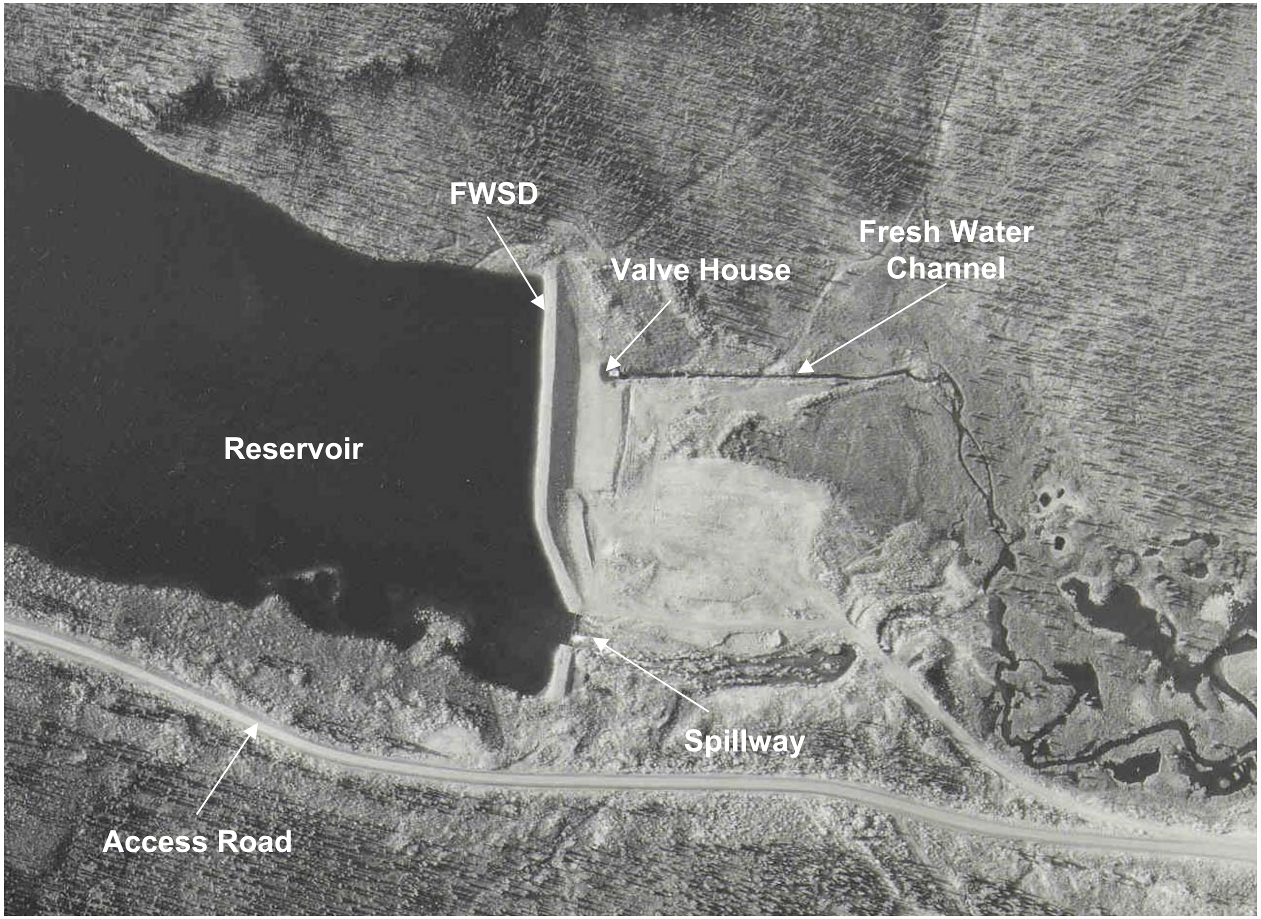
FWSD BREACH FARO MINE, YUKON TERRITORY

LOCATION MAP

PROJECT NO.	DATE	APPROVED	FIGURE
1CD003.20	FEB. 2003	GWF	2.1

Dwg Ref: Figure 1.1.dwg



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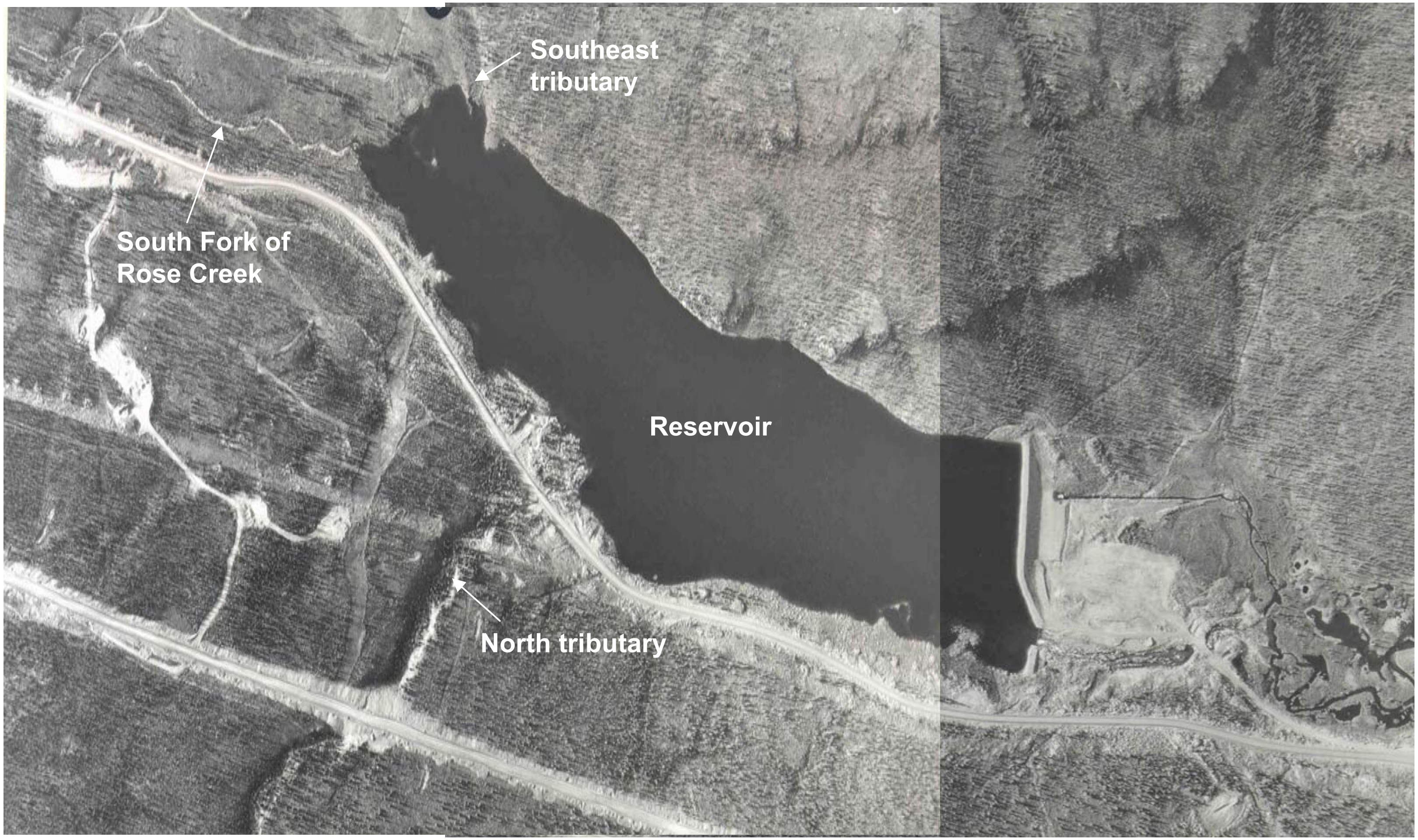
FWSD Breach
Faro Mine, Yukon Territory

FWSD Features

PROJECT:	DATE:	APPROVED:	FIGURE::
1CD003.20	February 2003	GWF	**2.2**



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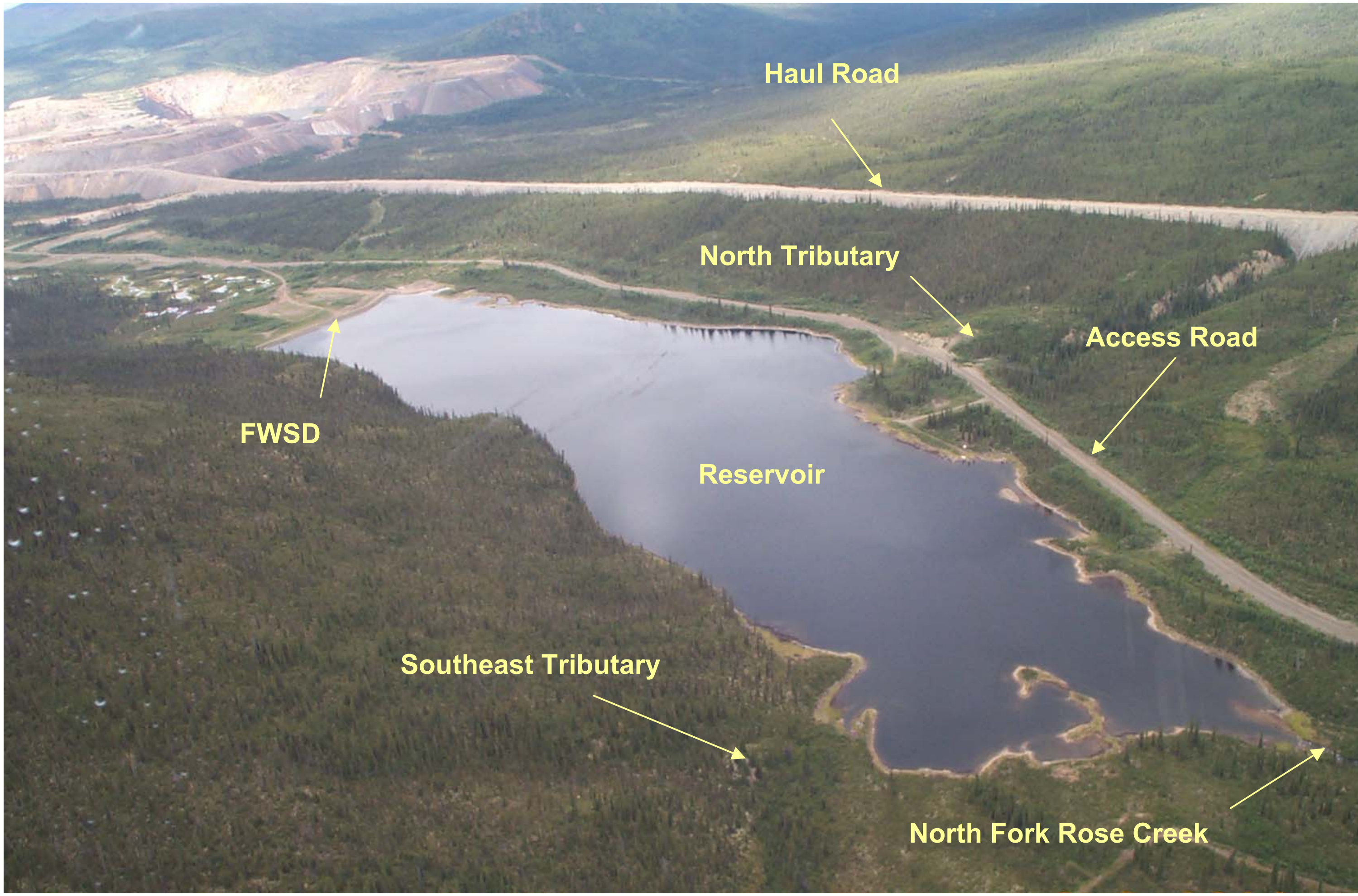
FWSD Breach
Faro Mine, Yukon Territory

Areal Photo of Reservoir

PROJECT:	DATE:	APPROVED:	FIGURE::
1CD003.20	February 2003	GWF	2.3



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FWSD Breach
Faro Mine, Yukon Territory

FWSD Reservoir

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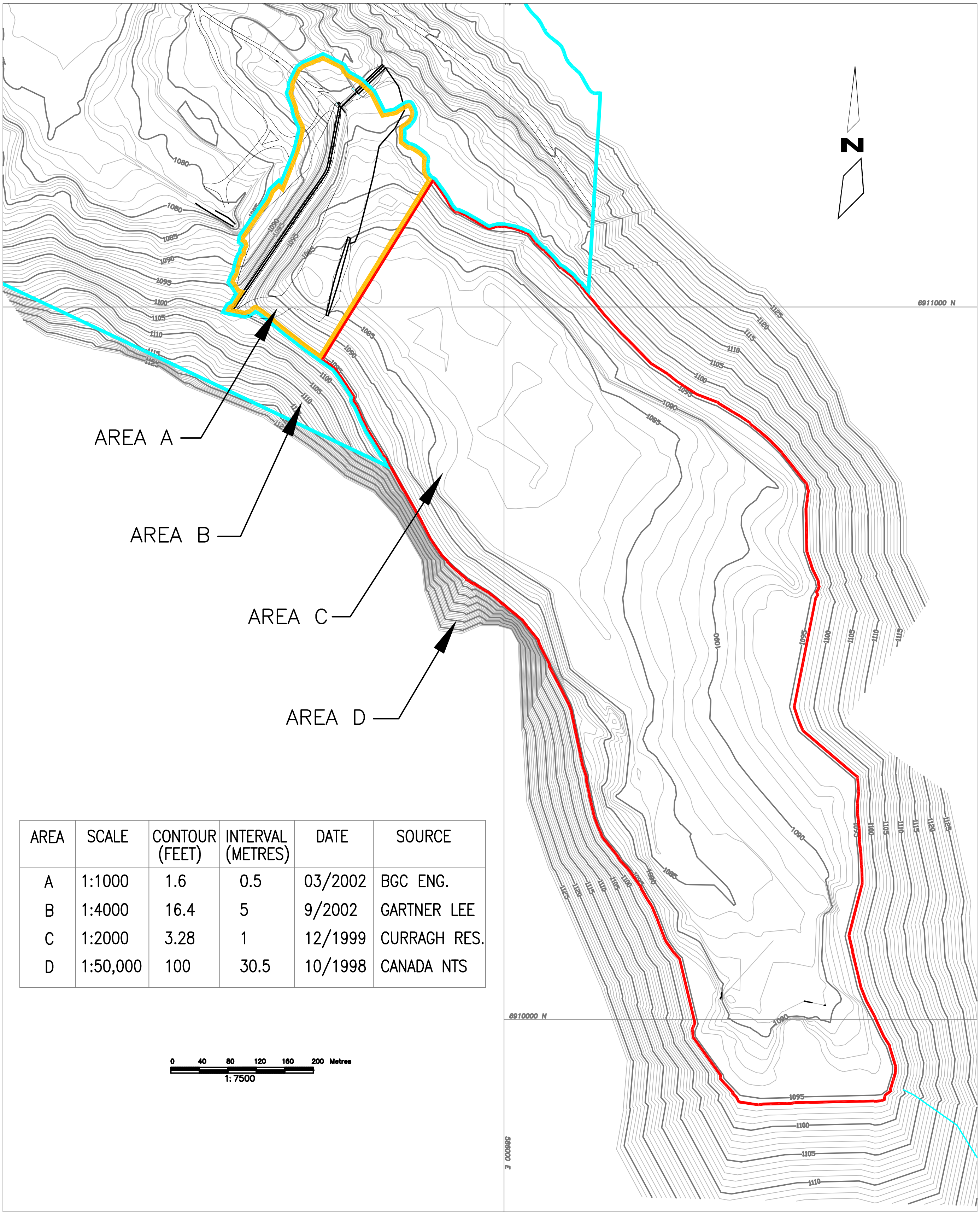
PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.20	February 2003	GWF	2.4



AREA	SCALE	CONTOUR (FEET)	INTERVAL (METRES)	DATE	SOURCE
A	1:1000	1.6	0.5	03/2002	BGC ENG.
B	1:4000	16.4	5	9/2002	GARTNER LEE
C	1:2000	3.28	1	12/1999	CURRAGH RES.
D	1:50,000	100	30.5	10/1998	CANADA NTS

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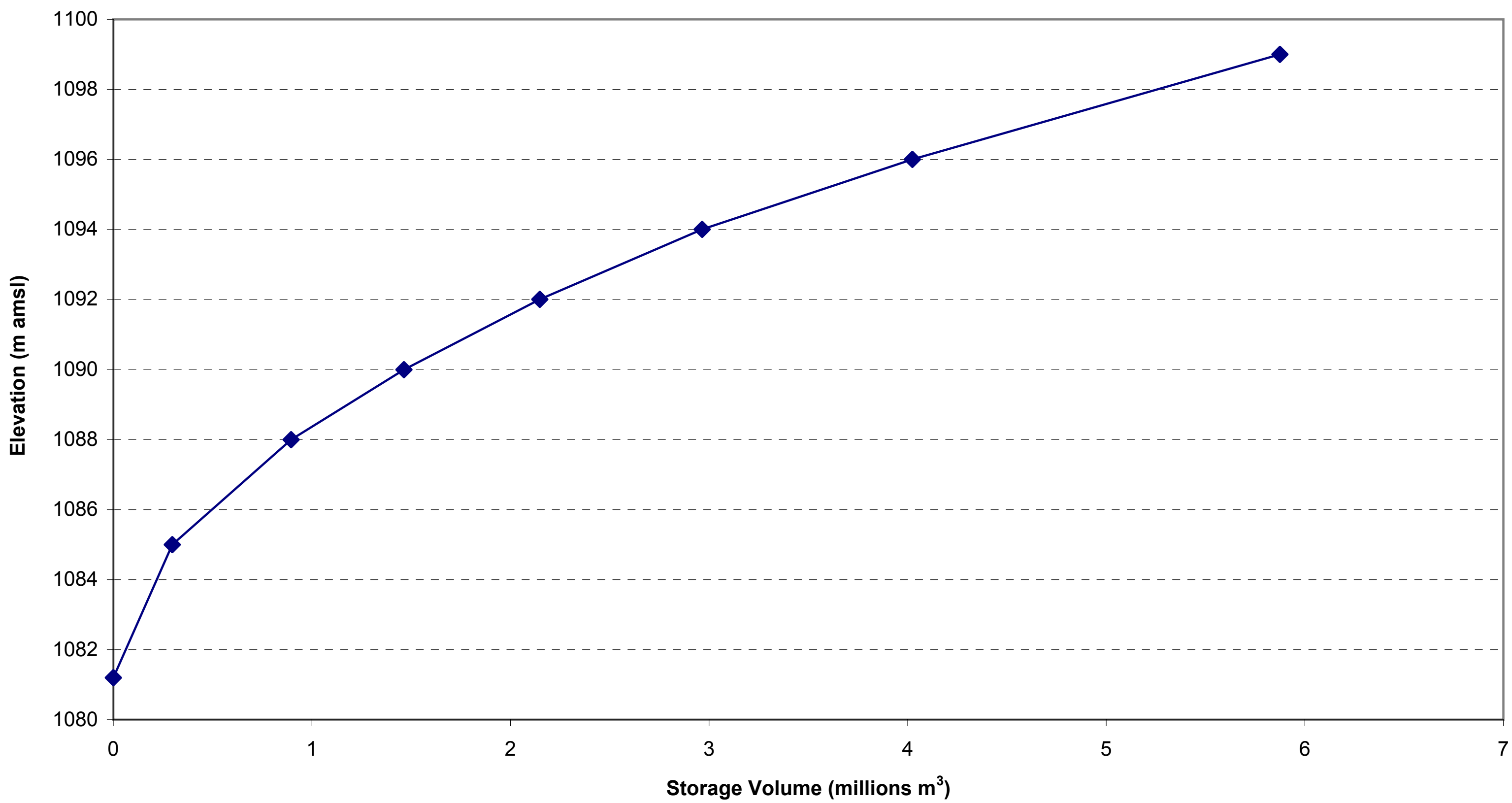
FWSD BREACH FARO MINE, YUKON TERRITORY

AVAILABLE SURVEY DATA
FOR PROJECT AREA

PROJECT NO.	DATE	APPROVED	FIGURE
1CD003.20	FEB. 2003	GWF	2.5

Dwg Ref: Figure 1.2.dwg

FILE REF: Figure 2.6.xls



Based on Bathymetric Survey performed August 6-7, 2002

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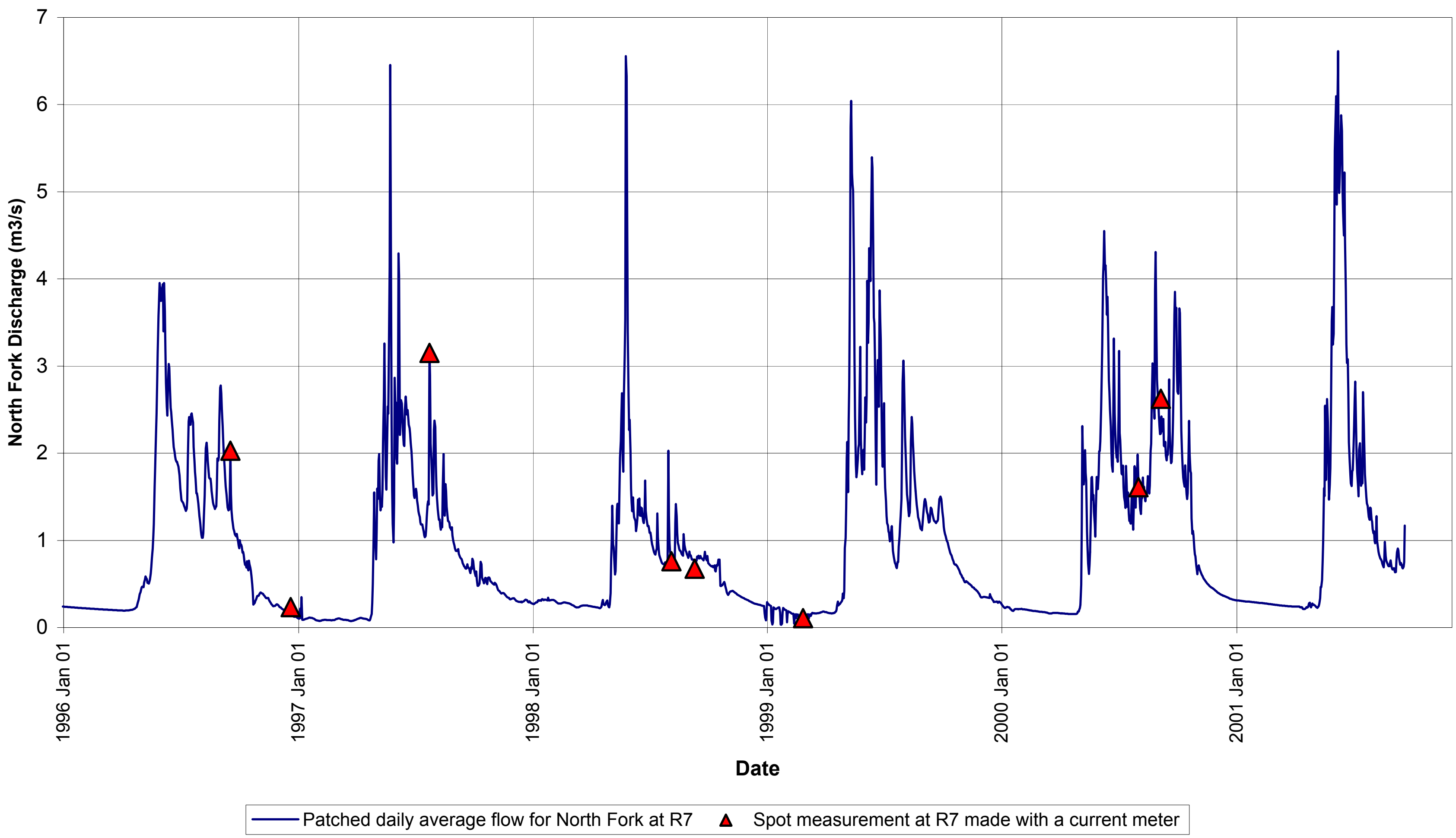
FWSD Breach
Faro Mine, Yukon Territory

FWSD Height Capacity Curve

PROJECT	DATE	APPROVED	FIGURE
1CD003.20	February 2003	GWF	**2.6**

FILE REF: Figure 2.7.xls



Source: Gartner Lee Limited

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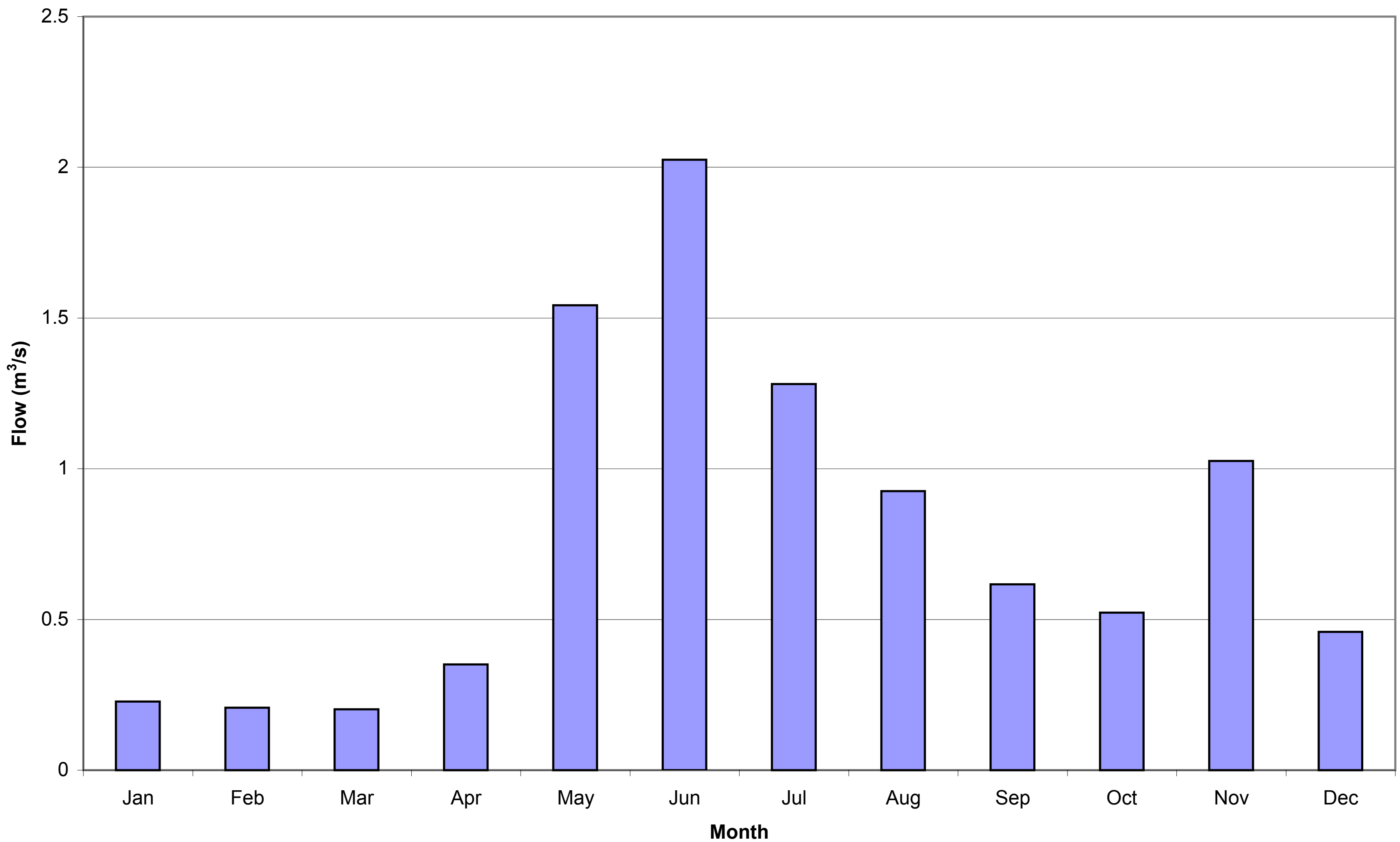
FWSD Breach
Faro Mine, Yukon Territory

Flow Measurement
R7 - North Fork of Rose Creek

PROJECT	DATE	APPROVED	FIGURE
1CD003.20	February 2003	GWF	**2.7**

FILE REF: Figure 2.8.xls



Source: nhc 201

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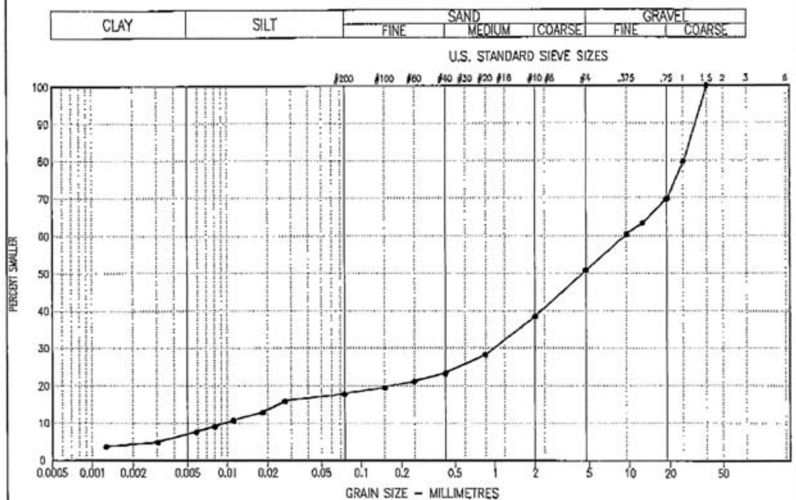
FWSD Breach
Faro Mine, Yukon Territory

Hydrograph of Average Flow Conditions for FWSD

PROJECT	DATE	APPROVED	FIGURE
1CD003.20	February 2003	GWF	**2.8**

PARTICLE SIZE – ANALYSIS OF SOILS

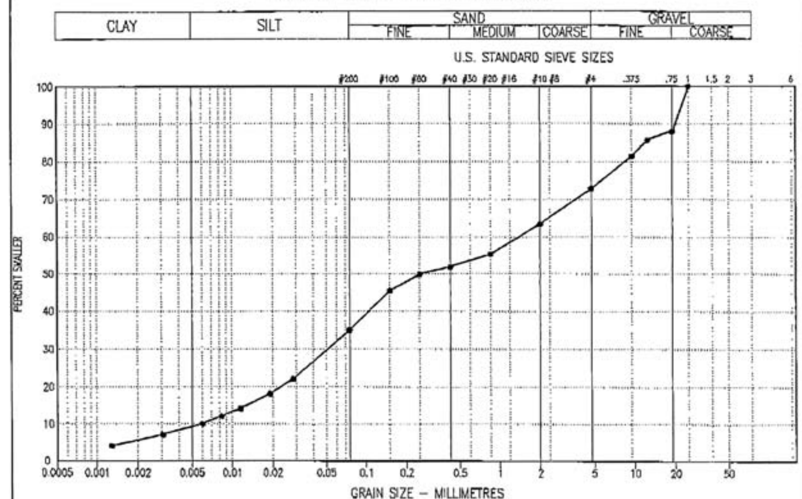


SYMBOL	BOREHOLE NUMBER	DEPTH (ft)	DESCRIPTION				Cu	Cc	U.S.C
			CLAY %	SILT %	SAND %	GRAVEL %			
	INTAKE1	0.00	6.8	11	33	49	-	12.2	GM

Project: 0201–1280001023 Date Tested: 09/01/03 BY: MCP

Tested in accordance with ASTM D422 unless otherwise noted.

PARTICLE SIZE – ANALYSIS OF SOILS



SYMBOL	BOREHOLE NUMBER	DEPTH (ft)	DESCRIPTION				Cu	Cc	U.S.C
			CLAY %	SILT %	SAND %	GRAVEL %			
	INTAKE2	0.00	9.0	26	38	27	-	0.4	SM

Project: 0201–1280001023 Date Tested: 09/01/03 BY: MCP

Tested in accordance with ASTM D422 unless otherwise noted.

Source: EBA Engineering Consultants Ltd., Fax dated January 14, 2003, Re: Intake Sediment Sample Grain Size

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SRK Consulting Engineers and Scientists

Deloitte & Touche Inc.

FWSD Breach
Faro Mine, Yukon Territory

Grain Size Testing

PROJECT:	DATE:	APPROVED:	FIGURE
1CD003.20	February 2003	GWF	**2.9**

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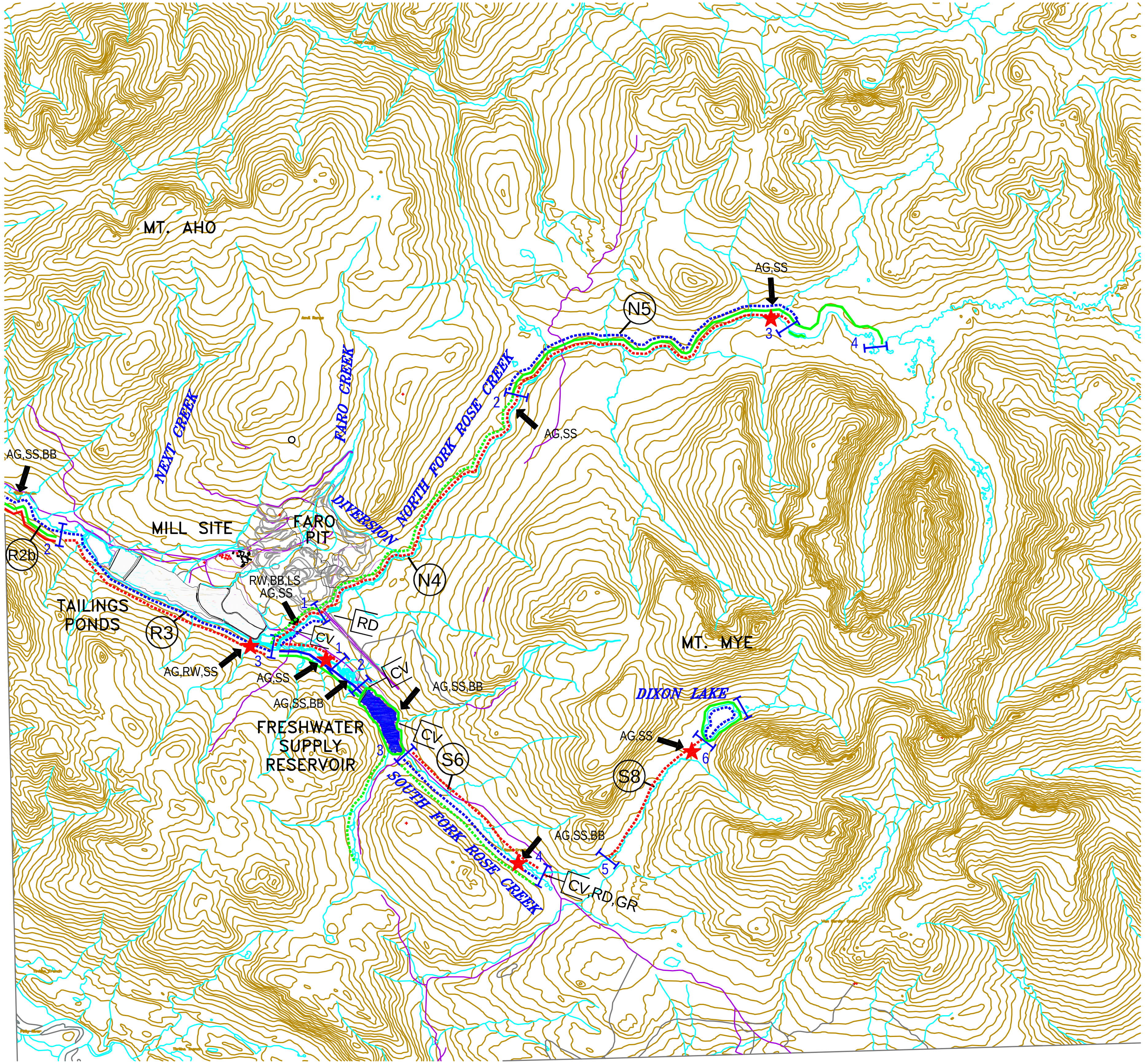
Deloitte & Touche Inc.

FWSD Breach
Faro Mine, Yukon Territory

South Fork of Rose Creek Downstream of FWSD

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.20	February 2003	GWF	2.10



LEGEND:

- ROAD
- SURFACE DRAINAGE
- REACH BREAK
- (BC) FISH SAMPLING LOCATION (2002)
- KNOWN SPAWNING IN STREAM REACH
- FISH SPECIES PRESENT (KNOWN) IN STREAM REACH
  - AG ARCTIC GREYLING
  - SS SLIMY SCULPIN
  - BB BURBOT
  - RW ROUND WHITEFISH
  - CH CHINOOK SALMON
  - LS LONGNOSE SUCKER
- CV POTENTIAL OR KNOWN BARRIER TO FISH PASSAGE
  - CV CULVERT
  - RD HAUL ROAD
  - GR GRADIENT
  - CANYON

ARCTIC GREYLING HABITAT VALUE IN ROSE WATERSHED

- HIGH REARING
- MODERATE REARING
- HIGH SPAWNING
- MODERATE SPAWNING
- HIGH OVER-WINTERING
- MODERATE OVER-WINTERING

Notes:

Base Map supplied by the Orthoshop, January 1991
North American Datum 1927
Contour Interval 100 m
Universal Transverse Mercator Projection
Map is a composite of 9 basemaps,
it was redigitized to simplify planimetric features.

SOURCE OF FIGURE:

DRAWING NO. 4.1 "OVERVIEW OF FARO MINESITE"
DIAND WASTE MANAGEMENT PROGRAM BY GARTNER LEE LIMITED
03/16/01 ADAPTED FROM ROBERTSON GEOCONSULTANTS INC.

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DATE: JAN. 2003	DATE: JAN. 2003
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DATE: JAN. 2003	DATE:
PROJ. MGR:	PROJ. ENGR.
DATE	DATE
SRK PROJECT NUMBER: 1CD003.20	

Deloitte & Touche Inc.

FWSD BREACH
FARO MINE, YUKON TERRITORY

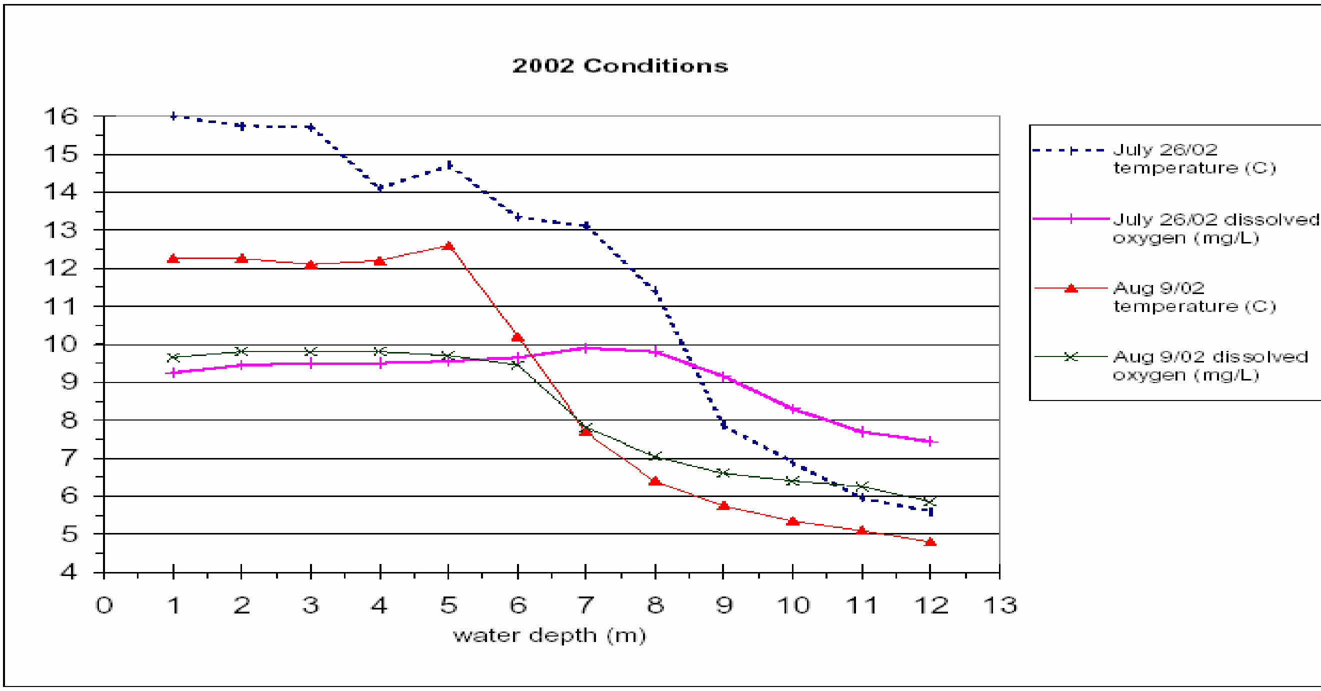
SCALE: 1:50,000
0 1.0 2.0 3.0 4.0 5.0 Km

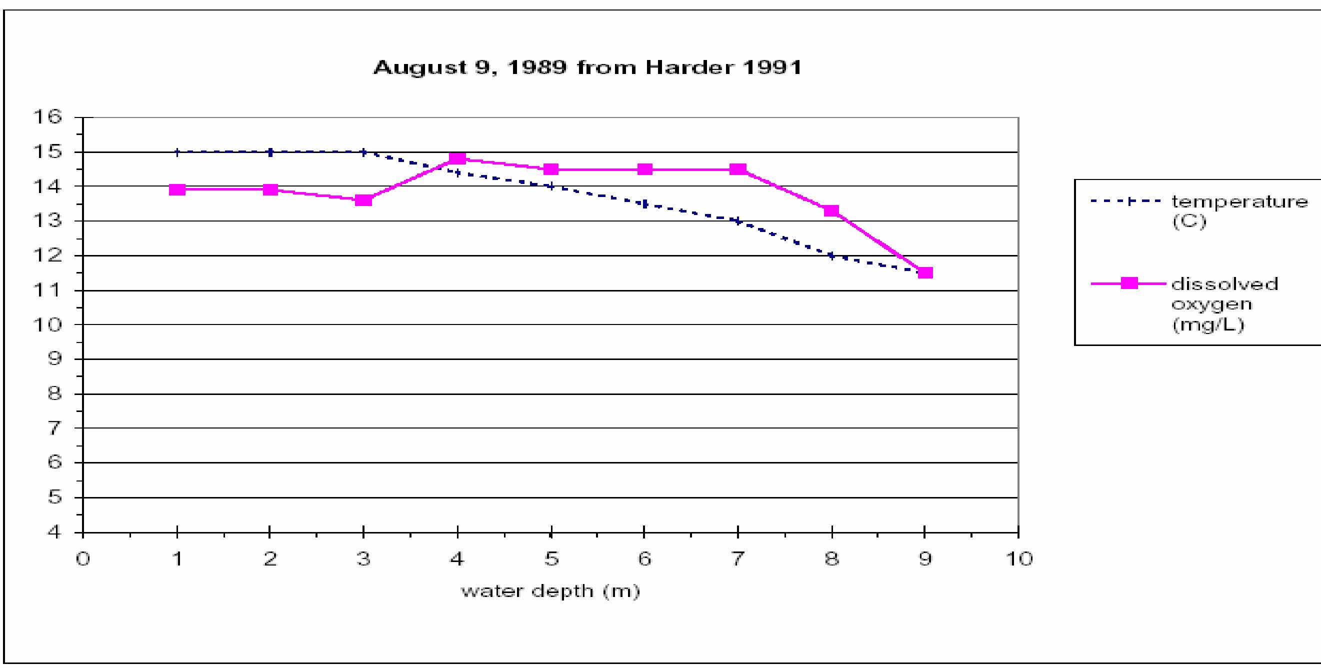
FISH HABITAT IN THE ROSE CREEK WATERSHED

DRAWING NUMBER 2.11

REV. A

FILE NAME: F:\Deloitte_touche\Acad drawings-Faro\faro FWSD\Figure GL2.5.dwg





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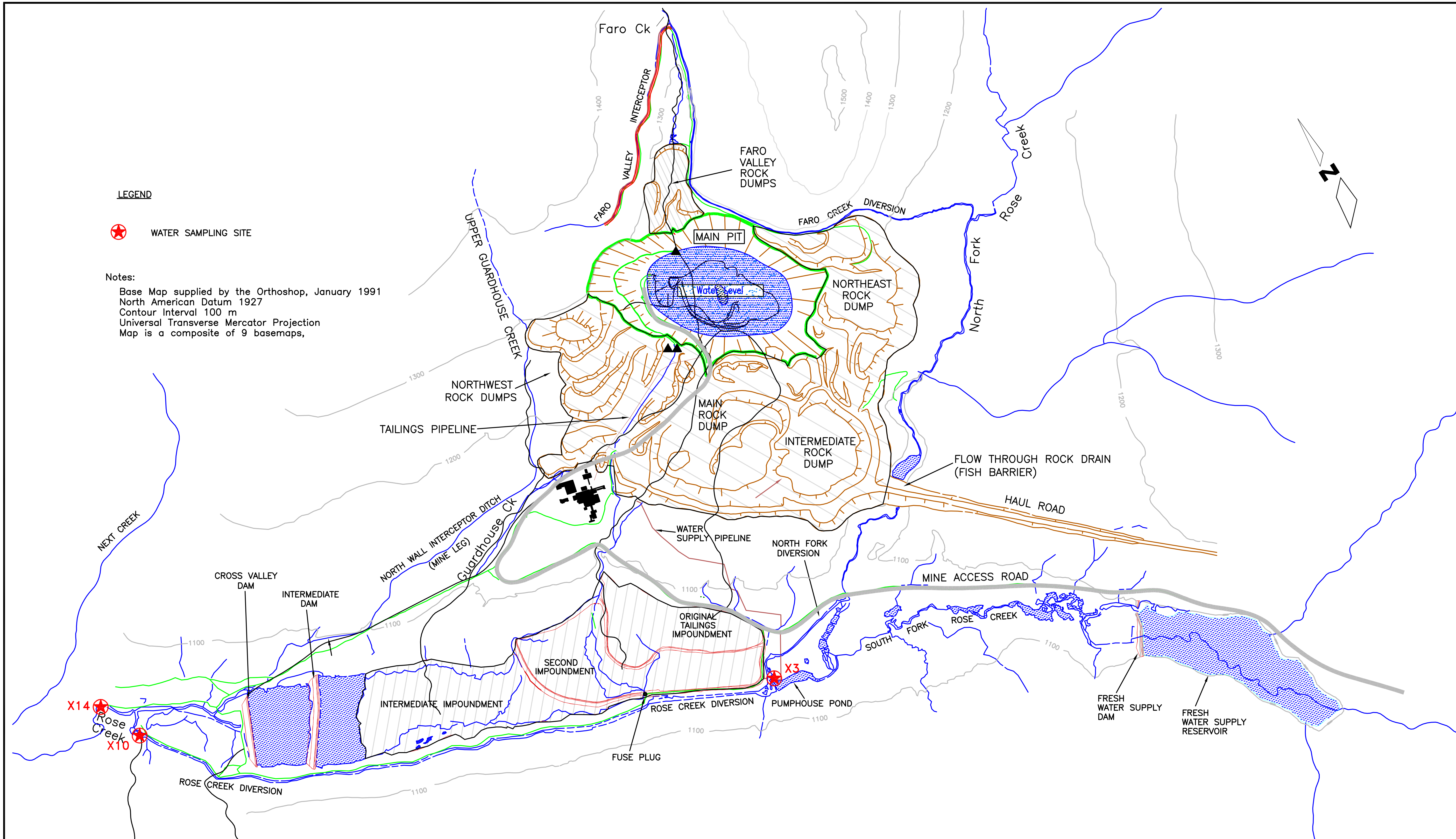
FWSD Breach
Faro Mine, Yukon Territory

Temperature/Dissolved Oxygen Profiles of the FWSD Reservoir

PROJECT:	DATE:	APPROVED:	FIGURE::
1CD003.20	February 2003	GWF	2.12



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CHECKED BY: CCS DATE: JAN. 2003
DISCIP. ENGR. DATE:
PROJ. MGR: DATE
PROJ. ENGR. DATE
SRK PROJECT NUMBER: 1CD003.20

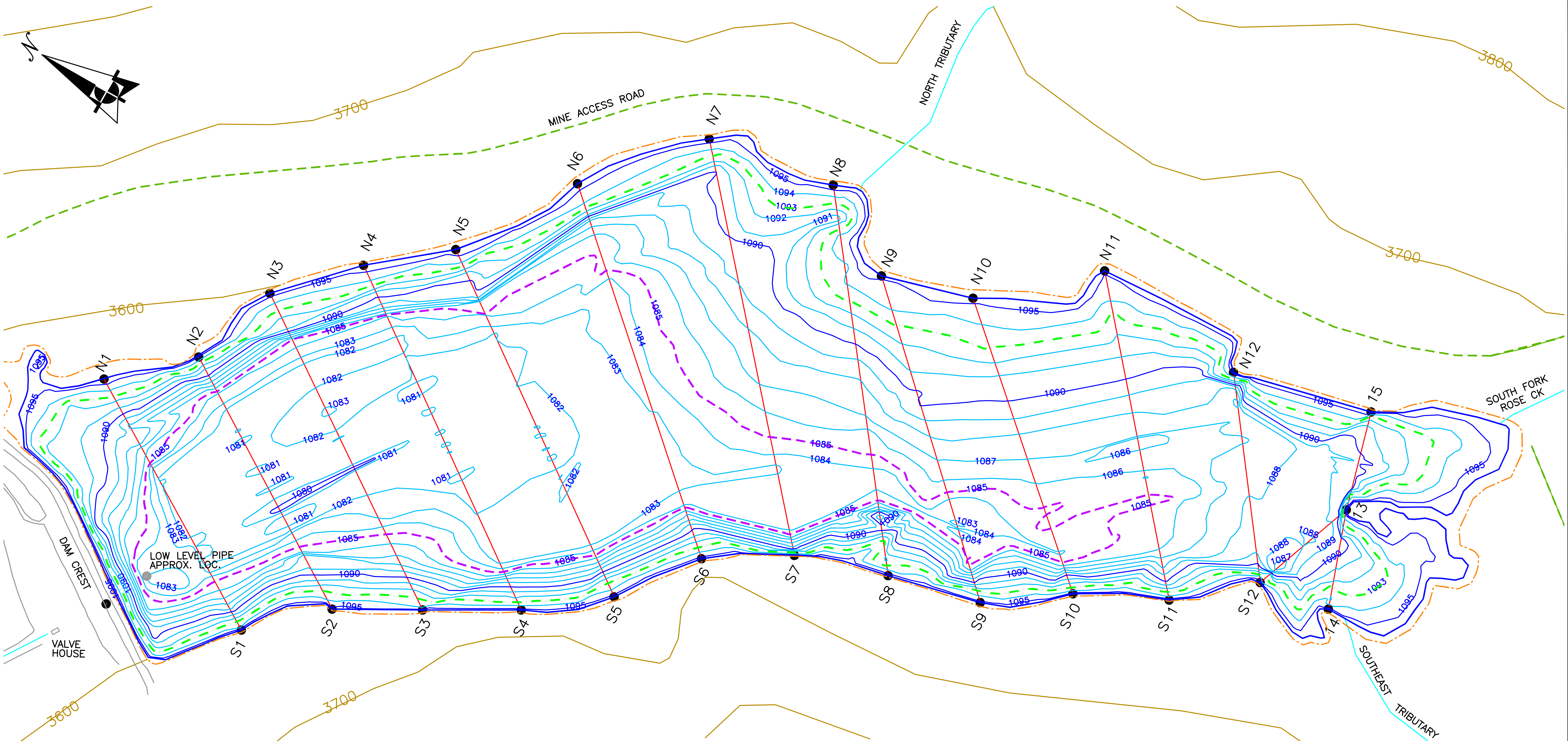
Deloitte & Touche Inc.

FWSD BREACH
FARO MINE, YUKON TERRITORY

SCALE: 1:25,000
0 200 400 600 800 1000

WATER SAMPLING LOCATIONS

DRAWING NUMBER 2.13
REV. A



0 100 200 Metres

SCALE

LEGEND:

- GROUND CONTOUR FROM 1:50,000 TOPOGRAPHIC MAP (CONTOUR INTERVAL 100 FEET)
- RIVER / STREAM FROM 1:50,000 CONTOUR MAP
- MINE ACCESS ROAD
- S1 — N1 LOCATION OF TRANSECT
- SHORELINE AT 1096 m ELEVATION
- SHORELINE DURING SURVEY CONDUCTED ON AUGUST 6–7, 2002 (1095.5 m ELEVATION)
- END OF WINTER WATER LEVEL WITH 1096 m SPILLWAY (CURRENT)
- END OF WINTER WATER LEVEL WITH 1090 m SPILLWAY (PROPOSED)
- MAJOR BATHYMETRIC CONTOUR (5m) DERIVED FROM AUGUST 6–7, 2002 SURVEY
- MINOR BATHYMETRIC CONTOUR (1m) DERIVED FROM AUGUST 6–7, 2002 SURVEY

DRAWING NO.	DRAWING TITLE	NO.	DESCRIPTION	CHK'D	APP'D	DATE	REV.	ISSUE PURPOSE	AUTH. BY	DATE
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DATE
DRAWN BY: CPW
DATE: JAN. 2003
DISCIP. ENGR.
DATE:
PROJ. ENGR.
DATE
SRK PROJECT NUMBER: 1CD003.20

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FWSD BREACH
FARO MINE, YUKON TERRITORY

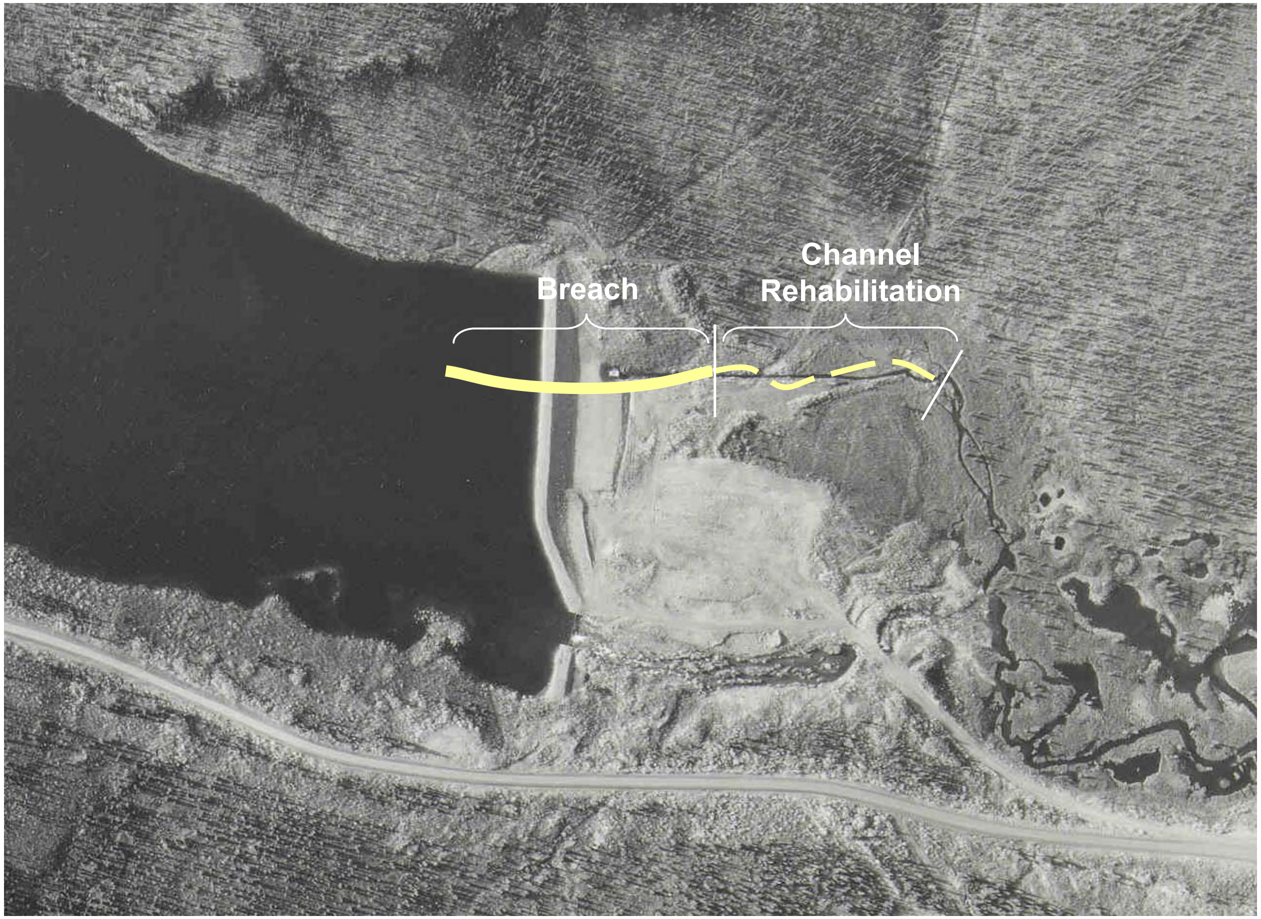
SCALE: AS SHOWN

BATHYMETRIC MAP OF FRESHWATER SUPPLY RESERVOIR

DRAWING NUMBER: 2.14

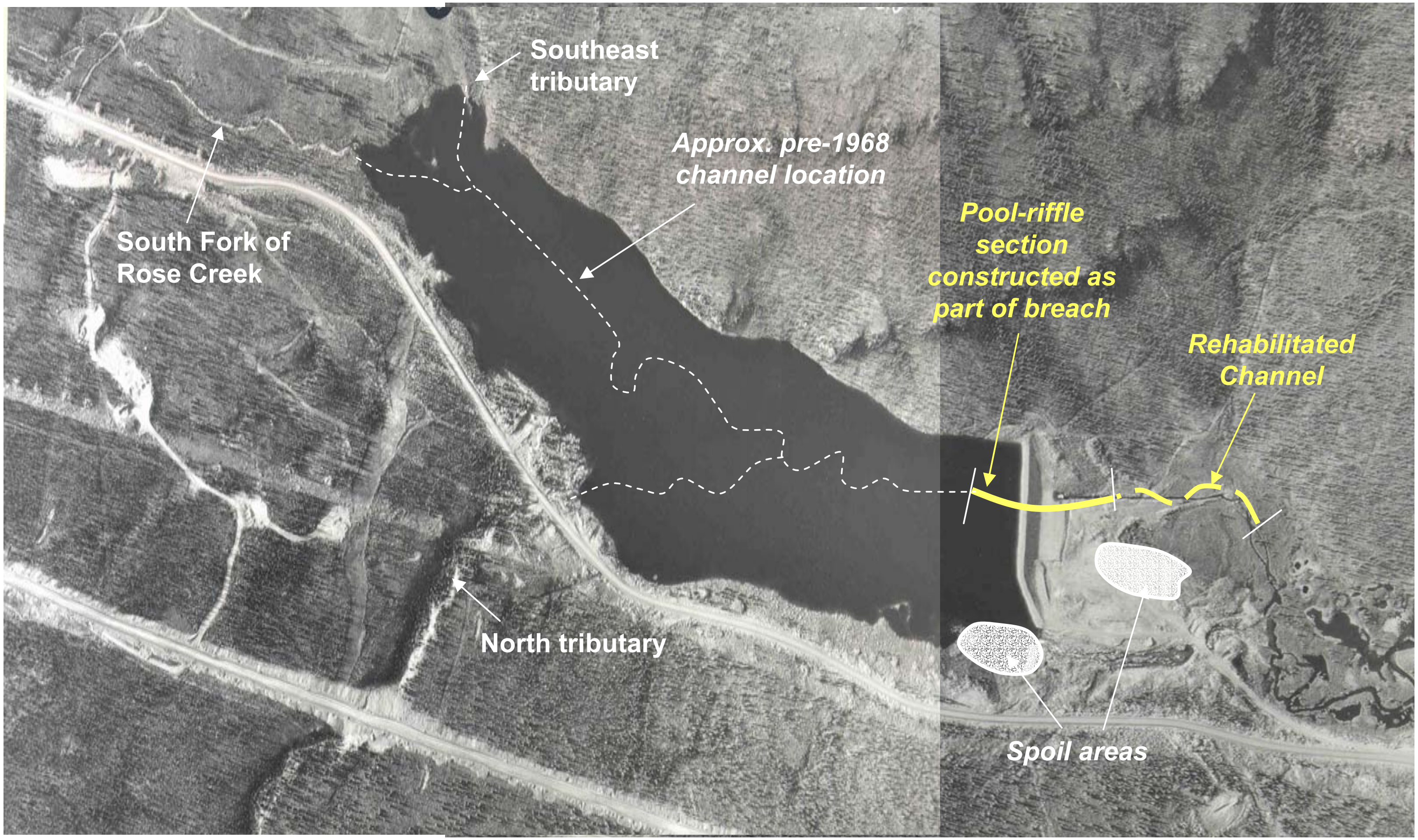
REV.: A

FILE NAME: F:\Deloitte_touche\Acad drawings-Faro\faro FWSD\figure 1.dwg



Source: Airphoto – OS9009015-1-12 & -13, Scale 1:10,000 Line 1-SE (1-15) 8700 ASL, September 17, 1990

Gartner Lee Limited — BGC Engineering Inc. AN APPLIED EARTH SCIENCES COMPANY — SRK Consulting Engineers and Scientists	FWSD Breach Faro Mine, Yukon Territory
	Breach & Rehabilitated Channel approx. Location
Deloitte & Touche Inc.	PROJECT: 1CD003.20 — DATE: February 2003 — APPROVED: GWF — FIGURE:: **5.1**



Source: Airphoto – OS9009015-1-12 & -13, Scale 1:10,000 Line 1-SE (1-15) 8700 ASL, September 17, 1990

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FWSD Breach
Faro Mine, Yukon Territory

Approximate Post Breach Channel Configuration

PROJECT:	DATE:	APPROVED:	FIGURE
1CD003.20	February 2003	GWF	**5.2**

Photos

Photo 1: FWSD

Photo 2: Spillway near North Abutment of FWSD

Photo 3: Valve House and portion of Fresh Water Channel

Photo 4: Fresh Water Channel

Photo 5: Culvert Discharge at downstream end of spillway

Photo 6: Cracking on the crest of the FWSD

Photo 7: South Fork of Rose Creek – upstream from the FWSD reservoir

Photo 8: North Tributary as it enters the FWSD Reservoir

Photo 9: Southeast Tributary at the entrance to the FWSD reservoir

Photo 10: South Fork of Rose Creek, downstream of reservoir

Photo 11: View of the exposed reservoir base near the south abutment of the FWSD

Photo 12: View of the exposed reservoir base, near the spillway

Photo 13: Exposed reservoir base

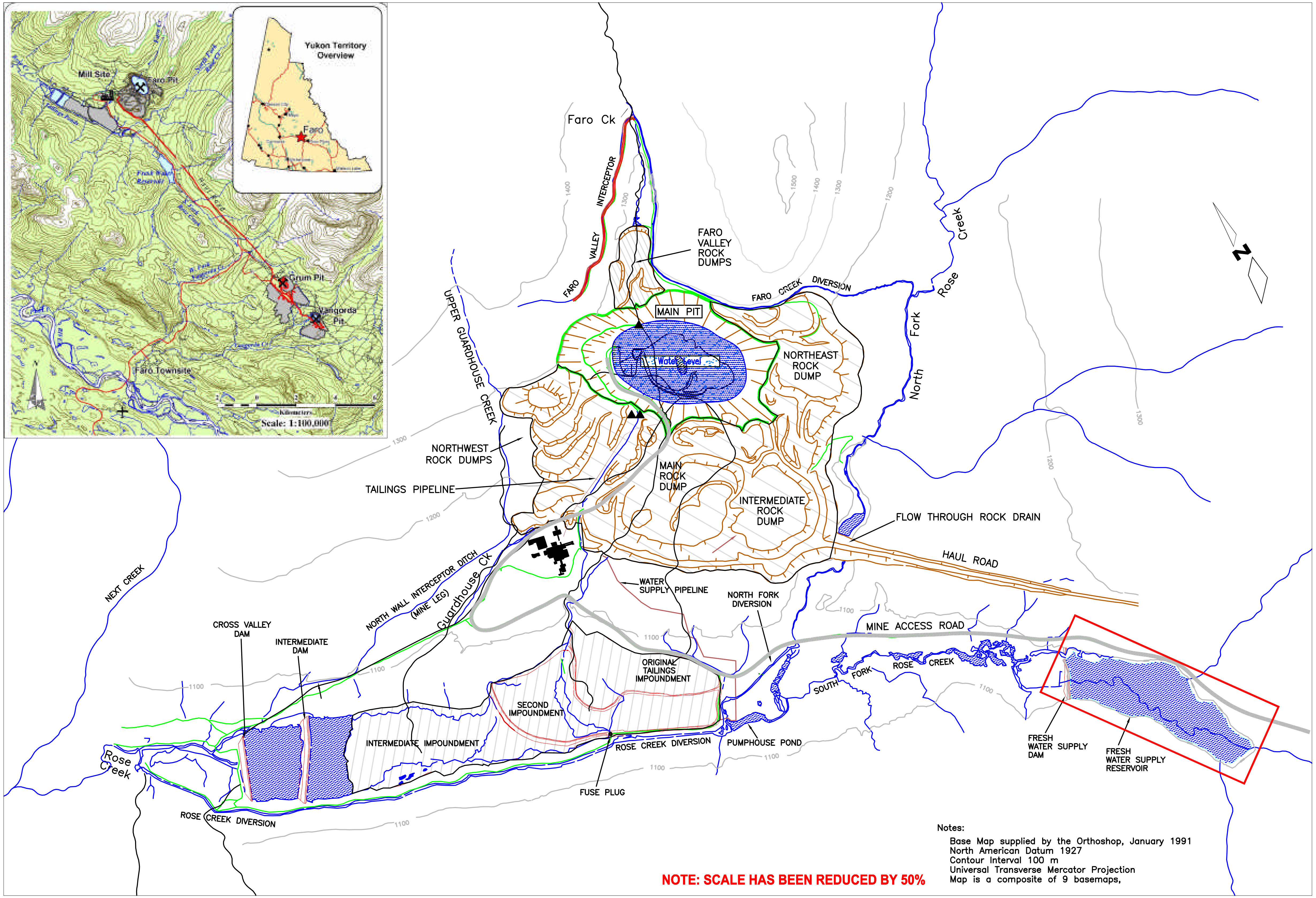
Photo 14: Exposed reservoir base

Photo 15: South Rose Creek inlet (July 26/02)

Photo 16: New inlet of Southeast Tributary

Photo 17: Near inlet of South Fork of Rose Creek

Reduced Drawings



Notes:
Base Map supplied by the Orthoshop, January 1991
North American Datum 1927
Contour Interval 100 m
Universal Transverse Mercator Projection
Map is a composite of 9 basemaps,

DRAWING INDEX:

REFERENCE DRAWINGS		REVISIONS					ISSUE AUTHORIZATION			
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SRK PROJECT NUMBER: 1CD003.20	

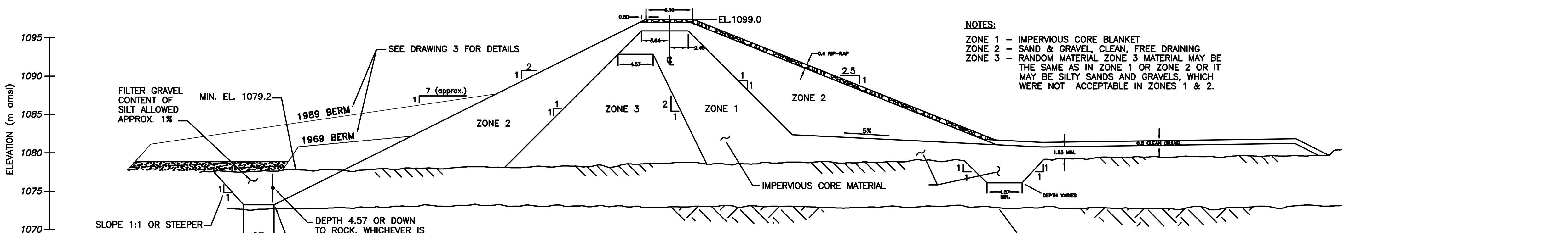
Deloitte & Touche Inc.

FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

SCALE: 1:15,000
0 200 400 600 800 1000

COVER SHEET

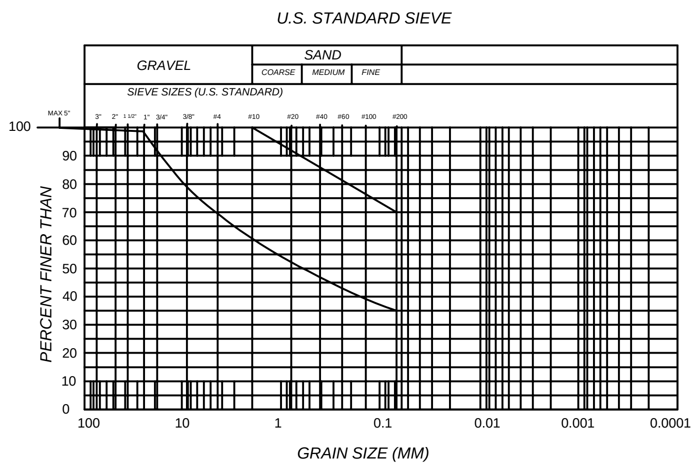
DRAWING NUMBER 1
REV. A



1968 DAM CONSTRUCTION DETAILS WITH 1989 AND 1969 BERMS

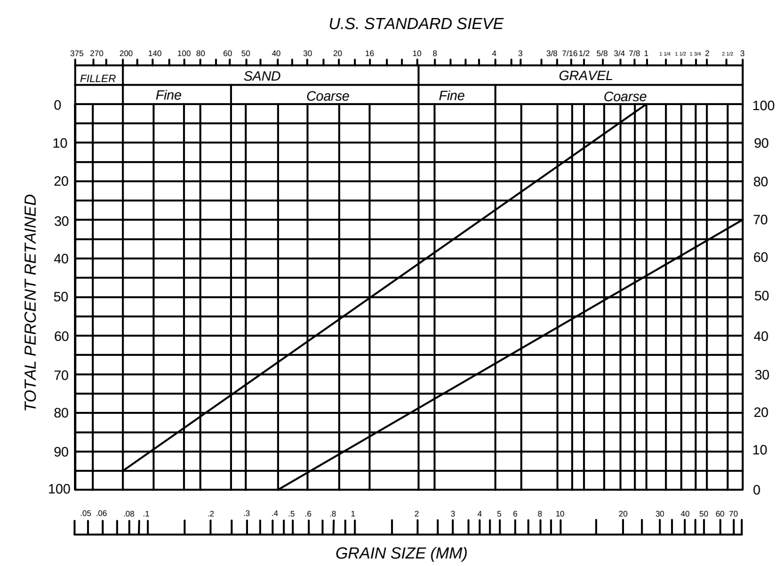
TYPICAL SECTION

SOURCE: AS BUILT DRAWING NO. D1575–058–006, REV.06
PARSONS–JURDEN CORPORATION, OCT. 1968



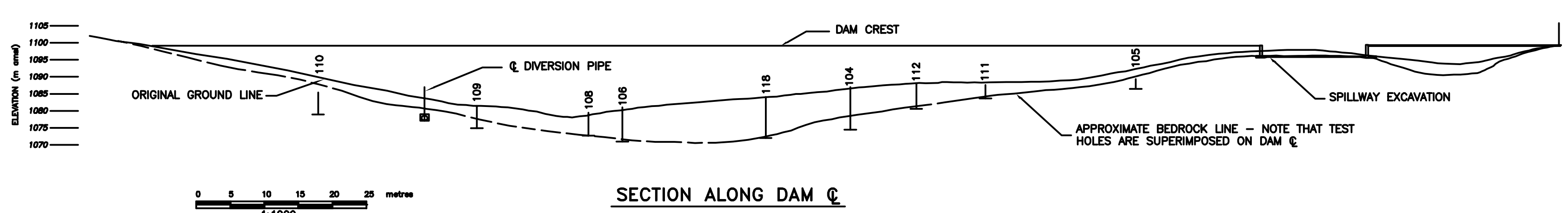
DESIGN GRADATION CURVE LIMITS - ZONE 1

SOURCE: AS BUILT DRAWING NO. D1575–058–006, REV.06
PARSONS–JURDEN CORPORATION, OCT. 1968



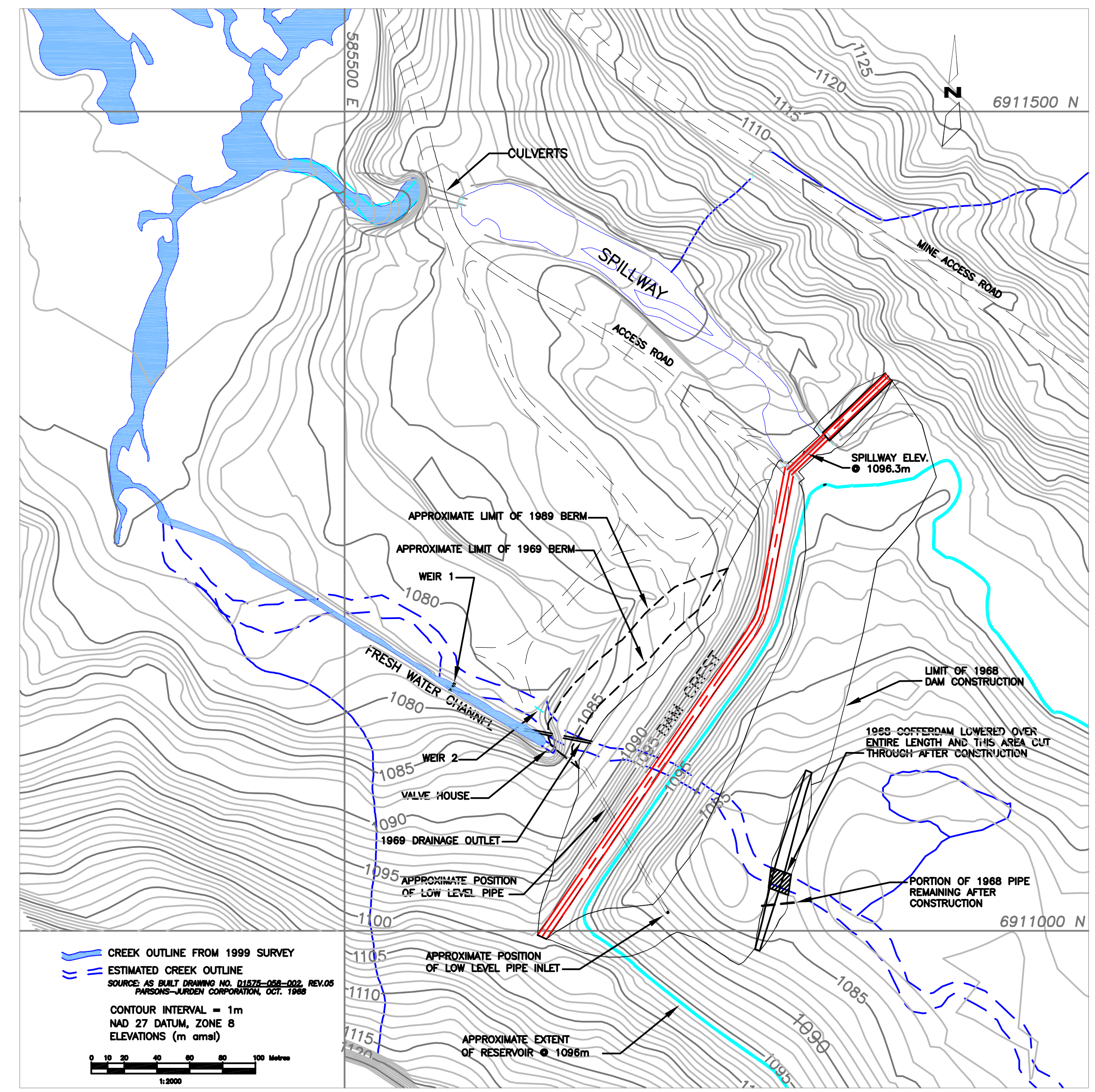
DESIGN GRADATION CURVE LIMITS - ZONE 2

SOURCE: AS BUILT DRAWING NO. D1575–058–006, REV.06
PARSONS–JURDEN CORPORATION, OCT. 1968



SECTION ALONG DAM ℄

SOURCE: AS BUILT DRAWING NO. D1575–058–004, REV.05
PARSONS–JURDEN CORPORATION, OCT. 1968



NOTE: SCALE HAS BEEN REDUCED BY 50%

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DATE | DATE
SRK PROJECT NUMBER: 1CD003.20

Deloitte & Touche Inc.

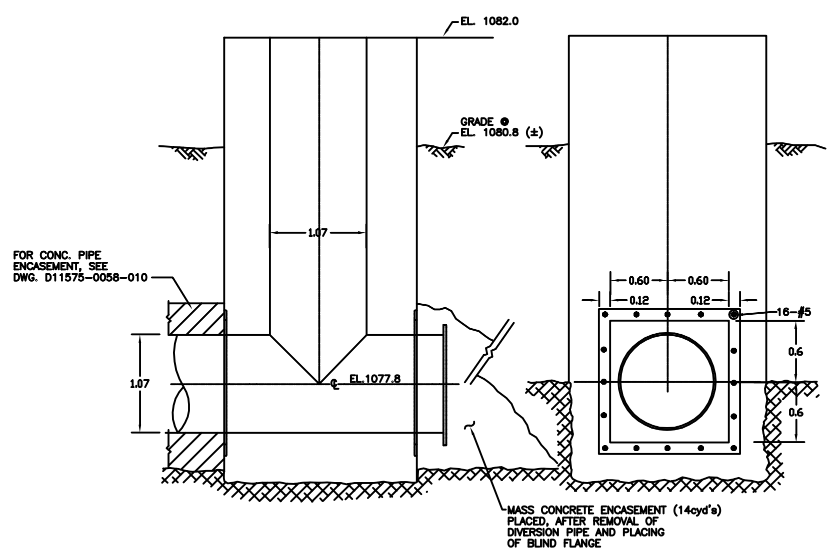
FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

SCALE: AS SHOWN

FWSD EXISTING CONDITIONS

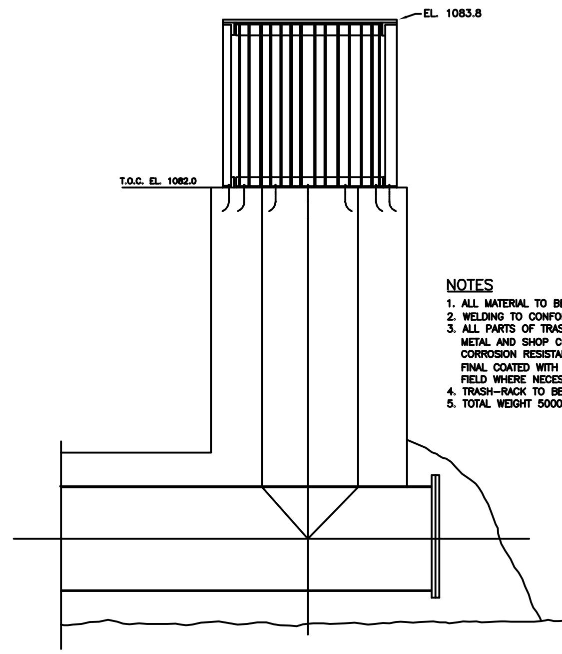
DRAWING NUMBER: 2

REV.: A



LOW-LEVEL PIPE INLET
SECTION ELEVATION
CONCRETE OUTLINE

SOURCE: AS BUILT DRAWING NO. D1575-058-013, REV.04
PARSONS-JURDEN CORPORATION, OCT. 1968

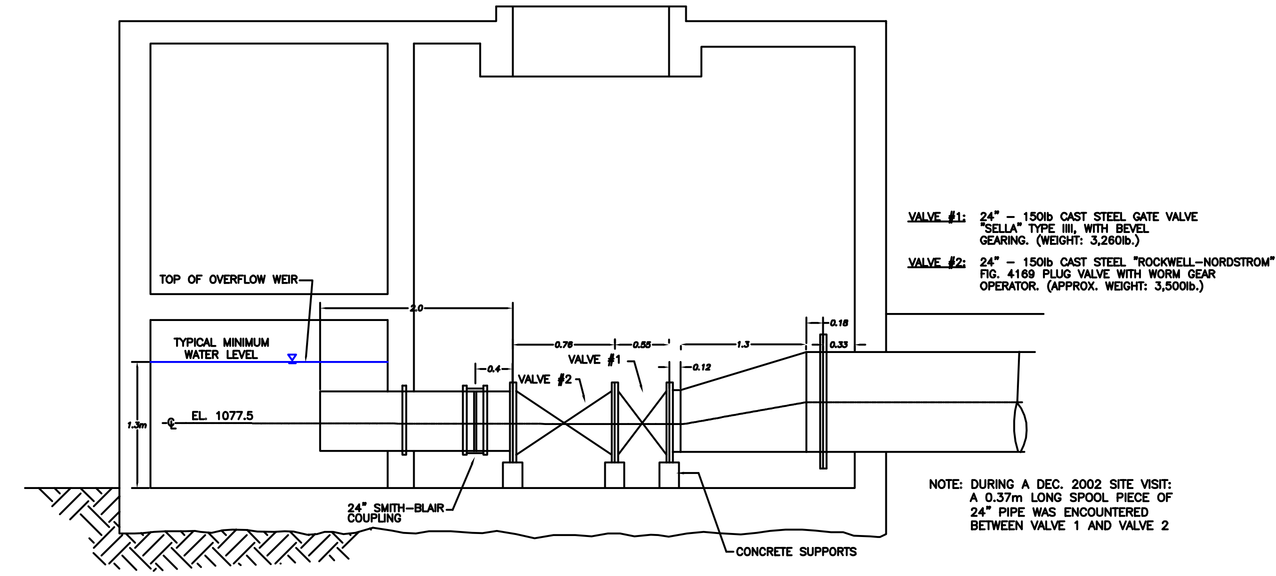


NOTES
1. ALL MATERIAL TO BE MILD STEEL.
2. WELDING TO CONFORM TO CSA STANDARDS. S16-1965.
3. ALL PARTS OF TRASH-RACK TO BE SAND-BLASTED TO WHITE METAL AND SHOP COATED WITH PCE-414 'FLINTGUARD' CORROSION RESISTANT COATING TO 20 MIL. THICKNESS AND FINAL COATED WITH 'CHEMROX' VINYL ENAMEL. TOUCH UP IN FIELD WHERE NECESSARY.
4. TRASH-RACK TO BE 4 PANELS AS SHOWN
5. TOTAL WEIGHT 5000LB.

TRASH RACK DETAILS

SOURCE: AS BUILT DRAWING NO. D1575-058-017, REV.04
PARSONS-JURDEN CORPORATION, OCT. 1968

0 .5 1 1.5 2 2.5 Metres
1:40



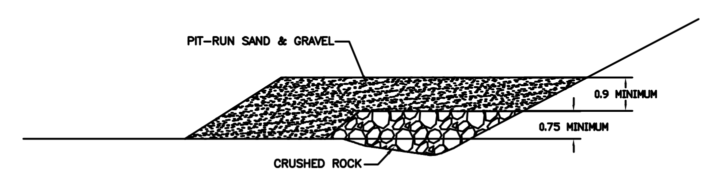
VALVE #1: 24" – 150lb CAST STEEL GATE VALVE "SELLA" TYPE IIII, WITH BEVEL GEARING. (WEIGHT: 3,260lb.)
VALVE #2: 24" – 150lb CAST STEEL "ROCKWELL-NORDSTROM" FIG. 4169 PLUG VALVE WITH WORM GEAR OPERATOR. (APPROX. WEIGHT: 3,500lb.)

NOTE: DURING A DEC. 2002 SITE VISIT: A 0.37m LONG SPOOL PIECE OF 24" PIPE WAS ENCOUNTERED BETWEEN VALVE 1 AND VALVE 2

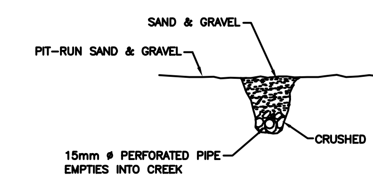
VALVE HOUSE ARRANGEMENT

SOURCE: AS BUILT DRAWING NO. D1575-058-014, REV.05
PARSONS-JURDEN CORPORATION, OCT. 1968

0 2 Metres
1:125



TYPICAL SECTION THROUGH DRAIN AND BERM

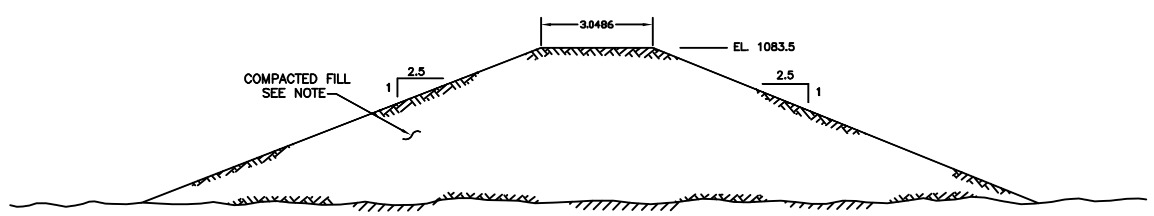


TYPICAL SECTION THROUGH
1969 DRAINAGE OUTLET DITCH

1969 TOE BERM CONSTRUCTION DETAILS

RECOMMENDED TOE DRAINAGE DRAWING B-1223-C
ACCOMPANYING LETTER OF JUNE 16, 1969
SOURCE: RIPLEY,KLOHN & LEONOFF INTERNATIONAL LTD.

0 1 2 3 4 5 Metres
1:100

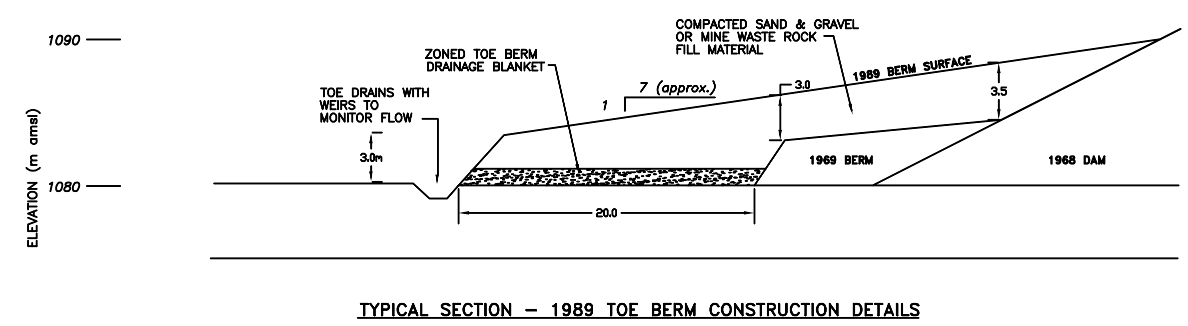


NOTES
1. ALL OVERBURDEN COMPOSED OF ORGANIC SOIL, DEBRIS, AND MUSKEG AND TO A MIN.2' DEPTH TO BE REMOVED OVER ENTIRE COFFERDAM.
2. COFFERDAM FILL MATERIAL TO BE COMPOSED OF ZONE 1 MATERIAL.
3. COFFERDAM FILL TO BE PLACED IN LIFTS OF 12 MEASURED AFTER COMPACTION TO 97% STANDARD PROCTOR DENSITY, BY FOUR PASSES OF A 50 TON STANDARD RUBBER TIRED ROLLER. (ALTERNATIVELY PLACE 6" LIFTS AND COMPACT TO ABOVE DENSITY USING A "SHEEPS FOOT ROLLER"

0 1 2 3 4 5 Metres
1:100

UPSTREAM COFFERDAM TYPICAL DETAIL

SOURCE: AS BUILT DRAWING NO. D1575-058-011, REV.06
PARSONS-JURDEN CORPORATION, OCT. 1968



TYPICAL SECTION – 1989 TOE BERM CONSTRUCTION DETAILS

SOURCE: 1988 ANNUAL INSPECTION OF THE FARO MINE.
GOLDER ASSOCIATES LTD.

0 4 8 12 16 20 Metres
1:250

NOTE: SCALE HAS BEEN REDUCED BY 50%

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DATE: JAN. 2003	DATE
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DATE	DATE
SRK PROJECT NUMBER: 1CD003.20	

Deloitte & Touche Inc.

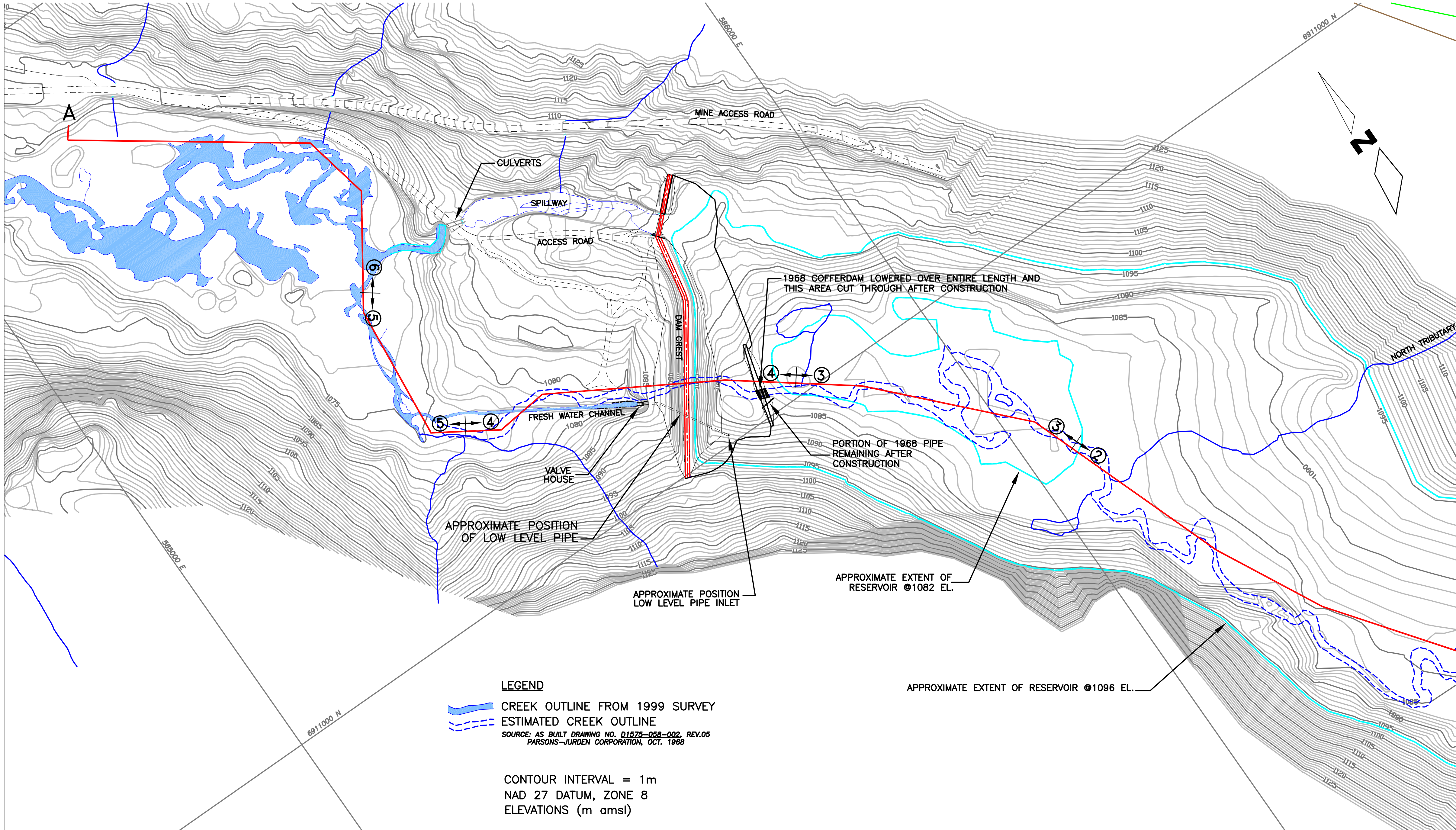
FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

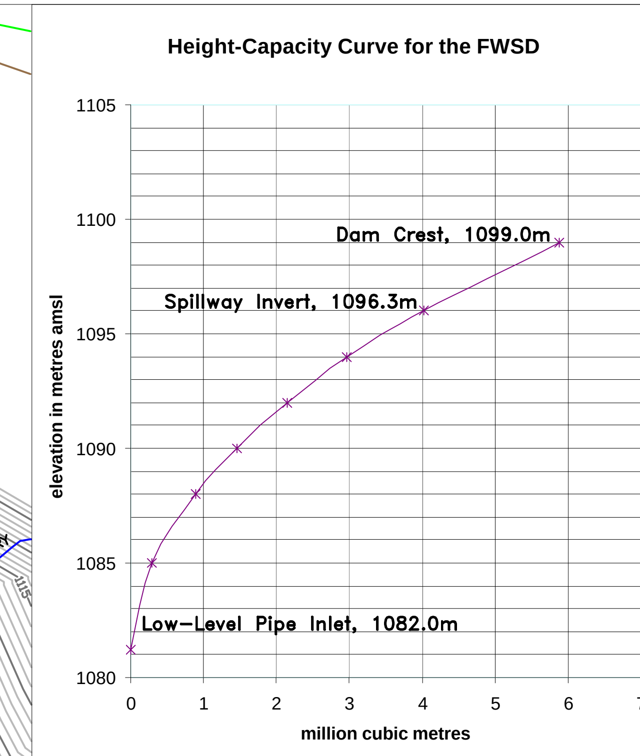
SCALE: 1:15,000
0 100 200 300 400 500m

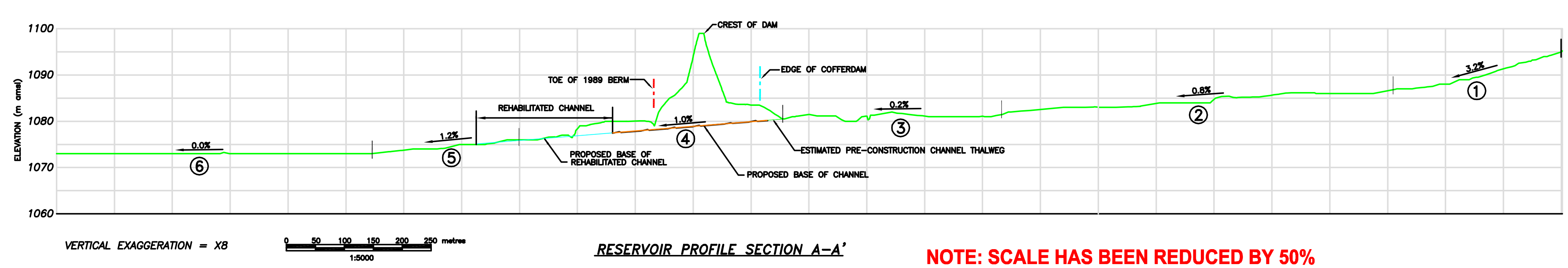
FWSD DETAILS: EXISTING CONDITIONS

DRAWING NUMBER: 3
REV. A

FILE NAME: F:\Deloitte_touche\Acad drawings-Faro\faro FWSD\Figure 3.dwg







RESERVOIR PROFILE SECTION A-A'

NOTE: SCALE HAS BEEN REDUCED BY 50%

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CHECKED BY: CCS	DISCIP. ENGR.
DATE: JAN. 2003	DATE
PROJ. DES. COORD.	PROJ. ENGR.
DATE	DATE
SRK PROJECT NUMBER: 1CD003.20	

Deloitte & Touche Inc.

FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

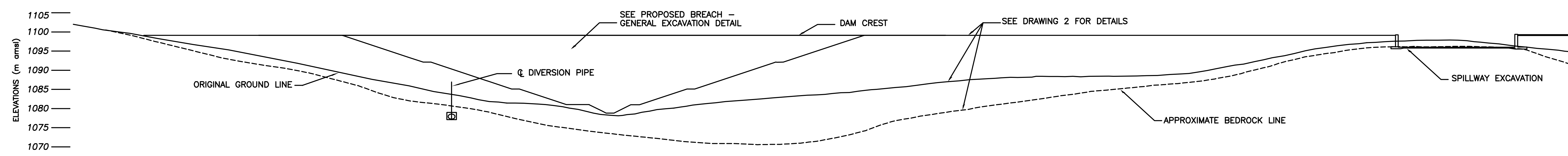
SCALE: 1:3000
0 50 100 150 200 250 metres

RESERVOIR SITE PLAN

DRAWING NUMBER 4

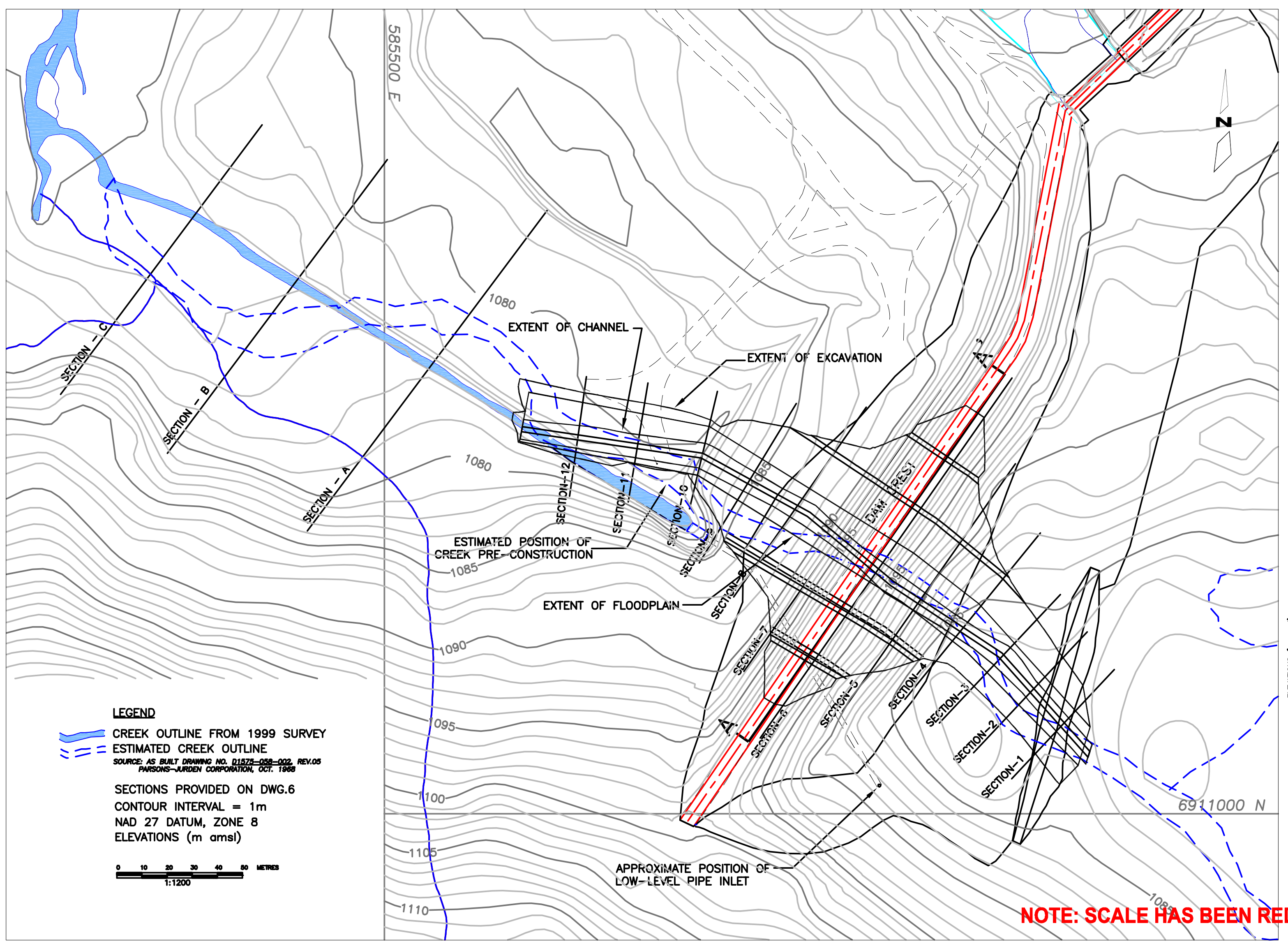
REV. A

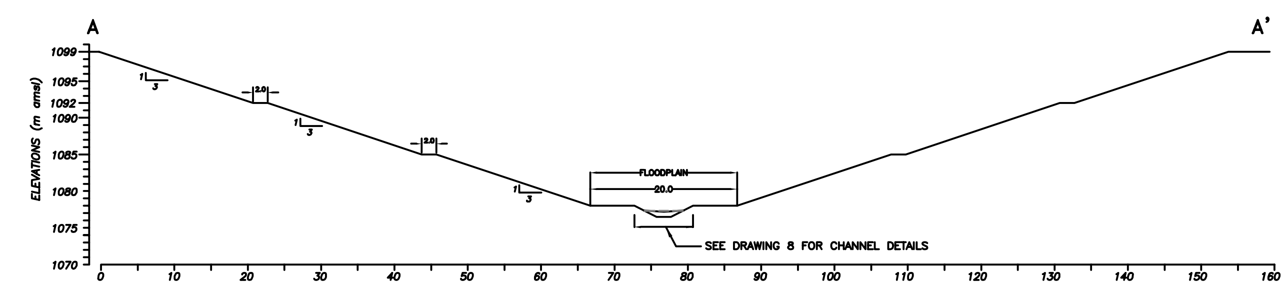
FILE NAME: F:\Deloitte_touche\Acad drawings-Faro\faro FWSD\Figure 4.dwg



DAM PROFILE – EXISTING CONDITIONS AT ℄ OF DAM AND PROPOSED BREACH LOCATION

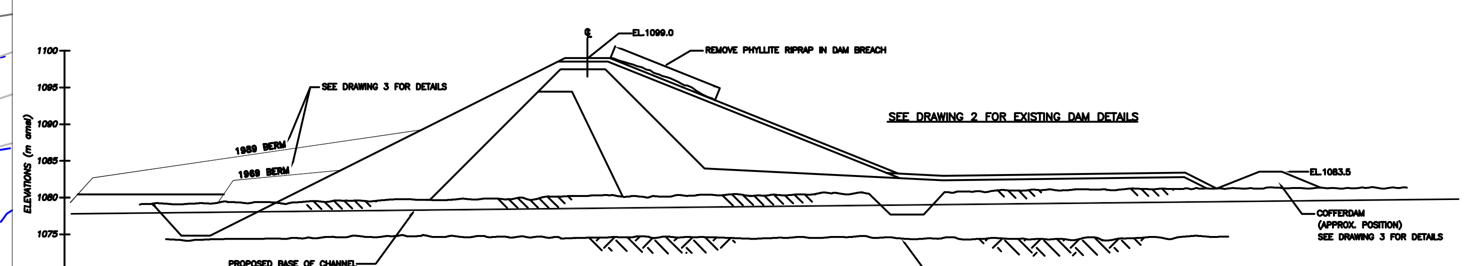
SOURCE: AS BUILT DRAWING NO. D1575–058–004, REV.05
PARSONS–JURDEN CORPORATION, OCT. 1968





SECTION A–A' – PROPOSED BREACH

GENERAL EXCAVATION DETAIL



TYPICAL DAM SECTION

PROPOSED CHANNEL BASE

NOTE: SCALE HAS BEEN REDUCED BY 50%

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							A	PRELIMINARY DESIGN	GWF	FEB 3/03

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BGC Engineering Inc. AN APPLIED EARTH SCIENCES COMPANY
SRK Consulting Engineers and Scientists

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DATE	DATE
SRK PROJECT NUMBER: 1CD003.20	

Deloitte & Touche Inc.

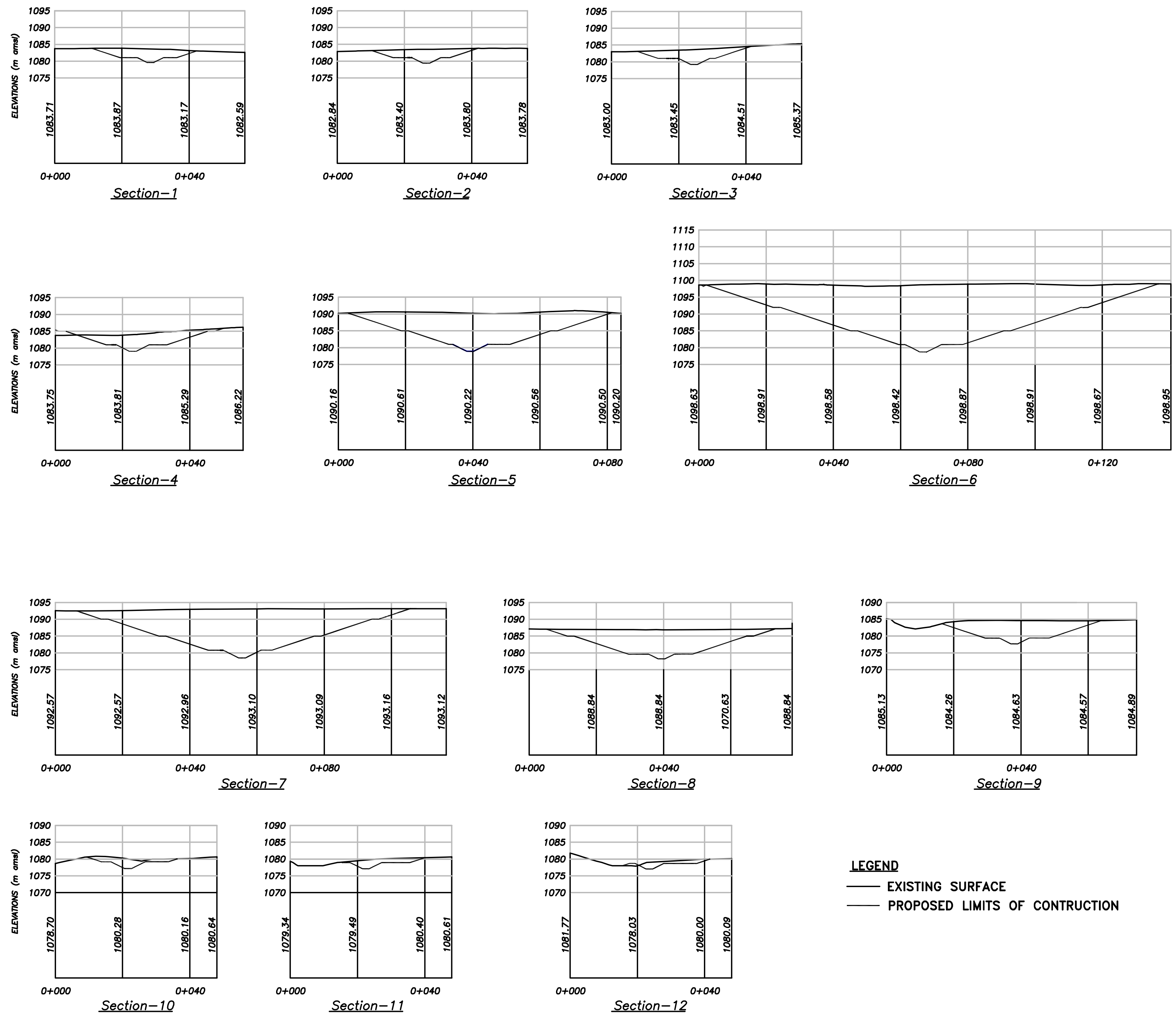
FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

SCALE: AS SHOWN

FWSD BREACH GENERAL ARRANGEMENT

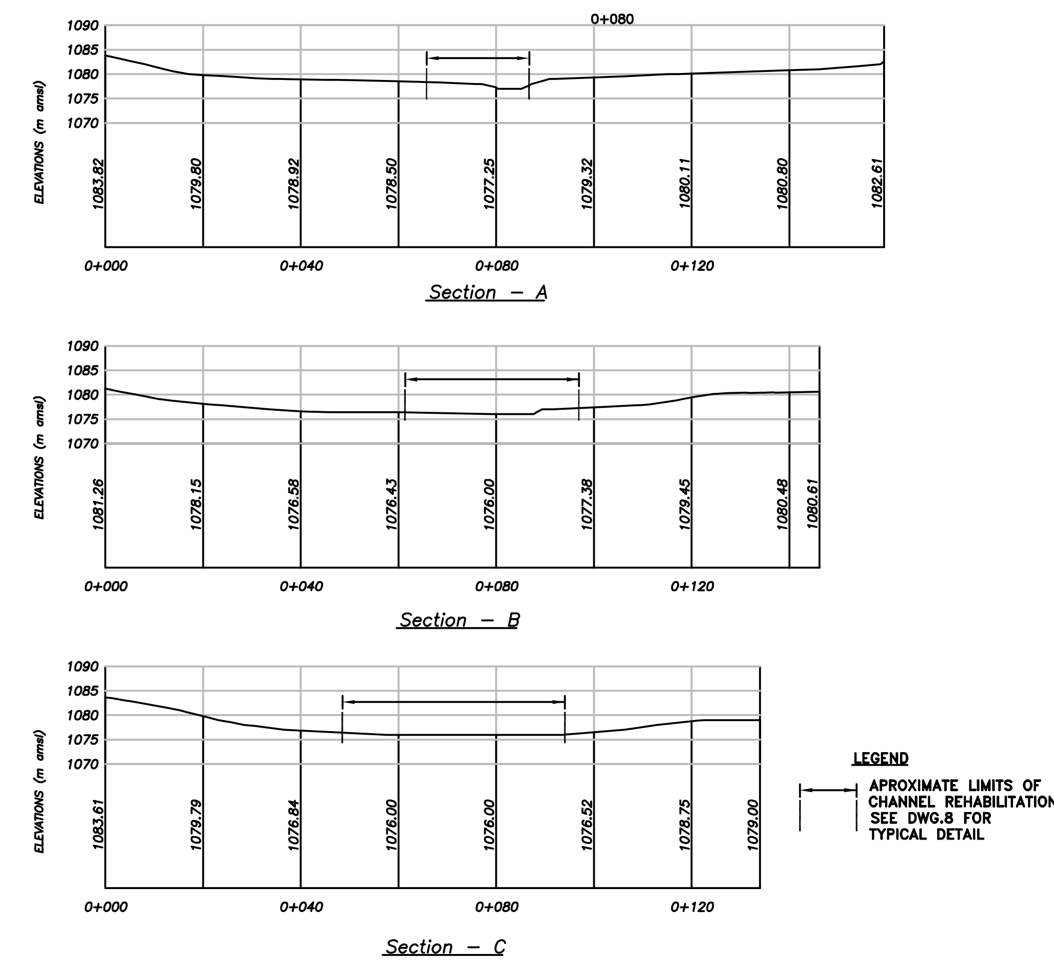
DRAWING NUMBER 5

REV. A



LEGEND
EXISTING SURFACE
PROPOSED LIMITS OF CONTRUCTION

PROPOSED BREACH SECTIONS
(1 – 12)



Existing Topography
(Proposed Rehabilitated Channel)
(Sections A to C)

NOTE: SCALE HAS BEEN REDUCED BY 50%

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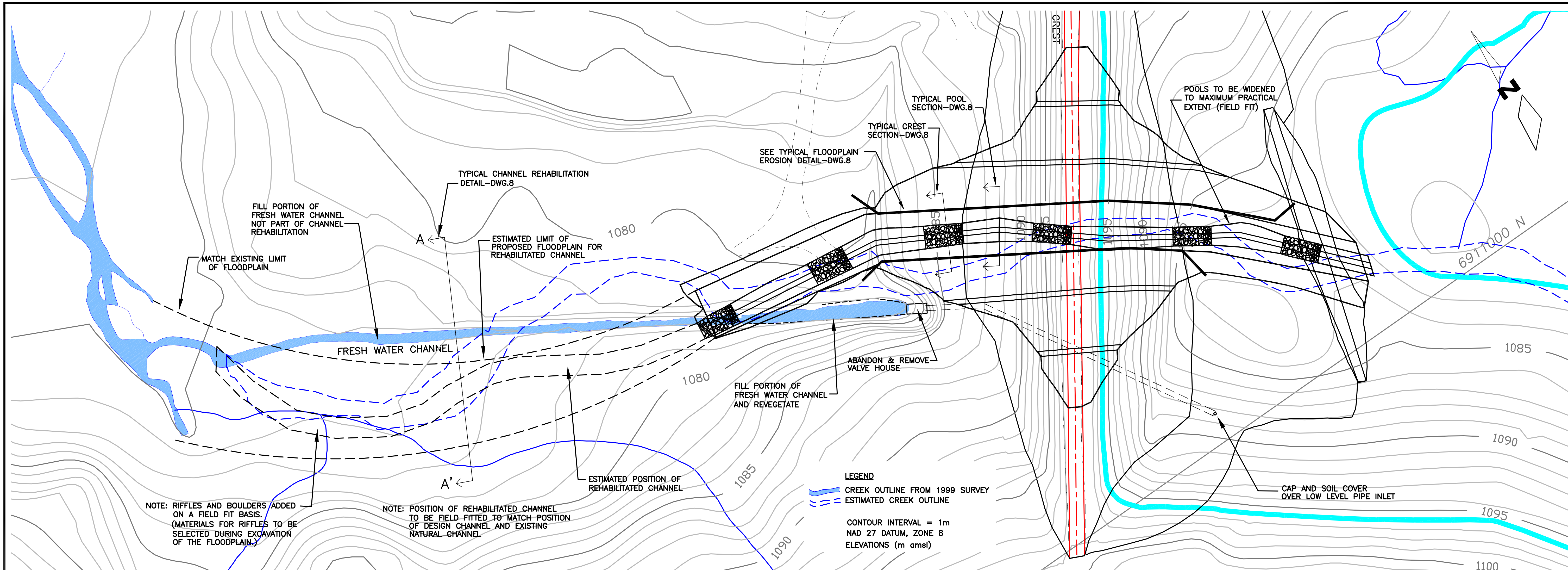
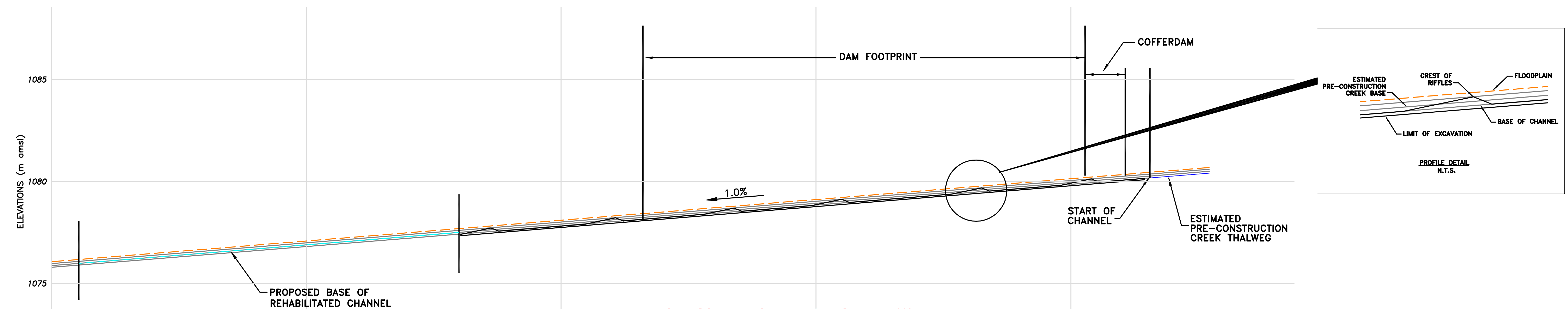
FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

SCALE: 1:750
0 10 20 30 40 50 Metres

FWSD BREACH SECTIONS

DRAWING NUMBER: 6
REV.: A





NOTE: SCALE HAS BEEN REDUCED BY 50%

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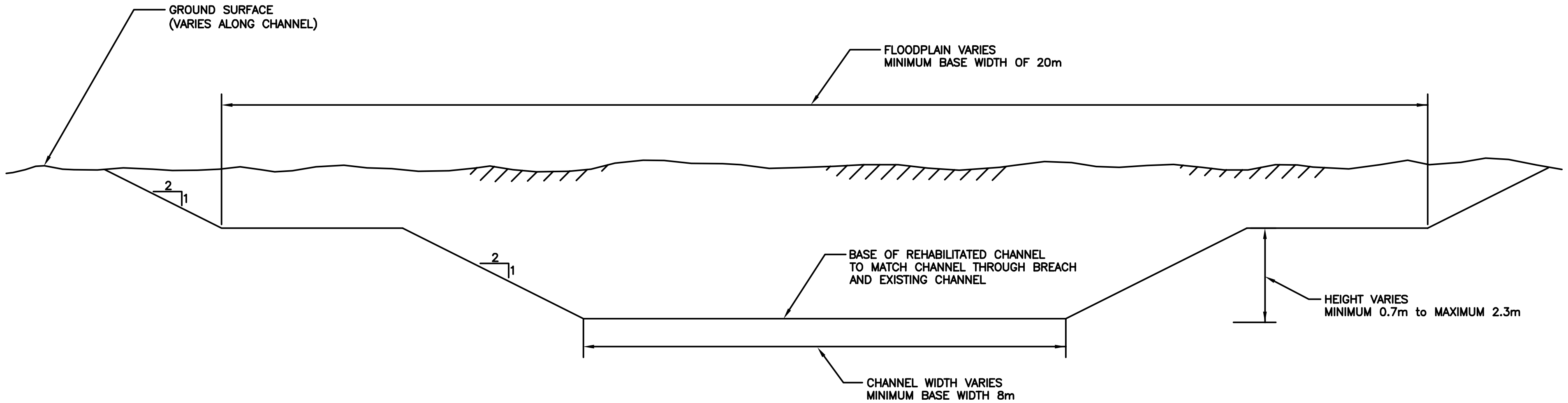
FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

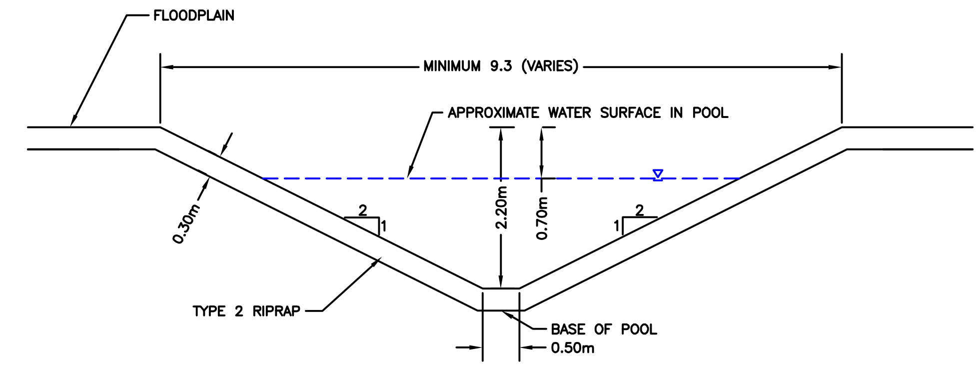
SCALE: 1:750
0 10 20 30 40 50 Metres

FWSD CHANNEL PLAN AND PROFILE

DRAWING NUMBER: 7
REV. A

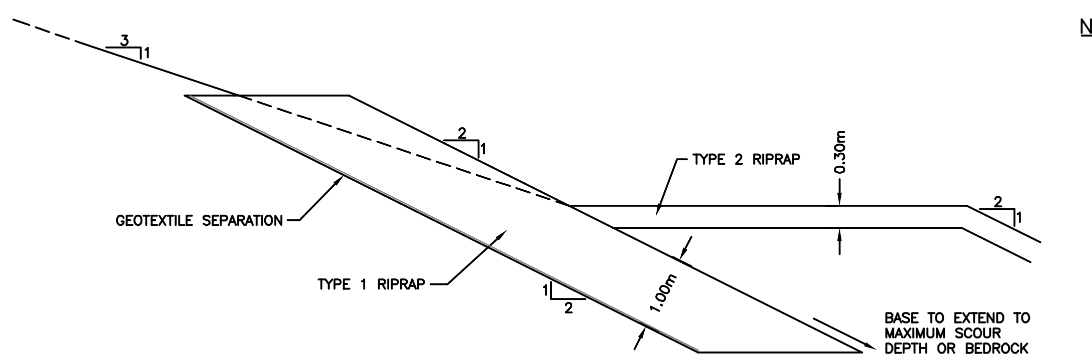


TYPICAL CHANNEL REHABILITATION DETAIL

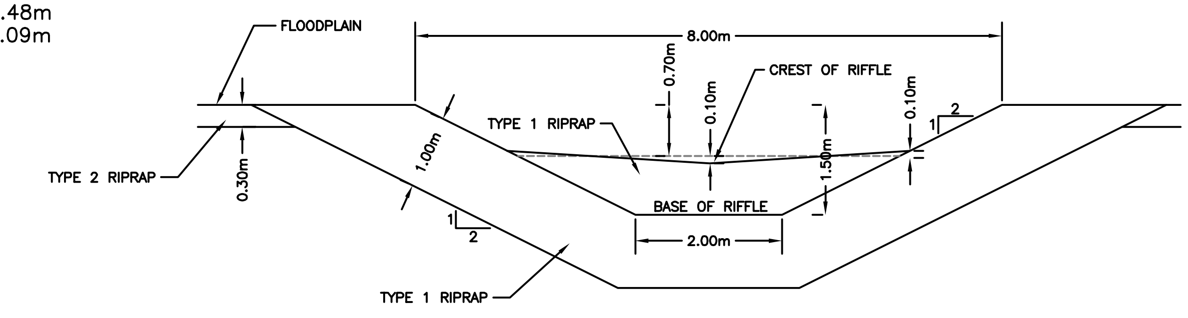


TYPICAL POOL SECTION

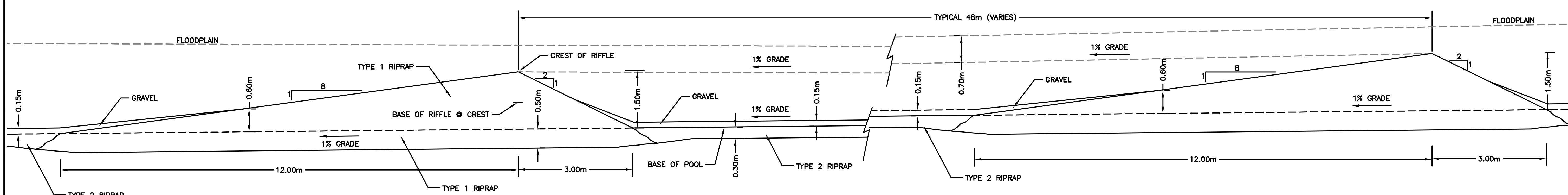
NOTE: TYPE 1 RIPRAP – D50 = 0.48m
TYPE 2 RIPRAP – D50 = 0.09m



TYPICAL FLOOD PLAIN EROSION PROTECTION (WITHIN DAM BREACH)



TYPICAL RIFFLE CREST SECTION



TYPICAL RIFFLE AND POOL PROFILE

NOTE: SCALE HAS BEEN REDUCED BY 50%

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DATE | DATE
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Deloitte & Touche Inc.

FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

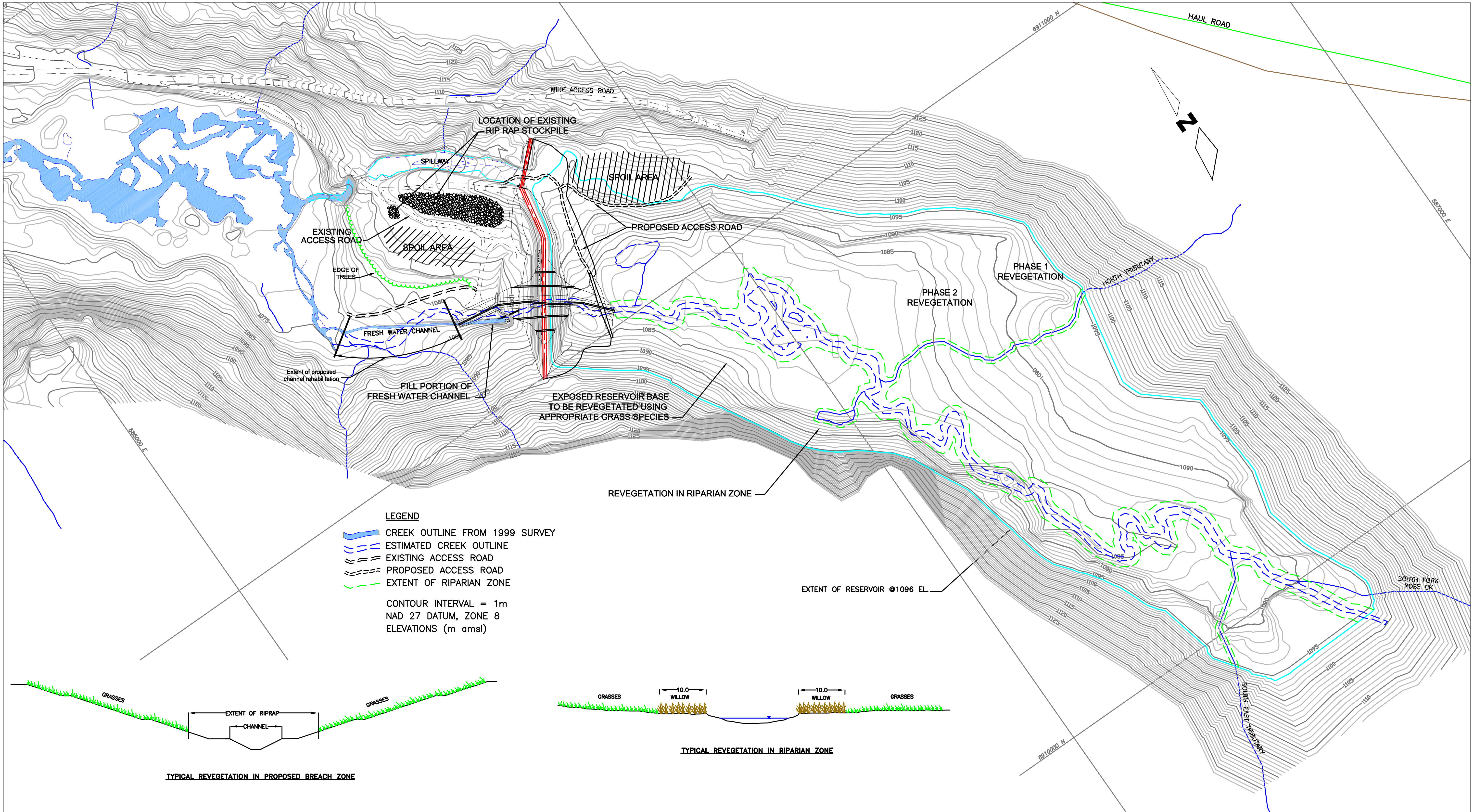
SCALE: 1:50
0 1 2 3 4 5 M

FWSD CHANNEL DETAILS

DRAWING NUMBER 8

REV. A

FILE NAME: F:\Deloitte_touche\Acad drawings-Faro\faro FWSD\Figure 8.dwg



NOTE: SCALE HAS BEEN REDUCED BY 50%

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DATE — DATE
SRK PROJECT NUMBER: 1CD003.20

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FRESH WATER SUPPLY DAM BREACH
ANVIL RANGE MINING COMPLEX, YT

SCALE: 1:3000
0 50 100 150 200 250 metres

CONSTRUCTION AREA AND REVEGETATION PLAN

DRAWING NUMBER: 9
REV.: A

Appendix A

Planning Meeting for Breach of Fresh Water Supply Dam

Meeting Dates: December 2/3, 2002
Meeting Location: Offices of SRK, Vancouver

Attendees:

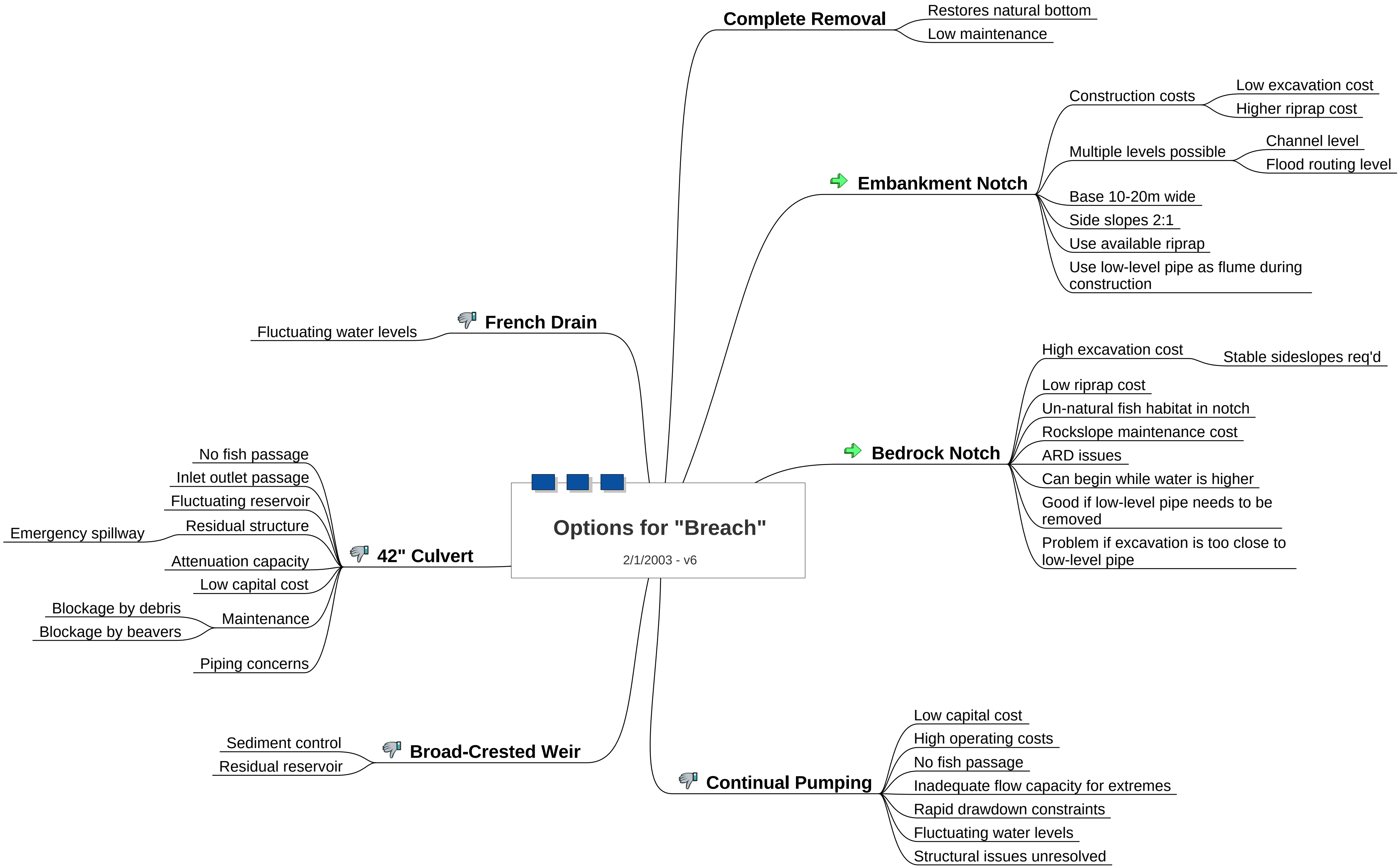
- DFO: Herb Klassen, Sandra Orban
- DIAND: Bud McAlpine, Bill Slater
- Hydroconsult: Wim Veldman
- Geo-Engineering: Milos Stepanek
- BGC: Jim Cassie, Gerry Ferris
- SRK: Daryl Hockley, Peter Healey, Cam Scott

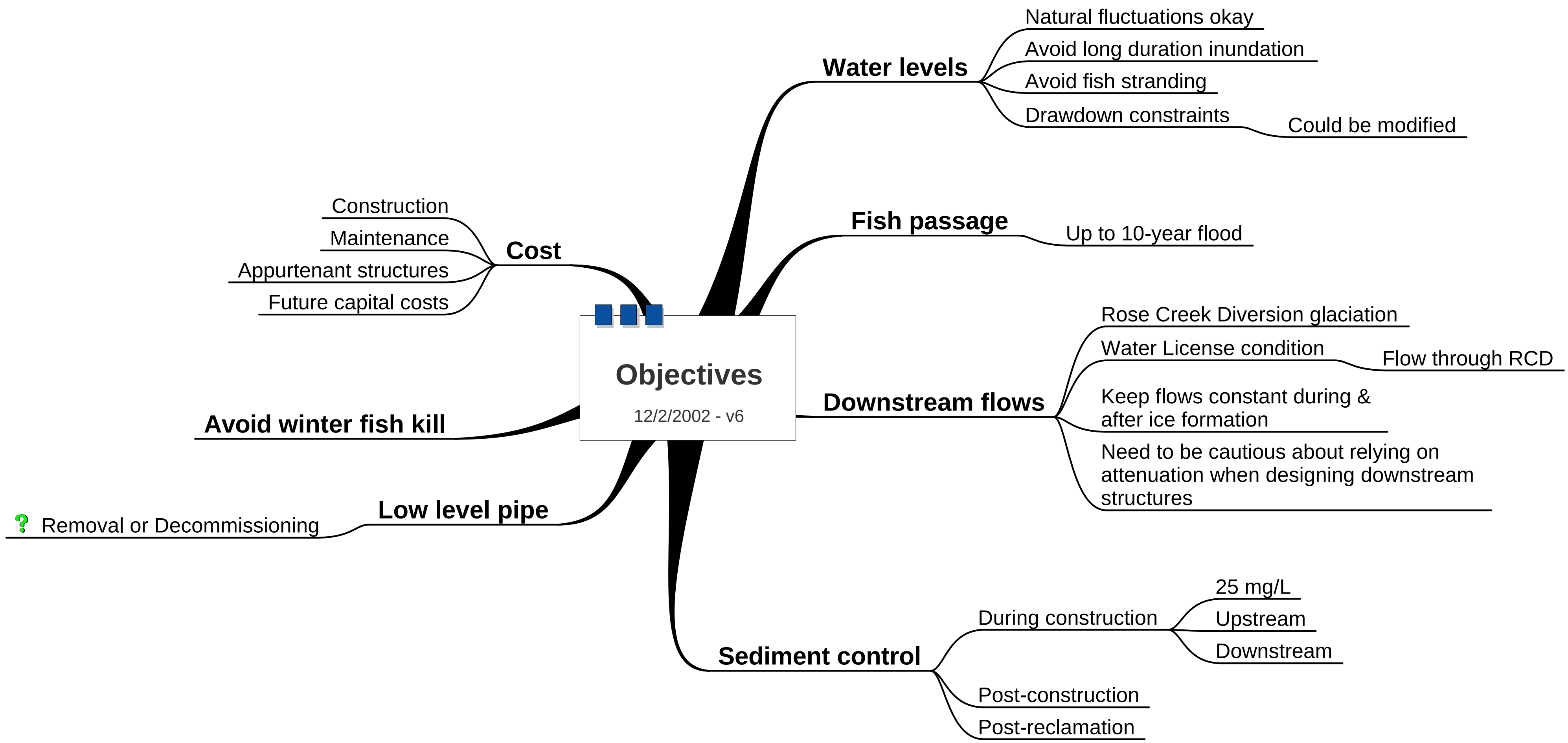
Conclusions:

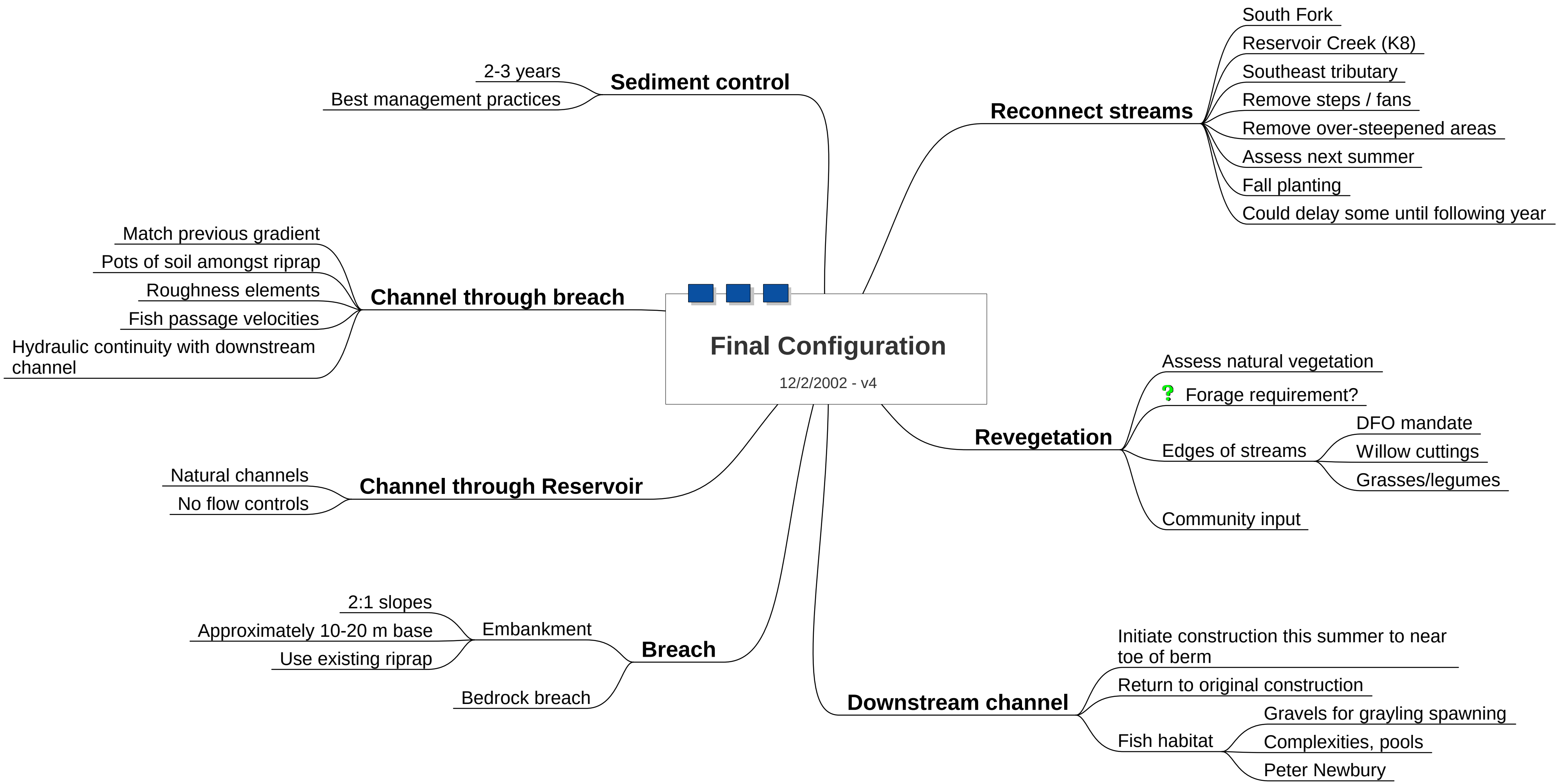
1. Seven options for the breach were considered (see attached Mindmap entitled "Options for Breach"). Four of them ("French drain", "42-inch culvert", "broad-crested weir", and "continual pumping") were rejected. Of the remaining three, "embankment notch" was preferred, "bedrock notch" should remain under consideration, and "complete removal" was judged to be a variant of embankment notch.
2. The objectives for the FWSD breach were clarified (see attached Mindmap entitled "Objectives"), as follows:
    - Keep water levels within the range of natural fluctuations.
    - Avoid long duration inundation.
    - Avoid fish stranding in higher ponds.
    - Respect dam stability constraints on dewatering rates.
    - Provide fish passage for up to 10-year flood.
    - Minimize risk of ice blockages, specifically by avoiding increases in flowrates after ice formation.
    - Control sediment releases.
    - Avoid creation of small volume pond that would result in winter fish kill.
    - Consider construction, maintenance, future construction, and appurtenant costs.
3. Further clarification is required regarding the DFO requests related to "removal" of the low level pipe and "attenuation" of floods.
4. Unnaturally maintaining flows through the Rose Creek Diversion is not an objective of this project. (The Water License requirement specifies the minimum Rose Creek Diversion flow, but does not require the FWSD to be the source of that flow. Furthermore, the License will be outdated prior to the start of construction.)
5. Upstream and downstream requirements associated with the breach were reviewed (see attached Mindmap entitled "Final Configuration"). Specific items to be considered in the design include:
    - Reconnecting streams – Need to remove obstacles to fish passage, including over-steepened delta areas. This work could be delayed until after freshet of 2004.

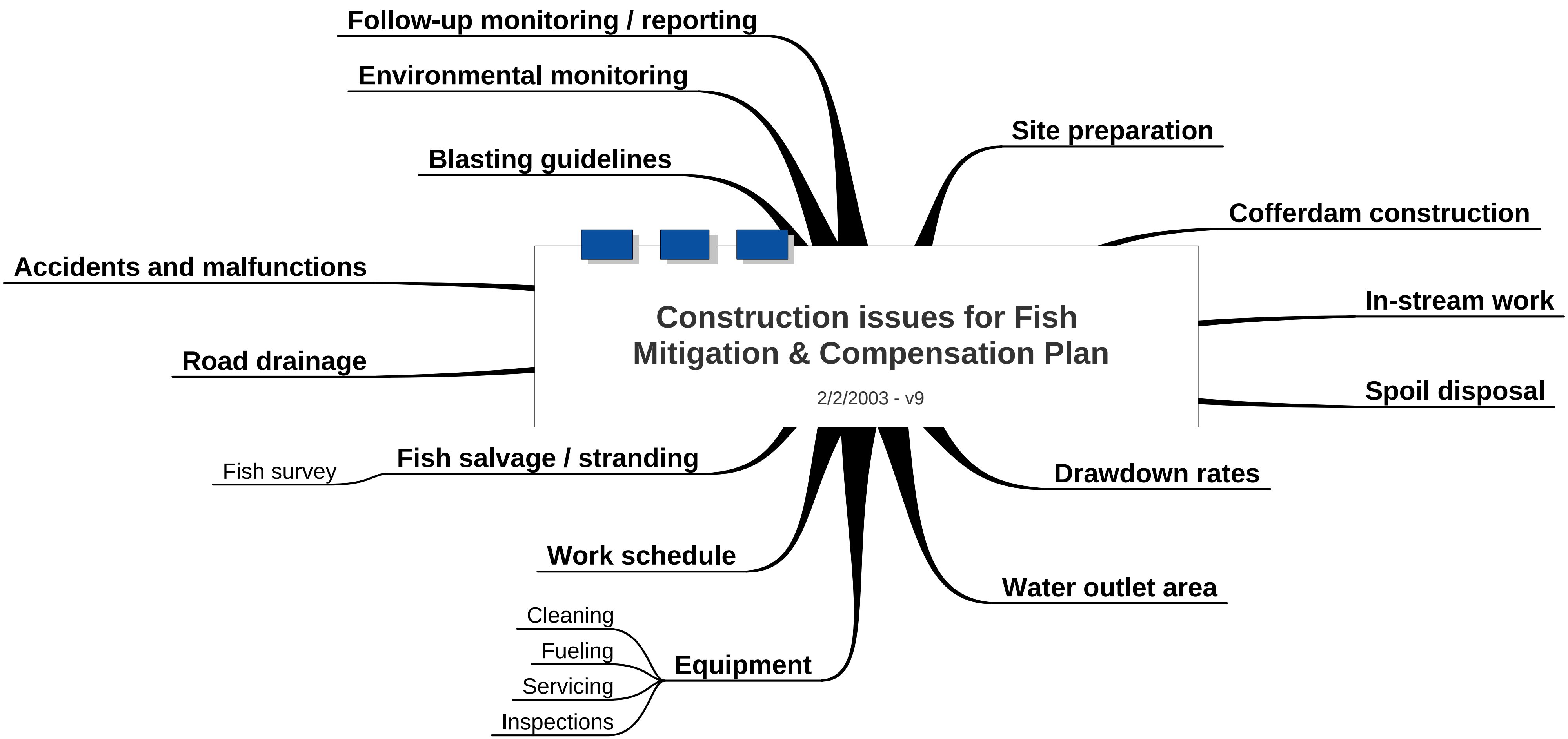
- Revegetation – Need to assess natural vegetation. Need community input. DFO mandate includes edges of streams, where willow cuttings and grasses/legumes should be considered.
- The channel through the reservoir should be natural, with no constructed flow controls.
- The channel through the breach should match previous gradients, should include roughness elements, and should be hydraulically continuous with downstream reaches.
- The downstream channel should link breach with original downstream alignment, and include fish habitat elements. Habitat creation here will compensate for loss of habitat elsewhere.

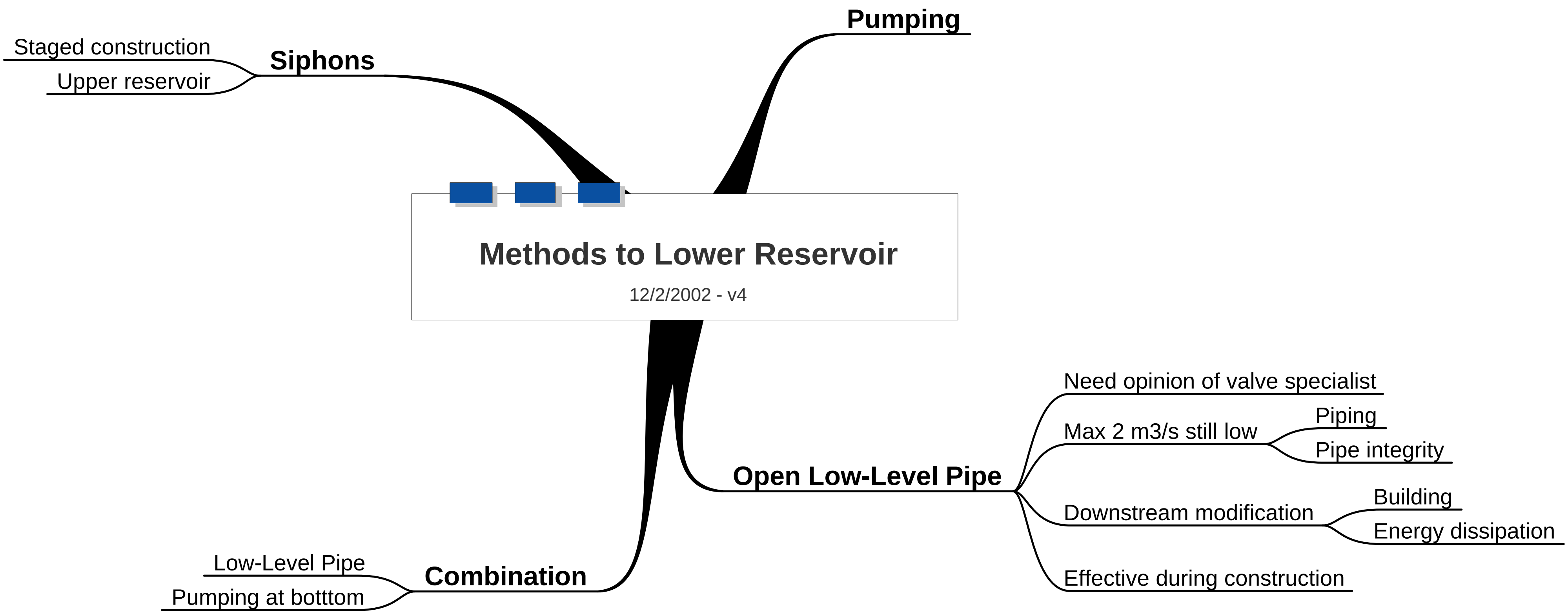
6. Requirements for the Fish Habitat Mitigation and Compensation Plan were reviewed (see attached Mindmap entitled "Construction Issues for Fish Mitigation & Compensation Plan"), and placed additional requirements on the design and construction planning:
   - A fish survey and fish salvage plan will be needed.
   - The plan should include commitments to environmental monitoring and follow-up monitoring and reporting.
   - Details of site preparation, coffer dam construction, in-stream work, spoil disposal, road drainage, water drawdown rates, and outlet point, where relevant to fish habitat will need to be provided.
   - A plan for cleaning, fueling, servicing and inspecting equipment that will be used in-stream will need to be provided.
   - Contingencies for accidents and malfunctions will need to be included.
   - Blasting guidelines will need to be provided.
7. Methods to lower the reservoir were discussed (see attached Mindmap entitled "Methods to Lower Reservoir"). The preference is to use the low level pipe to the maximum extent possible. There is an urgent need to evaluate the conditions of the valve and pump-house. There is no permitting concern with using either the low level pipe or properly screened pumps to lower water levels for one year, as long as there is fish salvage before the pond is completely removed.
8. A project schedule was developed. Key considerations are:
   - The Feb. 3 requirements from the DFO letter
   - Requirement for a Comprehensive Study
   - Requirements for a Water Licence
   - Dry construction and revegetation could be initiated in summer 2003
   - Breach construction should be completed by freshet 2004
   - Upstream construction and revegetation can be completed in summer 2004.
9. The permitting schedule is aggressive. Complications arising from YEAA have not been included in the current schedule.


Options for "Breach"
2/1/2003 - v6
Complete Removal
Restores natural bottom
Low maintenance
Embankment Notch
Construction costs
Low excavation cost
Higher riprap cost
Multiple levels possible
Channel level
Flood routing level
Base 10-20m wide
Side slopes 2:1
Use available riprap
Use low-level pipe as flume during construction
Bedrock Notch
High excavation cost
Stable sideslopes req'd
Low riprap cost
Un-natural fish habitat in notch
Rockslope maintenance cost
ARD issues
Can begin while water is higher
Good if low-level pipe needs to be removed
Problem if excavation is too close to low-level pipe
Continual Pumping
Low capital cost
High operating costs
No fish passage
Inadequate flow capacity for extremes
Rapid drawdown constraints
Fluctuating water levels
Structural issues unresolved
Broad-Crested Weir
Sediment control
Residual reservoir
42" Culvert
No fish passage
Inlet outlet passage
Fluctuating reservoir
Residual structure
Emergency spillway
Attenuation capacity
Low capital cost
Maintenance
Blockage by debris
Blockage by beavers
Piping concerns
French Drain
Fluctuating water levels


Objectives
12/2/2002 - v6
Water levels
Natural fluctuations okay
Avoid long duration inundation
Avoid fish stranding
Drawdown constraints
Could be modified
Fish passage
Up to 10-year flood
Downstream flows
Rose Creek Diversion glaciation
Water License condition
Flow through RCD
Keep flows constant during & after ice formation
Need to be cautious about relying on attenuation when designing downstream structures
Sediment control
During construction
25 mg/L
Upstream
Downstream
Post-construction
Post-reclamation
Cost
Construction
Maintenance
Appurtenant structures
Future capital costs
Avoid winter fish kill
Low level pipe
Removal or Decommissioning


Final Configuration
12/2/2002 - v4
Sediment control
2-3 years
Best management practices
Reconnect streams
South Fork
Reservoir Creek (K8)
Southeast tributary
Remove steps / fans
Remove over-steepened areas
Assess next summer
Fall planting
Could delay some until following year
Channel through breach
Match previous gradient
Pots of soil amongst riprap
Roughness elements
Fish passage velocities
Hydraulic continuity with downstream channel
Revegetation
Assess natural vegetation
Forage requirement?
Edges of streams
DFO mandate
Willow cuttings
Grasses/legumes
Community input
Channel through Reservoir
Natural channels
No flow controls
Breach
Embankment
2:1 slopes
Approximately 10-20 m base
Use existing riprap
Bedrock breach
Downstream channel
Initiate construction this summer to near toe of berm
Return to original construction
Fish habitat
Gravels for grayling spawning
Complexities, pools
Peter Newbury


Construction issues for Fish Mitigation & Compensation Plan
2/2/2003 - v9
Follow-up monitoring / reporting
Environmental monitoring
Blasting guidelines
Accidents and malfunctions
Road drainage
Fish salvage / stranding
Fish survey
Work schedule
Equipment
Cleaning
Fueling
Servicing
Inspections
Site preparation
Cofferdam construction
In-stream work
Spoil disposal
Drawdown rates
Water outlet area



E:\Deloitte_touche\1CD003.20_FWSD_Prelim_Breach_Design\Report\Appendix A\Methods to Lower Reservoir.mmp - 12/2/2002 - Daryl Hockley - dhockley@srk.com

Appendix B

Steffen Robertson and Kirsten (Canada.) Inc.
Suite 800, 1066 West Hastings St.
Vancouver, BC.
V6E 3X2

email: vancouver@srk.com
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Tel: 604.681.4196
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MEMORANDUM

DATE: February 2, 2003

TO: File 1CD003.20

FROM: Pat Bryan

RE: Hydraulic and Hydrology – Preliminary Design – FWSD Breach

A preliminary design for the breach of the FWSD has been prepared to provide a basis for initiating the permitting process. This memorandum documents the hydraulic and hydrology analyses that were undertaken in support of this preliminary design. These analyses included:

- estimation of average channel size and planform geometry for a restored channel that will be stable through the excavated notch and downstream of the dam;
- estimation of rock sizes to provide erosion protection of the dam abutments and to mimic the armouring process within the channel of the South Fork of Rose Creek;
- characterization of the reservoir inflows during the proposed drawdown period (August to November) and the construction period (December to March); and,
- characterization of the summer low flow regime of the South Fork of Rose Creek.

The rock size determinations required estimation of the hydraulics of the restored channel of the South Fork of Rose Creek. For the purpose of the preliminary design, the hydraulics were characterized using manual calculations or algorithms programmed in spreadsheets. This provided only a simplified representation of the complex hydraulics of the restored channel. A quasi-2d hydraulic model (HEC-RAS) will likely be used in the final design of the stream restoration to provide a more detailed and accurate characterization of the stream's hydraulics. The newest release of this model provides a means for estimating the shear stress imposed on the bed and banks of stream channels. This feature may be exploited to examine the stability of the constructed channel under a range of flows from bankfull to the 500-year peak instantaneous flood.

The various hydraulic and hydrologic analyses performed for the preliminary design are described below under separate headings.

Bankfull Discharge

An estimate of bankfull discharge was required to perform the interrelated tasks of: i) applying hydraulic geometry formulas; ii) simulating the hydraulics of the main channel of the restored South Fork of Rose Creek; and iii) estimating the size of rock required to form riffles.

For the purpose of the preliminary design, the bankfull discharge was assumed to be the 2-year peak instantaneous flood. A hydrology study prepared by Northwest Hydraulic Consultants (2001) estimated the magnitude of this flood at the FWSD to be 5.6 m^3/s.

Some evidence exists to suggest that the true bankfull discharge may be greater than 5.6 m^3/s. A soils investigation report prepared for the FWSD construction (Ripley, Klohn & Leonoff, 1968) describes the original stream at the dam site as being “incised in a shallow meandering channel that is about 20 ft wide and about 5 ft deep”. Given the valley slope of 1%, assumed channel sides lopes of 1.5H:1V and a Manning’s n of 0.05, these channel dimensions suggest a bankfull discharge of about 10 m^3/s. A discharge of 10 m^3/s has an estimated return period between 4 and 5 years (based on nhc 2001).

Stream Morphology

Preliminary values of channel size and planform geometry for the preliminary channel design were obtained from empirical relationships developed largely in western Canada. The empirical relationships provided the following guidelines for the design of the channel:

1) the slope of the channel (approx. 1%) is consistent with a riffle-pool morphology (Anon., 1996);
2) the pool-riffle-bar unit has a characteristic spacing of five to seven channel widths (Church, 1992);
3) for a stable channel, pools comprise about 50% to 75% of the channel length and have a wide range of depths (Anon., 1996);
4) the average width of a stable channel with a bankfull discharge of 5.6 m^3/s should fall in the range of 8 to 12 m, based on hydraulic geometry relationships for gravel rivers in Canada (Carson and Griffiths, 1987); and,
5) the soils investigation report referred to in the previous section suggests the actual channel width was about 6 m.

Based on the above information, an average width of 8 m was adopted for the main channel. This width set the average length of the pool-riffle units at 48 m (or six times the width).

Main Channel Depth

The depth of the main channel at the riffles will be a function of two factors: i) the minimum required depth of the pools to provide suitable over-wintering habitat for Arctic grayling and, ii) the water level just upstream of riffle during passage of the bankfull discharge.

For the first factor, based on the measured ice thickness on the reservoir and experience with river environments it was estimated that the minimum pool depth be 1.5 m. Thus, to provide the required pool depth during low flow periods, the crest of any riffle must be at least 1.5 m above the bottom of the upstream pool.

For the second factor, it was recognized that the riffles would act as trapezoidal weirs during passage of the bankfull discharge. The hydraulics of the riffle crest can be approximated by using a combination of equations for the broad-crested rectangular and triangular weirs. For channel side slopes of 2H:1V, the combination equation may be expressed (Smith, 1992):

$$\mathbf{Q = 1.7\ B\ H^{1.5} + 2.54\ H^{2.5}}$$

where: Q = discharge (m^3/s);
B = bottom width of trapezoidal shape formed by riffle crest and channel banks (m); and,
H = total head required to pass flow through vertical constriction caused by the riffle (m).

Assuming that the cross-sectional area of the upstream pool is so large that the velocity head is negligible, H can be taken as the depth of the pool relative to the crest of the riffle. Given a bottom width of about 6 m and a bankfull discharge of 5.6 m^3/s, the depth of the channel must be about 0.6 m above the crest elevation of the riffle to ensure that the flows do not spill over into the floodplain. Thus, the floodplain level should be at least 2.1 m above the deepest portion of the channel.

Minimum Rock Size for Riffles

The rock for the riffles was sized to just be stable during passage of the bankfull discharge. This was judged to be a critical condition in which the rock would probably be subjected to its greatest shear stress. The hydraulic gradient (and hence shear stress) in the vicinity of the riffles should decrease as the flows spill onto the floodplain.

The estimation of the rock size for the riffles entailed two sets of computations. Firstly, the depth and velocity of flow on the riffle were estimated. Secondly, the size of rock required to resist the imposed fluid and gravity forces was assessed. This was an iterative process because the flow hydraulics and the rock size depend on each other.

The hydraulics were simulated using the Manning's equation. The resistance caused by the rock (Manning's n) was estimated by an empirical equation that is a function of the rock's median diameter (Strickler equation). The rock size required to resist the imposed forces was estimated by two different techniques, one developed by the US Department of Agriculture (Robinson, Rice and Kadavy, 1998) and the other by the civil engineering department at the University of Saskatchewan (Smith and Kells, 1995). The two methods were selected because they: i) examine rock on steep slopes subject to downslope flow; and, ii) provide estimates of rock sizes that are just stable under the imposed forces (i.e., F.O.S. = 1).

Table 1 presents a printout of the spreadsheet calculations. For an assumed riffle slope of 10% and a design discharge of 5.6 m^3/s (bankfull), the required D_{50} for the riffle rock is about 240 mm (about 10 inches) for a factor of safety of one. Assuming a shape halfway between a cube and sphere, this size of rock weighs 58 lb. The USDA and University of Saskatchewan methods provided similar estimates of rock size.

The USDA method computes the stable median diameter for angular rock. A larger diameter is required for rounded rock (details are provided in Robinson et al, 1998).

Rock Size for Floodplain

The floodplain within the excavated notch of the dam will be protected with a constructed armouring layer. This layer will be designed to just be stable during the passage of the 500-year return period flood (estimated to be 63 m^3/s). A rough estimate of the required rock size was made using the same spreadsheet described above for the riffles.

For the purpose of the preliminary design, the spreadsheet was set up to provide a conservative (high) estimate of the rock size. In essence, the main channel and the floodplain were collectively represented as a single trapezoidal channel with a 20 m bottom width, 2H:1V side slopes and a low estimate for Manning's n. This likely had the effect of overestimating the depths and velocities that will actually be experienced in the floodplain (to be checked with a backwater model).

Output from the spreadsheet is presented in Table 2. The required size of rock on the floodplain will be about 90 mm (about 4 inches), with a weight of 1.4 kg (3 lb).

Rock Size for Erosion Protection of Dam Abutments

The riprap protection along the dam abutments will be designed to remain stable during passage of the 500-year peak instantaneous flood. A rough estimate of the required rock size was made using a two step process.

Firstly, a spreadsheet was developed that represented the restored channel as a simple compound channel (i.e., a trapezoidal main channel set within a trapezoidal flood plain). Various geometries and channel resistances were examined to estimate the average velocity in the main channel during passage of the 500-year flood. The depth and width of the main channel were adjusted to represent different degrees of potential scour during passage of this flood. This analysis indicated that the average velocity in the main channel would be about 3.1 m/s.

The second step was to estimate the size of rock that would resist the imposed fluid stress. The assumptions were made that: i) the main channel was impinging directly against the dam abutments; and, ii) the maximum velocity on the riprap was 4/3 of the average velocity in the main channel. The required rock size was estimated using the riprap design chart prepared by the B.C. MoT for banks with a side slope of 2:1. The curve presented on this chart can be expressed by a power function as follows:

$$\mathbf{D_{50} = 0.0249\ V^{2.09}}$$

where: D_{50} = median particle size (m); and,
V = velocity against bank (m/s).

With an estimated velocity of 4.1 m/s against the bank, the required median riprap size is approximately 480 mm (about 1½ ft). Assuming a spherical shape and a specific gravity of 2.7, this rock would weigh 159 kg (350 lb).

As a check, the riprap was also sized using the method recommended by the U.S. DOT (Richardson et al., 2001). Assuming a stability factor of 1.2, this method provided a D_{50} estimate of 500 mm, or a value comparable to that determined by the B.C. MoT technique. The mechanics of applying the U.S. DOT method are set out in Table 3.

High Inflow Rates to FWS Reservoir during Draining Period

The drawdown of the fresh water supply (FSW) reservoir is tentatively scheduled to begin in mid August. Allowance has been made for up to four months to complete the drain down. The actual time required for the drawdown will depend on the inflows to the reservoir and the controlled release rate from the low-level outlet. This section is aimed at estimating the potential inflow rates that could occur during the period August 1 to November 30 under wet climatic conditions.

The inflows were estimated by a technique known as Regional Analysis. This involved transposing hydrological information from regional streamflow gauging stations to the ungauged site of the FWSD. Four broad steps were required to develop the Regional Analysis.

The first step involved identifying streamflow gauging stations in the region that have reasonably long records (>10 years), command relatively small drainage areas and experience comparable climatic conditions as the mine site. A search of the networks of stations operated by the Water Survey of Canada (WSC) and Indian and Northern Affairs (IANA) revealed a total of five candidate stations for the Regional Analysis (see Table 4).

The second step entailed assembling raw flood data from the daily streamflow records of the five selected stations. From each of these records, a total of ten annual series of flood values were extracted. All of these annual series had one common characteristic, they contained a chronological list of the largest flow that occurred each year during the period August 1 to November 30. The differences in the ten annual series related to the period over which the largest flow was defined. The defined periods were 1, 2, 3, 4, 7, 10, 30, 60, 90 and 122 consecutive days. The last duration equals the length of the period from August 1 to November 30.

The third step involved fitting a theoretical frequency distribution (Log-Pearson Type III) to each annual series assembled in the second step. This meant a total of 50 fittings were undertaken (i.e., 5 stations with 10 annual series per station). The fitted frequency distributions were then used to predict the magnitude of the August to November flows at the regional stations for return periods ranging from 2 to 100 years and durations ranging from 1 day to 122 days. The second and third steps were performed using a suite of computer programs developed by the U.S. Geological Survey (USGS) for processing hydrological data (viz., IOWDM4.1, SWSTAT4.1 and ANNIE4.1). Table 4 summarizes the results of the third step. To facilitate comparison of the floods generated on the differently sized catchments, the flood values in Table 4 have been expressed as unit discharges in units of $L/s/km^2$ (i.e., the absolute flood discharge has been divided by the contributing catchment area).

The final step entailed selecting flood values from Table 4 to represent the inflows of the FWSD reservoir. Details for this final step are provided in the footnotes of the table.

For a typical year (return period of 2 years), the average inflow to the FWSD reservoir during the period August to November is estimated to be 500 L/s (i.e., 7.5 $L/s/km^2$ x 67 km^2). For an extremely wet year with a return period of 100 years, the average inflow over this same period is estimated to be 160% greater, or 1,270 L/s.

High Inflow Rates to FWS Reservoir during Construction Period

The excavation of the dam notch and restoration of the stream channel are slated for the period December 1 to March 31. This section presents estimates of potential high inflows to the reservoir during this period for a range of return periods.

The general procedure outlined above for the August to November period was adhered to in estimating the magnitude of potential high inflows during the construction period. Differences in the specific application of Regional Analysis to the construction period are outlined below:

1) The streamflow gauging station on Vangorda Creek could not be used because it was only operated seasonally during the open water season.
2) The streamflow record for a long-term station in the eastern interior of Alaska (Salcha River) was added to the analysis to include a data set that extends over a significantly longer period of record than available at the other selected stations.
3) The annual series were extracted for the period December 1 to March 31.
4) The longest duration examined was changed from 122 to 121 days (i.e., except for leap years, the period December 1 to March 31 is one day shorter than the period August 1 to November 30).

The results of the analysis are presented in Table 5. The unit flows predicted for all return periods and all stations are extremely low and are consistent with baseflow conditions (i.e., the streamflow is derived from groundwater discharge). This suggests that no significant rainstorm or early snowmelt was ever recorded in the winter months (December 1 to March 31) at the five regional stations. This includes Salcha River, which has operated for 53 years.

Based on the above discussion, the system for diverting water around the construction site will only require a capacity equal to a high baseflow rate. The baseflow can be expected to be highest at the beginning of the construction period and then follow a recession curve through the remainder of the winter.

In a typical year (return period of 2 years), the peak daily inflow to the FWR during the December 1 to March 31 period is estimated to be 200 L/s. The average over the full four-month construction period would be approximately 115 L/s. The corresponding flows for a 100-year wet condition are 560 L/s and 280 L/s, respectively.

Before finalizing the design capacity of the diversion system, an examination should be made of other streamflow records in the Yukon to confirm that the period December 1 to March 31 can reasonably be expected to be devoid of significant rainstorms and periods of snowmelt.

Summer Low Flows

The restored channel of the South Fork of Rose Creek will be required to provide fish passage during low flow periods in the summer, requirement of fish passage under most flow conditions. This section describes the steps undertaken to estimate the magnitude of low flows in this stream during the period June 1 to September 30 for a range of return periods.

The general procedure for estimating the low flows was similar to that used for estimating the August to November flood flows, with the following differences:

1) The streamflow record for the South MacMillan River station was rejected for the low flow analysis. The mean annual runoff for this station is roughly double that of the South Fork of Rose Creek (624 mm vs. 311 mm). As a result, the South MacMillan River sustains a considerably greater flow in summer dry periods than does the South Fork (when expressed as a unit discharge).
2) The streamflow record for the North Fork of Rose Creek (Station R7) was included in the low flow analysis. This record covers a period of six years, which is adequate for estimating events of frequent occurrence (say, up to a 20-year return period).
3) Preparation of the annual series involved extracting the lowest flow in each year of record for the period June 1 to September 30.
4) The fitted frequency distributions were used to estimate low flows for a smaller range of return periods than specified for the flood analyses (viz., 2, 5, 10 and 20 years).

The results of the low flow analysis are presented in Table 6. During a typical year, the 7-day low flow for the period June 1 to September 30 is estimated to be 470 L/s at the site of the

FWSD. The 7-day, 10-year low flow event for this same location and period is computed to be 290 L/s. For comparison, the long-term average annual flow at the FWR is estimated to be 660 L/s.

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Table 1 Estimation of stable rock size for riffles (assuming 10% slopes)

Channel slope =	0.1 m/m			
Bottom width =	6 m			
Side slope =	2 H:1V			
Depth of flow (guess) =	0.272427 m			
Wetted perimeter =	7.218329 m			
Flow area =	1.782993 m^2			
Hydraulic radius =	0.247009 m			
Manning's n =	0.039633			
Discharge =	5.600615 m^3/s			
Average velocity =	3.141132 m/s			
Riprap d_{50} (guess) =	0.28 m			
Riprap d_{50} (computed) =	0.278815 m	278.8153 mm (FOS = 1.2)	96.38894 lb (assuming midway between sphere and cube)	
conservative unit discharge =	0.933436 m3/s/m	232.3461 mm (FOS = 1)	55.78063 lb (assuming midway between sphere and cube)	
Top width of free surface =	7.089707			
Froude number =	1.999821			
average flow width =	6.544853 m			
approximate unit discharge =	0.855728 m3/s/m			
USDA D50 (S < 0.1) =	224.2505 mm (FOS=1)	50.15075 lb (assuming midway between sphere and cube)		
conservative unit discharge =	0.933436 m3/s/m			
USDA D50 (S < 0.1) =	234.8044 mm (FOS=1)	57.56996 lb (assuming midway between sphere and cube)		

Table 2 Estimation of stable rock size for floodplain

Channel slope =	0.01	m/m				
Bottom width =	20	m				
Side slope =	2	H:1V				
Depth of flow (guess) =	1.015424	m				
Wetted perimeter =	24.54111	m				
Flow area =	22.37065	m^2				
Hydraulic radius =	0.911558	m				
Manning's n =	0.033383					
Discharge =	62.99975	m^3/s				
Average velocity =	2.816178	m/s				
Riprap d_{50} (guess) =	0.1	m				
Riprap d_{50} (computed) =	0.104638	m	104.6377	mm (FOS = 1.2)	5.094981	lb (assuming midway between sphere and cube)
conservative unit discharge =	3.149988	m3/s/m	87.19812	mm (FOS = 1)	2.948484	lb (assuming midway between sphere and cube)
Top width of free surface =	24.0617					
Froude number =	0.932501					
average flow width =	22.03085	m				
approximate unit discharge =	2.859615	m3/s/m				
USDA D50 (S < 0.1) =	68.28358	mm (FOS=1)	1.41588	lb (assuming midway between sphere and cube)		
conservative unit discharge =	3.149988	m3/s/m				
USDA D50 (S < 0.1) =	71.8686	mm (FOS=1)	1.650802	lb (assuming midway between sphere and cube)		

Table 3 Estimation of riprap size using DOT recommended method

1) Estimation of Shear Stress

Depth of flow (y_0) =	2	m
Grain roughness (k_s) =	0.4	m
Average velocity in vertical (V) =	4.1	m/s
Shear stress (t_0) =	159.0696	N/m2

2) Estimation of riprap size

Sideslope angle (?) =	26.56505	degrees	0.463648	radians	2	H:1 V
Angle of repose (f) =	42	degrees	0.733038	radians		
Angle of velocity field relative to horizontal (?) =	0	degrees	0	radians		
Riprap diameter (D_s) =	500.8879	mm	0.500888	m		
Specific weight of riprap (S_s) =	2.65					
Tractive force (t_0) =	160	N/m^2	2.424366	m/s (approx. velocity against stone - n/a where flow decelerating)		
Density of water (?) =	1000	kg/m^3	3.394113	m/s (approx. velocity a distance D50 from bed)		
Acceleration due to gravity (g) =	9.81	m/s^2				

Computation of riprap size

? =	0.414424823	
ß =	0.395240484	
? =	0.286995436	
S.F. =	1.199954378	(recommended = 1.5)

Table 4 Estimation of High Inflow Rates to FWR during Reservoir Draining Period (August 1 to November 30)

Streamflow Gauging Station		Length of Record	Catchment Area	Mean Annual Runoff	Authority [b]	Return Period	Average high discharge in L/s/km^2 for the following number of consecutive days:									
ID No.	**Name**	**(years)**	**(km^2)**	**(mm)**		**(years)**	**1**	**2**	**3**	**4**	**7**	**10**	**30**	**60**	**90**	**122**
09BA001	Ross River at Ross River	36	7250	293	WSC	2	18	18	17	17	16	15	13	11	9.4	7.7
						10	28	27	27	26	24	22	19	16	14	11
						25	32	31	30	29	27	25	21	18	16	12
						50	35	34	33	31	28	27	22	20	17	14
						100	37	36	35	33	30	28	23	21	18	15
09AD002	Sidney Creek at km 46	12	372	350	WSC	2	18	17	16	16	14	14	11	10	9.2	8.1
	South Canol Road					10	36	33	31	30	27	25	19	16	15	13
						25	48	44	41	39	35	33	23	19	18	15
						50	58	52	49	46	42	39	27	22	20	17
						100	70	62	58	54	50	45	31	24	22	19
09AG003	South Big Salmon River	15	515	246	WSC	2	19	18	17	17	15	15	13	11	9.4	7.8
	below Livingstone Creek					10	37	34	32	30	28	26	20	17	15	12
						25	49	43	40	38	34	31	24	20	17	15
						50	59	50	46	43	39	36	27	22	19	16
						100	69	58	52	49	44	40	30	24	21	18
09BB001	South MacMillan River	22	997	624	WSC	2	58	55	52	50	44	41	30	25	21	17
	at km 407 Canol Road					10	82	75	70	66	59	55	44	36	29	23
						25	90	82	76	71	64	61	51	42	33	26
						50	95	86	79	74	68	64	56	47	36	28
						100	99	89	82	77	71	68	61	52	39	30
29BC003	Vangorda Creek at	16	91.2	235	IANA	2	14	13	13	13	12	11	9.4	8.2	7.5	6.4
	Faro Townsite Road [a]					10	26	24	23	22	21	20	16	13	11	9.3
						25	34	30	29	28	26	25	20	16	13	11
						50	40	36	34	33	31	29	23	19	15	12
						100	47	41	39	38	35	34	26	21	16	13
Estimated Inflow Rates for the		n/a	67	311	n/a	2	**17**	**16**	**16**	**15**	**14**	**14**	**11**	**10**	**8.9**	**7.5**
Fresh Water Reservoir (FWR) [c, d]						10	**37**	**34**	**32**	**30**	**28**	**26**	**20**	**17**	**15**	**13**
						25	**49**	**44**	**41**	**39**	**35**	**33**	**24**	**20**	**18**	**15**
						50	**59**	**52**	**49**	**46**	**42**	**39**	**27**	**22**	**20**	**17**
						100	**70**	**62**	**58**	**54**	**50**	**45**	**31**	**24**	**22**	**19**

Notes:

a) The gauging station on Vangorda Creek is operated on a seasonal basis. Missing data within the daily record of this station were patched using a correlation with WSC Station 09BC001 (Pelly River at Pelly Crossing). The data for most of October and all of November were usually missing.

b) WSC = Water Survey of Canada and IANA = Indian and Northern Affairs

c) Two observations were made about the regional flood data contained in this table. Firstly, the unit flood values appear to be independent of drainage area for the longer durations (>30 days) and only weakly correlated for the shorter durations. Secondly, the unit mean annual runoff (MAR) appears to explain a significant amount of variation in flood magnitude between the stations (i.e., the unit flood increases with increasing MAR). Because of this latter observation, the flood data for the South MacMillan River were not used to characterize the flood regime of the FWR. The MAR of the South MacMillan River is about double that of the FWR.

d) Two approaches were used to estimate the inflows to the FWR during the reservoir draining period. For a return period of 2 years, the inflow rate was taken as the average of the predicted flows at the regional stations. For each of the other return periods, the inflow to the FWR was assumed equal to the maximum of the flows estimated at the regional stations. In most cases, Sidney Creek provided the largest unit flow value.

Table 5 Estimation of High Inflow Rates to FWR during Construction Period (December 1 to March 31)

Streamflow Gauging Station		Length of Record	Catchment Area	Mean Annual Runoff	Authority[a]	Return Period	Average high discharge in L/s/km² for the following number of consecutive days:									
ID No.	Name	(years)	(km²)	(mm)		(years)	1	2	3	4	7	10	30	60	90	121
09BA001	Ross River at Ross River	35	7250	293	WSC	2	2.3	2.2	2.2	2.2	2.1	2.0	1.8	1.5	1.3	1.2
						10	3.1	3.1	3.1	3.0	3.0	2.9	2.5	2.1	1.8	1.6
						25	3.5	3.5	3.4	3.4	3.4	3.3	2.9	2.3	2.0	1.7
						50	3.8	3.8	3.7	3.7	3.6	3.6	3.2	2.6	2.1	1.8
						100	4.0	4.0	4.0	4.0	3.9	3.8	3.4	2.7	2.3	1.9
15484000	Salcha River near Salchaket	53	5618	261	USGS	2	2.0	2.0	1.9	1.9	1.9	1.9	1.7	1.5	1.3	1.2
						10	3.0	3.0	3.0	3.0	2.9	2.9	2.6	2.3	2.0	1.9
						25	3.5	3.5	3.4	3.4	3.4	3.3	3.0	2.7	2.4	2.2
						50	3.8	3.8	3.8	3.8	3.7	3.7	3.3	2.9	2.7	2.5
						100	4.2	4.1	4.1	4.1	4.0	4.0	3.6	3.2	3.0	2.7
09AD002	Sidney Creek at km 46 South Canol Road	11	372	350	WSC	2	3.9	3.9	3.9	3.8	3.8	3.7	3.5	3.1	2.9	2.7
						10	5.9	5.8	5.8	5.7	5.6	5.4	4.9	4.3	3.8	3.4
						25	6.9	6.8	6.7	6.7	6.5	6.3	5.7	4.9	4.1	3.7
						50	7.6	7.6	7.5	7.4	7.2	6.9	6.3	5.3	4.4	4.0
						100	8.4	8.3	8.2	8.1	7.8	7.6	6.9	5.8	4.7	4.1
09AG003	South Big Salmon River below Livingstone Creek	14	515	246	WSC	2	2.3	2.2	2.2	2.2	2.1	2.1	1.8	1.5	1.2	1.1
						10	4.7	4.7	4.6	4.6	4.5	4.4	4.0	3.3	2.7	2.3
						25	6.1	6.0	5.9	5.9	5.8	5.7	5.3	4.3	3.6	3.1
						50	7.1	7.0	7.0	6.9	6.7	6.7	6.3	5.1	4.3	3.6
						100	8.2	8.0	8.0	7.9	7.7	7.7	7.3	6.0	5.0	4.2
09BB001	South MacMillan River at km 407 Canol Road	21	997	624	WSC	2	4.3	4.2	4.2	4.1	4.0	3.9	3.4	2.8	2.4	2.1
						10	5.9	5.9	5.8	5.8	5.6	5.4	4.7	3.9	3.3	2.9
						25	6.7	6.6	6.6	6.6	6.3	6.1	5.2	4.4	3.7	3.2
						50	7.3	7.2	7.2	7.1	6.8	6.6	5.7	4.8	3.9	3.4
						100	7.9	7.8	7.7	7.7	7.3	7.0	6.1	5.2	4.2	3.6
Estimated Inflow Rates for the Fresh Water Reservoir (FWR) [b]		n/a	67	311	n/a	2	**2.9**	**2.9**	**2.9**	**2.9**	**2.8**	**2.7**	**2.4**	**2.1**	**1.8**	**1.7**
						10	**5.9**	**5.9**	**5.8**	**5.8**	**5.6**	**5.4**	**4.9**	**4.3**	**3.8**	**3.4**
						25	**6.9**	**6.8**	**6.7**	**6.7**	**6.5**	**6.3**	**5.7**	**4.9**	**4.1**	**3.7**
						50	**7.6**	**7.6**	**7.5**	**7.4**	**7.2**	**6.9**	**6.3**	**5.3**	**4.4**	**4.0**
						100	**8.4**	**8.3**	**8.2**	**8.1**	**7.8**	**7.7**	**7.3**	**6.0**	**5.0**	**4.2**

Notes: a) WSC = Water Survey of Canada and USGS = United States Geological Survey

b) Two approaches were used to estimate the inflows to the FWR during the construction period. For a return period of 2 years, the inflow rate was taken as the average of the predicted flows at the five regional stations. For each of the other return periods, the inflow to the FWR was assumed equal to the maximum of the flows estimated at the regional stations. In most cases, Sidney Creek or South Big Salmon River provided the largest unit flow value.

Table 6 Estimated Low Flows in the South Fork of Rose Creek During the Summer (June 1 to September 30)

Streamflow Gauging Station		Length of Record	Catchment Area	Mean Annual Runoff	Authority [c]	Return Period	Average low discharge in L/s/km^2 for the following number of consecutive days:					
ID No.	Name	(years)	(km^2)	(mm)		(years)	1	7	30	60	90	122
09BA001	Ross River at Ross River	36	7250	293	WSC	2	6.7	7.1	8.7	10	13	18
						5	4.9	5.2	6.2	7.6	9.5	14
						10	4.1	4.4	5.1	6.2	8.0	12
						20	3.4	3.7	4.3	5.3	6.8	11
09AD002	Sidney Creek at km 46 South Canol Road	12	372	350	WSC	2	6.7	7.0	8.3	9.4	13	22
						5	5.3	5.6	6.5	7.3	9.4	17
						10	4.7	4.9	5.8	6.5	8.0	14
						20	4.3	4.5	5.3	5.9	6.9	12
09AG003	South Big Salmon River below Livingstone Creek	14	515	246	WSC	2	6.6	7.2	8.5	10	12	16
						5	5.5	5.8	6.6	7.7	9.2	12
						10	5.1	5.2	5.9	6.8	7.9	10
						20	4.8	4.9	5.5	6.1	7.0	9.4
29BC003	Vangorda Creek at Faro Townsite Road [a]	15	91.2	235	IANA	2	4.8	5.2	6.4	7.6	9.2	13
						5	3.7	4.0	4.8	5.7	7.2	9.9
						10	3.2	3.5	4.2	5.0	6.4	8.6
						20	2.9	3.2	3.7	4.6	5.7	7.6
R7	North Fork of Rose Creek above Faro Creek Diversion Channel [b]	6	95	332	ARMC	2	7.6	8.2	11	13	14	19
						5	6.5	6.9	8.1	9.7	11.0	14
						10	6.1	6.4	7.1	8.5	9.7	12
						20	5.9	6.2	6.3	7.6	8.6	10
Estimated Summer Low Flows for South Fork of Rose Creek at Fresh Water Reservoir [d]		n/a	67	311	n/a	2	6.6	7.0	8.3	9.4	12	16
						5	4.9	5.2	6.2	7.3	9.2	12
						10	4.1	4.4	5.1	6.2	7.9	10
						20	3.4	3.7	4.3	5.3	6.8	9.4

Notes:
a) The gauging station on Vangorda Creek is operated on a seasonal basis. Missing data within the daily record of this station were patched using a correlation with WSC Station 09BC001 (Pelly River at Pelly Crossing).

b) The flow record for the North Fork of Rose Creek (NFRC) gauging station is probably less accurate than the records of the other stations in this table, largely because fewer direct discharge measurements are made at the NFRC station than at the government stations. Missing and suspect data within the NFRC daily record were patched using a correlation with WSC Station 09BA001 (Ross River at Ross River).

c) WSC = Water Survey of Canada, IANA = Indian and Northern Affairs, and ARMC = Anvil Range Mining Corporation (Interim Receivership)

d) The low flows for the South Fork of Rose Creek were estimated to equal the minimum of the flows predicted for the three WSC Stations. The adopted flows are bounded by the predicted flows for Vangorda Creek and NFRC.

Appendix C

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Engineers and Scientists

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MEMORANDUM

DATE: February 2, 2003

TO: File 1CD003.20

FROM: Jim Robertson / Cam Scott

RE: Stability Analysis – Preliminary Design – FWSD Breach

Preliminary stability analyses have been completed to evaluate the stability of the cut slopes on either side of the breach. The analyses were completed using the Bishop's method for circular failure surfaces within the slope stability program, Slope/W (GEO-SLOPE, 1998). The Bishop's method calculates the factor of safety by satisfying moment equilibrium.

Cases Considered

The slope stability analyses considered the critical section, i.e. through the exposed core of the dam at the location where the dam height is a maximum. In addition, the slope stability analyses considered two different side slopes within the breach (2.5H:1V and 3H:1V) and the following three basic loading conditions:

- Immediately following excavation of the breach (referred to as "end of construction"),
- Long term, under static loading, and
- Long term, under earthquake loading.

The end of construction case is considered to be a transient condition corresponding to relatively high pore pressures that are likely to exist as a result of the long-term seepage through the dam. The corresponding factor of safety is expected to increase as the phreatic levels fall in response to "drain down" of the slopes on either side of the breach.

The long-term conditions are applicable after phreatic levels have dropped and a new set of equilibrium conditions have been established at the structure.

Piezometric Assumptions

The "end of construction" piezometric condition assumed a relatively high surface, with the line running from the channel bottom up to the toe of the side slope, then up to a point 5m vertically below the crest. The assumed long-term piezometric line runs upward at a slope of about 8H to 1V from the toe of the slope. The piezometric lines are shown on the attached figures.

Earthquake Loading

The earthquake loading condition was evaluated using the pseudo-static method of analysis. The earthquake parameters included the 2003 estimations of the peak ground accelerations (PGA) compiled by the Pacific Geoscience Centre. Specifically, they included the PGA from the 1 in 475-year seismic event (0.06g) and, for comparison, the PGA from the 1 in 10,000-year event (0.16g).

Material Properties

The material properties used in the analyses are summarized in Table 1. In the case of the friction angle, the typical range of values that might be appropriate for each of the material types is also provided. Based on these values, the friction strengths associated with the core and foundation till are somewhat conservative. In the case of the shell material, the friction angle used in the analyses is generous. Assuming zero cohesion for the core and foundation materials is judged to be conservative for the "end of construction" case, but appropriate for the long-term cases.

Table 1
Material Properties

Material	**Description**	**Effective Friction Angle (°)**		**Cohesion (kPa)**	**Unit Weight (kN/m³)**
		Used in Analyses	Typical range (possible)		
Core (Zone 1)	Glacial till	27°	27° to 36°	0	20
Shell (Zone 2)	Sand and gravel	38°	32° to 40°	0	20
Foundation Till	Silty, sandy gravel	30°	28° to 36°	0	20

As a comparison, the values used in stability analyses by Golder Associates (1989) and BGC (2001) are summarized in Table 2 and 3, respectively.

Table 2
Soil Strength Parameters used by Golder (1989)

Material	**Effective Friction Angle Values Used**	**Effective Cohesion (kPa)**	**Unit Weight (kN/m³)**
Core, Random Fill and Shell Zones	33°	0	20
Foundation	30°	0	20

Table 3
Material Properties used by BGC (2001)

Material	Range of Effective Friction Angles	Effective Cohesion (kPa)	Unit Weight (kN/m^3)
Core and Random Fill Zones	27° – 36°	0	20
Shell	33° & 38°	0	20
Foundation	30°	0	20

Results

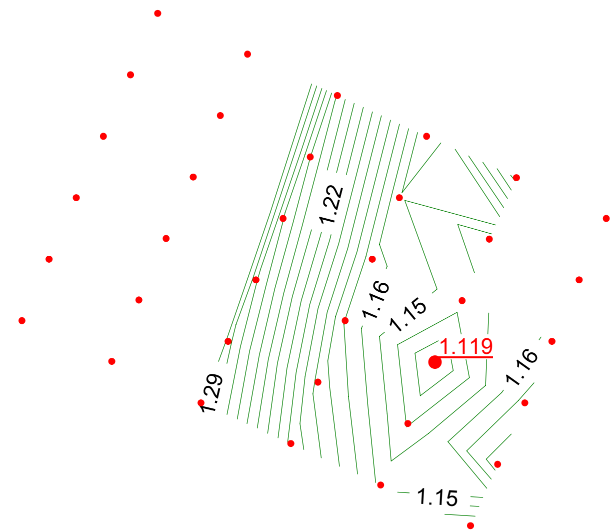
The results are summarized in Table 4, along with the minimum factor of specified in Section 3.7 of the main text. The outputs from individual results are attached.

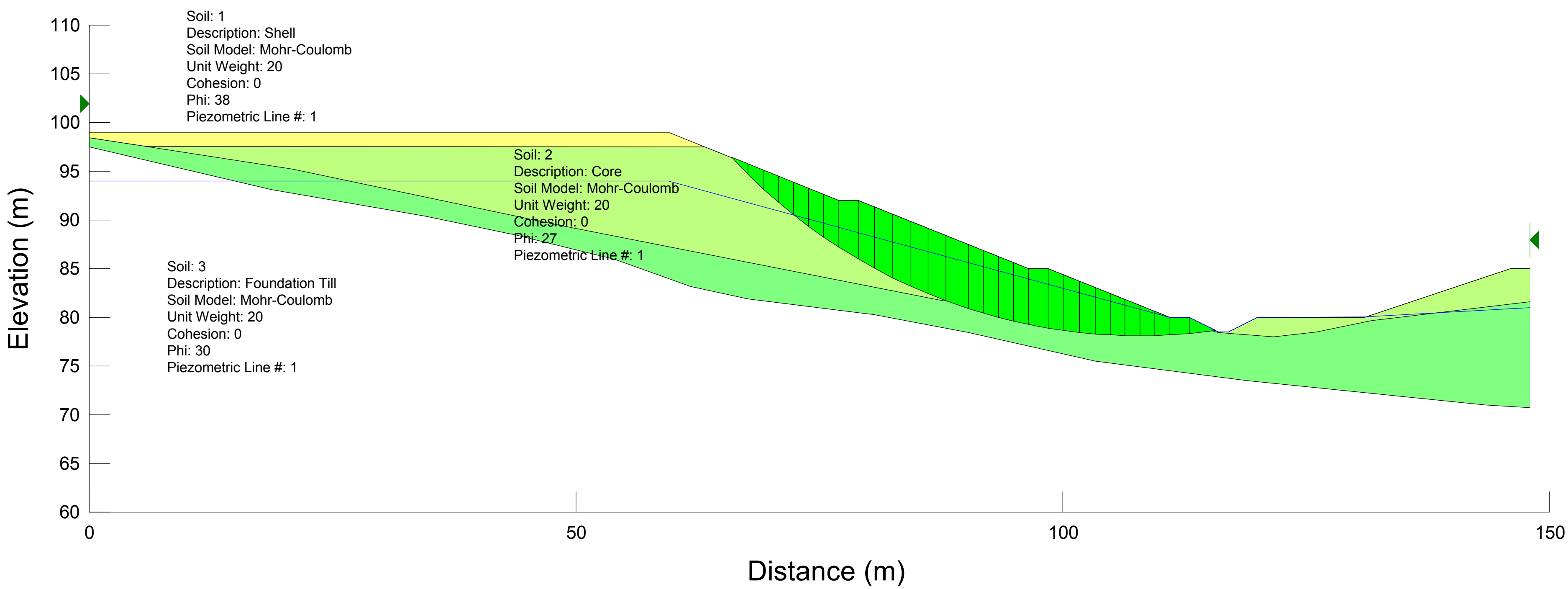
Based on this preliminary analysis the side slopes of the overall breach were chosen as 3H:1V. Further analysis will be performed as part of the final design. The final design slope stability analysis will include a sensitivity analysis of the soil properties.

Table 4
Results of Preliminary Stability Analysis

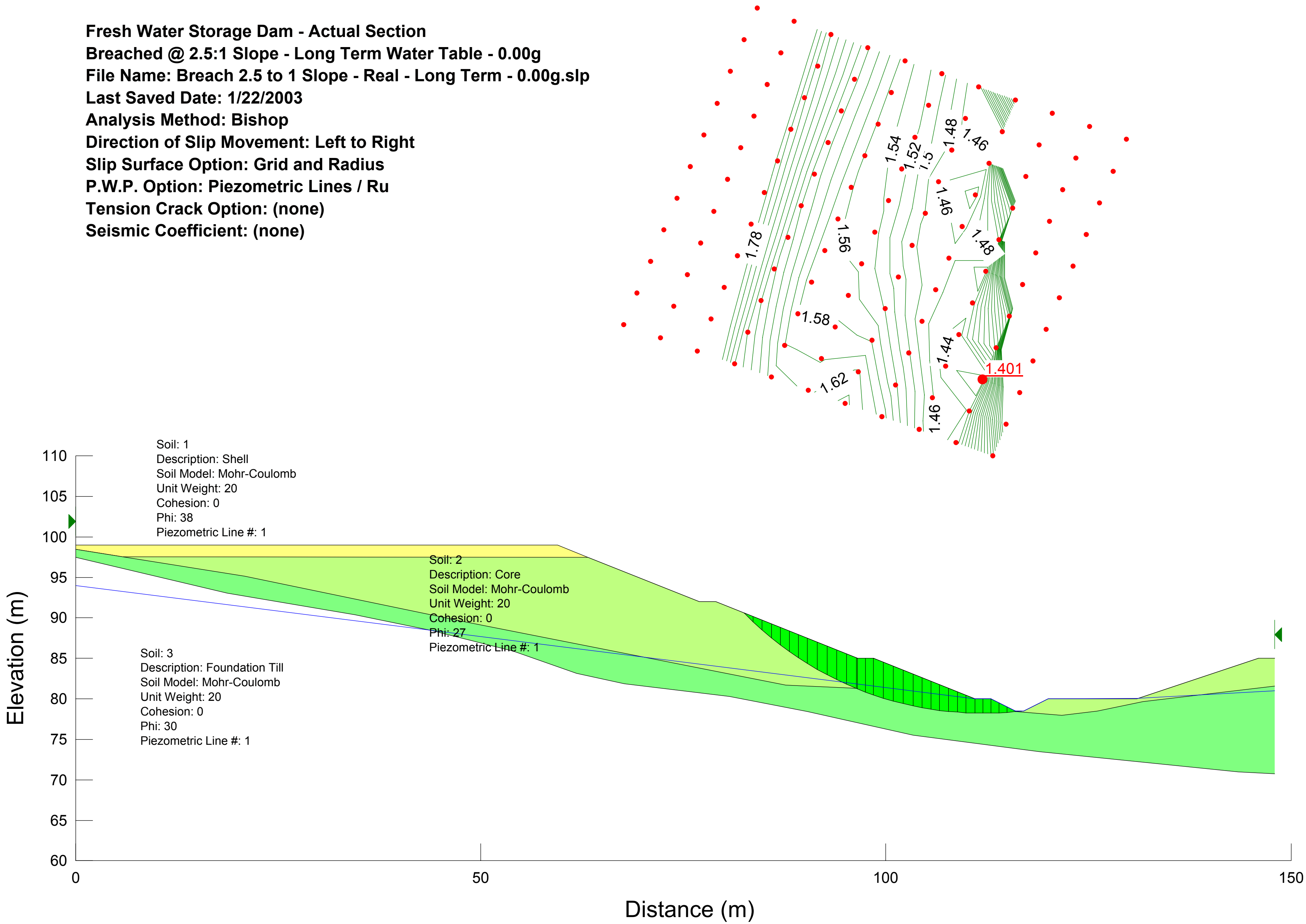
Condition Analysed	Calculated Factor of Safety		Minimum Factor of Safety
	Side Slope of Breach		
	2.5H:1V	3H:1V	
Static, immediately post-construction	1.12	1.29	1.3
Static, long term	1.40	1.59	1.3
Pseudo-static (g=0.06), long term	1.17	1.30	1.0
Pseudo-static (g=0.16), long term	0.90	0.98	1.0

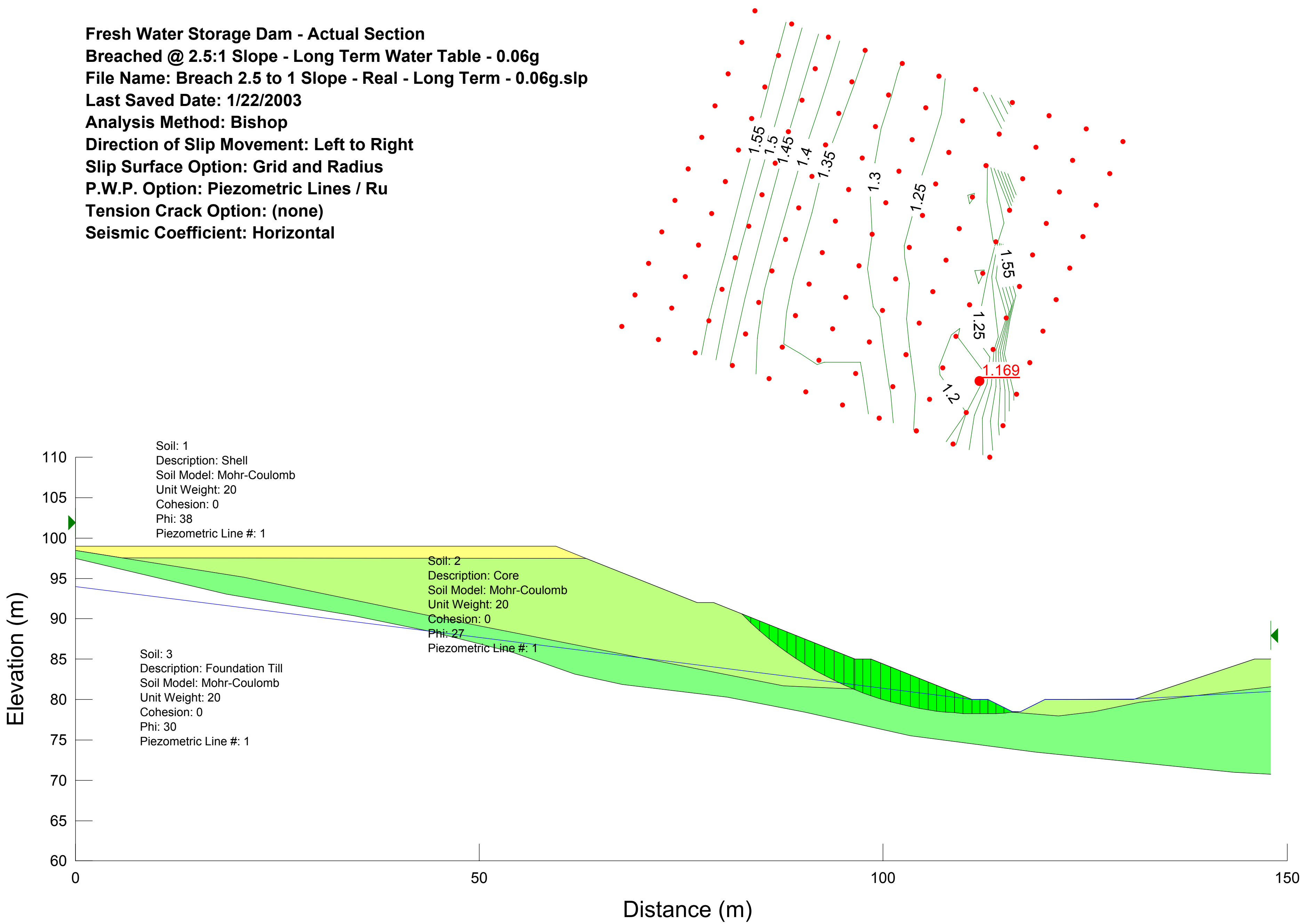
Fresh Water Storage Dam - Actual Section
Breached @ 2.5:1 Slope - Post Construction Water Table
File Name: Breach 2.5 to 1 Slope-Real-Post Con - 0.00g.slp
Last Saved Date: 1/22/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: (none)



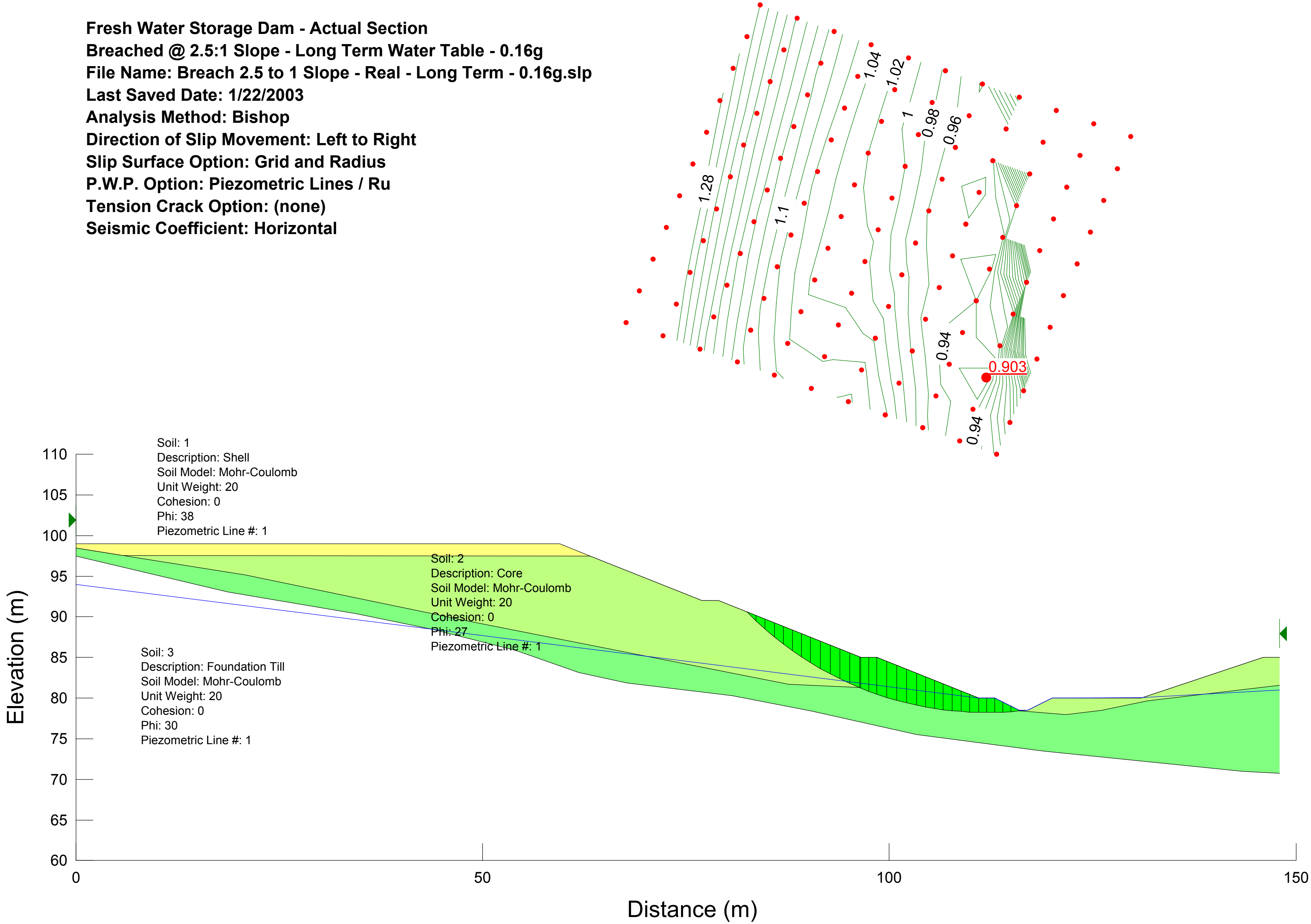


Fresh Water Storage Dam - Actual Section
Breached @ 2.5:1 Slope - Long Term Water Table - 0.00g
File Name: Breach 2.5 to 1 Slope - Real - Long Term - 0.00g.slp
Last Saved Date: 1/22/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: (none)

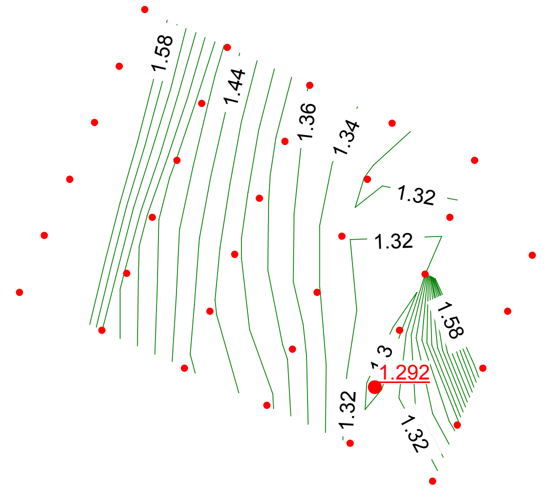


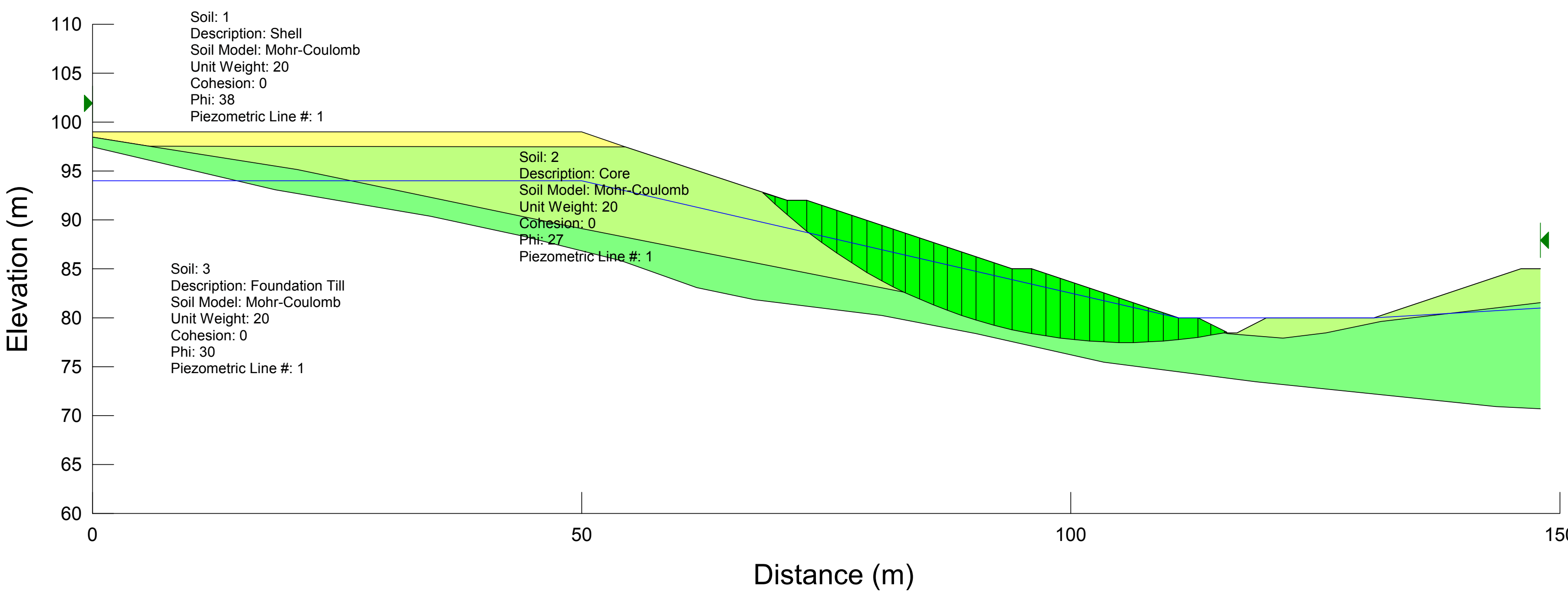

Fresh Water Storage Dam - Actual Section
Breached @ 2.5:1 Slope - Long Term Water Table - 0.06g
File Name: Breach 2.5 to 1 Slope - Real - Long Term - 0.06g.slp
Last Saved Date: 1/22/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: Horizontal
1.55
1.5
1.45
1.4
1.35
1.3
1.25
1.55
1.25
1.2
1.169
Soil: 1
Description: Shell
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 38
Piezometric Line #: 1
Soil: 2
Description: Core
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 27
Piezometric Line #: 1
Soil: 3
Description: Foundation Till
Soil Model: Mohr-Coulomb
Unit Weight: 20
Cohesion: 0
Phi: 30
Piezometric Line #: 1
Elevation (m)
110
105
100
95
90
85
80
75
70
65
60
0
50
100
150
Distance (m)

Fresh Water Storage Dam - Actual Section
Breached @ 2.5:1 Slope - Long Term Water Table - 0.16g
File Name: Breach 2.5 to 1 Slope - Real - Long Term - 0.16g.slp
Last Saved Date: 1/22/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: Horizontal

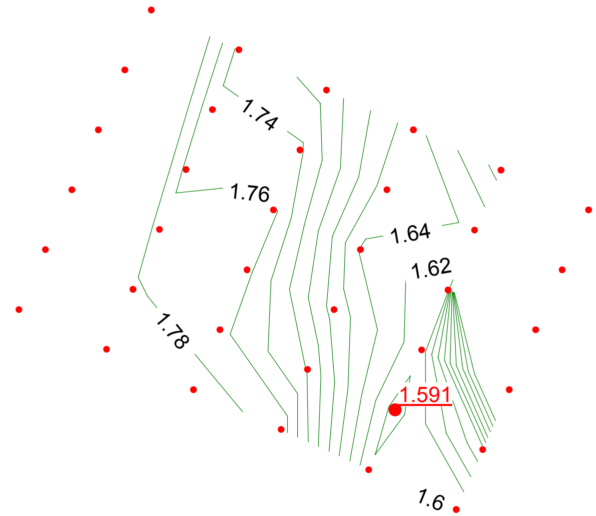


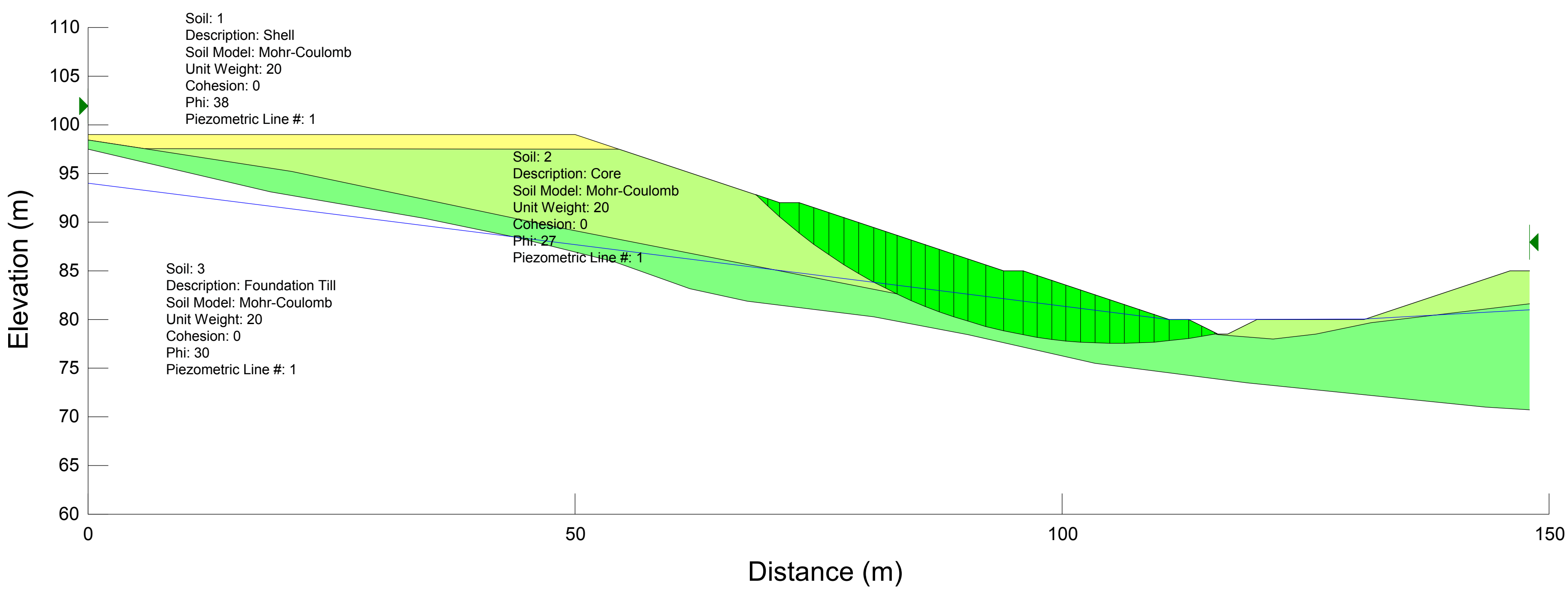
Fresh Water Storage Dam - Actual Section
Breached @ 3:1 Slope - Post Construction Water Table
File Name: Breach 3 to 1 Slope-Real-Post Con.slp
Last Saved Date: 1/21/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: (none)



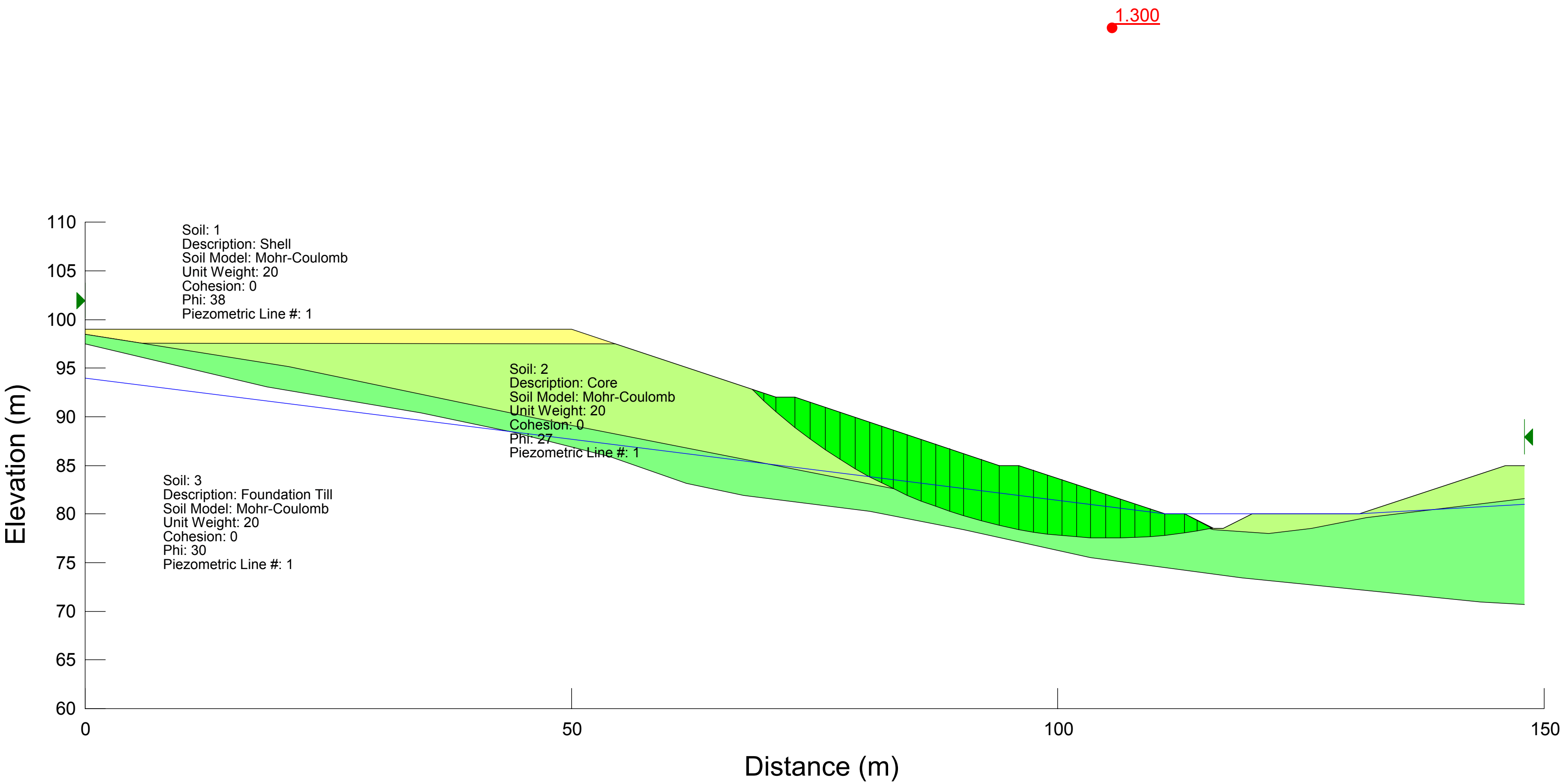


Fresh Water Storage Dam - Actual X-Section
Breached @ 3:1 Slope Long Term Water Table
File Name: Breach 3 to 1 Slope-Real-Long Term-3.slp
Last Saved Date: 2/3/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: (none)

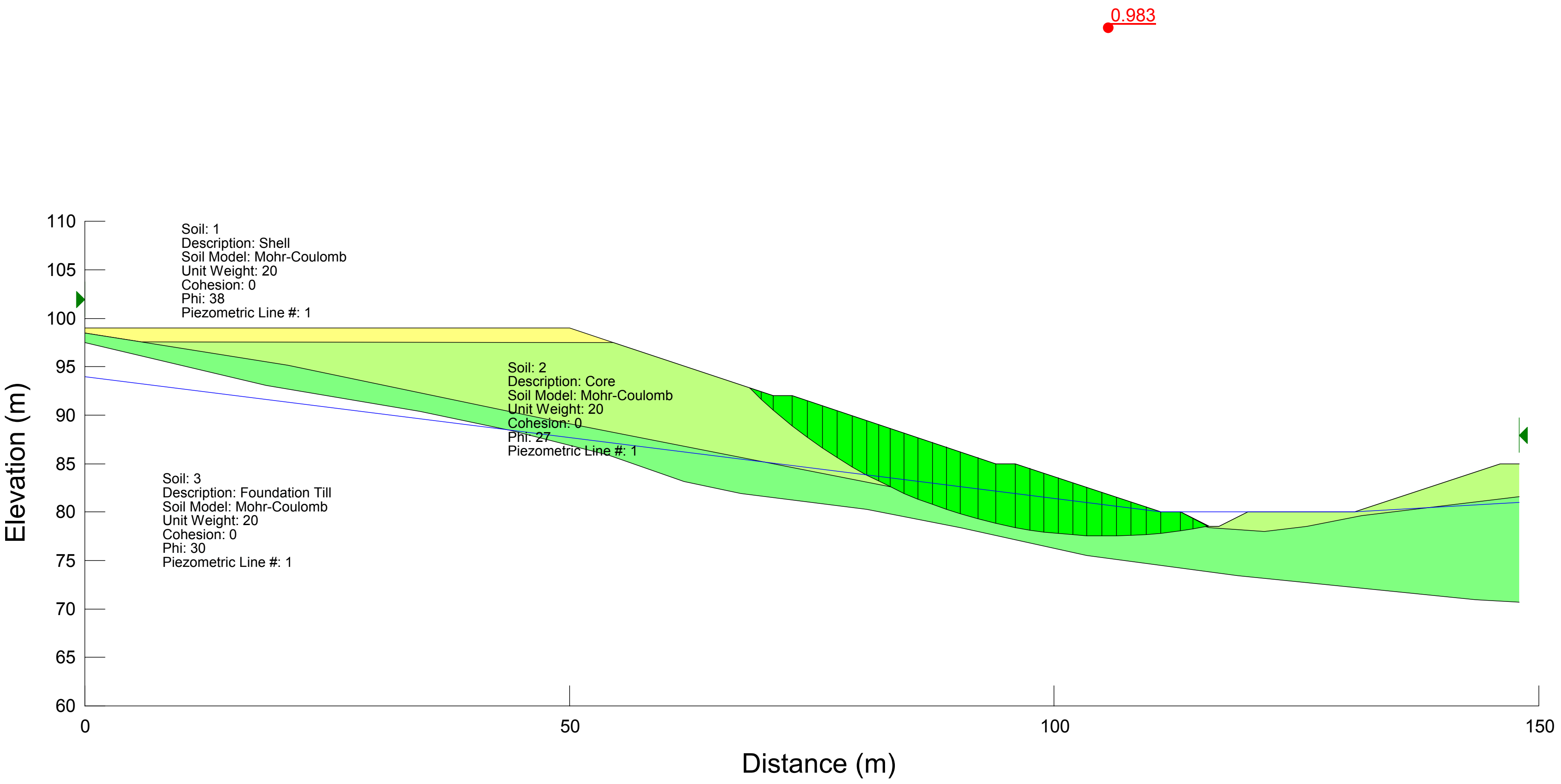




Fresh Water Storage Dam - Actual X-Section
Breached @ 3:1 Slope Long Term Water Table - 0.06g
File Name: Breach 3 to 1 Slope-Real-Long Term - 0.06g.slp
Last Saved Date: 1/21/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: Horizontal



Fresh Water Storage Dam - Actual X-Section
Breached @ 3:1 Slope Long Term Water Table - 0.16g
File Name: Breach 3 to 1 Slope-Real-Long Term - 0.16g.slp
Last Saved Date: 1/21/2003
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric Lines / Ru
Tension Crack Option: (none)
Seismic Coefficient: Horizontal



Appendix D

Table D.1 Estimated Capital Cost for the FWSD Breach						
Task	**Item**	**Units**	**Quantity**	**Unit Cost**	**Total**
1	Complete repairs to low-level outlet	l.s.	1	$70,000	$70,000
	Monitoring of drawdown performance	day	80	$90	$7,200
				Subtotal	**$77,200**
2	Live capture of fish	day	10	$3,000	$30,000
	transport the fish	km	720	$4	$2,880
	release the fish	day	10	$2,000	$20,000
				Subtotal	**$52,880**
3	Revegetation of the exposed reservoir base	ha	20	$1,500	$30,000
	Revegetate exposed riparian area	ha	3	$2,000	$6,000
	Prepare spoil areas	m^2	1000	$3	$2,500
	Prepare access roads	m^3	1500	$8	$12,000
				Subtotal	**$50,500**
4	Source and deliver gravel substrate	m^3	500	$28	$14,000
	Source and deliver Type I riprap	m^3	3000	$28	$84,000
	Source and deliver Type II riprap	m^4	1600	$28	$44,800
	Purchase geotextile and deliver	m^2	1300	$8	$10,400
				Subtotal	**$153,200**
5	Mob and Demob of the contractors	l.s.	1	$193,000	$193,000
	Install flume/pipe for pumping	l.s.	1	$8,000	$8,000
	Pump rental - 1 pumps (3 months)	month	3	$6,700	$20,100
	Spare pump rental – 1 pumps (3 months)	month	3	$4,700	$14,100
	Pump operating cost – 1 pump running continuous, 1 pump operating 20% of the time for 70 days	days	112	$185.50	$20,776
	On going fish capture and transfer	days	15	$5,000.00	$75,000
	Monitoring water quality during construction	days	75	$100.00	$7,500
	Preparation of access road	m^3	600	$8	$4,800
	Cofferdam foundation preparation	m^3	500	$14	$7,000
	Install main cofferdam	m^3	1,600	$20	$32,000
	Seepage collection ditch & berm	m^3	2,000	$8	$16,000
	Pumps for removing seepage	days	3	$260	$780
				Subtotal	**$399,056**
6	Excavation of riprap, and removal to Faro Pit	m^3	100	$16	$1,600
	Excavation, separation and local storage	m^3	72,400	$6	$434,400
				Subtotal	**$436,000**
7	Excavation of channel	m^3	2100	$15	$31,500
	Over-excavation for placement of floodplain erosion protection	m^3	1800	$6	$10,800
				Subtotal	**$42,300**
8	Excavation for residual structure erosion protection	m^3	1,300	$50	$65,000
	Place type I riprap at edge of residual structure	m^3	1,300	$13	$16,900
	Place type I riprap in riffles	m^3	925	$13	$12,025
	Place type II riprap	m^3	1,800	$13	$23,400
	Place gravel substrate	m^3	500	$12	$6,000
				Subtotal	**$123,325**
9	Rehabilitation of channel	m	274	$90	$24,660
				Subtotal	**$24,660**
10	Removal of cofferdam	m^3	3,500	$6	$21,000
	Final channel excavation	m^3	300	$8	$2,400
	Placement of remaining channel fill	m^3	350	$15	$5,250
	Abandon inlet of LLO	lump	1	$5,000	$5,000
	Abandon valve house	lump	1	$25,000	$25,000
	Fill placement in fresh water channel	m^3	3600	$8	$28,800
				Subtotal	**$87,450**
11	Revegetation of riparian area, not done in the fall	ha	27	$1,500	$40,500
	Revegetation of riparian area	ha	4	$2,000	$8,000
	Evaluate hydraulic elements and repair	l.s.	1	$40,000	$40,000
				Subtotal	**$88,500**
12	Perform water testing	day	6	1200	$7,200
	Perform fish survey	l.s.	1	15000	$15,000
				Subtotal	**$22,200**
				Total	**$1,557,271**

Task 1 - Reservoir lowering
Task 2 - Live fish transfer
Task 3 - Other pre-construction activites, including preliminary revegetation
Task 4 - Prepare fill materials and source supplies
Task 5 - Install cofferdam and maintain reservoir
Task 6 - Bulk excavation of breach
Task 7 - Excavation of channel
Task 8 - Fill placement, geotextile, riprap and gravel
Task 9 - Rehabilitation of the downstream channel
Task 10 - Removal of cofferdam and site clean up
Task 11 - Post 2004 freshet activities
Task 12 - Post-construction monitoring

Drawings