DELOITTE AND TOUCHE INC.

ROSE CREEK DIVERSION CANAL

CONCEPTUAL CLOSURE OPTIONS

(DRAFT)

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Project No. 0257-028-02
November 24, 2004

Mr. Douglas Sedgwick
Deloitte & Touche Inc.
Suite 1900, TD Centre
79 Wellington Street West
Toronto, Ontario M5K 1B9

Re: Rose Creek Diversion Canal – Conceptual Closure Options

Dear Doug:

The above referenced draft report has been uploaded to the Deloitte & Touche Inc. e-room. The access to the report has been set-up to allow stakeholder review of this report. This report presents our assessment of conceptual closure options for the Rose Creek Diversion Canal. These conceptual designs have been prepared based on commentary received on the previous conceptual designs and on additional site specific information. The original intent of this study was to provide a detailed cost estimate for one option only, that of extending the canal along the valley wall. As the study progressed, it became clear that additional conceptual options should be considered. These concepts were developed in general and then rough cost estimates prepared. It is the intention that further evaluation and cost estimation be performed for one of the conceptual options. However, an evaluation of the “fit” of the Rose Creek Diversion Canal into the overall closure plan and the closure philosophy should be performed prior to completion of this task. The conceptual closure option selected for further detailed evaluation may be a variation of one of the concepts presented.

This draft report has been issued for your comment and review as part of the next phase of Faro Mine closure planning. Once your, and the regulatory reviewers, comments are received a second draft report will be prepared. The second draft will take account of the comments received and will reflect the direction given at the January closure planning meeting. Following this the report will be issued in final form. We trust that this information meets with your requirements at this time. Should you have any questions or comments, please do not hesitate to contact me at the number listed above.

Yours truly,
BGC Engineering Inc.
per:

Gerry Ferris, M.Sc., P.Eng.
Geotechnical Engineer

GWF/sf

Attached: Draft Report

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LIMITATIONS OF REPORT

This report was prepared by BGC Engineering Inc. (BGC) for the account of Deloitte and Touche Inc., Interim Receiver for Anvil Range Mining Corporation. The material in it reflects the judgement of BGC staff in light of the information available to BGC at the time of report preparation. Any use which a Third Party makes of this report, or any reliance on decisions to be based on it are the responsibility of such Third Parties. BGC Engineering Inc. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

As a mutual protection to our client, the public, and ourselves, all reports and drawings are submitted for the confidential information of our client for a specific project and authorization for use and / or publication of data, statements, conclusions or abstracts from or regarding our reports and drawings is reserved pending our written approval.

1.0 INTRODUCTION

BGC Engineering Inc. (BGC) was retained by Deloitte & Touche Inc. (D&T), the Interim Receiver for Anvil Range Mine, to perform an assessment of closure options for the Rose Creek Diversion Canal (RCDC). This project is part of a series of reports prepared by BGC, SRK Consulting Inc., Gartner Lee Limited and others related to the closure of the Faro Mine.

The RCDC diverts the water flow of the Rose Creek around the tailings impoundment at Faro Mine. The Rose Creek Diversion consists of two segments; the upper reach, constructed in 1974 as part of the development of the second tailings impoundment and the lower reach, constructed as part of the 1980 Down Valley Tailings development. The 1980 portion of the canal was designed for the 50 year return period flood with a contingency to transmit the 500 year return period flood.

Recently the RCDC was evaluated to determine if it still met these original design requirements (BGC 2004a). This study revealed that sections of the RCDC canal dike crest were too low to transfer the water without overtopping. During 2004, construction was performed on the low sections of the canal dike bringing the canal back into compliance with its original design philosophy (as built report currently in preparation).

Previously three conceptual designs were prepared for the expansion of the existing RCDC to pass the Probable Maximum Flood (PMF) (Northwest Hydraulic Consultants (nhc) 2004). Based on discussions at the February 2004 closure meeting in Vancouver the three options previously presented were considered unsuitable. The options were considered unsuitable due to either requiring construction on tailings or the need for a concrete spillway. At the meeting it was decided among the stakeholders that the preferred option should not include a concrete spillway due to their long term maintenance requirements and finite life-span. Additionally the preferred option should not require construction on tailings due to the possibility of liquefaction or settlement of the tailings. At the February 2004 meeting the scenario thought to hold the most promise was to extend the canal along the valley wall downstream of the current steep sections. The extended canal would then discharge on the valley wall.

This report provides a summary of BGC's evaluation of concepts related to the expansion of the RCDC to handle the PMF flood, without the need for construction on the tailings or the use to concrete spillways.

1.1 Scope of work

In order to complete the general scope of work indicated above, a proposal was submitted which outlined the following tasks:

- Review the PMF for the RCDC. This would include determining the PMF if the North

Fork Rose Creek Rock drain was left in place.

- Compile existing subsurface information
- Perform ground truthing of the proposed canal route
- Perform geophysics along the new portion of the proposed canal route
- Prepare a preliminary design of the proposed channel and perform a cost estimate to complete the construction. This design task was to be taken to the level such that commentary on construction sequencing and considerations for care of water would be developed.
- Estimate the PMF peak flood for the Intermediate Dam and Cross Valley Dam spillways. This includes the assumption that the two existing dams would receive no contribution of flow from the expanded RCDC.
- Estimate sedimentation from the tailings area. Consider a number of possible cover scenarios and estimate the sediment leaving the tailings impoundment.

The original intent of this study was to provide a detailed cost estimate for one option only, that of extending the canal along the valley wall. As the study progressed, it became clear that additional conceptual options should be considered after performing the initial analysis.

The preliminary results were discussed with D&T personnel and the scope of work was modified. For each of the concepts preliminary cost estimates were prepared to allow a general sense of the scopes of work required for the two main options. Development of a work program and cost estimate such as envisioned in the proposal was not considered appropriate at this time. It was felt that for the current budget that the broad concepts should be taken to a preliminary level of evaluation, given the vast differences between the cost estimates. These concepts should then be evaluated in terms of the “fit” of the RCDC into the closure plan. Later the conceptual closure options could be further evaluated as envisioned in the proposal. It is considered likely that during the next round of closure discussions, following definition of closure objectives, that the plan for the RCDC will change. It would be therefore more important to have the broad concepts in place, rather than expending the budgeted resources to obtain a high degree of confidence in an option may be ultimately rejected.

Given the modification to the scope of work, a second draft report is envisioned. The second draft report would take into account the comments received on this first draft and discussion at the January 2005 meetings.

1.2 Authorization to Proceed

Authorization to proceed was provided by Mr. Doug Sedgwick of Deloitte & Touche Inc. the interim receiver for Anvil Range Mining Corp.

2.0 BACKGROUND

2.1 Site Location

Faro Mine is located in the central Yukon, approximately 200 km north-northeast of Whitehorse. The Faro Mine is situated approximately 22 km north of the Town of Faro, as shown in Figure 1. The RCDC is located to the south of the tailing impoundment, along the south side of the Rose Creek Valley, as shown in Figure 2. Photo 1 shows a panoramic photo mosaic of the entire RCDC, taken from the north side of the Rose Creek valley.

2.2 Rose Creek Diversion Canal

The RCDC transfers the water flow of Rose Creek around the tailings impoundments. The diversion was developed in two stages, referred to as the “original or 1974” and the “1980” reaches of the diversion. The original diversion was constructed in 1974 as part of the development of the Second Tailings impoundment (Figure 2). The 1980 diversion extended the original diversion around the concurrently constructed Down Valley Tailings impoundment, in 1980-81.

The South and North Forks of Rose Creek converge upstream of the entrance to the original reach of the RCDC and the combined flow enters the diversion immediately downstream of the pump house pond. The original reach is a predominately straight channel that is constrained by natural slopes on the south side and by a constructed dike augmented by an upper road/tailings dike on the north side. The canal was excavated with a bottom width of 15 m and had side slopes of 2H:1V (Gartner Lee 2002). The gradient of the initial portion of the canal was 0.23%, this portion of the canal remains. The gradient of the canal increased to 2% and finally to a grade of 5%, however this portion of the original RCDC has been abandoned. Throughout the canal weirs were designed to allow a 0.61 m (2 foot) head loss at each weir, resulting in a weir spacing of 30.5 m (100 foot) in the zone with 2% grade and a 12.2 m (40 foot) section where the grade was 5%. Additionally, buried weirs were installed on 152.4 m (500 foot) spacing where the grade was 0.23 %. Between the weirs, no rip rap protection was provided on the sideslopes in the two lower gradient sections (Sigma 1975). During the construction of the 1980 canal, the downstream (steeper) portion of this canal was abandoned, leaving only the initial 0.23% grade section. The condition of the weirs in the original reach was reviewed during the ground truthing inspection for this study. The above water portions of the weirs are visible on both banks, but their condition within the base of the channel is not known.

The 1980 portion passes water along the south side of the Intermediate Impoundment and returns flow into the natural Rose Creek channel downstream of the Cross Valley Dam. The 1980 reach includes a series of boulder-lined drop structures (“weir sections”) and a sharp corner at the downstream end. The 1980 reach is constrained by a cut slope into the natural hill side on the south side and by a till dike on the north side. Most of the 1980 channel has a gradient of 0.19%, with two drop weir sections having a gradient of about 5%. The canal was designed with a bottom width of 12.2 m and side slopes of 2H:1V in soil and 0.5H:1V in rock. The low gradient sections of the channel included a pilot channel 3.65 m wide by 0.6 m deep to control glaciation during winter low flows.

A Diversion Dam was included as part of the 1980 canal construction; the purpose of this structure was to divert the water from the original reach into the 1980 reach. In 1982 (one year following construction of the diversion canal), the crest of the Diversion Dam was lowered by approximately 0.5 m below the adjacent crest of the diversion canal dike. The lowered section was then armoured with rip rap (HydroCon 1982). The lowered crest (fuse plug) was installed to ensure that any flows in excess of the design flow overtop the Diversion Dam at that location and flow into the Intermediate impoundment in a controlled manner.

There is one primary tributary that enters the original section of the canal from the south side, just downstream of the pump house pond, or immediately upstream of the inlet to the diversion. There are two tributaries that enter the 1980 reach of channel from the south side; Goodall Creek and Cornish Creek.

The channel has been prone to ice build-up over the winter and clearing of ice has been required in the spring, on occasion.

Visual inspection and instrumentation have been used to monitor the condition of the canal. Generally, most of the permafrost in the backslope has thawed and no significant deformations have occurred. One portion of the canal dike just upstream from the Intermediate dam is still underlain by permafrost. As a result, continued thawing, cracking and deformations still occur within this area of the dike. A 2003 study was conducted to determine the capacity of the RCDC (BGC 2004a). The study revealed areas with settlement of the dike crest. This settlement required the reconstruction of a portion of the dike crest to increase the capacity of the canal back to its original stated capacity. The construction activities recommended based on the 2003 study were performed in 2004. The canal has the capacity to pass the 500 year return period flood event.

2.3 Previous Determination of Hydraulic Capacity

2.3.1 1975 Observations and Assessment

In 1974, construction began on the second tailings impoundment, which required the construction of the original Rose Creek Diversion. Construction of the diversion was completed in April of 1975. The diversion consisted of an upper segment, with a low gradient (0.23 %), a middle segment with a gradient of 2% and a lower segment with a high gradient (5%). As noted previously, the steeper reaches of the original diversion were abandoned as part of the construction of the 1980 diversion canal and are not discussed further.

The calculated peak flow which occurred in the spring of 1975 was 950 cfs (26.9 m^3/s). That spring freshet was 56% of the design flow of 1,700 cfs and no significant erosion was evident in the canal, other than minor erosion related to slumping of unstable banks (Sigma 1975).

2.3.2 1982 Capacity Assessment

Following the construction of the 1980 portion of the RCDC, the as-built conditions were reviewed to confirm that the final arrangement conformed to the design. This was accomplished by measuring the flow within the canal and determining the local and average water levels in the canal at the time of the flow measurement (Hydrocon 1982). During the assessment, the measured flow was 6.37 m^3/s, when this flow was correlated with the water levels in the channel it resulted in a calculated Manning's n of 0.032. The conclusion based on this result was that the assumed design Manning's n of 0.030 was realistic, and thereby the design capacity was realistic.

The 1982 report also included a design for the overflow at the diversion dam (fuse plug). The crest of the overflow was 0.5 m below the surrounding diversion dyke crest and flow would initiate at the 160 year return period flood. The maximum discharge capacity of the overflow weir was approximately 55 m^3/s.

2.3.3 2003 Capacity Assessment

Based on survey data collected by Yukon Engineering Services (Y.E.S.) during the summer of 2003 and visual condition surveys by BGC, a hydrotechnical assessment of the canal was performed (BGC 2004a).

The following is a summary of some of the key modelling parameters and methods used:

- Modelling was performed using the hydraulic model HEC-RAS.
- Ice-free conditions and 1.5 m of ice blockage in the base of the channel were modelled.
- The updated cross sections, 39 in total, were input into the model.
- The following Manning's n were used: 0.045 for the mildly sloped reach below the upper weir, 0.06 for the upper weir, 0.04 for the 1980 diversion and 0.037 for the original diversion.
- A Mannings n of 0.02 was used for ice.
- The 500 year flood used was 135 m^3/s, applied at the upstream end of the original diversion.
- The water levels were established with no water allowed to flow over the overflow section of the Diversion Dam.

The results of the hydraulic modelling during ice-free conditions were:

- Overtopping of the canal dike at a discharge of 82 m^3/s, with an estimated return period of about 90 years.

- Overtopping at the overflow in the Diversion Dam would begin at a discharge of 100 m^3/s, a peak flood value with an estimated return period of about 170 years.
- If the canal dike were raised to retain the 500 year flood of 135 m^3/s, 12 m^3/s would flow over the diversion dam.

The results of the hydraulic modelling for the ice filled channel depth are:

- Overtopping of the canal dike would begin at a discharge of 60 m^3/s, with an estimated return period of about 40 years.
- Overtopping of the overflow at the Diversion Dam would begin at a discharge of 73 m^3/s, a peak flood value with an estimated return of about 60 years.
- If the canal dike were raised to retain the 500 year flood of 135 m^3/s, 25 m^3/s would flow over the diversion dam.

It was concluded that the RCDC could not safely pass the 1:500 year flood in its early 2003 configuration. The canal dike would be overtopped under the design flood for a length of about 1,000 meters of the dike length, essentially upstream of the Intermediate Dam.

A construction program consisting of raising these low sections of the canal dike was recommended. The recommended crest elevation was selected on the basis of the canal with ice in it and not allowing any flow to pass over the Diversion Dam.

During the winter of 2003/2004 a very thick snow pack developed. Given the thickness of the snow pack and the knowledge that portions of the canal dike were low, a temporary dike raise was undertaken. The temporary raise was constructed by dumping and spreading a gravely sand on the surface of the existing dike. Compaction was nominal and the fill was spread on top of the existing frozen surface of the dike. This fill was placed only as a temporary measure, with the intention that it would be reworked as part of the later dike raise construction. The temporary fill was placed only in the two lowest areas of the dike and did not raise the crest of the dike completely to the recommended design elevation.

2.4 2004 Construction

The recommended construction for the RCDC canal (BGC 2004a) was completed in the summer of 2004. The ‘as-built’ report for this construction is currently under preparation, however the dike raise was completed as per the recommendations in the design report. The following provides a general summary of the construction program undertaken.

- When encountered, the spring placed fill was removed to expose the surface of the dike prior to placement of this temporary fill. The material removed was used as the general fill material to raise the dike surface.
- The existing surface of the canal dike surface was scarified to a minimum depth of 300 mm. The scarified material was compacted in a 300 mm maximum lift to 98% Standard Proctor Maximum Dry Density (SPMDD).

- Vegetation and other objectionable material (as defined by Field Engineer) was removed from the existing surface prior to placing new fill.
- The fill placed for the dike raise consisted of a well graded gravely sand.
- The fill was compacted in 300 mm maximum lift thickness to 98% of SPMDD. Moisture conditioning was required to achieve this level of compaction.
- A 0.5 m thick zone of rip rap was placed on the inside face of the newly placed fill and additionally in areas below the level of new fill, as required. The rip rap gradation was similar to the gradation of the existing rip rap.
- The surface of the canal dike was graded to drain towards the canal.

2.5 2003 Conceptual Closure Study

As indicated in Section 1, as part of the hydrotechnical study for closure planning undertaken in 2003, three conceptual closure options were examined (nhc 2004). In brief these options were:

Scenario 1 – increase size of RCDC by increasing the height of the canal dike. Three different options were examined to accomplish increasing the capacity of the RCDC.

Scenario 1 – raising the right dike height only

Scenario 1A – Widening of the existing RCDC invert by 5 m into the south bank in combination with raising the right dike height to a lesser degree

Scenario 1B – Similar to Scenario 1 except that a concrete spillway is utilized to convey flow down the steeply sloped rock drop weir section adjacent to the Cross Valley Dam.

Scenario 2 – Abandon the Rose Creek Diversion downstream of the Diversion Dam. Convey the PMF over the tailings (covered with a soil cover) in a swale lined with rip rap to the south abutment of the Intermediate Dam where a new spillway conveys flow to the downstream of the Cross Valley Dam.

Scenario 3 – Remove tailings from the Original, Second and Intermediate Impoundments to elevation 1042 m. Rose Creek flow to enter the impoundments immediately downstream of the Pump house pond. The attenuated PMF to pass down a spillway sited at the north abutment of the Intermediate Dam

Further details of these three closure scenarios are found in the previous report (nhc 2004), specifically Appendix C. As noted in the introduction these options were rejected either due to the requirement to construct a concrete spillway or the requirement to construct on tailings.

An assessment of the potential for the tailings to liquefy was performed in 2003 (Golder 2003), and it was concluded that much of the tailings would liquefy during the design earthquake. This result reinforces the decision not to proceed with any option that required construction on the tailings surface.

3.0 PMF REVIEW

In the 2003 Hydrotechnical Study for the Faro Mine (nhc 2004) a thorough review of the available information was undertaken and a new estimate of the PMF was developed. As indicated in the 2003 study the two most important factors in estimating the PMF flood value are the Probable Maximum Precipitation (PMP) and the time to peak (time from centroid of rainfall excess to time of peak discharge). The results of the study are summarized in Table 1.

Table 1 Estimated Probable Maximum Floods for the Faro Mine Site

Mine Site Sub-Basin	Drainage Area (km^2)	PMF Peak Discharge (m^3/s)
North Fork Rose Creek at Rock Drain	118	504
Fresh Water Supply Dam	67	354
Rose Creek above the RCDC	203	690
Rose Creek downstream of the Mine (X14)	230	783

The results of the 2003 study produced between a 47 to 64% reduction in the peak PMF discharge from the previous study (nhc 2001). The reasons for the different PMF peak values were related to the estimated PMP and the times to peak.

In the 2001 study the PMP was conservatively estimated on the basis of rainfall measured in south-west Yukon, which is influenced by coastal rains. For the 2003 study the PMP for the Faro Mine site was estimated on the basis of a PMP study conducted for Mayo, Yukon. The PMP study for Mayo considered the measured rainfalls in the Yukon but also accounted for the differences in elevation and general climatic conditions between the areas affected by the coastal rainfall and the location of Mayo. The results of the Mayo PMP were arbitrarily increased by 50% for the Faro mine site and resulted in a 24 hour point PMP of 200 mm.

In the 2001 study the time to peak for all estimated PMF was 2 hours. For the 2003 study this assumption was revised. Although no data was directly available from the Faro Mine site a comparison between the rainfall measured at the Faro Airport and the stream flow measurements at Vangorda Creek was undertaken. This resulted in times to peak of approximately 24 hours. This was considered to be unreasonable given the steep terrain, rock outcrops and permafrost. Therefore, a relationship between the size of the drainage area and the time to peak was developed. This resulted in times to peak varying between 3 and 6 hours.

Recommendations were provided in the 2003 study for determining these two key parameters (PMP and time to peak) with a greater degree of confidence. The present study envisioned a brief review of newly available information related to PMF estimation. However only limited data was available and it was concluded that until additional data becomes available no update is possible. Therefore the PMF estimates provided in the 2003 study are the best available.

Two weather stations were established and maintained at the Faro Mine site by Yukon Government personnel. One of the stations is located on the Faro Waste Dumps and the second at Vangorda/Grum. The rainfall data combined with stream flow measurements can be used to provide better estimates for the time to peak. The only flow station on the Faro side of the site located upstream of mine disturbance, R7 did not collect data during 2004. Therefore, no data is available with respect to time to peak for the Faro side of the mine site. Based on the rainfall and flow measurements made on the Vangorda/Grum side of the mine site, a measured time to peak of 5 hours was made for one rainfall event in June 2004 (personal communication YTG) for an event with a total rainfall of 28 mm occurring over 13 hours.

4.0 SURFACE AND SUBSURFACE DATA

To complete the layout and evaluation of the new conceptual design options various data sources were combined. For this study the data sources included airphoto interpretation, ground truthing, a review of existing drilling and testing pitting information and a review of the as-built records for the 1980 portion of the RCDC.

Prior to proceeding with the 2004 field program a preliminary alignment was selected for the proposed channel. This alignment shown on Figures 4 and 5 as the main geophysics line was selected on the basis of matching the slope of 1980 portion of the RCDC. The preliminary alignment selected matched the existing slope to keep the rip rap at a manageable size as well as ensuring that discharge from the canal would be downstream of the tailings facilities and near the existing Rose Creek.

4.1 Topography

As part of the previous study of the RCDC (BGC 2004a) a detailed topographical plan was developed. In 2003 airphotos were taken of the entire mine site and a topographic plan developed for the area covered by the airphotos (Othoshop 2003). Variances of up to 3 m were encountered between the ground surface that was based on direct survey information as compared to information base on the airphotos. Since the area under study for this project was not part of the ground survey the information from the airphotos was used.

4.2 Surface Information

Surface information was derived from two sources; air photo interpretation and ground truthing.

Airphoto interpretation was performed and a new terrain map was developed as shown in Figure 3. The airphotos used in this study were the new 1:10,000 scale airphotos taken in 2003 (The Othoshop 2003). The information from the airphoto interpretation was added to the topographic plan as shown in Figures 4 through 7.

Ground truthing consisted of a walking inspection of the existing RCDC and the area along the proposed alignment. The alignment shown on Figures 4 and 5 as the geophysics line was used as the basis for the ground truthing. The ground truthing was performed by Mr. Gerry Ferris, P.Eng. of BGC between September 10 and 14, 2004. During the ground truthing exercise about 300 individual observations were made, about 40 photographs were taken and some soil samples collected. The locations of the key observations from the ground truthing are shown on Figures 4 through 7. The observations included locations with exposed bedrock or locations where thin bedrock was expected. On the basis of the ground truthing observations the terrain map was updated, as required.

Included on Figures 4 through 7 are: a plan view of the area, a profile (which is either along the existing base of the RCDC or along the main geophysics line) and sections at 100 m spacing.

4.3 Subsurface Information

Three different sources were used to create a preliminary estimate of the subsurface conditions with the emphasis was on locating the top of bedrock. At this preliminary stage, this was the main difference considered important.

Information contained in the 1980 summary of investigation and design (Golder 1980) provided locations and elevations for the occurrence of bedrock in the 1980 portion of the RCDC. The location and elevation of bedrock from this study has been transferred to Figures 4 through 7. This information was derived from both boreholes and test pits.

During the construction of the 1980 portion of the RCDC a detailed record was kept concerning the instances when bedrock was encountered (Golder 1982). The information contained in the as-built report, related to the occurrence of bedrock, was transferred on the overall site plan as shown in Figures 4 through 7.

Geophysical surveys were performed at the Faro Mine site for programs undertaken by both BGC and SRK in 2004. The geophysical contractor, Aurora Geophysics Ltd. (Aurora) from Whitehorse Yukon was on site from October 7 to 18, 2004 for all the programs. The work undertaken for the RCDC upgrade project consisted of two seismic refraction lines basically centered on two creeks crossed to be crossed by the proposed channel and three EM surveys. One of the EM surveys was conducted along the entire length of the proposed canal length with two additional lines running downslope from the main line to the edge of Rose Creek. The location of the main line is shown in Figures 4 and 5.

The purpose of the geophysical assessment was to determine the bedrock contact. The preliminary results of the geophysics study are attached in Appendix I. The results of the EM survey were noted to be difficult to interpret in the short time available for the preparation of the preliminary report. It is expected that interpretation will be provided in the final report from

Aurora. The EM preliminary traces, shown in Appendix I were provided by Aurora with no interpretation.

Seismic line SL-1 (Figure 4), provided a fairly clear indication of depth to bedrock. The top of bedrock elevation in this area ranged from 1030 m increasing to 1040 m on the northwest end of the line. Given the ground surface elevation in this area was about 1053 m on the southeast part of the line, the interpreted depth to bedrock is greater than 20 m. On the northwest portion of the seismic line, the ground surface elevation is about 1047 m yielding a depth to bedrock of between 12 to 17 m.

At seismic line SL-2 (Figure 4), the interpretation was not as clear as for SL-1 for the top of bedrock. The velocities measured were in the typical range of either permafrost or weathered bedrock, with some data returning high velocities more indicative of bedrock. This line extended for about 230 m. In the eastern 100 m of the seismic line the depth to the reflector was between 2 and 3 m. In the western 130 m of the seismic line the depth to the reflector level varied from 4 to 12 m but averaged about 8 m. Although the geophysical interpretation of this area is not yet clear, for the purposes of this preliminary assessment, the reflector found in SL-2 was assumed to be the bedrock surface.

Prior to proceeding to feasibility type designs a drilling program will be required in the area demarked by the main geophysical line to confirm bedrock elevations and overburden stratigraphy.

5.0 CONCEPTUAL OPTIONS

As noted in the introduction, the original purpose of the report was to study the possibility of widening the existing RCDC into the hillside and then extending the entire widened canal downstream along the valley wall. Following the numbering provided in the 2003 study (nhc 2004) this new option is denoted Scenario 4. A preliminary hydrotechnical analysis of this option was performed and resulted in a very significant resizing of the canal, and therefore high costs. The preliminary result indicated that the cost of the expanded RCDC would be at least $100 million dollars higher than indicated in the Scoping Studies (SRK 2003). A fifth scenario, Scenario 5, was added to the study following this preliminary result. Scenario 5 consists of using structures upstream of the RCDC to attenuate the peak flood and then upgrading the RCDC to pass these smaller peak floods.

Development of these two scenarios has been left at a very cursory level. The difference in the scope of work and the rough cost estimate between scenario 4 and 5 is so vast that it is felt that a decision from the project team is required. Each of the scenarios results in some different risks and difficulties but more importantly a difference in philosophy. Scenario 4 takes the idea that one structure should be used to pass the peak floods at the site, and scenario 5 brings forward the idea that multiple structures could be used to attenuate the flood.

Given the differences in philosophy and the very large cost differences between the two scenarios, only a preliminary scoping and costing of the two scenarios was performed. It is felt that guidance with respect to the closure philosophy and objectives is needed prior to moving beyond this cursory level. The estimates have been prepared in a modular (new reach, 1980 reach and original (1974) reach) fashion so that these preliminary estimates could be used as part of a discussion of these two scenarios but also for additional options currently not under consideration.

5.1 Scenario 4

Scenario 4 consists broadly of expanding the existing RCDC to the south, into the hillside such that the water level in the design flood is contained within the canal without requiring raising of the existing canal dike.

As noted in the introduction of Section 5, this option has been only taken to a preliminary level of evaluation. It is expected that further refinements will be possible, but as explained, a decision is required to select either Scenario 4 or Scenario 5. Following selection of either scenario then optimization of the construction can be performed. Some of the more obvious improvements that could be considered include: partial raise of the existing RCDC surface, variations on the expanded section of the proposed RCDC (such as lowering the invert of the expanded section, increasing the base gradient of the expanded section and steepening the cut slope angle in bedrock from 2H:1V to near vertical).

The following provides a general summary of the main considerations for this scenario. The new portion of the expanded RCDC will have the same gradient in the flat upper part of the 1980 reach. The RCDC is to be expanded to completely contain the PMF peak flood and therefore the existing overflow at the diversion dam will be abandoned. At the divergence of the existing route and the proposed route a small structure will be required to maintain a minimal flow down the existing steep sections for fish passage. This flow capacity will be sized such that the majority of “normal” flows pass through the existing steep sections. The downstream limit of the expanded RCDC will end in a side overflow weir. Water that over tops the side overflow weir will spill down the side of the mountain joining Rose Creek in the base of the valley.

The hydrotechnical routing was performed by nhc. A copy of this analysis is contained in Appendix II. The analysis was performed using a PMF peak value of 730 m^3/s. The results of the analysis indicated that in the 1980 reach of the RCDC, the base of the channel would have to be extended 60 m in to the hillside. In the original reach, the required expansion was 100 m. The new section of the expanded RCDC was assumed to have the same base slope as the 1980 reach and therefore the same total channel base was used. Figures 8, 9 and 10 respectively are section views of the expanded section for the new, 1980 and original reaches of the RCDC. Included on each of these sections is the area of excavation required at each section to expand the canal.

For proposed expanded RCDC selected sections were chosen from the 47 sections shown on Figures 4 through 7 for quantity estimation purposes. For the new reach, sections 8 and 13 were selected, largely since depth to bedrock was available from the seismic lines. Sections 22, 30 and 36 were selected to represent the 1980 reach and sections 43 and 46 were selected to represent the original reach. For each of the seven representative sections, the position of the top of bedrock was estimated. The proposed channel section was then superimposed and the area of excavation was calculated as shown on the figures.

In keeping with the cursory level of evaluation only the clearing area and excavation volumes were determined. In order to complete the expansion of the RCDC, clearing of about 67 hectares and the excavation, to neat lines, of 6.5 million m^3 of material will be required. Table 2 provides a summary of the calculated volumes and the contribution of each of the three reaches to the overall project.

Table 2 Excavation Volumes for RCDC Expansion, Flow Rate of 730 m^3/s

730 m^3/s			**Volume**			
RCDC Reach	***Total Excavation Area (m^2)***	***Thermal Berm (m^3)***	***Topsoil (m^3)***	***Common (m^3)***	***Rock (m^3)***	**Total (m^3)**
1974	119,100	0	35,730	673,666	385,046	1,094,442
1980	332,675	126,000	99,803	992,440	2,318,760	3,537,003
New	215,325	0	64,598	1,249,075	556,010	1,869,683
Total	667,100	126,000	200,130	2,915,181	3,259,816	***6,501,127***

As noted in Table 2 the excavation includes the removal of the existing thermal protection berm in the 1980 reach of the RCDC. The estimated volume of topsoil excavation is based on an assumed topsoil thickness of 0.3 m. This construction project would require almost equal excavation of rock and common material, therefore a blended cost of excavation of \$20/m^3 (0.5 x \$14/m^3, soil and 0.5 x \$26/m^3, rock) was used to estimate the excavation cost of **\$130 million**. It must be pointed out that this cost is preliminary only and is only for the excavation portion of the construction. Further refinements are likely to bring this excavation cost lower. However, the cost to upgrade the existing structure, place a seepage barrier, filter and rip rap and place a new thermal protection berm has not been included which will bring the cost estimate up.

5.2 Scenario 5

Scenario 5 option broadly meets the considerations discussed in the February 2004 technical meetings, that is, no construction on the tailings and no use of concrete. This scenario depends on structures upstream of the inlet of the RCDC to attenuate the peak flood event.

As stated previously, the high volume of excavation for scenario 4 combined with the results of the study of the Rose Creek Rock Drain (BGC 2004b) led to the inclusion of scenario 5. The rock drain report included analysis of the routing of the PMF flow through the structure. Based on this analysis the PMF peak flood at the rock drain was reduced from 540 m^3/s to about 15 m^3/s.

As a conservative assumption the peak flood used to estimate the required construction the 15 m^3/s flood value from the NFRD was directly added to the peak flood value from other parts of the basin. It is expected that the peak flows from the attenuation structures will not be additive to the direct run-off downstream of structures. However for this preliminary analysis this assumption was made.

Nhc performed a hydrotechnical analysis to determine the appropriate size of the RCDC expansion assuming that the rock drain was left in place. This analysis used a peak flood of 460 m^3/s as shown in Appendix II. The results indicated a smaller canal would be required. The assumptions made for the canal construction of this reduced flood were the same as those made for the full PMF flood peak. Designing for this peak flood value would result in the clearing of an additional 39 hectares of land and a total excavation of 2.7 million m^3. Additional details are provided in Table 3.

Table 3 Excavation Volumes for RCDC Expansion, Flow Rate of 460 m^3/s

460 m^3/s		**Volume**				
RCDC Reach	***Total Excavation Area (m^2)***	***Thermal Berm (m^3)***	***Topsoil (m^3)***	***Common (m^3)***	***Rock (m^3)***	**Total (m^3)**
1974	65,225	0	19,568	197,883	115,894	333,345
1980	196,086	126,000	58,826	383,144	995,600	1,563,570
New	132,825	0	39,848	592,418	137,295	769,561
Total	394,136	126,000	118,241	1,173,445	1,248,789	***2,666,475***

Given that the excavation volumes of common and rock are similar the \$20 / m^3 was again used as a cost, resulting in a cost of excavation of **\$53 million**.

Following completion of this analysis, it was assumed that a similar attenuation structure would be installed on the South Fork of Rose Creek. For the purposes of this conceptual design it was assumed that a rock drain would be installed at the location of the former fresh water supply dam. The attenuation characteristics on the south fork were assumed to be same as that achieved on the north fork, or a 15 m^3/s contribution to the peak flood received at the entrance to the RCDC.

When an attenuation structure is added on the south fork the resulting design peak flood was 230 m^3/s for the RCDC, as shown Appendix II. Similar to the assumption made previously the design flood consisted of the peak flood from the small basin downstream of the FWSD and the RCDC combined with the peak outflow from the FWSD and the RCDC. Further work will be required on the proper combinations of the peak flood values if this scenario is selected for further evaluation.

The assumptions regarding the canal construction were the same as the first two cases. The size of the canal required to transfer this peak flood value is shown on Figures 8 through 10 for the new, 1980 and original reaches of the canal. It is considered likely that the existing steep sections of the RCDC could be modified to handle this flood value, removing the need to extend the RCDC beyond its current alignment. However, the estimate presented is for the situation where the proposed RCDC will extend along the valley wall similar to the previous options. This construction project will require the clearing of about 19 hectares and the excavation of 780,000 m^3. Additional details are provided in Table 4.

Table 4 Excavation Volumes for RCDC Expansion, Flow Rate of 230 m^3/s

230 m^3/s			**Volume**			
RCDC Reach	***Total Excavation Area (m^2)***	***Thermal Berm (m^3)***	***Topsoil (m^3)***	***Common (m^3)***	***Rock (m^3)***	**Total (m^3)**
1974	27,000	0	8,100	30,100	22,572	60,772
1980	96,675	126,000	29,003	78,596	275,192	508,791
New	62,700	0	18,810	186,925	6,102	211,837
Total	186,375	126,000	55,913	295,621	303,866	***781,400***

For this project the common excavation is about 60% of the total volume the cost per unit excavation was \$18.8 / m^3 (0.6x\$14/m^3, soil plus 0.4x\$26/m^3, rock)was again used as a cost, resulting in a cost of excavation of **\$15 million**.

Additional options that are considered variations of scenario 5 that should be considered as part of the January 2005 closure meetings include:

- Additional studies to determine actual flood peak sizes with consideration of the variation in the time to peak in the attenuation structures.
- Attenuated flood as described for 230 m^3/s with the existing weir sections upgraded to pass the larger flood size.
- Consideration of the installation of a third attenuation structure upstream of the inlet to the RCDC to further reduce the peak flood size.

5.3 Effects of Peak Flood Size

In Section 3 a discussion was provided concerning the value of the peak PMF flood. Based on that discussion the analysis of the expanded RCDC channel was designed on the basis of a peak un-attenuated flood of 730 m^3/s for the PMF. Given that there remains some disagreement concerning the value of the peak flood from the PMF and the possibility of using various attenuation structures, a simple correlation of the estimated excavation volume and the value of the peak flood was performed, this is presented in Figure 11. The information in Figure 11 can be used to evaluate, at least on a preliminary basis, the amount of excavation required for a given size of the peak flood. If a cost per unit of excavation of $20 /$m^3$ a preliminary cost estimate can also be obtained.

6.0 EVALUATION OF CONCEPTUAL ALTERNATIVES

The following provides a brief evaluation of the two closure scenarios presented in this study.

As noted throughout this document the two closure scenarios have only been taken to a preliminary level of detail. The difference between each of the options, even at this preliminary level points to a large difference in the cost and the overall philosophy of the closure. As noted previously, BGC felt that given the differences and the variation from the cost indicated in the scoping studies that guidance should be provided before proceeding to a more detailed level of conceptual engineering.

6.1 Design Criteria

In order to provide a frame of reference for evaluating the project the following design criteria or minimum project specifications were developed:

- Design Flood: Safely pass the peak flood of the PMF, or 730 m^3/s. Alternatively the attenuated peak PMF flood value.
- Ice: Minimal ice damming due to winter flows.
- Seismic: Safely withstand the peak acceleration of 0.56g, the maximum credible earthquake.
- Design Life: 1,000 years
- Maintenance and Monitoring: Minimal maintenance and monitoring.

6.2 Evaluation

Scenario 4:
As outlined, constructing an expanded RCDC canal for the full value for the peak PMF will be more expensive than scenario 5 and about $100 million more expensive than originally indicated in the scoping level studies.

This scenario is consists of a relatively simple design and construction, however there will be a number of issues to be dealt with as part of the design and long term performance of such a structure, including:

- Design freeboard for the canal dike. The existing crest of the canal dike had settled by as much as 65 cm (between 1981 and 2003), based on the recent crest raise project. Settlement of the existing dike may be an ongoing concern, but this will be a larger issue for the new segment of the dike as this will create new areas for thaw settlement.

- Protection of the canal from thawing permafrost upslope of the canal. The 1980 reach included installation of a thermal protection berm. The purpose of the berm was to slow the thawing of permafrost and to thereby stop slope instability due to thawing permafrost. For the most part the thermal berm worked as designed, however, some erosion of the berm has occurred and also there have been at least three instances of slope instability in the berm.
- Long term movement of sediments from the small tributary drainage courses into the canal.
- Large scale channel blockage due to movement in the debris torrent area identified in the airphoto.
- Landslide movements from slope above the canal section.
- Earthquake induced movements of either the canal dike or the slopes above the canal section.
- Water spilling over the side overflow weir at the end of the canal section will cause considerable erosion and additional sediment loading to Rose Creek during any overtopping event, upto the PMF flood. Following spill events a review of the performance will likely be required and reconstruction may be required.

Scenario 5:
Attenuating the peak PMF flood value provides very significant savings in the construction cost over designing the RCDC for the un-attenuated flood peaks. This scenario requires structures to attenuate the peak flood and requires good performance of those structures to maintain the safe passage of the predicted peak flood. Currently the rock drain located on the North Fork of Rose Creek provides the only attenuation for peak flood events.

As indicated in the report on the rock drain (BGC 2004b) the PMF flood results in a water elevation in the pond only about 40 cm higher than the maximum water level previously experienced at the drain. Therefore it is expected that the drain will perform adequately under the PMF flood event. Note this does not include considerations of sedimentation of the drain. Larger volume spring melt events result in much higher seepage gradients and high pond levels and may be the design flood for the structure if it remains. Based on the larger spring floods, it was recommended that the downstream slope of the rock drain be flattened by the addition of large diameter rock at the toe to provide greater protection due to failures induced by seepage action. It is expected that this construction could be accomplished for about $100,000.

For the analysis shown for scenario 5, it was assumed that a rock drain will be installed at the location of the former fresh water supply dam. The envisioned structure would fill the recently completed breach and have similar characteristics to the rock drain on the north fork. In order to complete the construction of a dumped fill rock drain at this location the estimated construction cost would be about $800,000.

Although the current assessment performed for scenario 5 consists of attenuation structures using flow-through rock drains this need not be the case. Other types of structures are possible but for the purpose of this discussion only rock drain type structures were estimated. Any decision to proceed with the construction of attenuation structures must include an assessment of the failure mode of the structure used, such as clogging of a rock drain or blockage of a spillway.

The canal constructed for the attenuated peak flood values has many of the same risk considerations as the larger canal. However, if the peak flood value could be lowered enough in the attenuation structures the canal could conceivably be left in its current location in the two weir sections, thereby removing the following risks:

- settlement of the downstream dike in the new section of the canal
- induced movement from thawing permafrost in the new canal
- possibility of canal blockage from the debris in the debris fan area.

The above risks will be replaced with the risk of blockage of the rock drains by sediments. Additional effort needs to be made, but on a preliminary basis it is thought that the risk presented by drain blockage would be similar to those presented from the debris fan.

7.0 PMF FOR EXISTING SPILLWAYS

Various assumptions were made in determining the size of the PMF for the Intermediate and the Cross Valley Dams spillways. The basis of the PMF estimate presented in Table 5 was the methodology presented in the 2003 study (nhc 2004). This method included determining the peak flood value on the basis of the PMP and the developed relationship between size of the drainage basin and the time to peak. There has been no account taken of the nature of the basins, that is, the peak PMF was estimated purely on the size of the basin. Therefore the size of the PMF at the Cross Valley Dam spillway does not take into account the potential routing of the peak flood through the Intermediate Dam. This is a conservative assumption.

The estimated peak flood values for the Intermediate Dam and the Cross Valley Dam are provided in Table 5, as detailed in Appendix III.

Table 5 PMF Estimate for Existing Tailings Dam Spillways

Dam	Drainage Area (km^2)	PMF Peak Discharge (m^3/s)
Intermediate Dam	9.7	95
Cross Valley Dam	11.5	110

The existing spillways of these dams have a stated capacity of 100 m^3/s. Although this analysis is very preliminary, this indicates that only minor upgrading of the existing spillways would be required to upgrade them to "closure" spillways.

8.0 ESTIMATE OF SEDIMENT LOADING

As part of this study an estimate amount of sediment generated in the tailings impoundments was made. This assessment considered the current configuration as well as some future scenarios for tailings cover. This assessment included determining the effects of the current pond on the amount of sediment that would leave the tailings area, if the water was allowed to be released from the existing spillways.

Five different scenarios have been considered for this preliminary analysis. The scenarios are as follows:

1. The existing arrangement;
2. The existing arrangement with the Polishing Pond removed;
3. The existing arrangement with the Intermediate Pond and Polishing Pond removed;
4. Tailings with a earth fill cover; and
5. Tailings with a water cover.

The analysis considered three main factors in the estimation of the total sediment loading: erodibility of tailings (amount of sediment generated in the tailings impoundment), sedimentation of tailings in Intermediate Pond and the amount of tailings released to the environment.

The amount of sediment generated within the tailings impoundment was calculated based on the *Revised Universal Soil Loss Equation for Applications in Canada* (RUSLEFAC), prepared by Agriculture and Agri-Food Canada.

The portion of the sediment that will settle in the intermediate and polishing pond was calculation based on settling velocities of particles in still water (Droste 1997).

8.1 Scenario 1, 2 and 3 – Tailings Not Covered

In this series of analysis, scenario 1, 2, and 3, it was assumed that the tailings remained uncovered. Scenario 1 considers leaving the ponds in there current arrangement. Scenario 2 considers the situation if the Polishing Pond is removed and scenario 3 considers the case if the existing intermediate pond is filled.

The R-factor in the RUSLEFAC was unavailable for the Faro Mine Site. Typically charts of R-factor are calculated and provided for all regions of southern Canada by agriculture departments. The R-factor was the main unknown in the analysis, the effects the choice in the R-factor are shown in Table IV-1 and Figure IV-1 in Appendix IV. Based on a comparison to

Northern Alberta and British Columbia a conservative R-Factor of 400 was chosen as our best estimate. It is recommended that a review of the site specific weather information should be undertaken to determine the site specific R-factor.

The estimated amount of sediment generated in the tailings impoundment for an R-factor of 400 is 3,150 tonnes per year. This amount of sediment is rated as a moderate soil loss, Figure IV-1 (RUSLEFAC 1997). If the rate of erosion is maintained at this constant rate the intermediate pond will fill with sediment in about 380 years. This estimate was based on the recent bathymetry of the pond provided by Gartner Lee Ltd., shown on Figure IV-2.

Based on the current surface area of the intermediate pond a theoretical particle size that would settle out of suspension was determined, Table IV-2. The settling velocity was determined based on the peak flow velocity from the PMF flood in the basing, Table 5 and the surface area of the intermediate pond. The settling velocity of individual particles was based on the assumptions that the surface area of the pond was constant, the particles are spheres, laminar flow in the pond, specific gravity of the particles was 2.65 and there was steady state uniform dispersion of particles. The grain size used in this analysis was based on fine tailings (Golder 2004).

For scenario 1 the sediment generated from the uncovered tailings was 3,150 tonnes, and about 5% of the clay sized particles (150 tonnes) would leave the intermediate pond and none would leave the polishing pond.

In scenario 2 the 150 tonnes of sediment leaving the intermediate impoundment would be discharged in to the environment.

In scenario 3 the entire 3,150 tonnes would be discharged to the environment.

These calculations for amount of sediment discharged to the environment depend on the assumption that the size of the intermediate pond is constant, however this pond will fill overtime, decreasing the settling time and increasing the proportion of the sediment discharged past the pond. The estimates of the velocity in the pond are very conservative, because they were calculated on the basis of the peak velocity in the PMF and only the fine portion of the tailings. It would be expected that all the sediments would have time to settle out under normal flow conditions and no sediment would leave the Intermediate pond.

8.2 Scenario 4 – Tailings with Earth Cover

In scenario 4 the tailings area is completely covered with an earth fill cover (or rockfill). If this earth/rock cover is assumed to be installed perfectly no tailings sediment will be generated. It was assumed that some potential problems could develop over time and expose between 1% and 10% of the surface with tailings. As detailed in Table IV-1, a total sediment load of tailings between 31 and 315 tonnes per year could result for this level of cover failure. This scenario also includes the assumption that

8.3 Scenario 5 – Water Cover For Tailings

For scenario 5 the tailings are covered in water. According to the methodology presented in Adu-Wusu 2001, when the water cover is 1 m deep, winds of 8 m/s (29 km/h) would be required to cause enough shear stress on the tailings to cause re-suspension. The required wind speed to cause re-suspension increases to 12 m/s (43.2 km/h) for a water cover of 2 m.

From the Canadian Climate Normals the mean wind speed for south-easterly winds are between 7.3 to 10.2 km/h for May to October. Maximum hourly wind speeds range from 29 to 37 km/h (nhc 2001). Little re-suspension of tailings would be expected if the depth of water cover was greater than 1 m and none if the water cover was greater than 2 m. Therefore no release of tailings sediment would be expected for a properly designed water cover.

9.0 CLOSURE

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

These options are only conceptual in nature and their technical feasibility has yet to be fully assessed. The conceptual design provided herein was undertaken as a scoping level study to potential options, costs and issues to be used as a basis for closure planning. The study has highlighted the need for stakeholders to establish an understanding of what needs to be achieved for the closure plan. Final decisions can not be made unless the closure objectives and goals are clearly identified. It is likely that some of these options will be removed from consideration because they fail to meet the basic objectives.

Yours truly,

BGC Engineering Inc.
Per

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Reviewed by:

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The Orthoshop 2003 1:20,000 and 1:10,000 scale airphotos of the Faro Mine, Project completed for SRK Consulting Inc.

PHOTOGRAPHS

This photo mosaic was taken from the north side of the Rose Creek Valley. Tailings impoundment in the foreground.

CLIENT:

Deloitte & Touche

REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED
	-	- -	-	-	-
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SCALE:	N/A
DATE:	NOVEMBER 2004
DRAWN:	SLF
DESIGNED:	GWF
CHECKED:	HHH
APPROVED:	

PROJECT
RCDC - CONCEPTUAL CLOSURE OPTIONS

TITLE
OVERVIEW OF RCDC

PROJECT No.	PHOTO No.	REV.
0257-028-02	1	0

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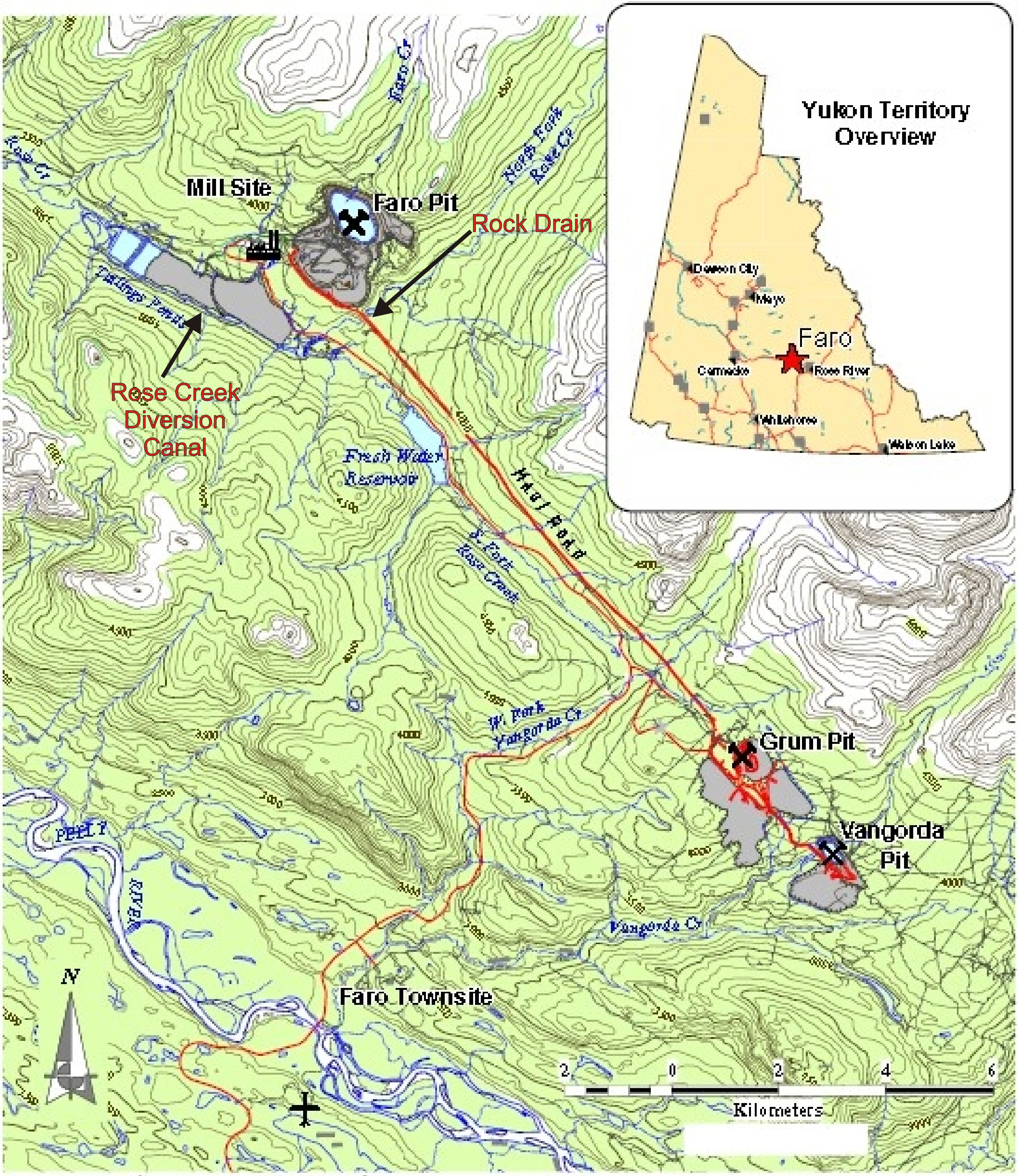
FIGURES



Note: Base map figure provided by Gartner Lee Limited.

SCALE:	As Shown	DESIGNED:	GWF
DATE:	November 2004	CHECKED:	HHH
DRAWN:	JMS	APPROVED:	

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PROJECT
RCDC - CONCEPTUAL CLOSURE OPTIONS

TITLE
SITE LOCATION PLAN

PROJECT No.	FIGURE No.	REV.
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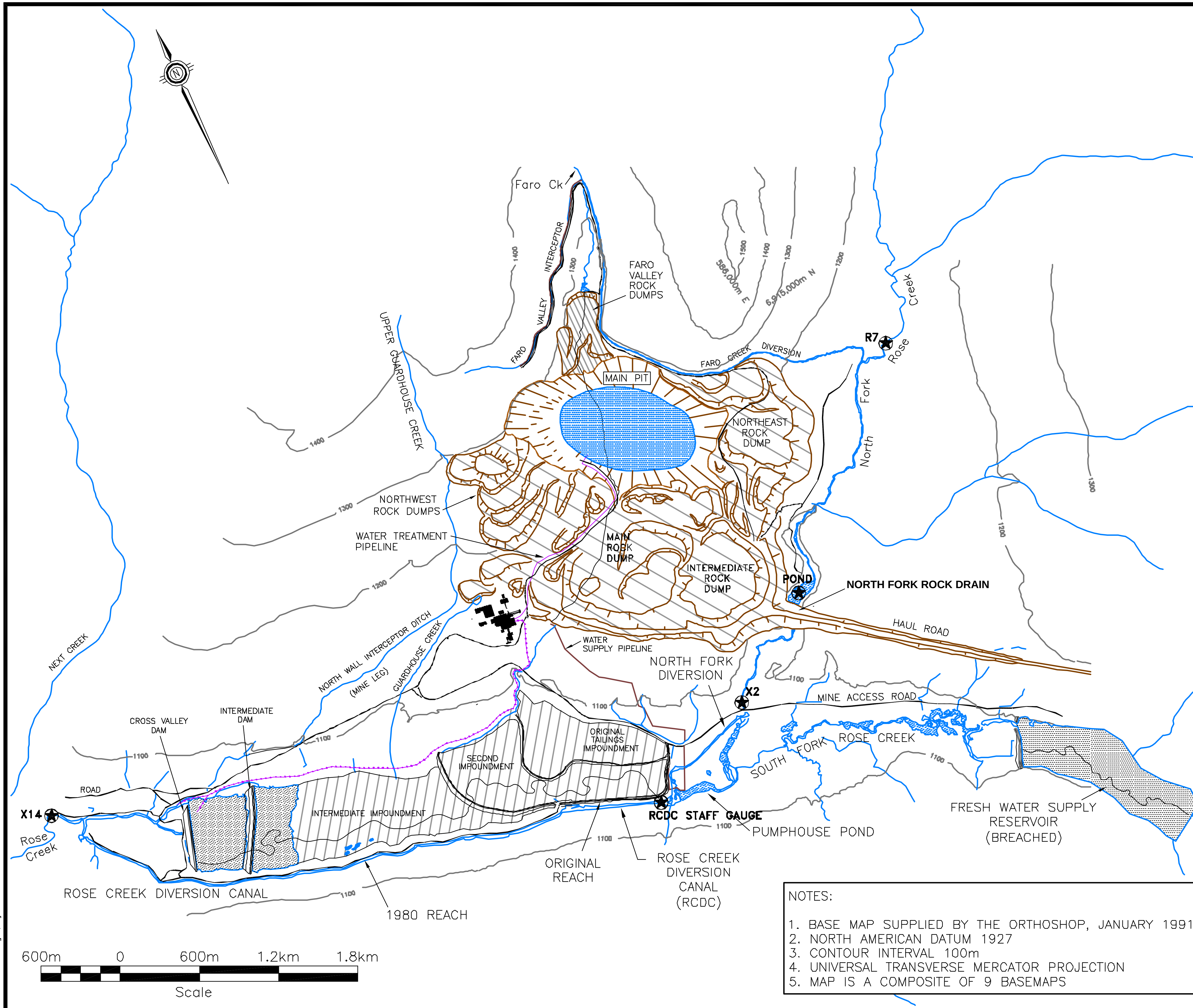

NOTES:

1. BASE MAP SUPPLIED BY THE ORTHOSHOP, JANUARY 1991
2. NORTH AMERICAN DATUM 1927
3. CONTOUR INTERVAL 100m
4. UNIVERSAL TRANSVERSE MERCATOR PROJECTION
5. MAP IS A COMPOSITE OF 9 BASEMAPS

CLIENT: Deloitte & Touche

LEGEND:

- ROADS
- EXISTING DRAINAGE
- ORIGINAL DRAINAGE
- EFFLUENT PIPELINE
- PIPELINE
- WATER MONITORING SITE

REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

SCALE:	AS SHOWN
DATE:	NOV 2004
DRAWN:	GCB
DESIGNED:	GWF
CHECKED:	HHH
APPROVED:	

DRAFT

PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS

TITLE: SITE PLAN

PROJECT No.	FIG No.	REV.
0257–028–02	2	0

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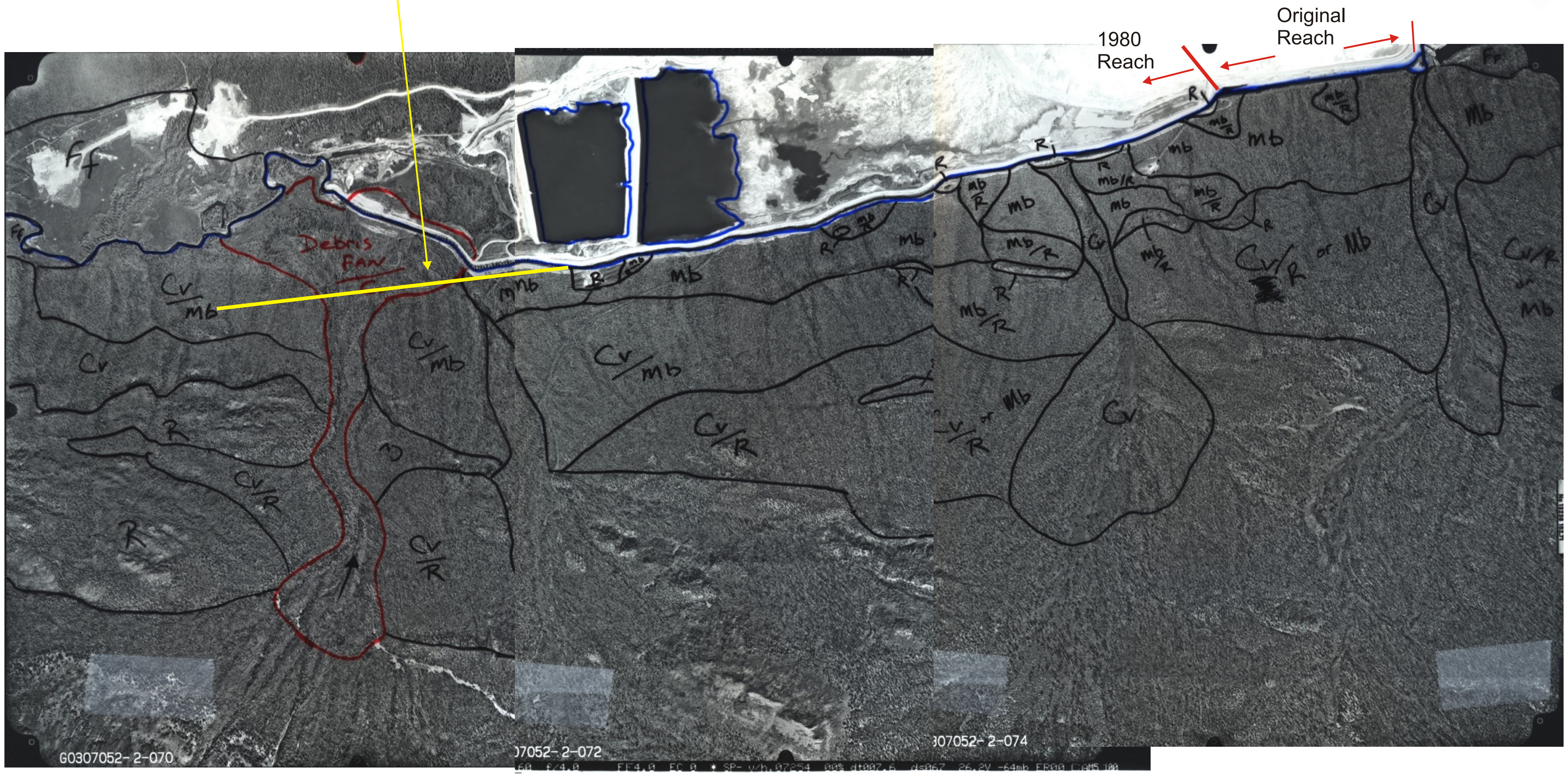
Approximate location of proposed canal.



Source: Airphotos G0307052-2-(070,072,074)
Scale 1:10,000 taken 07-25-03
The Orthoshop, Calgary, AB

Legend:
Ff- Fluvial Fan
Cv - Colluvium
R - Rock
Cv/R - Thin Colluvium Overlying Rock
Mb - Moraine
Mb/R - Thin Moraine overlying Rock
Cv/Mb - Thin Colluvium Overlying Moraine

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SCALE: N/A
DATE: NOVEMBER 2004
DRAWN: SLF
DESIGNED: GWF
CHECKED: HHH
APPROVED:

PROJECT
RCDC - CONCEPTUAL CLOSURE OPTIONS

TITLE
AIRPHOTO INTERPRETATION

PROJECT No.	FIGURE No.	REV.
0257-028-02	3	0

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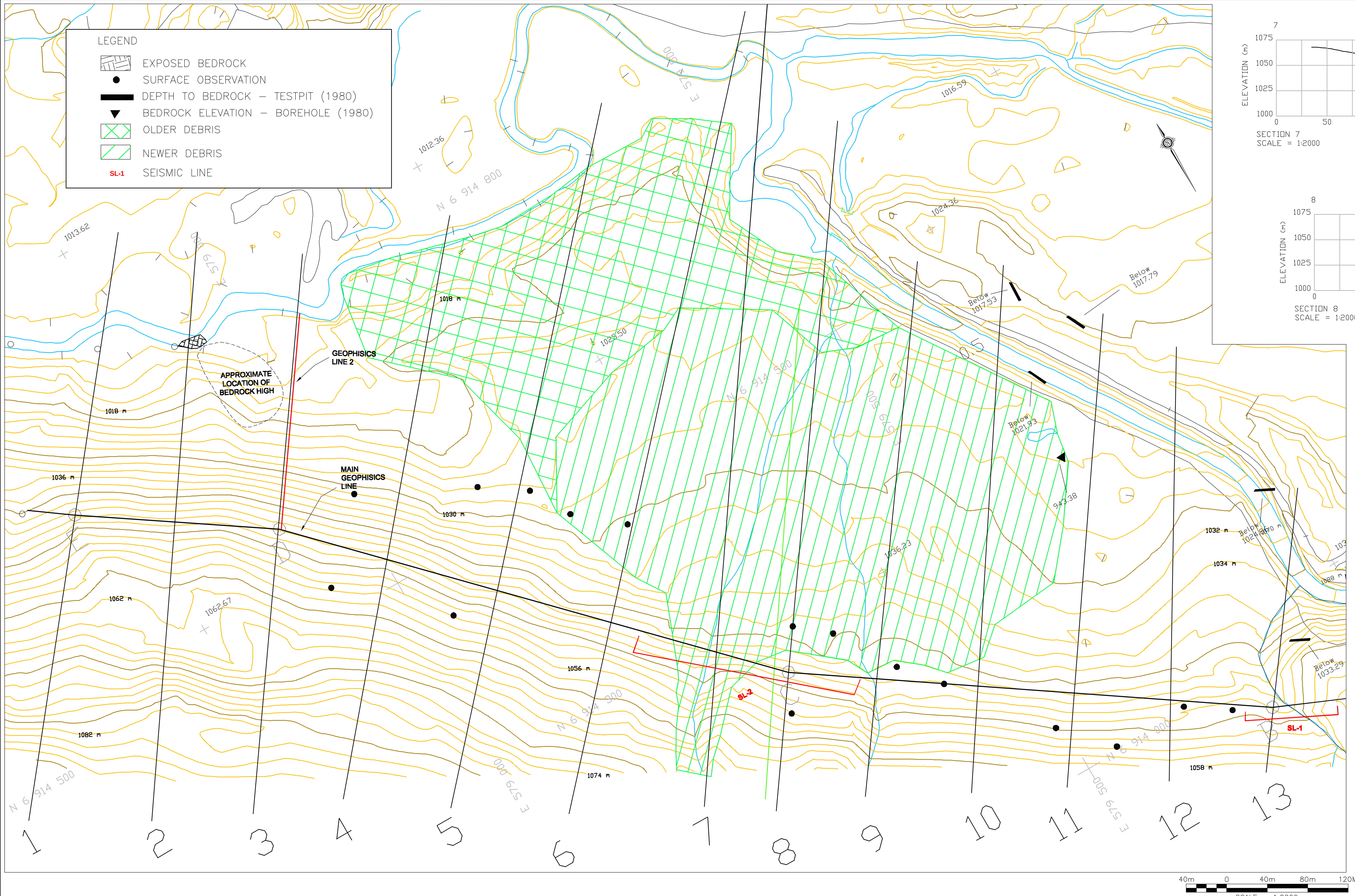

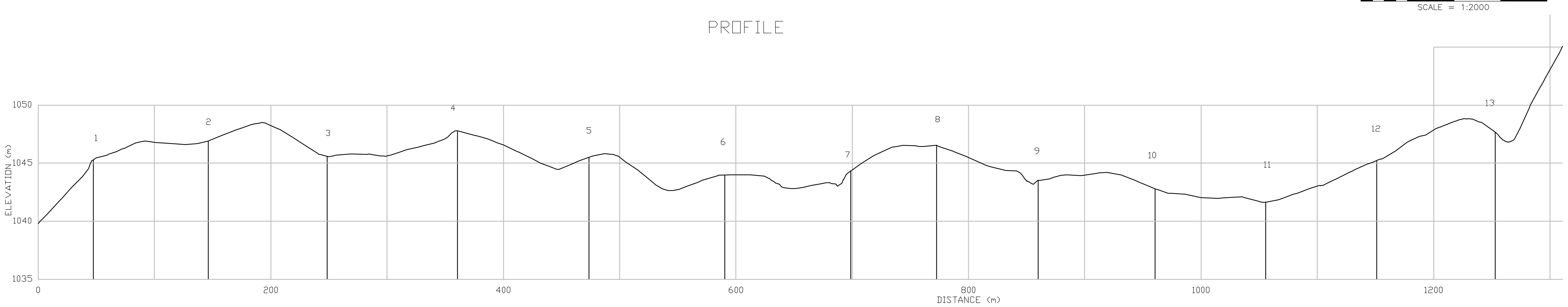

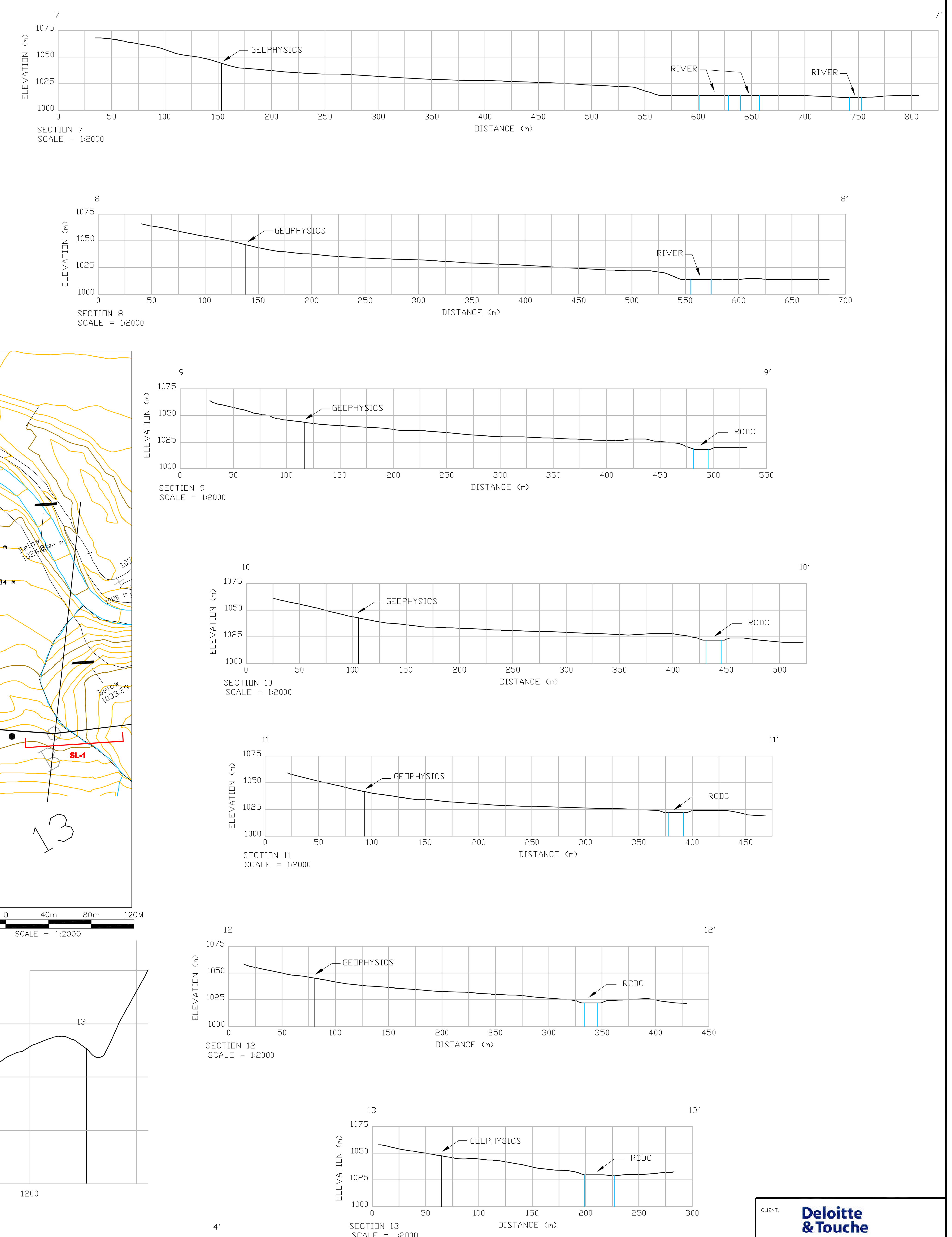

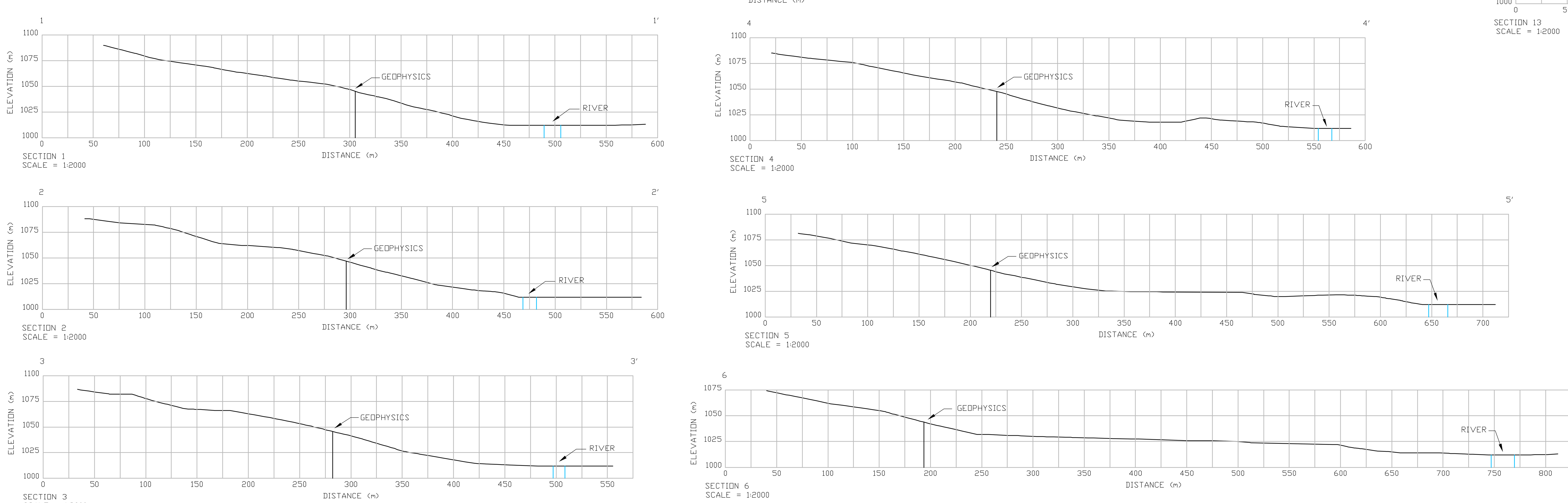

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REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

SCALE:	AS SHOWN
DATE:	NOV 2004
DRAWN:	CJT
DESIGNED:	GWF
CHECKED:	HHH
APPROVED:	

DRAFT

PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS

TITLE: PLAN AND SECTION 1

PROJECT No.	FIG No.	REV.
0257-028	4	0

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SCALE:	AS SHOWN
DATE:	NOV 2004
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PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS

TITLE: PLAN AND SECTION 2

PROJECT No.	FIG No.	REV.
0257-028	5	0

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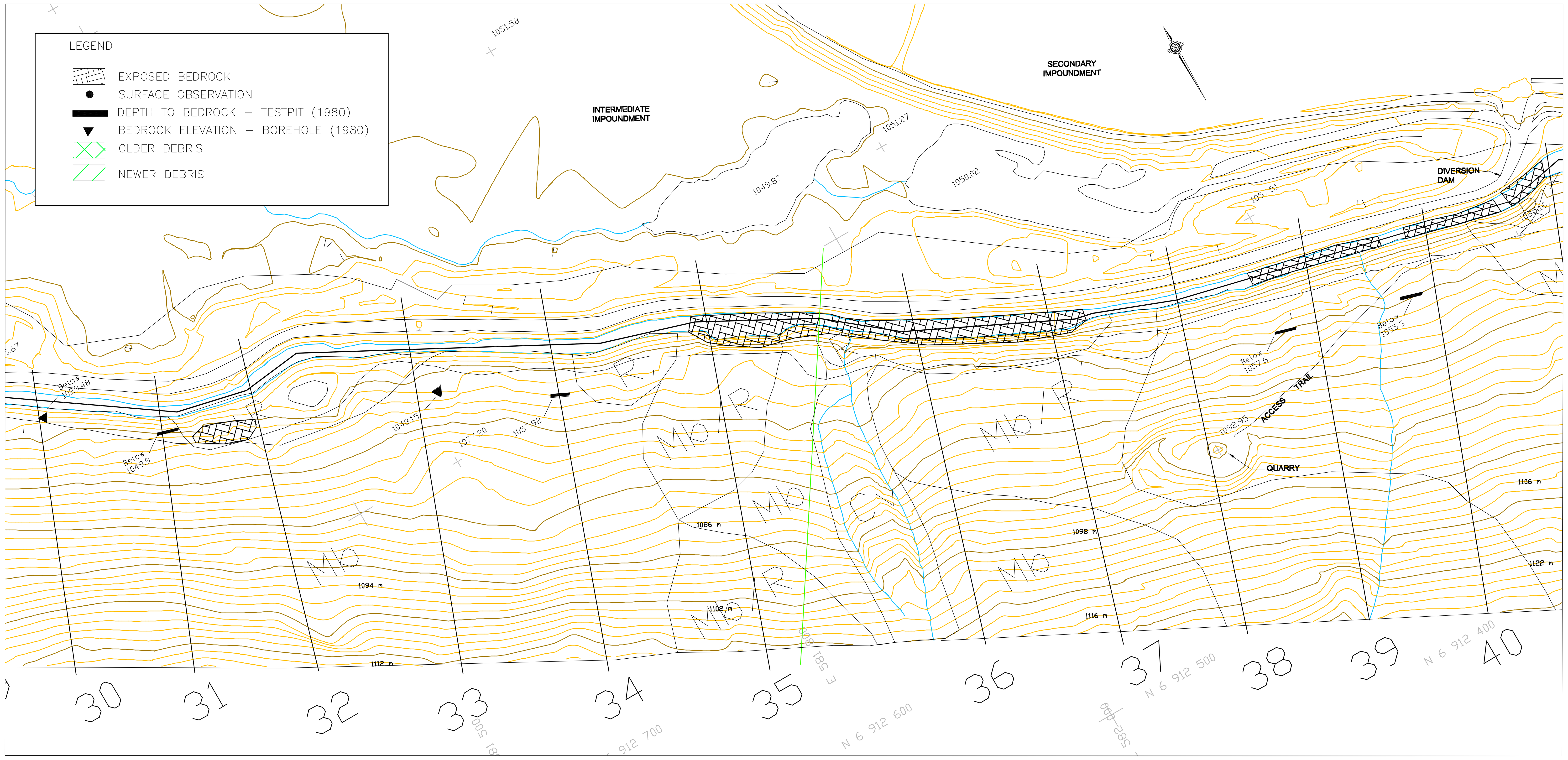

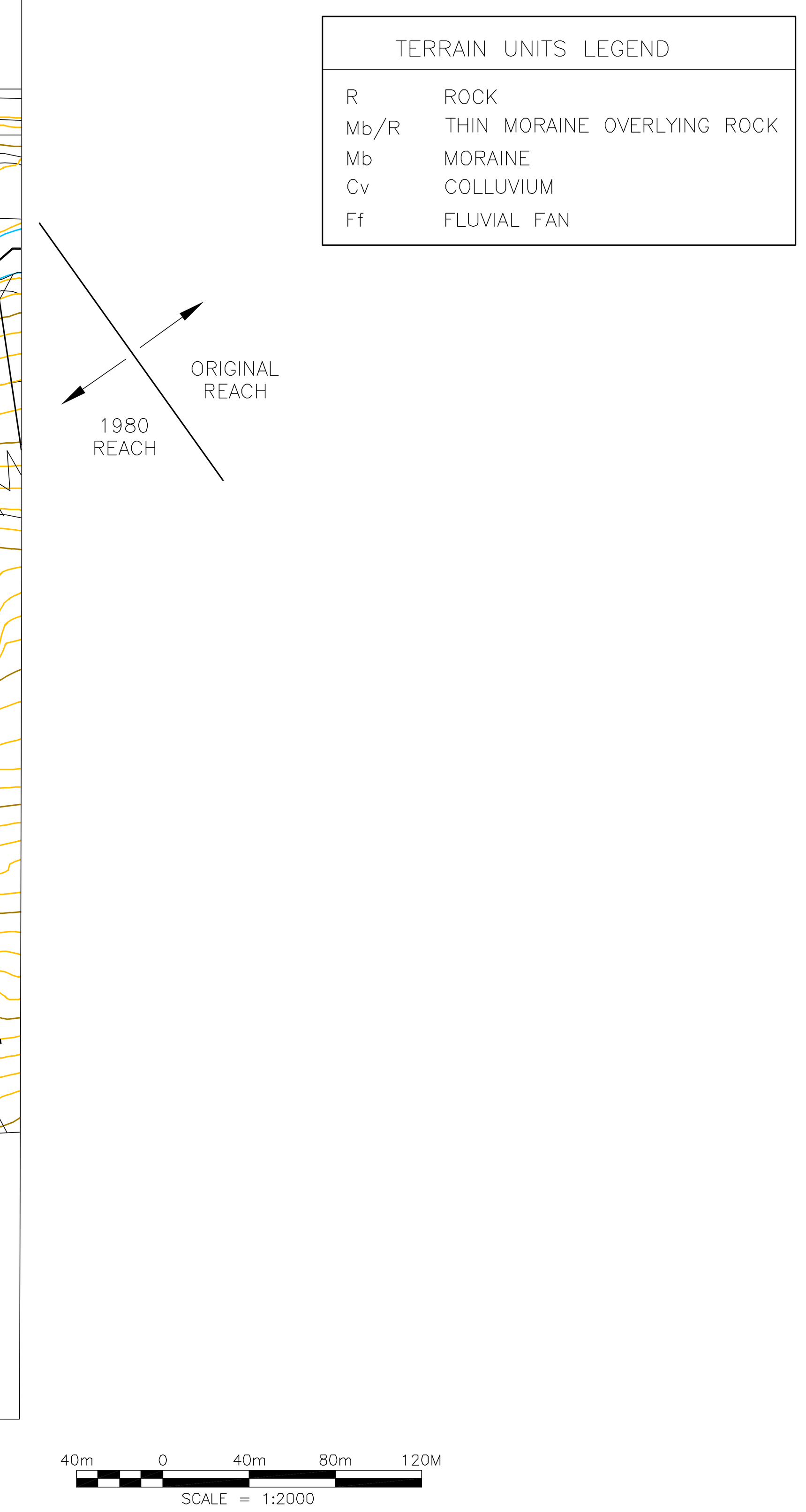

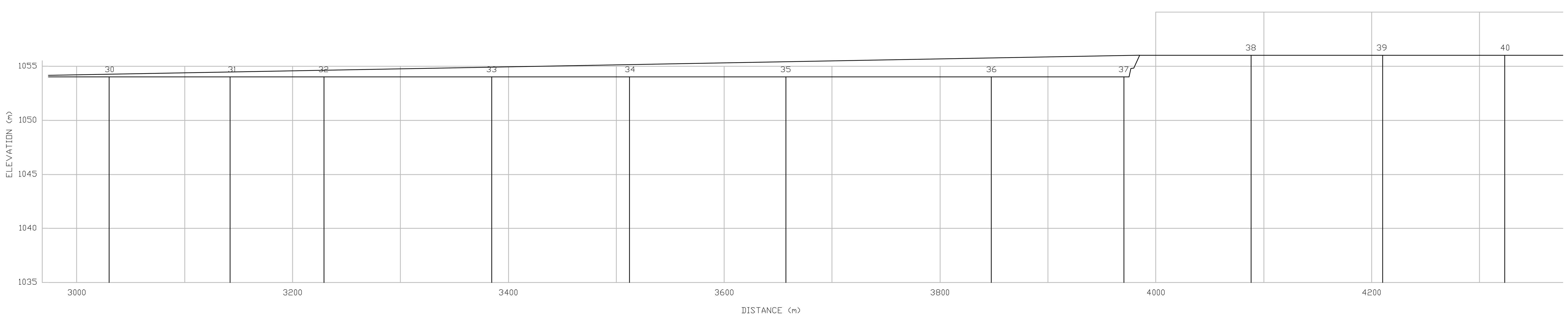

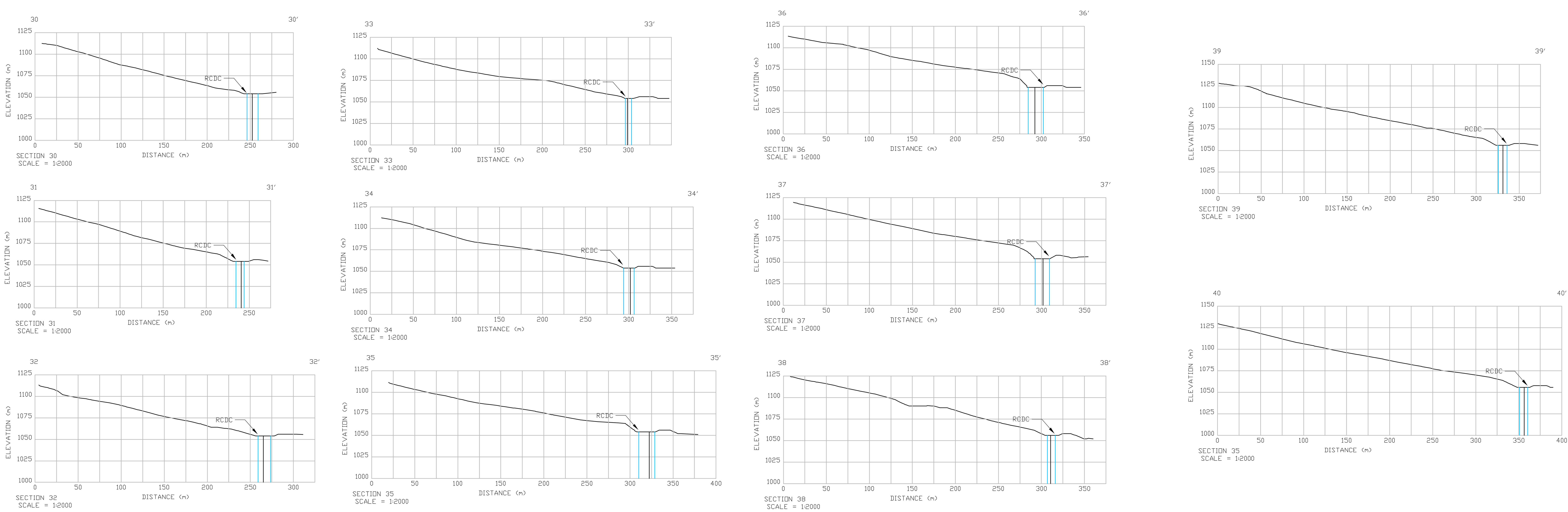

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REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

SCALE: AS SHOWN
DATE: NOV 2004
DRAWN: CJT
DESIGNED: GWF
CHECKED: HHH
APPROVED:

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PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS
TITLE: PLAN AND SECTION 3

PROJECT No.	FIG No.	REV.
0257–028	6	0

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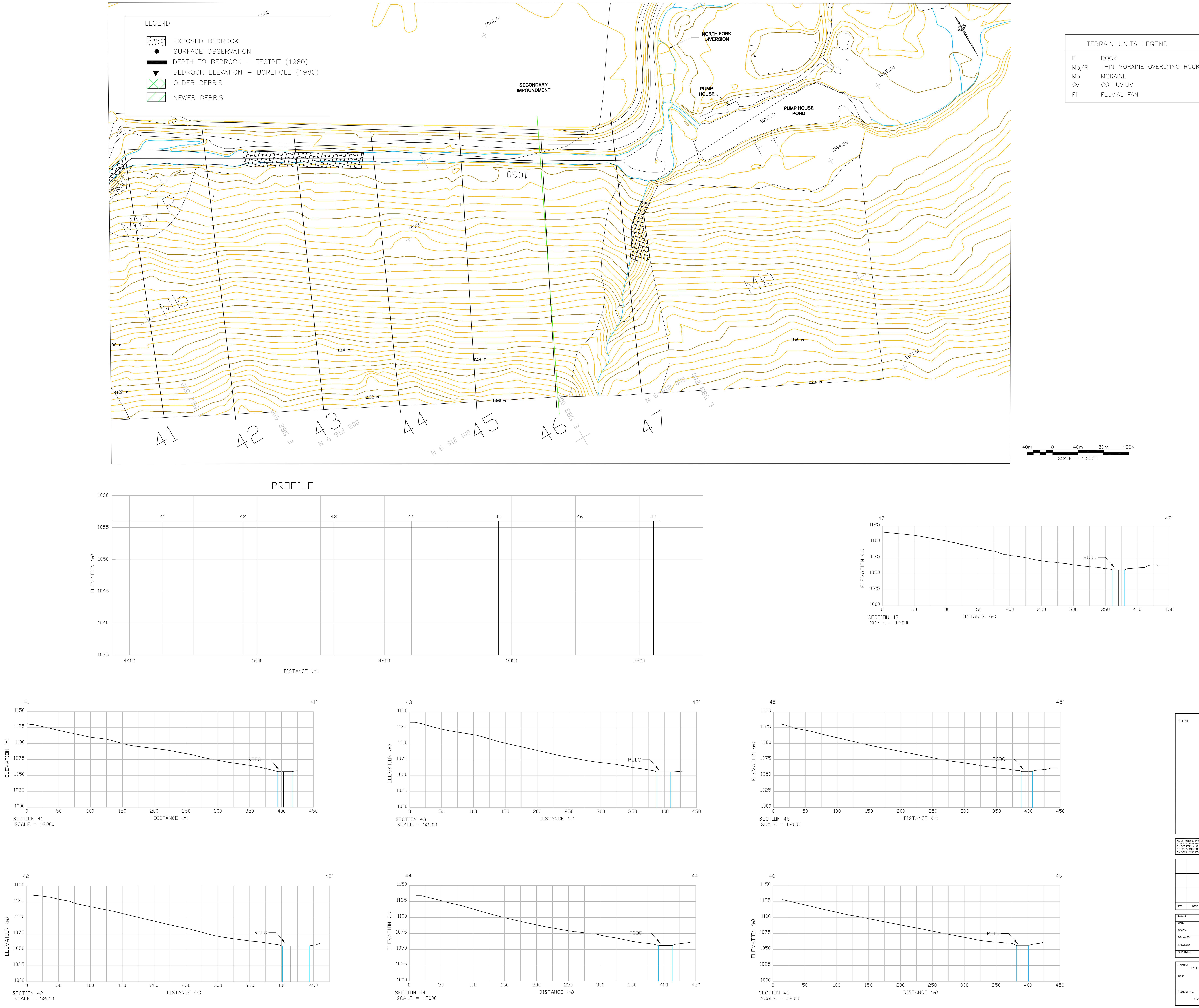

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REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

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DATE:	NOV 2004
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DESIGNED:	GWF
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DRAFT

PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS

TITLE: PLAN AND SECTION 4

PROJECT No.	FIG No.	REV.
0257-028	7	0

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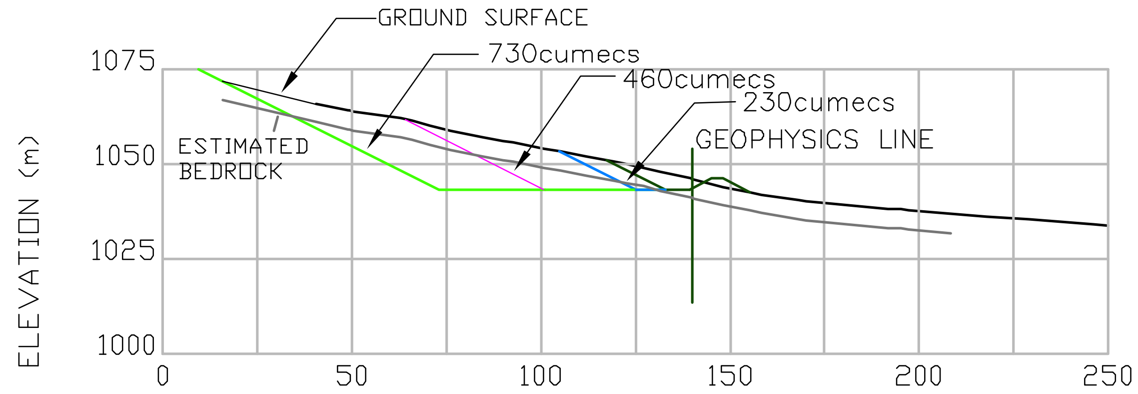
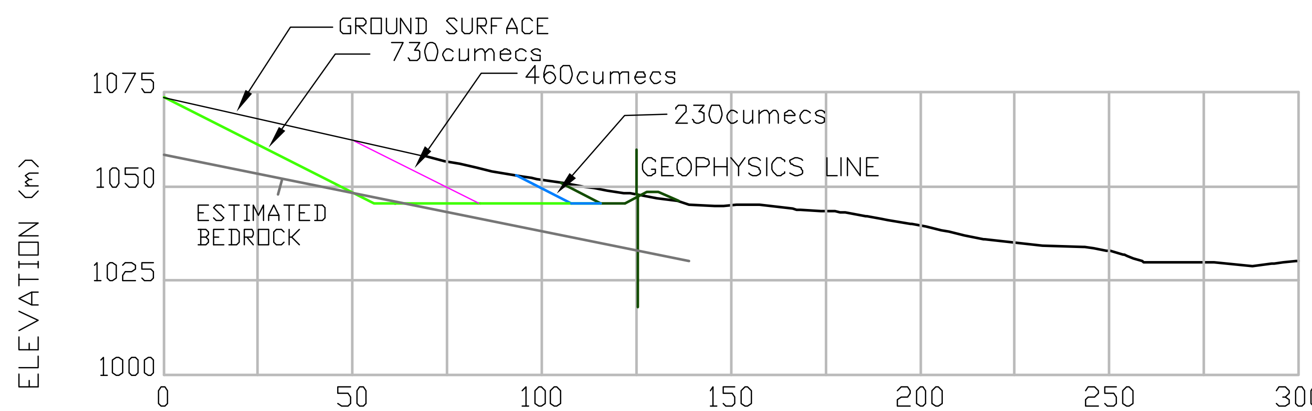


SECTION 8
SCALE = 1:2000

730 cumecs	460 cumecs	230 cumecs	120 cumecs
Rock: 540 m2	Rock: 135 m2	Rock: 6 m2	Rock: 0 m2
Common: 564 m2	Common: 330 m2	Common: 120 m2	Common: 61 m2
Total Area: 1104 m2	Total Area: 465 m2	Total Area: 126 m2	Total Area: 61 m2



SECTION 13
SCALE = 1:2000

730 cumecs	460 cumecs	230 cumecs	120 cumecs
Rock: 10 m2	Rock: 0 m2	Rock: 0 m2	Rock: 0 m2
Common: 989 m2	Common: 376 m2	Common: 95 m2	Common: 44 m2
Total Area: 999 m2	Total Area: 376 m2	Total Area: 95 m2	Total Area: 44 m2

NOTE:
1. SECTIONS SHOW EXCAVATION SECTION AND EXCAVATION VOLUMES FOR VARIOUS PMF FLOW SCENARIOS ($230m^3/s$, FOR $60m^3/s$, and $730m^3/s$).

0257-028-02 003 FIG8-9-10.dwg

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REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

SCALE:	AS SHOWN
DATE:	NOV 2004
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DRAFT

PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS

TITLE: SCENARIO 4/5 – NEW CANAL REACH

PROJECT No.	FIG No.	REV.
0257-028-02	8	0

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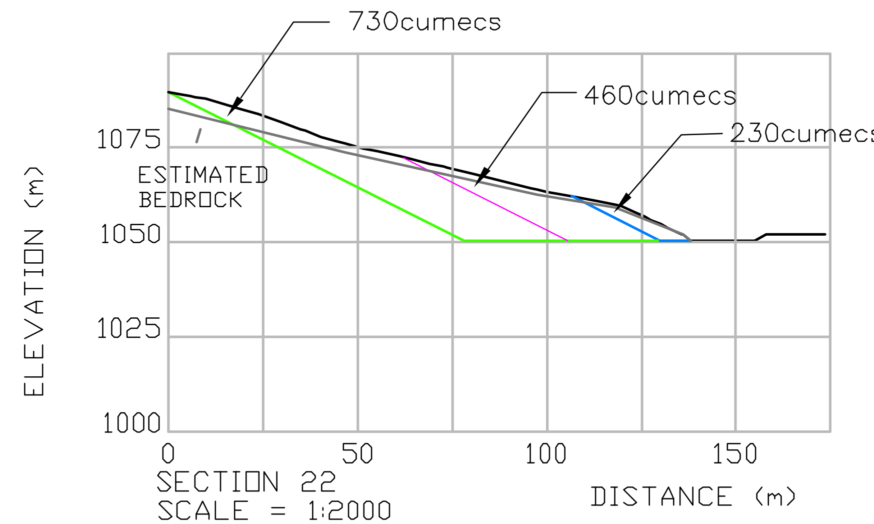
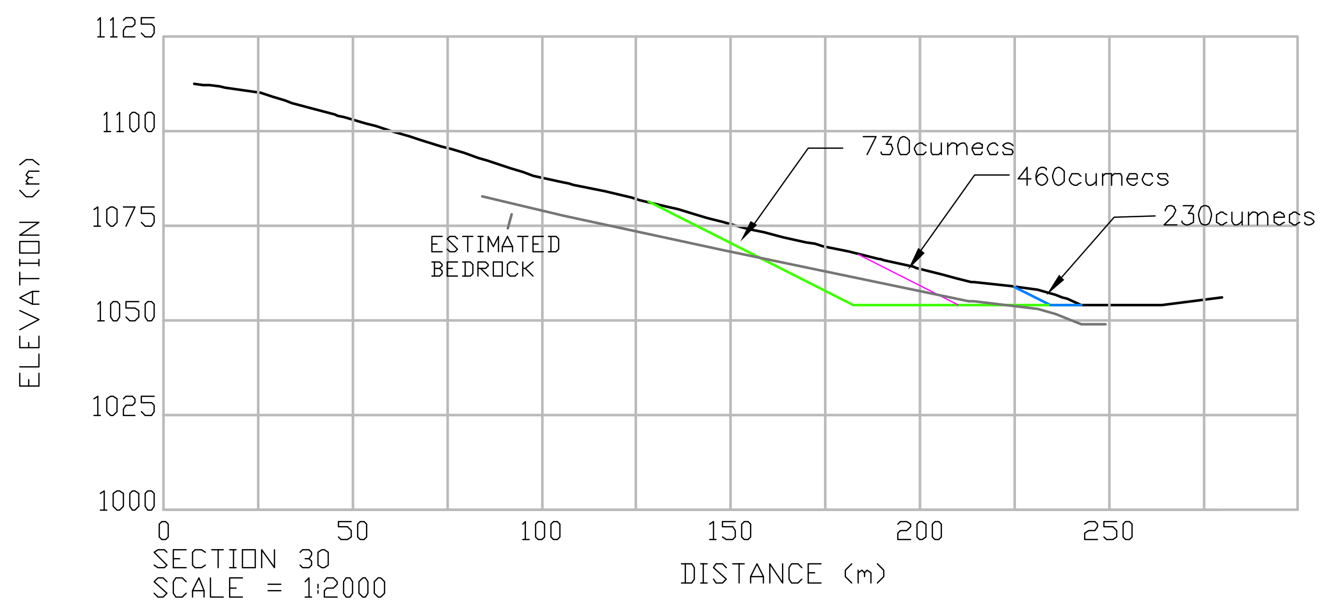
NOTE:
1. SECTIONS SHOW EXCAVATION SECTION AND EXCAVATION VOLUMES FOR VARIOUS PMF FLOW SCENARIOS ($230m^3/s$, FOR $60m^3/s$, and $730m^3/s$).



730 cumecs
Rock: 1090 m2
Common: 254 m2
Total Area: 1344 m2

460 cumecs
Rock: 396 m2
Common: 90 m2
Total Area: 486 m2

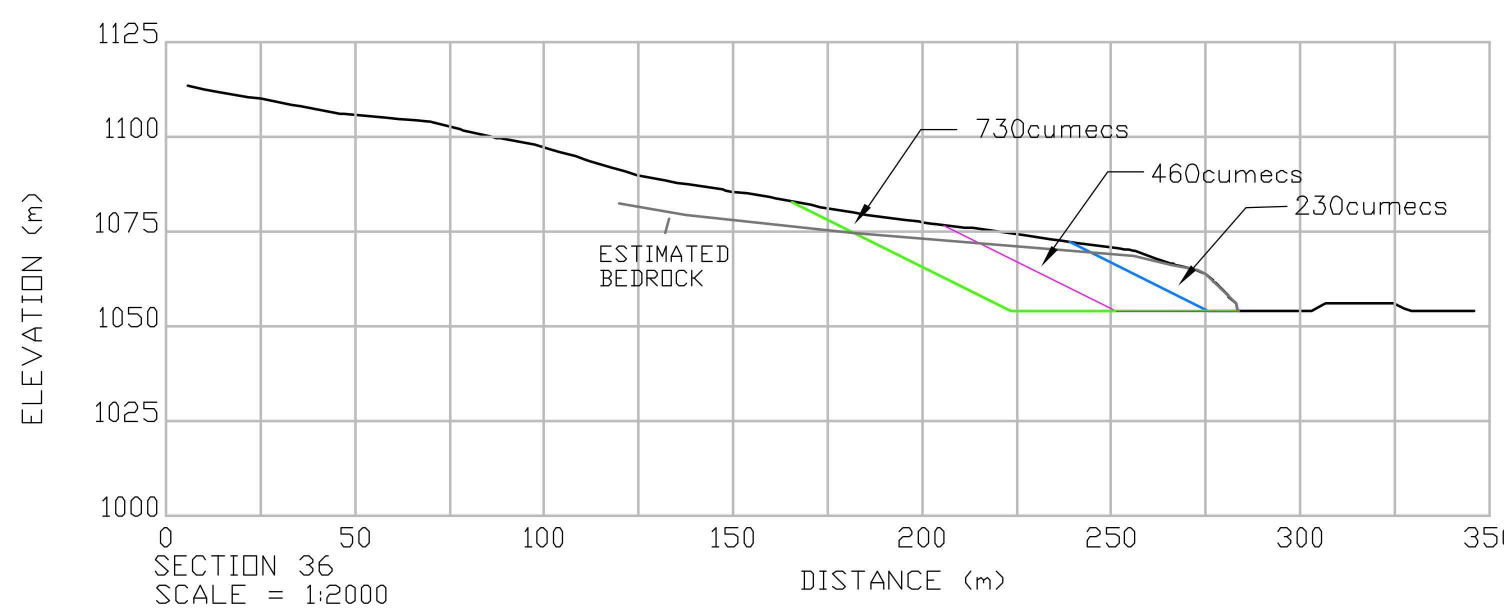
230 cumecs
Rock: 65 m2
Common: 16 m2
Total Area: 81 m2



730 cumecs
Rock: 226 m2
Common: 566 m2
Total Area: 792 m2

460 cumecs
Rock: 14 m2
Common: 208 m2
Total Area: 222 m2

230 cumecs
Rock: 0 m2
Common: 29 m2
Total Area: 29 m2



730 cumecs
Rock: 1144 m2
Common: 312 m2
Total Area: 1456 m2

460 cumecs
Rock: 613 m2
Common: 135 m2
Total Area: 748 m2

230 cumecs
Rock: 209 m2
Common: 40 m2
Total Area: 249 m2

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REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

SCALE:	AS SHOWN
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PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS

TITLE: SCENARIO 4/5 – 1980 CANAL REACH

PROJECT No.	FIG No.	REV.
0257–028–02	9	0

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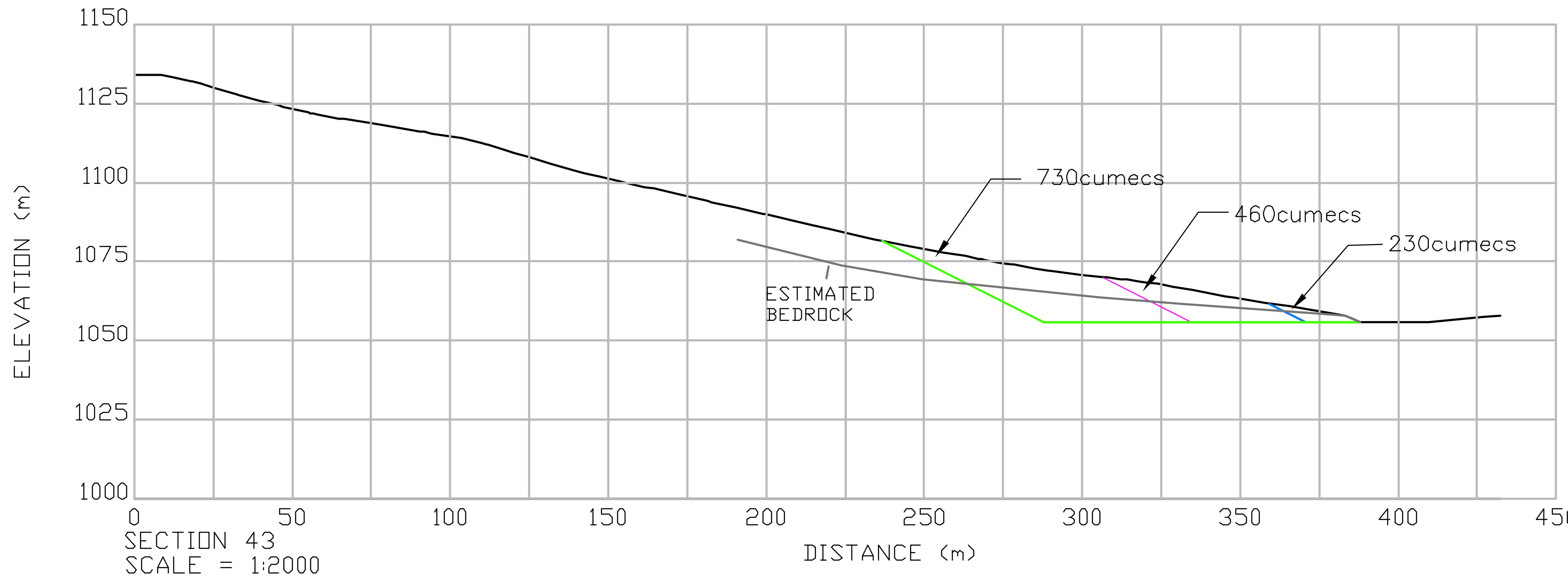
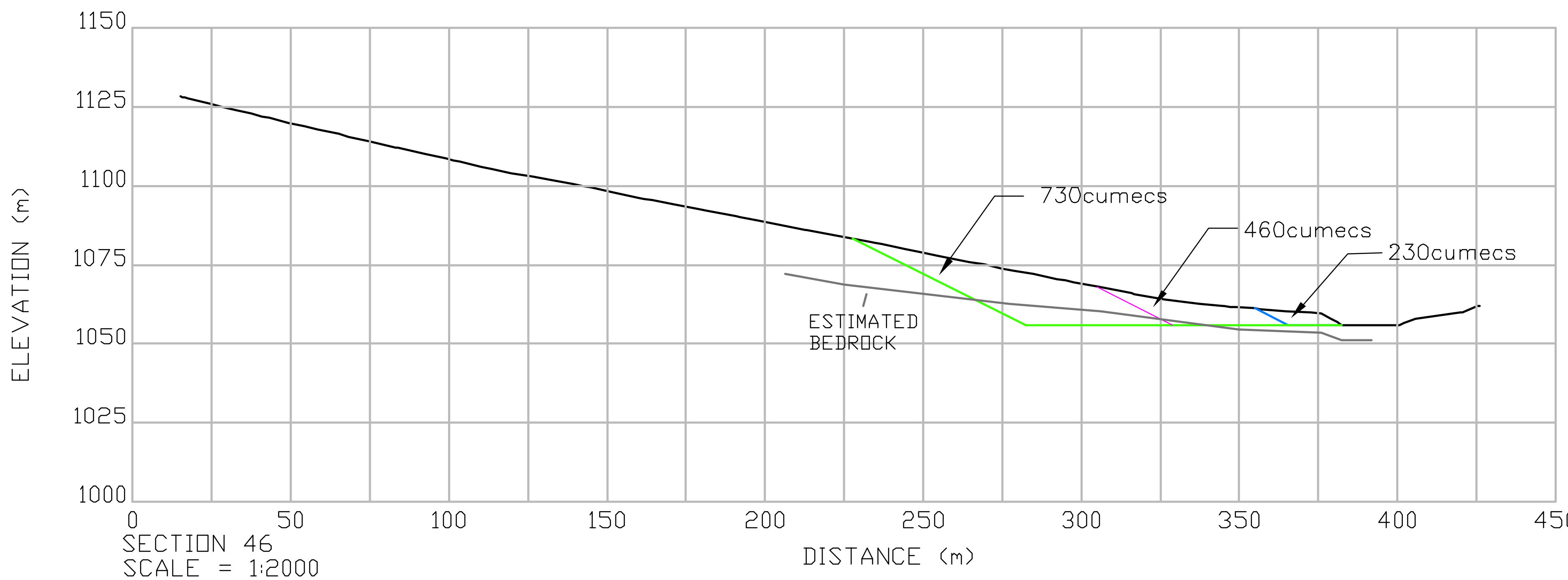


730 cumecs
Rock: 640 m2
Common: 797 m2
Total Area: 1437 m2

460 cumecs
Rock: 222 m2
Common: 214 m2
Total Area: 436 m2

230 cumecs
Rock: 44 m2
Common: 21 m2
Total Area: 65 m2



730 cumecs
Rock: 226 m2
Common: 1055 m2
Total Area: 1281 m2

460 cumecs
Rock: 8 m2
Common: 351 m2
Total Area: 359 m2

230 cumecs
Rock: 0 m2
Common: 77 m2
Total Area: 77 m2

NOTE:
1. SECTIONS SHOW EXCAVATION SECTION AND EXCAVATION VOLUMES FOR VARIOUS PMF FLOW SCENARIOS ($230m^3/s$, FOR $60m^3/s$, and $730m^3/s$).

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REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

SCALE: AS SHOWN
DATE: NOV 2004
DRAWN: CJT
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DRAFT

PROJECT: RCDC – CONCEPTUAL CLOSURE OPTIONS
TITLE: SCENARIO 4/5 – ORIGINAL CANAL REACH

PROJECT No.	FIG No.	REV.
0257–028–02	10	0

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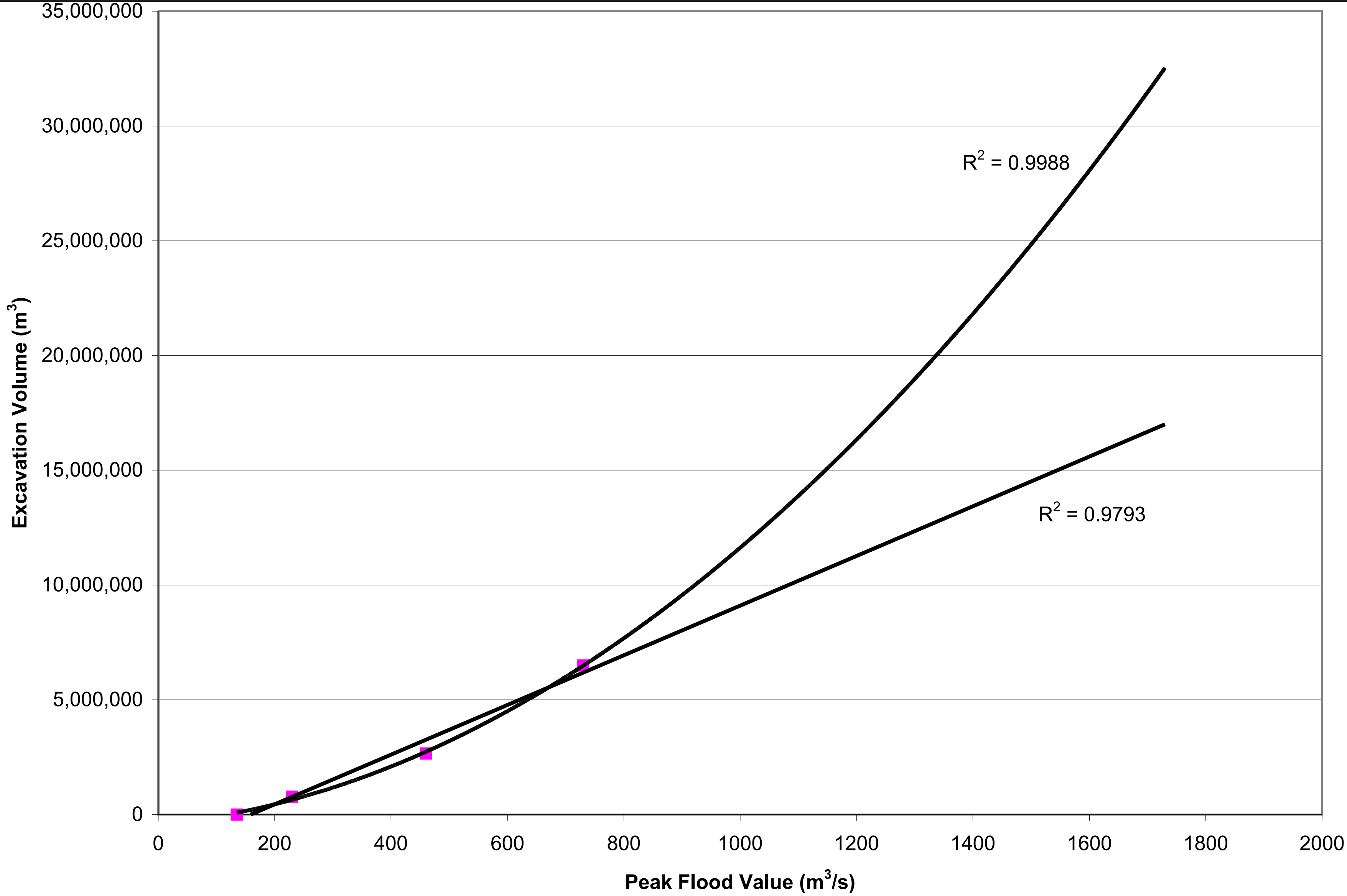
0257–028–02 003 FIG8–9–10.dwg



DATE: NOV 2004	DRAWN SLF

REFERENCED DRAWING DESCRIPTION

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CLIENT: Deloitte & Touche

PROJECT: RCDC - CONCEPTUAL CLOSURE OPTIONS

TITLE: PEAK FLOOD AND EXCAVATION VOLUME RELATIONSHIP

PROJECT No.	FIGURE No.	REV.
0257-028-02	11	0

APPENDIX I
GEOPHYSICAL ASSESSMENT

APPENDIX II
PRELIMINARY HYDRAULICS FOR ENLARGED RCDC

nhc
Memorandum

northwest
hydraulic
consultants

4823 - 99th street
edmonton, alberta T6E 4Y1
tel: (780) 436-5868
fax: (780) 436-1645

to:	Gerry Ferris	***sent:***	by email
company:	BGC Engineering Inc.	***date:***	9 November 2004
from:	Barry Evans	***pages:***	7

subject:	Faro Mine Site: North Fork Rose Creek - Required enlargement of RCDC to convey PMF without overtopping the right side dike

File: Memo6472/2_1.doc

1. INTRODUCTION

The Rose Creek Diversion Channel (RCDC) was originally designed in 1980 to have a hydraulic capacity equivalent to the 50-year return period flood and contingency capacity for the 500-year snowmelt flood, the latter assuming no freeboard. The 500-year flood estimate for the RCDC is 135 m^3/s. In October 2003, it was shown that the right dike would be overtopped in places during the 500-year flood event and it was recommended that the dike be raised to adequately convey the flood[1]. The right dike was subsequently raised during the summer of 2004[2].

In the June 2004 hydrotechnical study for closure planning, various options for conveying the probable maximum flood (PMF) of 730 m^3/s down the RCDC were reviewed[3]. One of the options considered included significantly increasing the height of the right side dike together with increasing the RCDC width by 5 m by excavating into the left side valley wall.

This memorandum provides estimates of the required increase in RCDC width to convey the PMF without overtopping the existing right dike, that is, conveying the PMF at a water level equivalent to the 500-year flood of 135 m^3/s. Three values for the PMF peak discharge were used representing three different upstream conditions, as described on the following page.

[1] *Northwest Hydraulic Consultants Ltd.(**nhc**) October2003. Rose Creek diversion channel at Faro Mine, Yukon. Hydrotechnical investigation. Prepared for BGC Engineering Inc.*
[2] *Verbal communication with Gerry Ferris of BGC Engineering Inc. summer 2004.*
[3] ***nhc** June 2004. Hydrotechnical study for closure planning, Faro Mine Site area, Yukon, final report. Prepared for SRK Consulting Inc.*

RCDC PMF (m^3/s)	Upstream Conditions
730	Haul Road embankment with its flow-through rock drain removed. This is the condition assumed in the majority of earlier studies.
460	Haul Road embankment rock drain retained thereby attenuating the North Fork Rose Creek PMF peak[4]
230	Haul Road embankment rock drain retained, plus construction of a similar rock drain across the Fresh Water Supply Dam (FWSD) to attenuate the South Fork Rose Creek PMF peak. The concept of attenuating South Fork flows is described below.

Potential South Fork Rose Creek PMF attenuation The preliminary assessment of the Haul Road embankment flow-through rock drain showed that the rock drain reduces the North Fork Rose Creek PMF peak discharge from 504 m^3/s to about 15 m^3/s. Currently, the Fresh Water Supply Dam (FWSD) does not attenuate South Fork flows[5]. Upgrading the FWSD to allow ponding of water behind the dam and incorporating a flow-through rock drain could realize a similar reduction in the PMF peak discharge to that attained by the Haul Road embankment.

Assuming that a rock drain installed in the FWSD reduces the South Fork Rose Creek PMF peak from 354 m^3/s to about 15 m^3/s - similar to the estimated reduction resulting from the Haul Road rock drain - the PMF peak discharge in the RCDC reduces to about 230 m^3/s. (The RCDC discharge is for the downstream end of the channel below the Cross Valley Dam.)

2. ROUTING OF RCDC PMF ESTIMATES

The HEC-RAS backwater model of the RCDC[6] was used for routing the three PMF values of 730, 460 and 230 m^3/s along the RCDC. The Figure 1 plan shows the RCDC and the location of cross-sections used in the model.

In operating the model, the RCDC channel bed width was gradually widened on the left side until the computed water levels did not overtop the right side dike, that is, the computed levels matched the levels for 500-year flood of 135 m^3/s in the existing channel.

[4] ***nhc**, November 4, 2004. Faro Mine Site: North Fork Rose Creek - Extreme flood hydrograph attenuation by Haul Road flow-through rock drain and effect on downstream RCDC flood peak. Memorandum to BGC Engineering Inc.*

[5] *A notch was recently cut through the FWSD embankment for dam safety reasons.*

[6] *From: **nhc**, October 2003. (See Footnote 1 on page 1.)*

3. RESULTS

The results of the required bed width increases showed that the channel could be divided into an upper and lower reach as there was a marked difference in results for the two reaches. Figure 2 shows the two reaches and Table 1 summarizes the required increases in bed width. For the 730 m^3/s PMF, the bed width needs to be increased by about 100 m and 60 m in the upper reach and lower reaches respectively. These bed width increases reduce to about 17 m and 8 m for the 230 m^3/s PMF.

For the purpose of determining excavation areas, CS24 was chosen as being representative of the lower reach and CS36 was chosen for the upper reach. The Figure 2 plots show the cross-sections and illustrate the bed width increases required to maintain the water levels at the 500-year flood level for the three PMF values. Excavation areas were measured from these cross-sections and are listed in Table 2. For the 730 m^3/s PMF, the required excavation areas are about 2085 m^2 and 736 m^2 in the upper reach and lower reaches respectively. These areas reduce to about 72 m^2 and 42 m^2 for the 230 m^3/s PMF.

Prepared by:

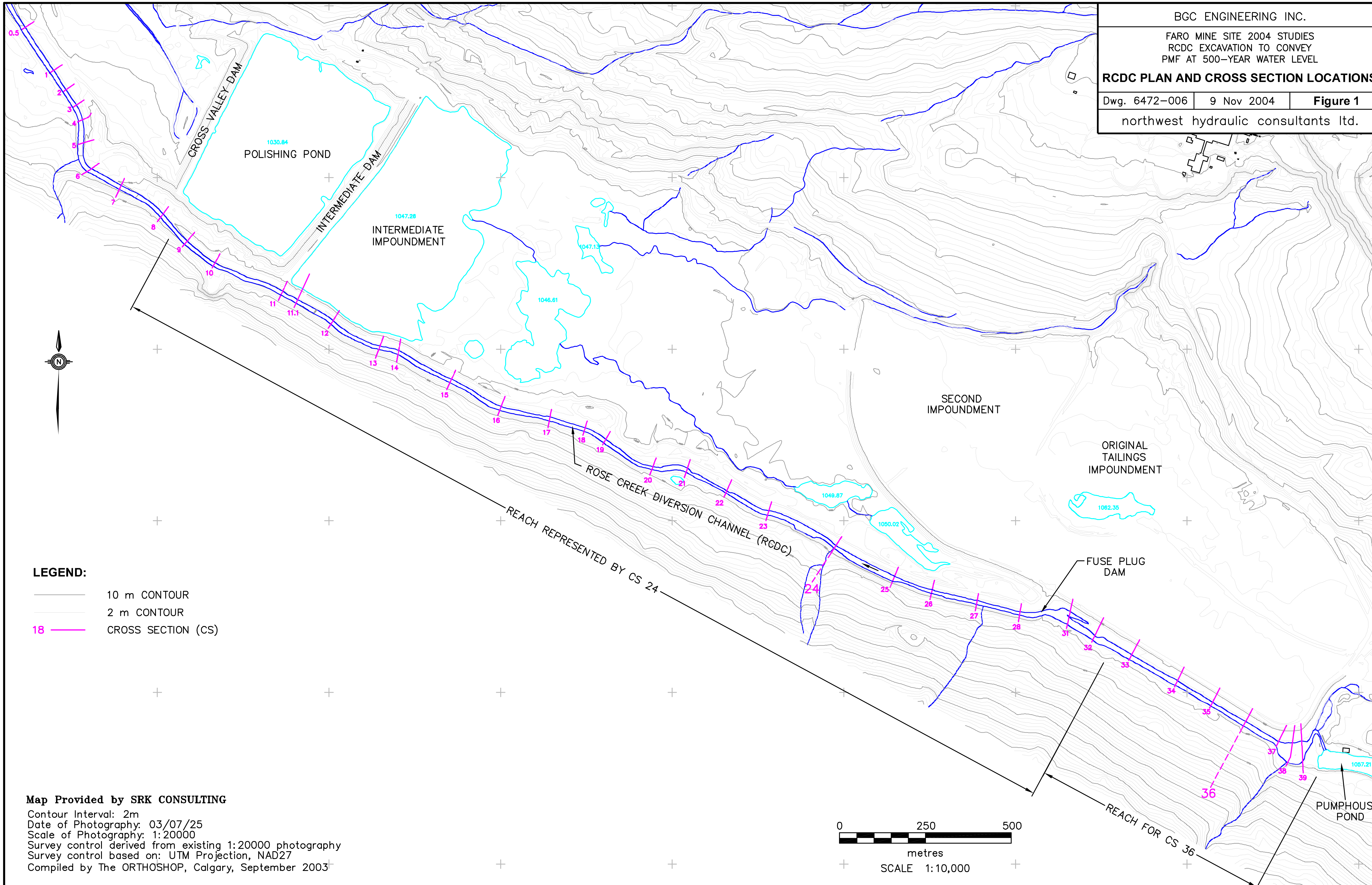
B.J. Evans, P.Eng.
Senior Engineer

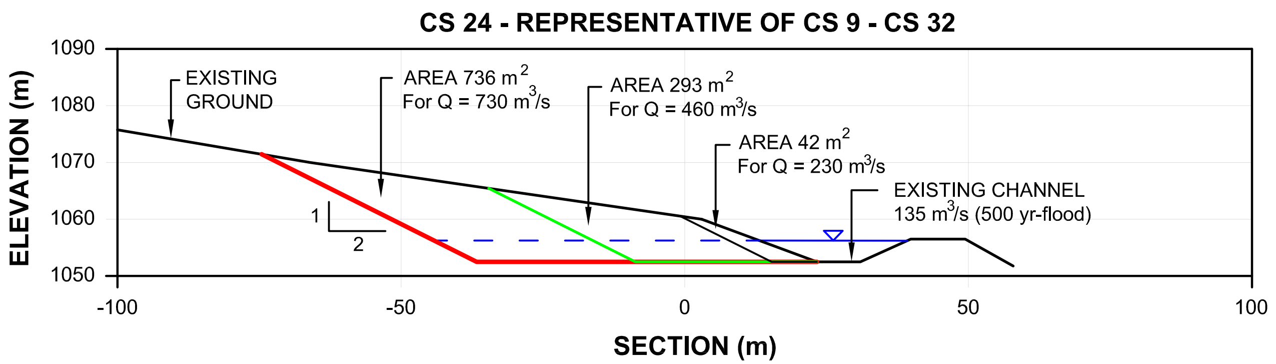
Table 1. Estimated required increase in RCDC bed width to limit water level from overtopping the right side dike

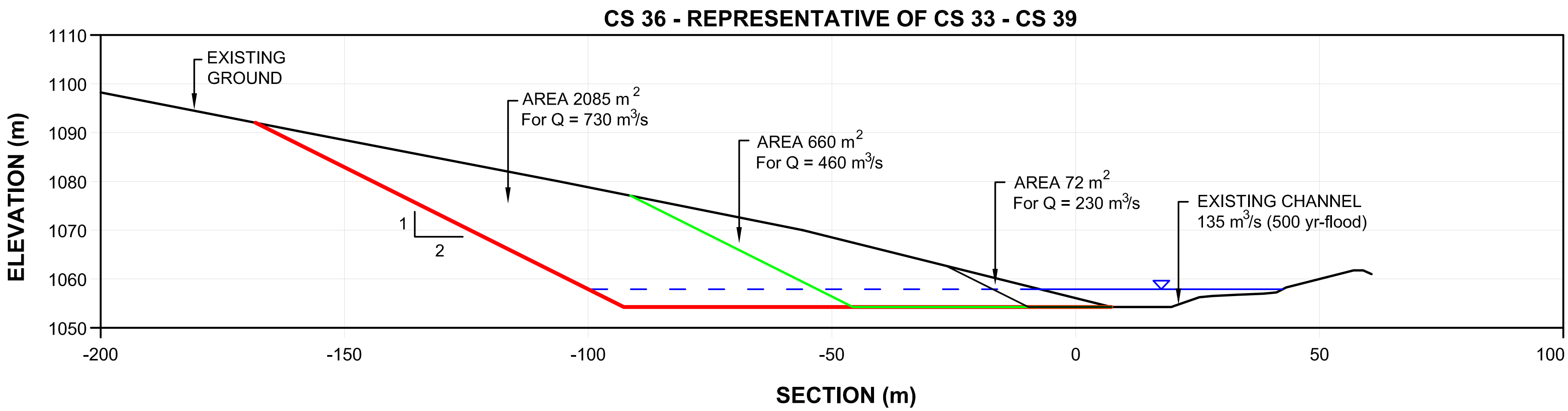
PMF Peak (m^3/s)	Approximate increase in bed width (m)	
	Lower Reach (CS9 - CS32)	Upper Reach (CS33 - CS39)
230	8	17
460	32	53
730	60	100

Table 2. Estimated increase in RCDC cross-sectional area required to limit water level from overtopping the right side dike

PMF Peak	Approximate excavation area (m^2) for	
(m^3/s)	Lower Reach (CS9 - CS32)	Upper Reach (CS33 - CS39)
230	42	72
460	293	660
730	736	2085


BGC ENGINEERING INC.
FARO MINE SITE 2004 STUDIES
RCDC EXCAVATION TO CONVEY
PMF AT 500-YEAR WATER LEVEL
RCDC PLAN AND CROSS SECTION LOCATIONS
Dwg. 6472-006
9 Nov 2004
Figure 1
northwest hydraulic consultants ltd.
CROSS VALLEY DAM
POLISHING POND
INTERMEDIATE DAM
INTERMEDIATE IMPOUNDMENT
SECOND IMPOUNDMENT
ORIGINAL TAILINGS IMPOUNDMENT
FUSE PLUG DAM
ROSE CREEK DIVERSION CHANNEL (RCDC)
REACH REPRESENTED BY CS 24
REACH FOR CS 36
PUMPHOUSE POND
LEGEND:
10 m CONTOUR
2 m CONTOUR
18 CROSS SECTION (CS)
Map Provided by SRK CONSULTING
Contour Interval: 2m
Date of Photography: 03/07/25
Scale of Photography: 1:20000
Survey control derived from existing 1:20000 photography
Survey control based on: UTM Projection, NAD27
Compiled by The ORTHOSHOP, Calgary, September 2003
0 250 500
metres
SCALE 1:10,000





BGC ENGINEERING INC.

FARO MINE SITE 2004 STUDIES
RCDC EXCAVATION TO CONVEY
PMF AT 500–YEAR WATER LEVEL

EXCAVATION AREAS FOR RCDC

Dwg. 6472–701	8 Nov 2004	**Figure 2**

northwest hydraulic consultants ltd.

APPENDIX III
PMF PEAK FLOOD ESTIMATES FOR EXISTING DAM SPILLWAYS

nhc
Memorandum

northwest hydraulic consultants

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edmonton, alberta T6E 4Y1
tel: (780) 436-5868
fax: (780) 436-1645

to:	Gerry Ferris	***sent:***	by email
company:	BGC Engineering Inc.	***date:***	12 November 2004
from:	Barry Evans	***pages:***	2

subject: Faro Mine Site: PMF estimates for Cross Valley and Intermediate Dam Spillways

File: Memo6472/3_1.doc

PMF ESTIMATES

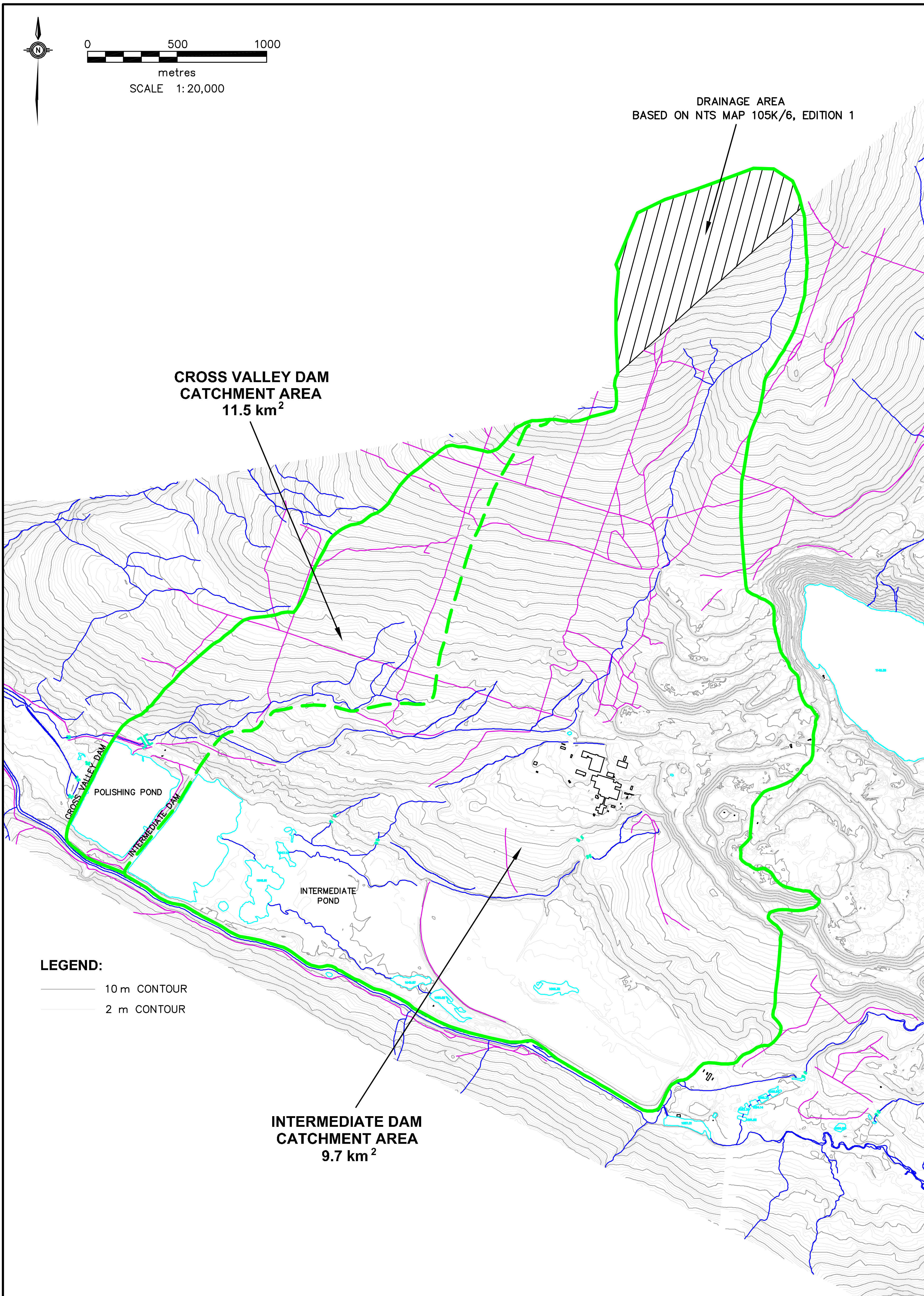
Figure 1 shows the delineated drainage areas for the Cross Valley and Intermediate Dams. It is assumed that flow in the Rose Creek diversion channel (RCDC) is contained within the RCDC and does not overflow into the ponds of either dams.

The PMF peak discharge estimates for the spillways of the two dams are:

Dam	Drainage Area (km^2)	PMF Peak Discharge (m^3/s)
Intermediate Dam	9.7	95
Cross Valley Dam	11.5	110

Prepared by:

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



Map Provided by SRK CONSULTING

Contour Interval: 2m
Date of Photography: 03/07/25
Scale of Photography: 1:20000
Survey control derived from existing 1:20000 photography
Survey control based on: UTM Projection, NAD27
Compiled by The ORTHOSHOP, Calgary, September 2003

BGC ENGINEERING INC.		
FARO MINE SITE 2004 STUDIES PMF ESTIMATES FOR CROSS VALLEY AND INTERMEDIATE DAMS **DRAINAGE AREA**		
Dwg. 6472–002	10 Nov 2004	**Figure 1**
northwest hydraulic consultants ltd.		

APPENDIX IV
CALCULATIONS FOR SEDIMENT FROM TAILINGS

Revised Universal Soil Loss Equation							Best Estimate							
A= R*K*L*S*C*P														
	Values	Values	Values	Values	Values	Values	Values	Values	Values	Values	Values	Values	Values	Values
R=Rainfall Factor (MJ mm/ ha*h)	100	150	200	250	300	350	400	450	500	550	600	650	700	750
K=Soil Erodibility (t h/ MJ*mm)	0.054	0.054	0.054	0.054	0.054	0.054	0.054	0.054	0.054	0.054	0.054	0.054	0.054	0.054
LS=Slope length and steepness (dimensionless)	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84
C=Cropping-management (dimensionless)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
P=Support practice (dimensionless)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
A=Tonnes/(hectare*year)	4.536	6.804	9.072	11.34	13.608	15.876	18.144	20.412	22.68	24.948	27.216	29.484	31.752	34.02
Soil Erosion Class (Table 1.1)	Very Low	Low	Low	Moderate	Moderate	Moderate	Moderate	Moderate	High	High	High	High	High	Severe
Total Surface Area in Hectares	173.4	173.4	173.4	173.4	173.4	173.4	173.4	173.4	173.4	173.4	173.4	173.4	173.4	173.4
Total Surface Area in Hectares with 10% exposed tailings.	17.34	17.34	17.34	17.34	17.34	17.34	17.34	17.34	17.34	17.34	17.34	17.34	17.34	17.34
Total Surface Area in Hectares with 1% exposed tailings.	1.734	1.734	1.734	1.734	1.734	1.734	1.734	1.734	1.734	1.734	1.734	1.734	1.734	1.734
Total Tonnage lost per year with current conditions.	787	1180	1573	1966	2360	2753	3146	3539	3933	4326	4719	5113	5506	5899
Total Tonnage lost per year with 10% exposed tailings.	79	118	157	197	236	275	315	354	393	433	472	511	551	590
Total Tonnage lost per year with 1% exposed tailings.	8	12	16	20	24	28	31	35	39	43	47	51	55	59
Total volume of tailings eroded (m^3) assume 1 tonne= $1m^3$	787	1180	1573	1966	2360	2753	3146	3539	3933	4326	4719	5113	5506	5899
Total volume of Intermediate Pond (m^3)	1191570	1191570	1191570	1191570	1191570	1191570	1191570	1191570	1191570	1191570	1191570	1191570	1191570	1191570
Number of years to fill Intermediate pond with Tailings	1515	1010	757	606	505	433	379	337	303	275	252	233	216	202

Settling Velocity Tailing Pond									
Vo=Q/As									
	Values								
Q= Flow (m^3/s)	95	PMF Peak Discharge (NHC)							
As= Surface Area of Basin (m^2)	236200	Surface area of tailings pond (GLL)							
Vo= Settling Velocity (m/s)	4.02E-04								
If V<Vo particle will exit basin									
V=SQRT(((1.33gd/Cd)((pp-p)/p))									
	Value	Value	Value	Value	Value	Value	Value	Value	Value
g= gravity (m/sec2)	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81
d= diameter of sphere (m)	5.00E-05	4.00E-05	3.00E-05	2.00E-05	1.00E-05	9.00E-06	8.00E-06	2.00E-06	1.00E-07
Cd= drag coefficient (24/Re)*	24	24	24	24	24	24	24	24	24
pp= density of particle	2.65	2.65	2.65	2.65	2.65	2.65	2.65	2.65	2.65
pp= density of water	1	1	1	1	1	1	1	1	1
V= Settling velocity of particle (m/s)	6.70E-03	5.99E-03	5.19E-03	4.24E-03	2.99E-03	2.84E-03	2.68E-03	1.34E-03	2.99E-04
Assumption:									
As is constant									
Particles are spheres									
Laminar flow => Reynold's # = 1									
Desity of particles are constant									
Density of water is constant									
Steady State									
Uniform dispersion of particles.									
Settling is ideal discrete particle sedimentation.									
Particles move forward at the same velocity of the liquid.									

Figure IV - 1
Revised Universal Soil Loss Equation for the Tailings Pond at Faro

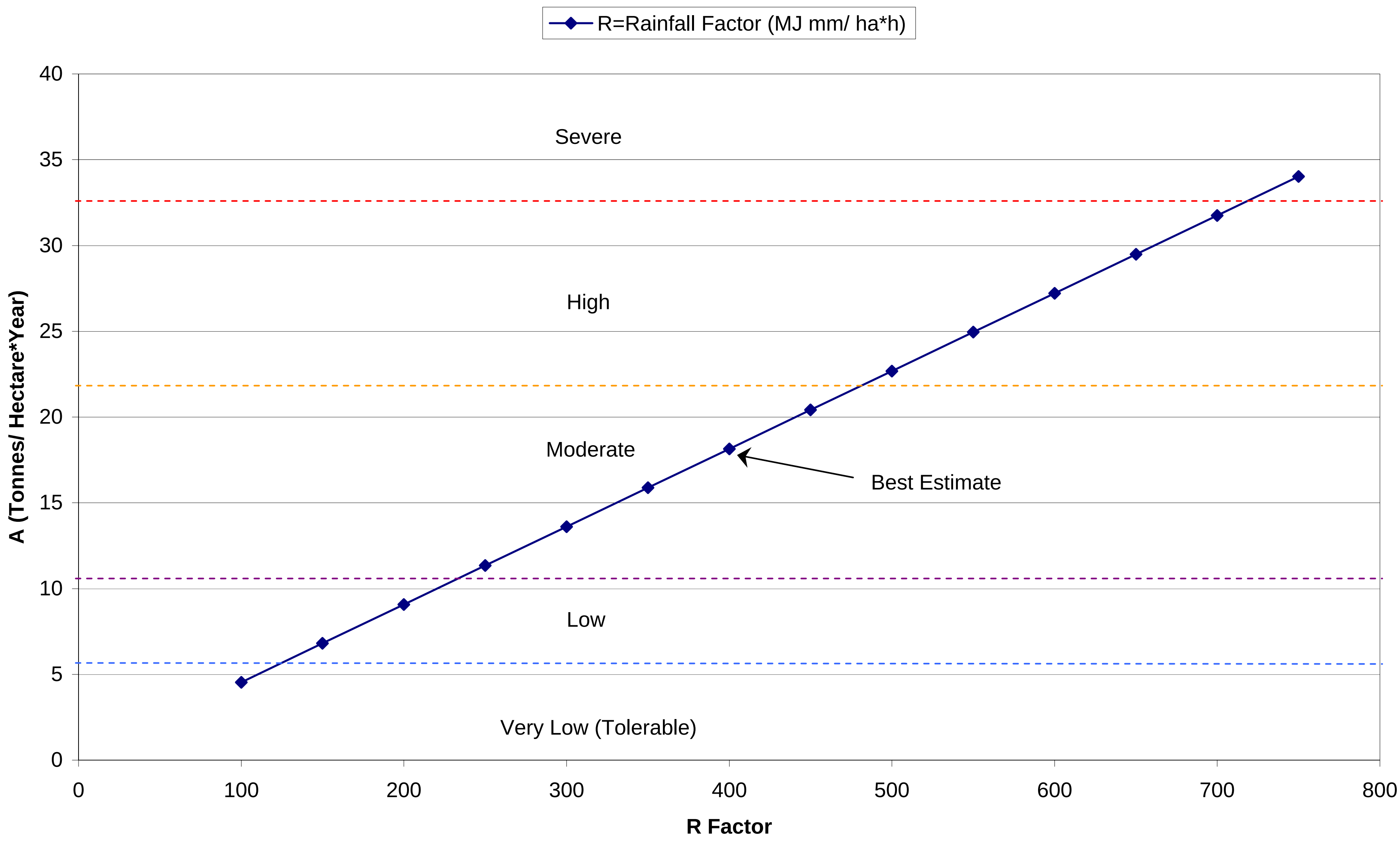


Figure IV - 2
Intermediate Pond Volume

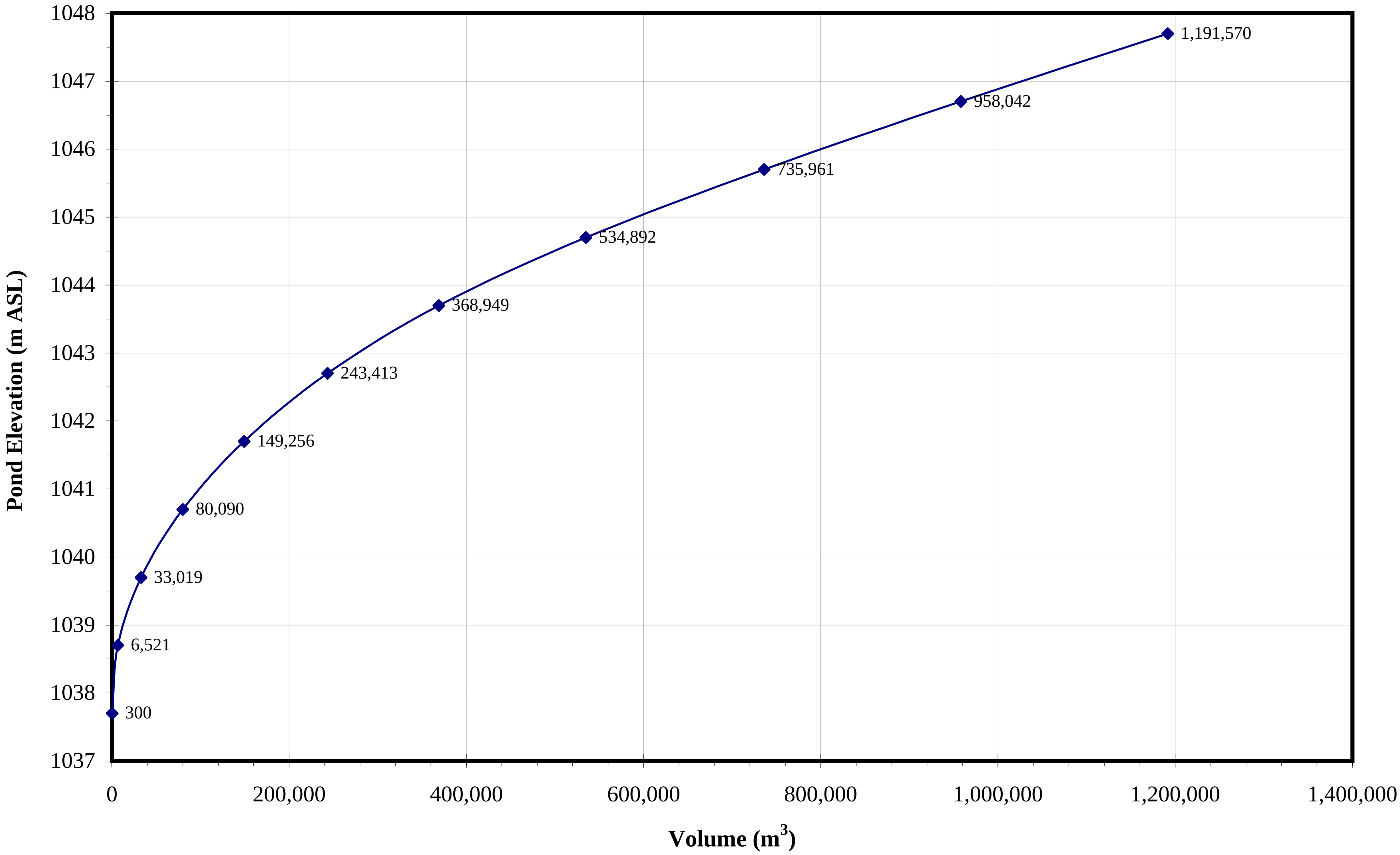


Figure IV - 3
Particle Settling Time Vs. Diameter