DELOITTE & TOUCHE INC.

NORTH FORK ROCK DRAIN

GEOTECHNICAL
EVALUATION

DRAFT REPORT

PROJECT NO.: 0257-023-03

DATE: NOVEMBER 22, 2004

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Project No. 0257-023-03
Date: November 22, 2004

Mr. Douglas Sedgwick
Deloitte & Touche Inc.
Suite 1400, BCE Place
181 Bay Street
Toronto, Ontario
M5J 2V1

Re: North Fork Rock Drain, Geotechnical Evaluation

Dear Doug:

The above reference draft report has been uploaded to the Deloitte & Touche Inc. e-room to allow stakeholder review. This report presents our evaluation of the North Fork rock drain as outlined in our proposal dated June 1, 2004. 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 reviewers comments are received, the report will be issued in final form. This is expected to be following the January 2005 closure planning meeting.

Should you have any questions or comments, please contact me at the number listed above.

Yours truly,
BGC Engineering Inc.
per:

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

encl: Draft Report

GWF/sf

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LIMITATIONS OF REPORT

This report was prepared by BGC Engineering Inc. (BGC) for the account of Deloitte & Touche Inc. 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 is 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 Mining Corporation, to provide an evaluation of the North Fork Rock Drain (NFRD) at Faro Mine. The NFRD was designed to transmit the water flow of the North Fork Rose Creek through the haul road. The haul road was constructed in 1987 to provide access to the Vangorda mining area. The performance of the rock drain, specifically the capacity of the drain to pass water from the creek flow, was evaluated in 1988 and 1993. The ongoing physical performance of the rock drain is evaluated on a yearly basis as part of the annual geotechnical review.

A review of the typical year to year hydraulic performance of the NFRD was recommended in the 2003 Annual Geotechnical Inspection Report (BGC 2004). In the February 2004 closure planning meetings the project team identified the need to evaluate the NFRD as part of closure planning. The NFRD was installed with the intention that it would be removed upon mine closure (Golder 1986). However, if left in place the NFRD could be used to attenuate the peak flow of the probable maximum flood (PMF) and thereby reduce the required canal upgrades around the tailings area.

Therefore the dual purpose of this assessment is to evaluate the performance of the NFRD under the existing operating conditions and to predict the performance under extreme conditions (floods and earthquakes) for possible use in closure.

1.1 Scope of Work

In order to complete the above noted general requirements, the following specific tasks were identified in BGC's June 1 proposal:

1. Review previous reports prepared for the NFRD.
2. Provide technical support for the temporary installation of monitoring systems (self contained piezometer and datalogger) in the pond on the upstream side of the NFRD and in the channel bed of the North Fork Rose Creek downstream of the rock drain.
3. Perform a site inspection of the NFRD.
4. Provide technical support for the installation of a permanent monitoring system (consisting of a piezometer sensor and the new datalogger/remote access capability) in the pond on the upstream side of the NFRD and in the North Fork of Rose Creek downstream of the rock drain. *Note that the initial instruments installed were used throughout the year.*
5. Perform flood routing.
6. Perform a stability assessment of the rock drain.
7. Compare present and historical flow-through rates.
8. Prepare and submit a summary report of the analyses conducted on the rock drain.

1.2 Authorization to Proceed

Authorization to proceed was provided via a letter dated July 16, 2004 from Mr. Doug Sedgwick of Deloitte & Touche Inc.

2 SITE DESCRIPTION

2.1 Site Location

Faro mine is located in the central Yukon, approximately 200 km north-northeast of Whitehorse. The Faro mine site is situated approximately 22 km north of the Town of Faro, as shown in Figure 1. The NFRD is located on the south-west side of the Intermediate rock dump at the beginning of the haul road. The NFRD was constructed in 1987 across the North Fork of Rose Creek, location shown in Figure 2.

2.2 1986 Design

2.2.1 Design Summary

The haul road between the Vangorda mining area and the Faro mill area was constructed as a dumped rock fill structure. This road crossed several creeks between the Faro and the Vangorda/Grum mine areas. The method selected crossing the North Fork Rose Creek was to construct the haul road as a "rock drain". The rock drain was constructed such that coarse fragments of clean waste rock were at the base of the road structure. These rocks would have the appropriate capacity to pass water through the void space (Golder 1986a). A copy of the original design drawing for the NFRD is shown in Figure 3. The following summarizes key design conditions:

1. The drain is to be constructed from calc-silicate rock. The remainder of the haul road could be constructed of schistose rock.
2. The design flood for this structure was the 100 year return period flood, 70 m^3/s.
3. The width of the drain was to be 70 m, centered on the pre-existing creek channel. That is, the portion constructed from calc-silicate rock.
4. The construction of the NFRD was to be accomplished by end-dumping the rock from the final road elevation, the final height was noted to be approximately 55 m. This method would result in natural sorting of the rock, with the largest rocks being at the base of the road and the fine material near the top.

5. The assumed slope of the upstream and downstream faces of the drain was 37°, the angle of repose for the calc-silicate rock.
6. For prediction purposes, the design considered that the drain would consist of the lower 3.6 m of the causeway. The grain size was assumed to be 0.3 m, given the likely maximum particle size and an allowance for particle breakage due to the overlying weight of the rock fill.
7. In determination of the capacity of the drain, no flow was considered to occur within the upper portions of the rock drain (above 3.6 m). The routing analysis indicated that the 100 year flood would produce a 40 m deep pool on the upstream side of the drain and that the mean annual flow would result in an 11 m deep pool.
8. The downstream face of the drain (if left at the assumed angle of repose, 37º) was considered to be unstable under high flow rates, due to seepage forces. The design included the construction of a "fillet" of large diameter rock to be installed at the downstream toe. The "fillet", which was intended to stabilize the toe with respect to seepage forces was to have a minimum slope of 3H:1V and extend at least to 15m above the toe of the drain. *Note that this fillet at the toe was not constructed*.
9. The original design intent was that the NFRD would be abandoned by construction of an emergency overflow spillway.

During the design phase, considerable discussion was provided concerning potential failure modes for rock drains, given the relative newness of the concept (Golder 1986a, Golder 1986b). The conclusions of the designer were that the drain would perform adequately, and that the flow capacity was conservatively selected.

2.2.2 Foundation Conditions

The foundation soil conditions for the NFRD were not considered as key design parameters given the relative flatness of the ground (Golder 1986a) but were later investigated prior to construction (EBA 1987). Prior to the 1987 investigation the estimated foundation conditions for the drain were based on boreholes drilled approximately 400 m downstream of the site. The depth to bedrock at the site 400 m downstream was between 7.9 and 10.5 m (Golder 1986b).

The 1987 investigation (EBA 1987) consisted of 5 test pits, completed to a maximum depth of 6.0 m. The sub-soils were reported as consisting predominately of glacial till. A buried peat layer was encountered in 3 of the 5 test pits and alluvial silt, sand and gravel was encountered in test pit 1. Numerous boulders were encountered at all locations. At the time of the investigation (April), the ground encountered in the test pits was frozen. However, only at test pit 3 was the ground considered frozen below 2.5 m (the estimated depth of seasonal frost penetration). The seasonal frost contained up to 20% ice by volume. The permafrost at test pit 3 consisted of a "pliable soil" matrix with stratified and randomly oriented clear ice formations (up to 15% by volume). A copy of the site plan and borehole logs from the 1987 investigation is included in Appendix I. The estimated position of these test pits is also shown on Figure 4.

2.3 1987 Construction

During construction, one site visit was conducted by the design engineer (Golder 1987). During the inspection, placement of rock for the drain was temporarily halted. The face of the dump was about 20 m from the edge of the creek, within the proposed footprint of the NFRD. The rock encountered by the inspector was calc-silicate.

A review of the grain size of the material making up the NFRD was undertaken. A summary of the results is provided in Table 1 and a copy of the photos collected during this inspection is included in Appendix II. The rock within the lower 55 m of the advancing face of the NFRD was noted to be "remarkably clean". Fines were noted only in the upper 10 m of the advancing face.

Table 1 Summary of Observed Particle Sizes During Construction

Location	D_{max}	D_{65}	D_{50}
Toe of dump	2 m[1]		1 m[1]
7 m above toe		0.5 m	0.3 m
10 m above toe	0.8 m[1]		0.4 m[1]
17 m above toe		0.5 m	0.15 m
55 m above toe		0.4 m[1]	0.2 m[1]

Note 1: Rock sizes estimated from 1987 photos by BGC for this study. Other estimates from the 1987 Golder report.

D_{max} is the maximum particle size.

D_{65} – 65% of the observed particles are smaller than this.

D_{50} is the medium particle size.

3 VISUAL AND HYDRAULIC CAPACITY ASSESSMENTS

Annual visual assessments of the NFRD have been made since its construction in 1987 as part of the overall Faro mine geotechnical inspection (BGC 2003). Prior to this study, three specific visual and hydraulic capacity assessments of the NFRD were undertaken, the results were presented in reports dated 1988, 1990 and 1993.

3.1 1988 Assessment

3.1.1 Observations

The site visit was conducted in May 1988 (Golder 1988a, 1988b), following the spring freshet. The water level during freshet was reported to be at least 2.5 m higher than measured during the inspection, the maximum water depth in the pond was about 4 m. It should be noted that the location for the water depth measurement was not given. The flow rate (measured downstream of the NFRD) was estimated to be 3 m^3/s when the pond had a depth of 1.5 m. Photographs were taken during the inspection and indicated that the boulders at the base of the drain were greater than 1 m diameter and that the fill was essentially devoid of fines except for the top 9.1 m (30 feet) below the crest.

3.1.2 Hydraulic Capacity

Studies conducted in 1986 (Golder 1988) indicated that the 100 year return period flood would have a peak flood value between 36 and 38 m^3/s, as compared to the value of 70 used in the design. Based on the tentative relationship between rate of discharge and pool depth, discussed below, this flood value would result in a pool depth of between 5.5 and 8 m.

Based on measurements of rock size, estimates of the void ratio and flow measurements collected by site staff two relationships were developed to describe flow through the NFRD (Golder 1988a, 1988b). These two relationships were based on the same dataset of pond depths and flow rates. The June equation was determined based on theoretical considerations for the exponent and then determination of the constant 1.45 via curve-fitting. The July equation was derived on the basis of curve fitting to the available data. The equations developed were:

$$Q = 1.45h^{1.8} \text{ (June equation)}$$

$$Q = 2.428h^{1.598} \text{ (July equation)}$$

Where: Q is the rate of discharge through the rock drain (m^3/sec)

h is the depth of the pool at the inlet of the drain

These equations indicate that the flow-through capacity of the drain is greater than that predicted at the time of the design. For example, based on the 1986 design relationship a flow rate of 70 m^3/s would have resulted in a water level in the pond 40 m above the base of the pond. This relationship predicts a water depth of 8.5 m.

This report (Golder 1988a and 1988b) also describes the possibility of plugging of the drain via sediments carried by the creek, and concluded that the velocity of the water flowing through the drain is sufficient to carry the particles through the drain. The grain size of the sediment transported by the creek was not measured, but it was noted that the majority of the bed load materials was deposited near the upstream limit of the pond. The materials transported to the face of the drain consisted of silt with 100% finer than 0.15 mm and 76% finer than 0.074 mm.

3.2 1990 Assessment

3.2.1 Observations

The site visit was performed in July (Golder 1991b), following spring freshet. The water depth in the pond was about 1.0 m (reported as 0.5 m lower than observed in 1988) and the flow was estimated to be 1.5 m^3/s. The water depth was based on a staff gauge installed by site staff near the upstream limit of the pond. The maximum water depth measured was about 4 m, during the spring freshet on June 1. It is not clear if the location of the staff gauge produced results comparable to previous water depth measurements or could be considered as the wetted depth at the toe of the drain.

3.2.2 Hydraulic Capacity

The single measurement of pond water depth and discharge rate measured in July was compared to the June 1988 relationship. A good correlation was achieved between the July measurement and the June 1988 relationship.

3.3 1993 Assessment

3.3.1 Observations

This assessment (Golder 1993) was largely concerned with the hydraulic capacity and no commentary was provided on the rock within the drain or physical stability issues.

3.3.2 Hydraulic Capacity

The assessment was based on data for flow and pond heights measured in 1991. The flow measurements were based on a rating curve developed at monitoring station X2 (near the crossing of the North Fork Rose Creek and the main access road). The pond depths were based on data collected using a data logger. Unfortunately, the data logger was not placed at the same location as the staff gauge used in previous assessments and no survey of the sensor elevation was undertaken. The deepest pond level measured was about 2.6 m on July 20th and had an associated maximum flow rate of 5.6 m^3/s.

Based on this new larger dataset a new relationship was developed for the flow quantity versus pond depth. Using a Least-Squares fit of the pond elevation and flow the following relationship was developed (Golder, 1993):

$$Q = 0.1557h^2 + 1.99h + 0.836$$

Where: Q is the rate of flow (m^3/sec)

h is the pond surface level (m)

A comparison between the four pond level-flow through relationships is shown in Figures 5 and 6. The plot in Figure 6 shows the comparisons below a pond level of 6 m. As seen in this figure, the initial design estimates (1986) were significantly lower than actual measurements. Comparison of these relationships in the low pond depth region (Figure 6) reveals that all of the relationships are relatively close. It must be noted that the largest pond depth measured at this time was about 4 m.

4 2004 ASSESSMENT

The assessment performed for this study consisted of installing two new data-loggers and performing a visual assessment of the drain. The data-loggers were installed in the pond, and at Station X2 in the creek, as shown in Figure 4. The position and elevation of the data-loggers was surveyed in geodetic elevation and using NAD27 control so as to allow comparison to future data.

4.1 Observations

The pond data logger was installed on May 27, 2004 near the time of the spring freshet. Given the high water levels at the time, the data-logger was installed on a barge tied to a tree. The sensor was installed at an elevation of 1088.257 m amsl and the barge was allowed to float keeping the data-logger out of the water. During installation the sensor was weighted to hold it in a single position and approximately 3 m of slack was allowed in the cable to help maintain a constant position of the sensor. The initial sensor position was not located within the limits of the pond at low creek flows (no trees were standing in the low pond area), this required the probe to be moved to a second position within the pond. The second sensor elevation was 1087.617 m amsl. The second sensor position is shown on Figure 4. A copy of the recorded pond elevations during 2004 is contained in Appendix III.

The pond sensor was installed about one or two days after of the peak pond elevation during the 2004 spring freshet. The high water mark prior to installation was estimated, later this was surveyed to be at elevation 1093.56 m amsl.

A sensor and data logger was installed at Station X2, upstream of the main access road as shown on Figure 4 and Photos 1 and 2. This instrument was installed on July 15, 2004. The first reliable data from this instrument was collected on August 14, 2004. The gap in the data collection was apparently due to static shock of the instrument. When the instrument failure was recognized the data logger was reset. The datalogger failed, apparently for similar reasons, a couple more times during the remainder of the monitoring season. The instrument measures the depth of water above the sensor, and this is converted to flow via a rating curve. Given the failures of the data-logger system this year, the rating curve for this station was based on only two data points at very low flows (the flow measurements were also of questionable validity). Therefore a rating curve was developed from HEC-RAS modeling. The model was constructed on the basis of survey information collected in 2004. The measured flows at Station X2, based on the developed rating curve are included in Appendix III.

During the July field visit the approximate boundary between the calc-silicate rock and the schistose rock was estimated. The eastern limit was between GPS way point 54 and 55, as shown on Figure 4. The western limit was estimated to be at GPS way point 57, as shown on Figure 4. These limits indicate that the NFRD was entirely constructed from calc-silicate rock, with no schistose rock placed as shown in the original design drawings (Figure 3).

No signs of overall instability problems were encountered during the 2004 inspection, similar to the observations in the annual geotechnical inspection. Surficial sloughing of the fine grained material stockpiled at the edge of the haul road has occurred (Photos 3 and 5). In some cases this material as flowed down the face of the rock dump completely from the crest to the base.

During the July inspection the water level in the pond was below elevation 1088.3 m amsl, which is below the elevation of the first position of the sensor (Photo 4).

The grain size of the rock near the base of the outlet of the NFRD was estimated from digital photographs using Split Net technology from Split Eng, as shown in Figure 7. The Split Net methodology consists of taking a digital image of the rocks with sizing balls in the photo (Photos 6 and 7). The balls are a standard size and are used to both resolve the sloped surface to a flat surface for processing and then are used as to estimate rock size. The results plotted in Figure 7, show the interpreted grain size at the base of the NFRD at GPS way-point 51 (referred to as Photo 118 and 119 in Figure 7 and as shown in Photo 6). The grain size curve for slightly higher up on the downstream side of NFRD at GPS way-point 52 is also shown (referred to as Photo 122 in Figure 7 and as shown in Photo 8). The locations of the two GPS way-points are shown in Figure 4.

Two views of the water exiting the downstream base of the NFRD are shown in Photos 8 and 9.

4.2 Hydraulic Capacity

The equation generally used to determine flow rate through rockfill was developed by Wilkins in 1956 (Hansen et al. 2004), the equation is written as:

$$Q = n\ A\ W\ m^{0.5}\ i^{0.54}$$

Where:

- Q – flow (m^3/s)
- n - porosity,
- A – area (m^2)
- W – Wilkins' empirical constant, 5.243
- m – hydraulic mean radius (m)
- i – hydraulic gradient

The hydraulic mean radius can be calculated according to the following expression:

$$m = e\ d\ /\ 6\ r_e$$

where:

- e – void ratio
- d – "dominant" particle diameter
- r_e – particle surface-area-efficiency, typically about 1.3 for coarse angular rock

The hydraulic gradient was estimated according to the size of the drain and empirical expressions developed in model testing (Hansen et al. 1995a).

The above set of equations essentially develops an equation that relates the applied head in the upstream pond to the flow rate through the drain. In the development of this relationship both assumed and measured values were used as detailed in Table 2.

Table 2 Parameters Used to Estimate Pond Versus Flow Relationship

	Value	
Parameter	Measured	Estimated
Height	55 m	
Length of NFRD	208 m	
Width of Crest	30 m	
Void ratio		0.7
Dominate rock diameter		0.4 m
Particle surface-area-efficiency		1.2
Width of drain		90 m

The relationship between pond elevation and flow through the drain is plotted in Figure 8 along with data collected in 2004 and 1993. The pond elevation data plotted from 1993 is based on a rough estimate of the elevation of the sensor, which was developed by comparing the low pond water levels measured in 1991 to those measured in 2004. Given the limited data collected in 2004, due to data-logger failures, and the rough correlation to the 1993 data set, this equation describing the relationship between pond elevation and flow should be considered as tentative. Additional pond elevation data should be collected in 2005 along with a careful program of flow measurement to confirm the relationship. A comparison between the relationship for pond elevation and flow developed in 1993 and 2004 is shown in Figure 9. Again, the 1993 curve was modified to pond elevations based on the comparison of low pond levels in the summer and fall.

Note that the above relationship and discussion between pond level and flow ignores the component of seepage under the rock drain through the overburden in the foundation. It can be assumed that the hydraulic conductivity of the rockfill is many orders of magnitude greater than the hydraulic conductivity of the foundation, and therefore the contribution of foundation seepage to the flow downstream of the rock drain would be insignificant with respect to the flow through the rock drain.

Given the gaps in the 2004 data collection, it is recommended that a program of data collection be undertaken in 2005. Such a program should include contingencies for potential datalogger failures, such as manual recordings from staff gauges or direct survey of various water levels and flow measurements during key high pond events during the spring freshet.

In order to test the sensitivity of the flow through relationship a number different assumptions were tested for the main unknowns. A comparison of the measured results and four of the developed relationships based on Wilkins' equation are shown in Figure 10. The parameters used in these equations matched, as best possible, the actual conditions with the remainder of the properties selected conservatively (realistically). Inspection of these relationships indicates that only minor differences in the calculated maximum water levels would result. The relationship used provided the best match to the measured flow and pond elevations. Based on this comparison it was concluded that although there were gaps in the data collection for 2004 it is considered unlikely that future data collection will significantly alter the main conclusions of this assessment. It is recommended that additional data be collected and the recommended relationship reviewed once this data becomes available. Additional data is expected to finalize the relationship between pond elevation and flow through capacity.

5 HYDRAULIC MODELLING

Hydraulic modelling for this project was performed by Northwest Hydraulic Consultants Inc. (nhc) and is attached in Appendix IV. The modelling used the relationship between pond elevation and flow through capacity developed in 1993 and 2004, discussed in Section 4.2. The routing was performed using five different floods; mean annual, 100 year return period, 500 year return period, 1,000 year return period and the Probable Maximum Flood (PMF). The first four floods are related to snow melt events and have a 20 day hydrograph, whereas the PMF is based on a rainfall event and a 16 hour hydrograph.

The routing was performed using two different assumptions; a near empty pond, Figure 3 and then assuming the pond level corresponding to the peak level during the mean annual flood. These assumptions regarding the initial conditions provide bounds to the likely conditions. This modeling was performed using the 2004 flow through relationship developed above.

Table 3 Results of Hydraulic Modeling, Low Pond Elevation

Initial conditions: Pond WL at El. 1086.5 m
(minimum pond level)

Event	Peak Inflow, Q_{IN} (m^3/s)	Peak Outflow, Q_{IN} (m^3/s)	Peak Pond Level (m)
1:100 yr	54.0	28.5	1099.5
1:500 yr	81.0	40.4	1102.4
1:1000 yr	93.0	45.4	1103.5
PMF	504	6.33	1092.0

Table 4 Results of Hydraulic Modelling, High Pond Elevation

Initial conditions: Pond WL at El. 1092.4 m
(Maximum pond level for MAF)

Event	Peak Inflow, Q_{IN} (m^3/s)	Peak Outflow, Q_{IN} (m^3/s)	Peak Pond Level (m)
1:100 yr	54.0	28.5	1099.5
1:500 yr	81.0	40.4	1102.4
1:1000 yr	93.0	45.4	1103.5
PMF	504	10.67	1093.9

The results of the analysis indicate that the initial pond elevation made little difference to the peak outflow or the peak pond elevation for the snow melt events. This result is related to the length of the design hydrograph, where the approximate eight or nine days prior to peak flood conditions allow the effect of the initial pond level to be lost. Table 5 presents the results of the modelling if the 1993 flow through relationship is used, along with the assumed high pond levels prior to the routed flood. The results presented in Table 4 and 5 show the effects of the different flow through relationships. The results presented indicate about a 2 m elevation difference for the snow melt events and almost no elevation difference for the PMF event. As can be seen, the results indicate that the longer term snow melt relationships result in both a much higher flood value and a deeper pond.

Table 5 Results of Hydraulic Modelling, Using 1993 Relationship

Initial conditions: Pond WL at El. 1092.4 m
(Maximum pond level for MAF)

Event	Peak Inflow, Q_{IN} (m^3/s)	Peak Outflow, Q_{IN} (m^3/s)	Peak Pond Level (m)
1:100 yr	54.0	33.5	1097.6
1:500 yr	81.0	47.5	1100.3
1:1000 yr	93.0	53.5	1101.3
PMF	504	17.32	1093.9

The results of pond level monitoring and of the hydraulic routing described above reveal a number of key points:

- The measured peak pond elevation in 2004 was 1093 m. The estimated peak was 1093.3 m.
- The pond level seen in 2004 was slightly higher than normally observed (personal communication with site staff). Based on the vegetation patterns around the pond, especially the destruction of trees, the ‘normal pond’ levels would be within about 0.5 to 1.0 m of the maximum pond (1093 m) elevation observed in 2004.
- The measured pond elevation in 2004 was about 0.5 m lower than the maximum pond elevation that had occurred in the past, based on the debris on the face of the drain.
- The predicted pond elevation for the mean annual flood of 1092.4 m matches, in general, these site observations.
- The predicted maximum pond elevation of 1093.9 m for the PMF flood is only 0.4 m higher than the estimated maximum pond elevation that has already occurred. Therefore the rock drain has experienced pond elevation only marginally lower than it will need to retain under the PMF design flood.
- Long duration events related to the spring freshet produce higher pond elevation and greater flows in the creek downstream of the drain than the PMF.

6 GEOTECHNICAL STABILITY

The stability analyses undertaken in this study were performed using the Generalized Equilibrium method of analysis in the commercially available software program SLOPE/W.

The methodology followed for the analyses included estimating a range of strength properties, on the basis of the observed soil conditions and measured geometery. The stability analyses considered information related to pond levels from the hydraulic routing, Section 5 and measured water levels from the field.

Three different conditions were analyzed; static stability, pseudo-static stability under earthquake loading and stability under seepage forces.

6.1 Configuration and Parameters

The cross-section used in the slope stability analysis was based on Section A-A', Figure 4. This profile through the deepest section of the NFRD was developed on the basis of topography provided by SRK from the 2003 airphotos (SRK 2003).

The material strength and unit weight properties were estimated for the rockfill on the basis of measured angle of repose on the existing waste rock dumps and of the NFRD and on experience with waste rock densities. The properties of the foundation material were estimated on the basis of previous analyses of waste dumps at Faro and published correlations between material strength properties and index properties (Carter & Bentley 1991) along with engineering judgment. Table 6 lists the material properties used in the analysis.

During construction of the NFRD the measured angle of repose was 37º. The current condition of the NFRD is shown on Figure 10. The pond side of the NFRD has an angle of 34º. The upper and lower portions of the downstream side of the NFRD are at 35º and 29º, respectively. Typical ranges for angle of repose for a rock dump constructed of clean rock ranges between 35º and 40º (BCMWRPRC 1991).

The foundation soil consists of a varied mix, but generally sand and gravel with some cobbles. Given the location of the test pits adjacent to the original creek bed, the varied soil conditions encountered during the 1987 investigation (Appendix I) would be expected. The investigation encountered both buried peat layers and the volcanic ash layer, commonly encountered at the Faro site. These soils (peat & ash) would normally be assigned lower strength properties than listed for the foundation soil in Table 6. Conversely, the sand and gravel (alluvial and till) encountered would typically be assigned higher friction angles, especially considering the cobbles and boulders encountered in the matrix. Given these differing materials and the likelihood that continuous layers of any material would be unlikely a reasonable lower bound blended strength property was selected.

Table 6 Material Properties for Slope Stability

Soil Type	Parameter	Value
Rockfill	Unit weight	20 kN/m^3
	Effective Friction Angle	37°
	Effective cohesion	0 kPa
Foundation Soil	Unit weight	21 kN/m^3
	Effective Friction Angle	30°
	Effective cohesion	1 kPa

The pore pressures used in the analysis were derived from measured water elevations on the upstream and downstream side of the NFRD or from the predicted pond levels during high floods calculated by hydraulic routing and assumptions regarding the water elevation at the outlet.

6.2 Static Stability

Limit equilibrium analysis was performed for a variety of pond conditions and assumed downstream exit water elevations. Shown in Figure 10 is the analysis using typical winter and summer pond levels. It should be noted that the failure surface shown in this figure is one of a family of failure surfaces that were analyzed, with the failure surface indicated being the one with the minimum factor of safety. The analysis shown in Figure 10 is based on measured pond elevation and measured water exit elevation on October 5, 2004, which were considered to be typical of summer/fall/winter conditions. Shown on Figure 11 is the result for the analysis of pond elevation and water exit elevation on May 27, 2004, which was near the peak of the 2004 spring freshet.

The stability analysis was repeated for pond elevations of 1095, 1100, 1105 and 1110 m amsl. The results were similar to those shown in Figures 10 and 11 and the resulting calculated factors of safety are listed in Table 7. The elevation of the water exiting the toe of the NFRD was selected in this analysis based on the relationship between pond and exit water elevations measured in May and October 2004. The results of this analysis assume static water levels. A discussion of the effects of seepage on the stability of the toe is included in Section 6.4. It should be noted that predicted water elevation during the 1,000 year return period flood is less than 1104 m amsl, so the results presented for a pond elevation 1110 m amsl are well above what will need to be withstood by this structure. Therefore the static stability for a deep seated failure on the downstream face is about 1.2. Given that the downstream face is near the angle of repose, the high water levels would likely induce shallow failures on the face of the dump. Further discussion concerning this type of failure is provided in Section 6.4.

Table 7 Stability Analysis Results

Stability Case	Factor of Safety
Typical Pond Level	1.40
2004 Freshet Pond Levels	1.38
Pond 1095	1.38
Pond 1100	1.34
Pond 1105	1.20
Pond 1110	1.14

6.3 Earthquake Loading

A site specific detailed seismic hazard assessment has recently been completed for the Faro Mine (Atkinson 2004). The results of this assessment indicate that for an annual probability of exceedence of 0.0001, the mean value of peak ground acceleration (PGA) is 546 cm/s^2 (0.56 g), and the median PGA is 343 cm/s^2 (0.35 g). These ground motions correspond to an earthquake of approximately M7, at a distance of 10 to 20 km from the site.

The seismic stability analysis performed for the NFRD considers that none of the foundation soils would be subject to liquefaction under the design earthquake. This assessment was based on a consideration of the soil conditions encountered in the foundation (Appendix I) and a preliminary assessment of the geological conditions under which sediments would have been deposited in the area. No testing of the foundation soils has ever been performed with respect to determining liquefaction resistance. Testing of the foundation materials may be warranted, depending on the results of the next round of closure planning.

The NFRD was constructed as a dumped fill structure at the angle of repose. Given that the upstream side and the upper portion of the downstream side of the structure are currently near the angle of repose, any significant earthquake acceleration will move these slope faces to a failure condition. A pseudo-static stability analysis was performed for the NFRD to determine the stability with respect to larger deep seated failures, like those shown in Figure 10 and 11.

The analysis was performed using the pseudo-static methodology, where horizontal accelerations are applied to potential failure surface and the conditions of limiting equilibrium are checked to determine the factor of safety. The pseudo-static method of analysis is suited as a preliminary evaluation tool (USACE 1995), assuming that the foundation is not susceptible to liquefaction. An indication of failure using pseudo-static methods means that a review of the resulting deformations resulting from the earthquake loading should be performed.

The soil properties listed in Table 6 were used along with the typical pond levels, as shown in Figure 10. Results of the factor of safety versus the applied acceleration (% g) are plotted in Figure 12. These results indicate that the structure will “fail”, that is a calculated factor of safety less then one, during an earthquake that produces acceleration greater than about 0.27g.

The overall safety of the NFRD is generally related to the height of water as compared to the crest elevation. Specifically, the possibility of the NFRD being overtopped during a flood. Overtopping would lead to a catastrophic failure due to seepage and erosion of the downstream face. The current crest elevation is 1144 m aml, and the maximum predicted water level is 1104 m amsl, providing a freeboard of 40 m. Given such a large freeboard it was considered unnecessary to do any further detailed analysis for earthquake loading, such as Newmark’s sliding block analysis. The crest settlement for the NFRD was estimated according to two different equations developed on the basis of dam settlement during an earthquake loading. The first relationship defines the Earthquake Severity Index (ESI) (Bureau et al. 1985) and then compares this to measured crest settlements. The ESI is defined as:

$$ESI = A\,(M\text{-}4.5)^3$$

Where:

A – Peak ground acceleration at the site
M – Earthquake Magnitude

Given the design earthquake for the Faro site, the ESI is 8.6 and based on the updated empirical relationship shown in Lo & Klohn (1992) this predicts a crest relative crest settlement of 4%. The ESI relationship was developed from data from both earth and rock fill dams and is known to over-predict settlement below an ESI of about 10 (Fell et al. 1992). An updated empirical relationship has been proposed by Swaisgood (1998) where:

$$CS = SEF \times RF$$

Where:

CS = Crest Settlement (percent)
$SEF = e^{(0.72\,M + 6.28\,PGA - 9.1)}$
$RF = 0.12\,D^{0.61}$ for rockfill embankments

And:

M – earthquake magnitude (
PGA – peak ground acceleration at the site (%g)
D – distance from source (km)

Given the predicted earthquake for the site and the above equation developed specifically from case histories of rock fill embankments, the crest settlement predicted is 0.28%.

These two results when combined with the embankment height of 55 m, indicate the expected crest settlement would be between 0.15 and 2.2 m. According to these predictions of crest settlement following a 10,000 year earthquake event the freeboard for the NFRD would be reduced from about 40 m to 37.8 m for the 1,000 year flood event. This amount of settlement would not be expected to affect the drain performance.

6.4 Hydraulic Stability

As indicated in Section 6.2 the stability analysis performed considered static waters levels and no consideration of seepage forces. Experience with dams, cofferdams and model embankments (Garga et al 1995, Leps 1971) indicate that rock drains will fail through unravelling and eventually to a complete breach under high seepage forces. The key consideration in this analysis is the hydraulic gradient acting on the rock near the toe of the NFRD. The critical gradient is actually a local gradient acting in the region of water exit from the rock. However, determination of this gradient is difficult due to the unknown geometry beneath the drain. During this study two separate estimates of the seepage gradient were made. Calculated gradients based on Wilkins equation and the pore pressure distribution within the embankment estimated during stability analysis. The first estimate is based on assumptions with respect to the geometry of both the NFRD and the valley bottom. The second estimate was developed on the basis of measured inlet and outlet water elevations. Both of these estimates determined the overall gradient, not the local gradient. A summary of the determined gradients are provided in Table 8.

Table 8 Estimated Hydraulic Gradients

	Average Hydraulic Gradient	
Pond Elevation (m amsl)	Wilkins Equation	Water exit elevation
1088.6	0.0015	**0.013**
1093 (2004 freshet)	0.0085	**0.030**
1095	0.012	0.036
1100	0.022	0.056
1105	0.034	0.075
1110	0.048	0.094

Note: Bold entries are based on measured values.

The measured average hydraulic seepage gradients were based on the following data. On October 5, 2004, the pond elevation was measured as 1088.6 m amsl and on the same day the maximum water height measured at the downstream toe (top of seepage face) was 1085.99 m amsl. These elevations combined with the overall length of the drain (208 m) results in an average seepage gradient of 0.013 m/m. During the spring freshet (May 27, 2004) the measured pond elevation was 1093 m amsl and the elevation of the water exiting the toe of the NFRD was 1087.03 m amsl. These measurements were not made at the peak of the spring freshet but the results indicate that the average seepage gradient was about 0.03 m/m.

As discussed in Section 6.2 the exit water elevation was estimated on the basis of related rates between these two data points. The validity of this approach is suspect given the increasing area available as the water levels on the upstream side become higher. Measurements at the toe of the slope should be collected to confirm these overall gradients, since only two measurements were collected at the toe.

The overall hydraulic gradients predicted for NFRD are quite low and when compared to the experience of past performance (Leps 1971). The Dix River Dam was a dumped rockfill dam with a downstream face with a slope of 1.4H:1V (35º). During construction it had to act as a flow through drain, when a large flood event occurred. The average hydraulic gradient through the structure was 0.057 and it experienced no stability problems.

Leps predicted that for a dumped angle of repose rock structure that failure would occur with a local seepage gradient of about 0.7.

The stability of the toe of the NFRD is related to overall hydraulic gradient but the most vulnerable area of the toe is the exit point of water from the rock fill. In order to estimate the stability of the toe of the NFRD under the actions of flowing water a comparison of the specific discharge calculated for the NFRD and allowable specific discharge (Knauss 1979) was made. Specific discharge is the discharge per unit width. The specific discharge estimated for the NFRD was based on the total flow calculated from the Wilkins equation and estimates of the width of the discharge area at the toe. Table 9 compares the approximate allowable specific discharge (Knauss) with the estimated specific discharge. The allowable specific discharge varies with the size of the rocks at the toe. The allowable specific discharge equations were based on allowable flow rates for overtopped rock structures.

Table 9 Specific Discharge Comparison

	Specific Discharge (m3/s m)	
	Allowable for different size rocks	
Pond Elevation	for 0.5 m, 1.0 m, 1.5 m stones	Estimated
1088.6	1, 4, 7	0.03
1093.3	1, 4, 7	0.17
1095	1, 4, 7	0.28
1100	1, 4, 7	0.66
1105	1, 4, 7	1.16
1110	1, 4, 7	1.79

The analysis indicates that rocks smaller than about 0.5 m may start to move at the toe when the 1,000 year flood event occurs.

The key to stability under seepage is the local gradients at the exit. If the local gradients start to induce movements the drain could begin to fail by ravelling. Predictions of the local seepage gradients will be possible with a clearer definition of the flow through relationship, measured water levels at the toe and measurements of seepage velocity. Given the importance of the local seepage gradients to the overall stability and the current uncertainly it would be prudent to flatten the downstream slope. Slope flattening will increase the stability of the toe for the same flow.

7 DISCUSSION

Two factors that will have an impact on the long term performance of the NFRD that have not been discussed in this report, until now, are the effects of the material soughing on the upstream face and the potential for sediments to eventually fill the pond on the upstream side. Both of these factors, although different will have the same effect, that of producing a partial seepage cut-off on the upstream face and raising the pond level.

The blockage or partial blockage of the upstream face will depend on a number of factors including;

- Amount of sediment brought to the pond and time required to completely fill the pond. Currently the pond size is such that only silt and clay sized sediment reach the inlet of the drain, and these small particles can not fill the large voids between the rock at the base of the drain. Once the pond is completely filled with sediment the larger sand, gravel sizes will impact directly on the inlet of the drain, increasing the potential for plugging.

- Size and distribution of the voids in the drain and the size distribution of the sediment. For the drain to become plugged the above two factors must combine in such a way so the sediment is not sluiced through the voids.

With respect to the plugging concern, some positive preliminary indications that the performance of the NFRD will be appropriate in the long term include:

- As shown on Figure 14 prior to development of the rock drain two channels were present in the location of the future pond. Based on the airphotos that were taken in 2003 it appears that these channels still exist, indicating a very low sedimentation rate of the pond.
- A rock drain in SE British Columbia was studied for a number of years as part of a sponsored research project (BCMWRPRC 1999). Detailed flow through capacity measurements (including velocity measurements using dye/salt tests) were made both before and after a high flow event that covered the inlet of the drain with sediment. No significant change in the pond level for the same outflow was recorded after the flood, and no change in the velocity was measured.
- In model testing performed at the University of Ottawa (Hansen et al. 1995b) the effects of a complete cut-off on the upstream face had only a small effect on the relationship between the pond elevation and the seepage through the model (above the top of the cut-off element).
- The case history presented by Leps (Leps 1971) included dams with an upstream seepage cut-off (earthfill and concrete elements). The water overtopped the seepage cut-off element and then fell in the rockfill structure and exited the downstream toe at water heights likely very similar to those that would be experienced if no seepage cut-off element was present.

Based on this preliminary information it is concluded that the critical part of the drain is the downstream toe and that a raised pond level on the upstream side would have little effect on the outflow of seepage at the toe and therefore the overall performance of the drain.

8 RECOMMENDATIONS

A detailed analysis for the potential plugging of the upstream face of the drain needs to be performed prior to making a final recommendation. The preliminary indications are that plugging of the drain would not be a major concern. Based on the assessment performed to date it appears that the NFRD would perform adequately under either earthquake or flood design condition. Therefore the NFRD could be left in place as part of closure. However, additional data collection should be performed to confirm this assessment.

The downstream toe of the NFRD is the key part of the structure that will control its stability and therefore its long term performance. Although this assessment indicates that the toe is stable the expected variation in flow velocities at the toe must be considered. These variations in flow could induce local failures, leading to general failures. Therefore, it is recommended that the downstream toe of the NFRD be flattened (to have a slope angle between 3H:1V to 5H:1V) to provide protection against seepage exit gradients. The flattening of the downstream toe will add the fillet that was shown in the original design and will provide a factor of safety against failure for unanticipated flow conditions.

As discussed, the assessment has been based on a limited dataset in 2004, due to datalogger failures. Additional data should be collected in 2005 to provide confirmation of the relationship developed for pond elevation and flow through capacity. Future data collection should include the following:

- Measurements of the pond elevation.
- Measurement of the flow into the pond (potentially using flow results from Station R7).
- Measurements of the water elevation at the downstream toe of the drain, to determine exit elevations.
- Measurement of the flow quantities from the drain, at station X2.
- Measure the velocity of the water flowing through the drain (dye tests).

The measurements listed above should be done as accurately as possible and should include redundancies to check the measurements made. The 2005 data collection should include a detailed site review during the period of peak flows (2005 spring freshet). Measurements at this time will be most critical to the confirmation of the flow through relationship.

In order to assess the potential plugging of the drain, analysis of the sediment in the creek needs to be performed and estimates of the sediment loading to the pond should be made. This combined with bathymetry of the pond should allow preliminary predictions of the time required to fill the pond. Measurements of the sediment movement both upstream and downstream of the pond could further add to the understanding of the potential plugging of the drain. These measurements along with estimates on the size of the voids and velocity of flow through during the high flow events could be combined to estimate the potential for particle clogging.

9 CLOSURE

This report summarizes the details regarding an evaluation of the North Fork Rock Drain as conducted by BGC Engineering Inc.

We trust that this report meets your needs at this time. Should you have any questions or comments concerning the information provided within this report, please contact the undersigned.

Respectfully Submitted,
BGC Engineering Inc.
Per:

Gerry Ferris, M.Sc., P.Eng. (AB)
Geotechnical Engineer

Reviewed by:

Holger Hartmaier, M.Eng., P.Eng. (AB)
Senior Geotechnical Engineer

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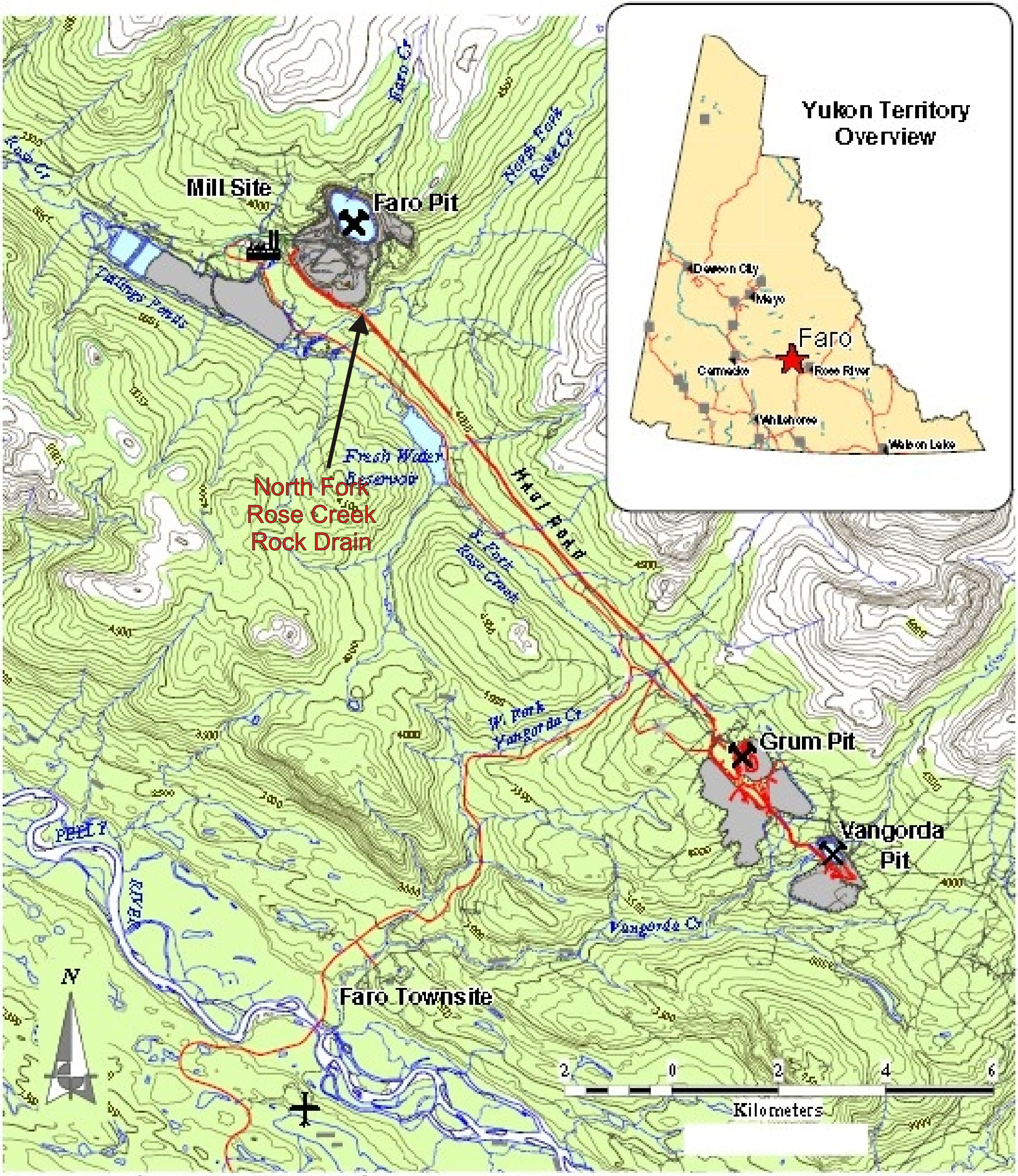
FIGURES



Note: Base map figure provided by Gartner Lee Limited.

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DATE:	November 2004	CHECKED:	GWF
DRAWN:	JMS	APPROVED:	JWC

CLIENT: Deloitte & Touche

BGC Engineering Inc.
AN APPLIED EARTH SCIENCES COMPANY
Calgary, Alberta. Phone: (403) 250-5185

PROJECT
NORTH FORK ROCK DRAIN EVALUATION

TITLE
SITE LOCATION PLAN

PROJECT No.	FIGURE No.	REV.
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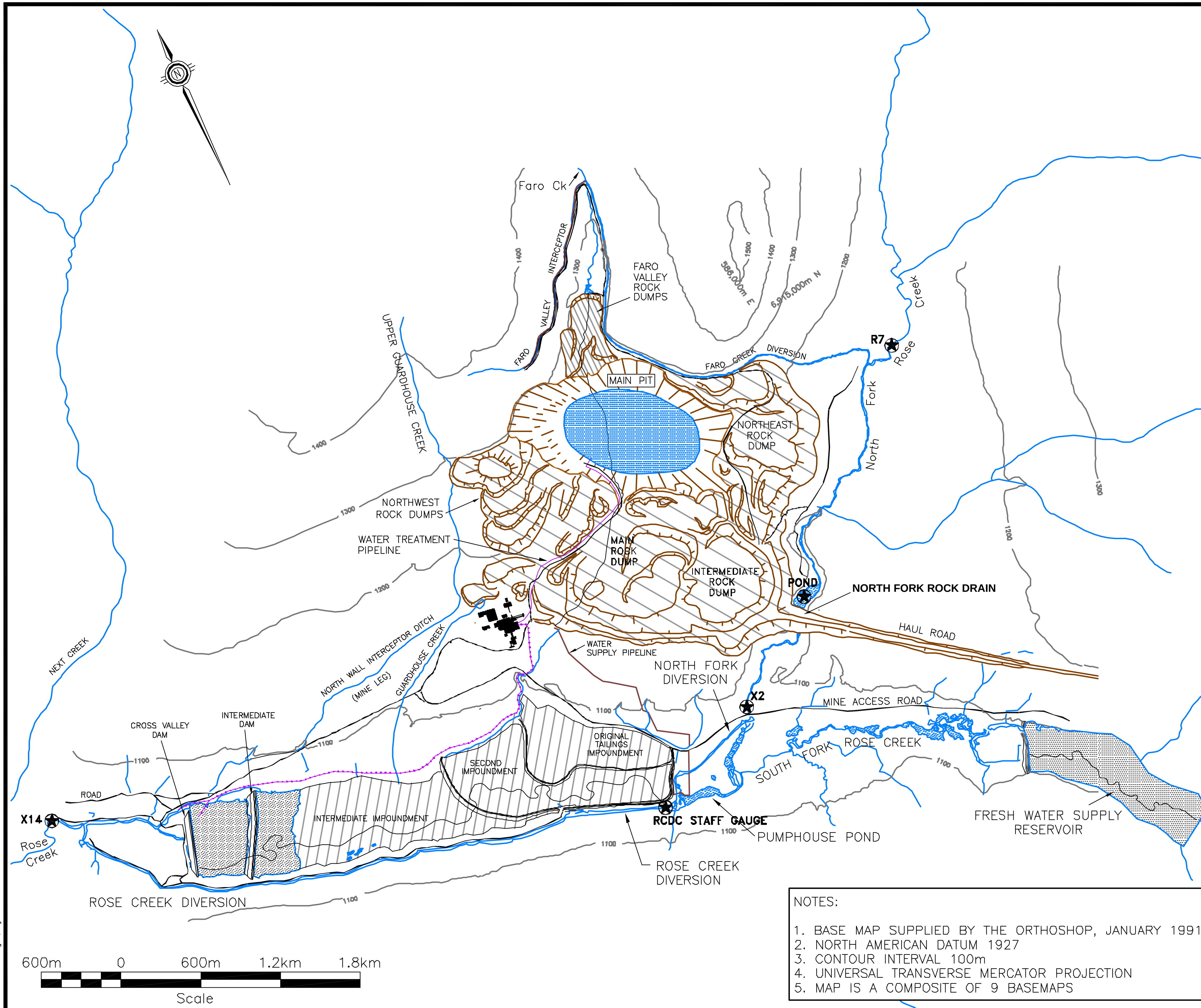

NOTES:

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:	OCTOBER 2004
DRAWN:	GCB
DESIGNED:	GWF
CHECKED:	GWF
APPROVED:	JWC

PROJECT: NORTH FORK ROCK DRAIN EVALUATION

TITLE: SITE PLAN

PROJECT No.	FIGURE No.	REV.
0257-023-03	2	0

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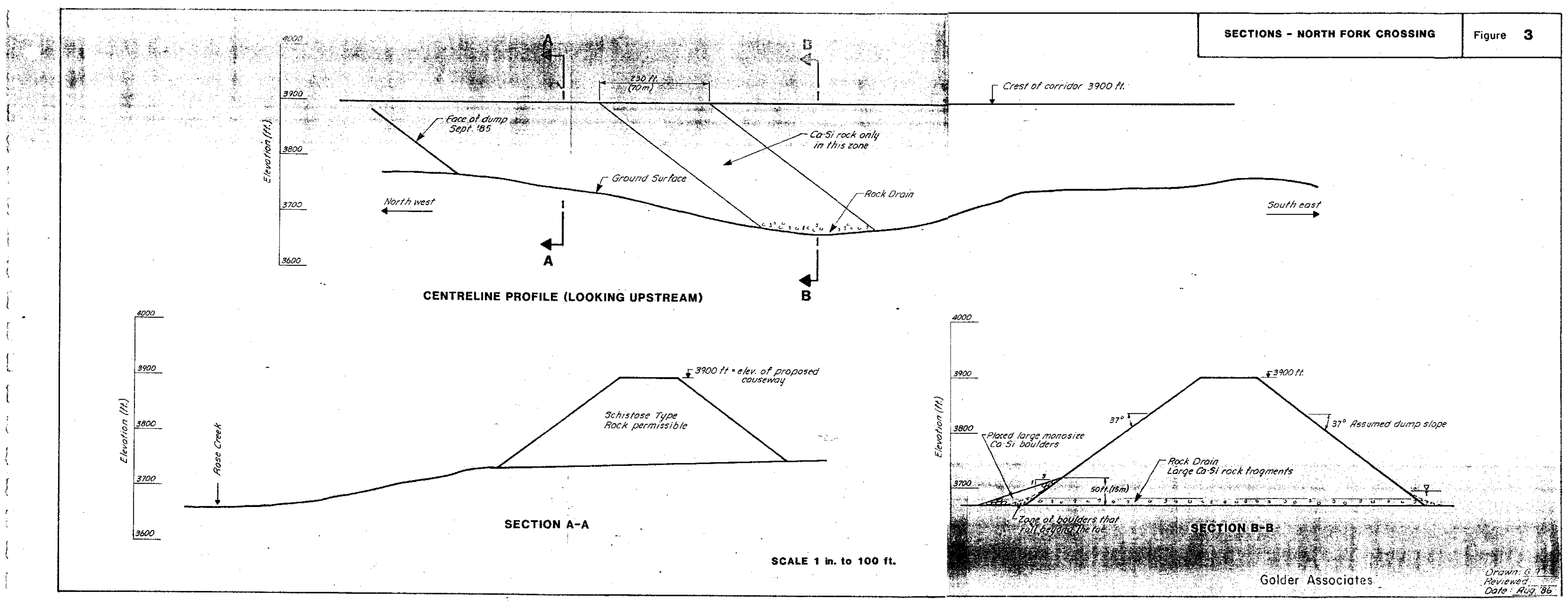

Source: Report No. 2 to Curragh Resources Corporation Re: Proposed rock drain, north fork, Rose Creek Faro, Yukon Golder Associates, September 1986.

CLIENT: Deloitte & Touche

REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

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

PROJECT: NORTH FORK ROCK DRAIN EVALUATION

TITLE: 1986 DESIGN PLAN AND SECTION

PROJECT No.	DWG No.	REV.
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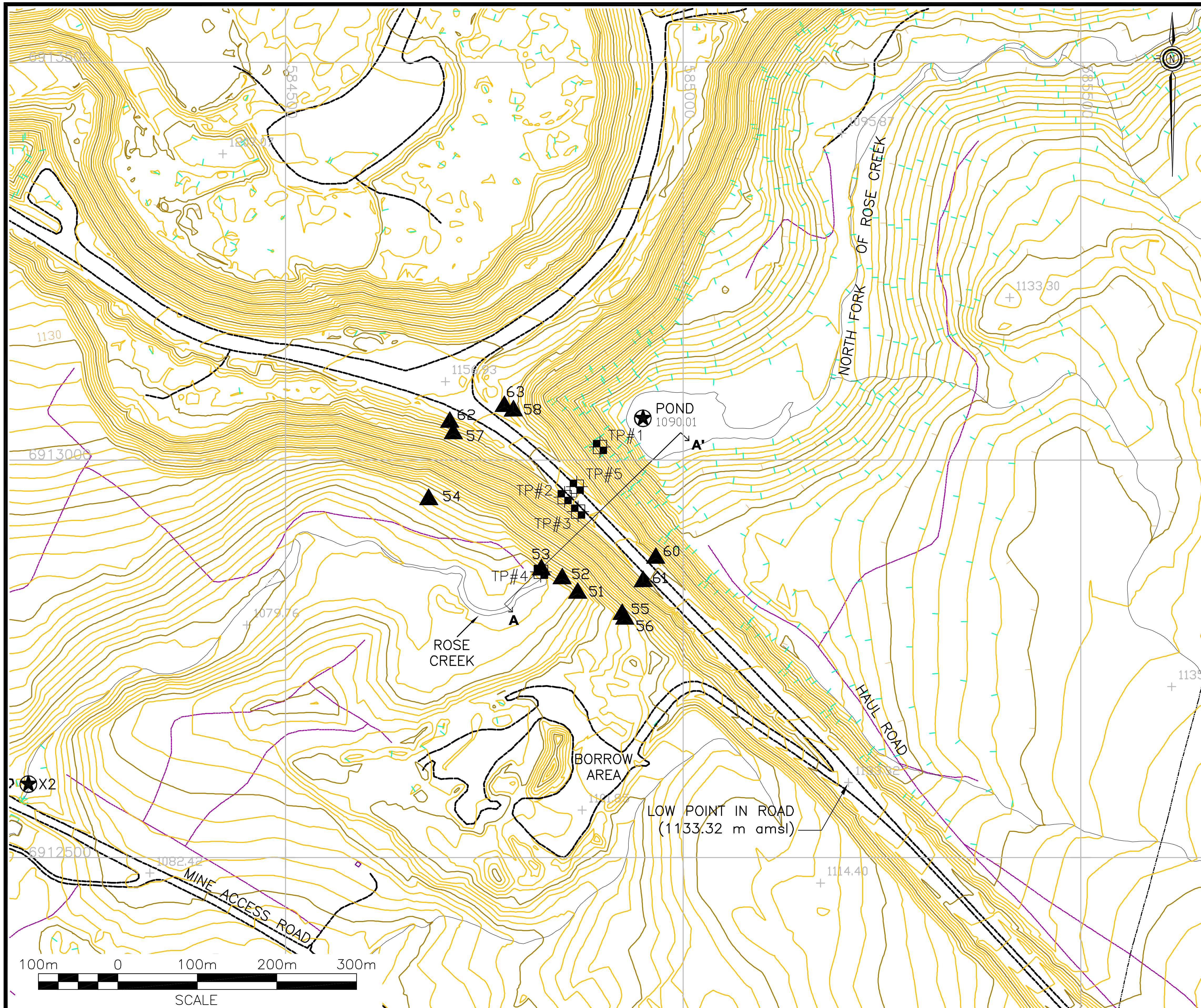

CLIENT: Deloitte & Touche

LEGEND:

WATER LEVEL MONITORING

TP#3 1987 TEST PIT

GPS WAYPOINT

REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED

SCALE:	AS SHOWN
DATE:	OCT 2004
DRAWN:	CJT
DESIGNED:	GWF
CHECKED:	GWF
APPROVED:	JWC

PROJECT: NORTH FORK ROCK DRAIN EVALUATION

TITLE: TOPOGRAPHIC DETAIL AT NFRD

PROJECT No.	DWG No.	REV.
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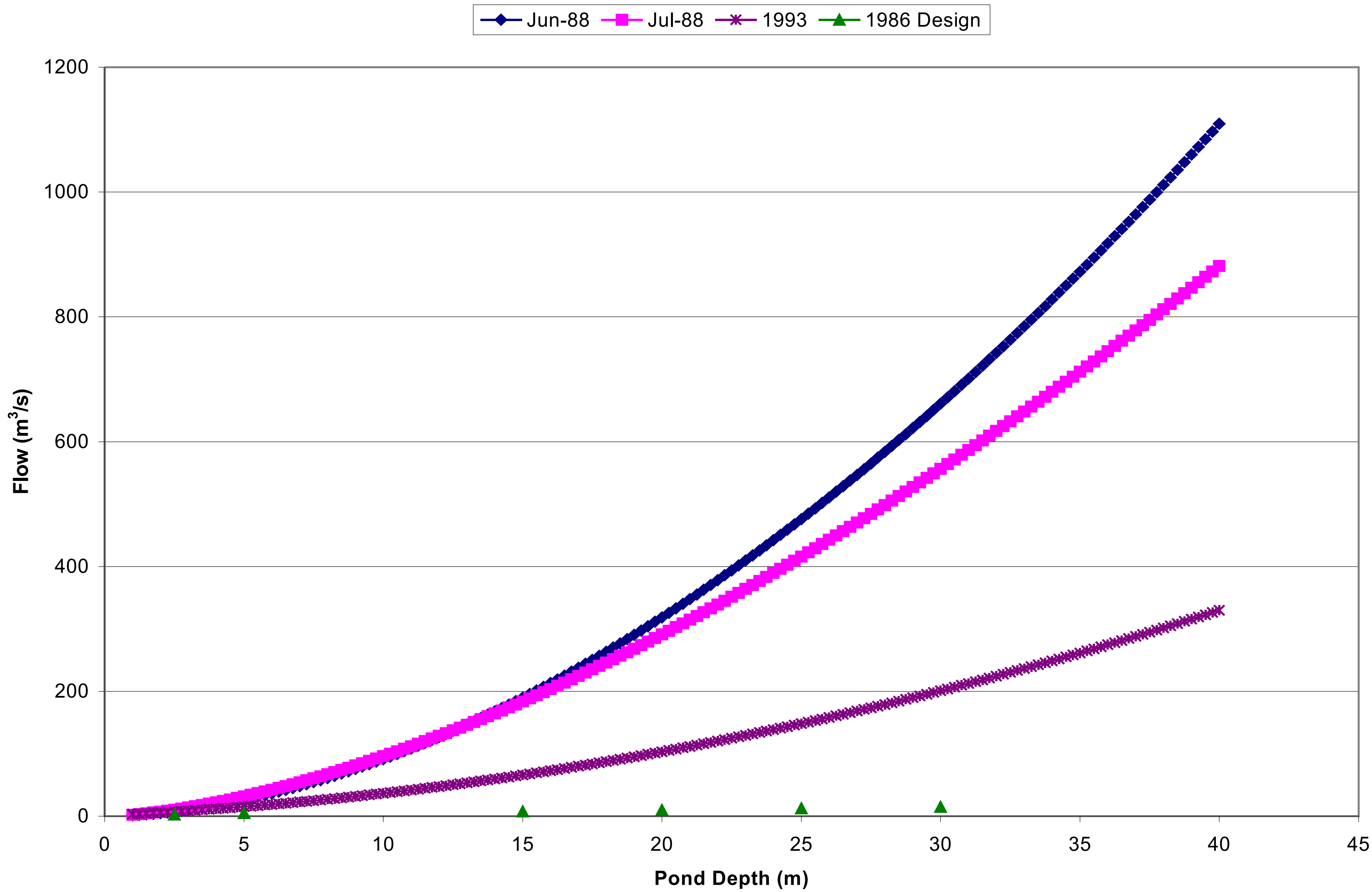

DATE: OCT 2004	DRAWN SLF

REFERENCED DRAWING DESCRIPTION

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CLIENT Deloitte & Touche

PROJECT NORTH FORK ROCK DRAIN EVALUATION

TITLE PREVIOUS RELATIONSHIPS BETWEEN POND DEPTH AND FLOW CAPACITY

PROJECT No.	DWG. No.	REV.
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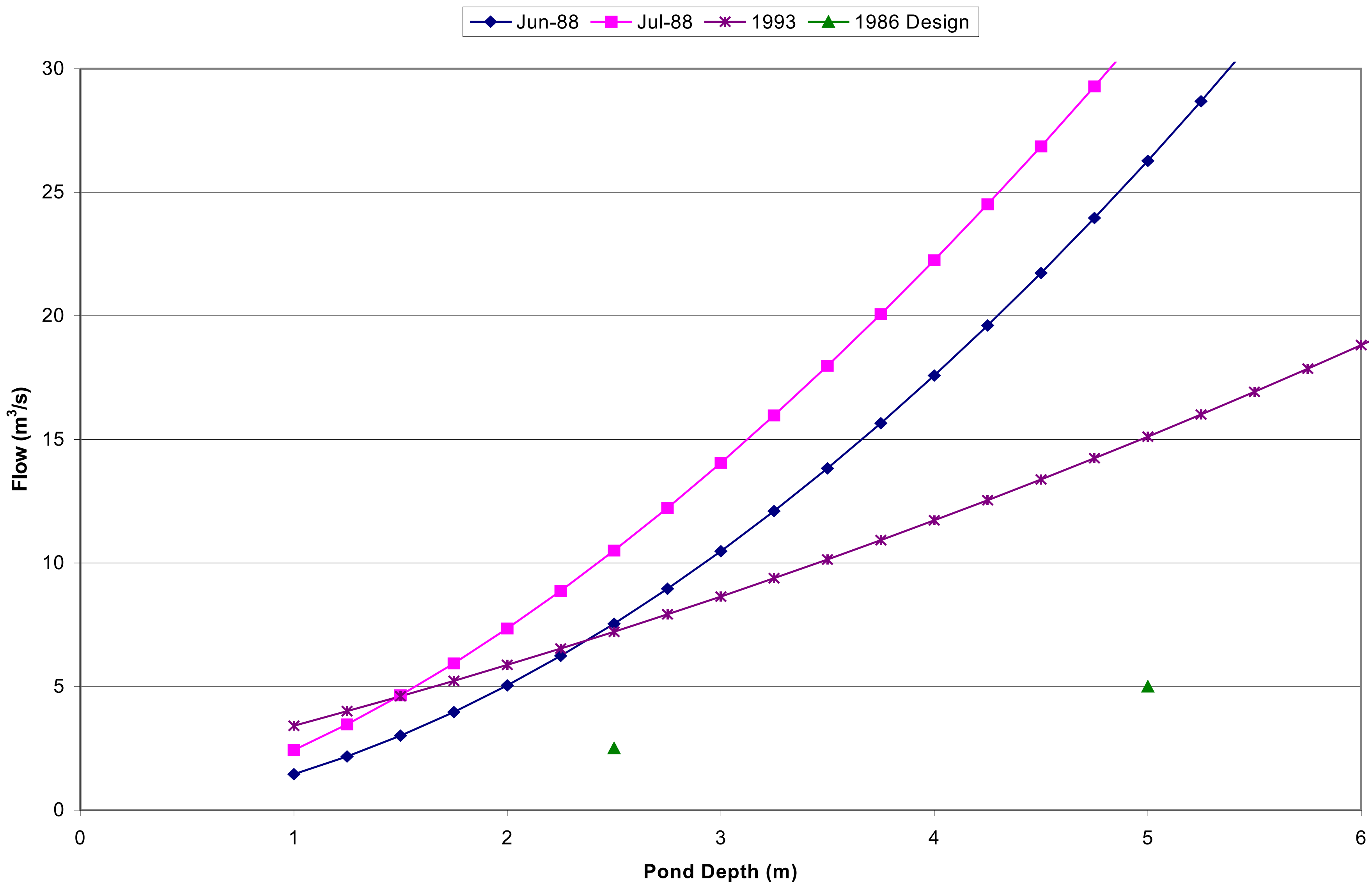

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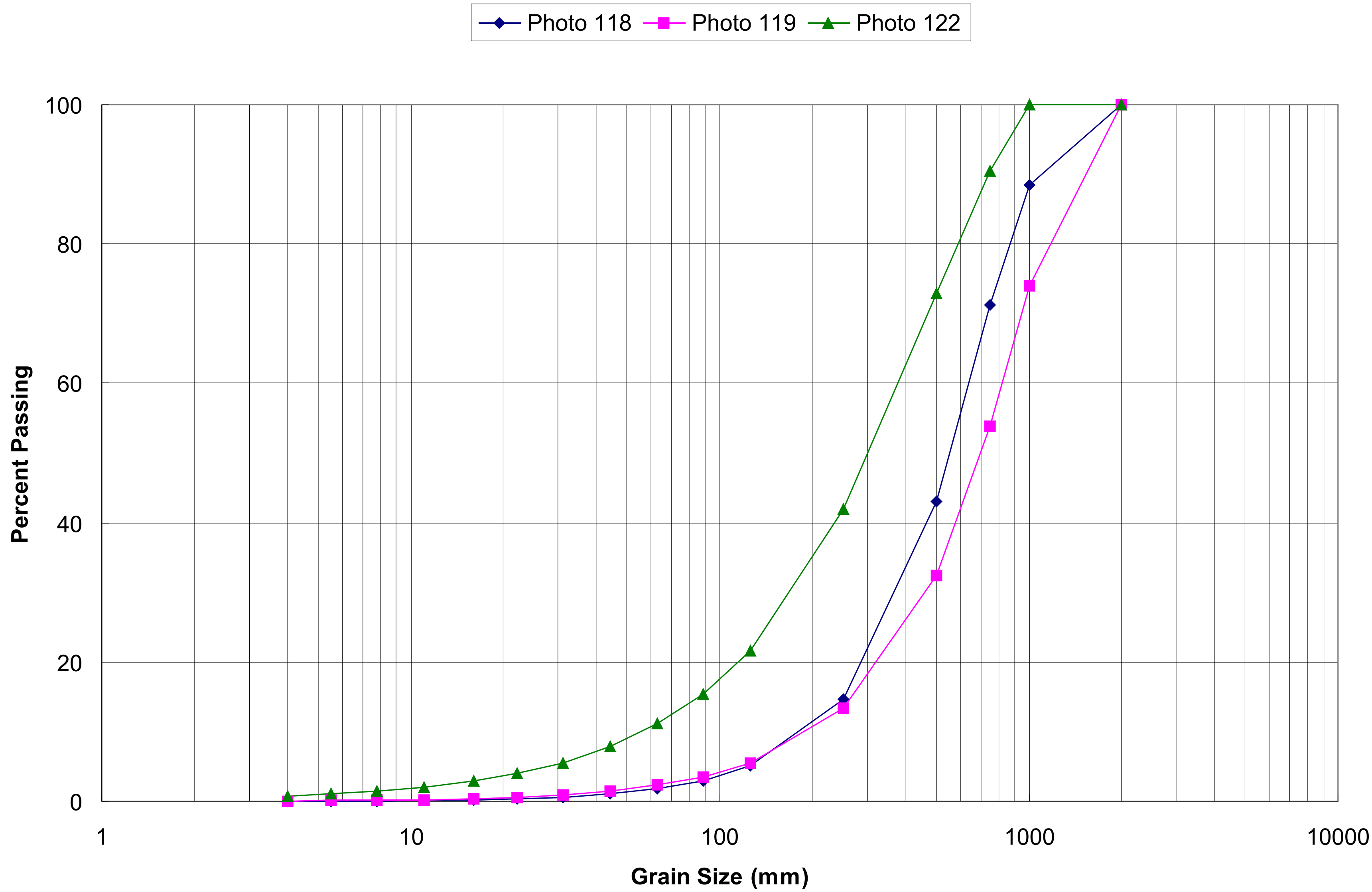

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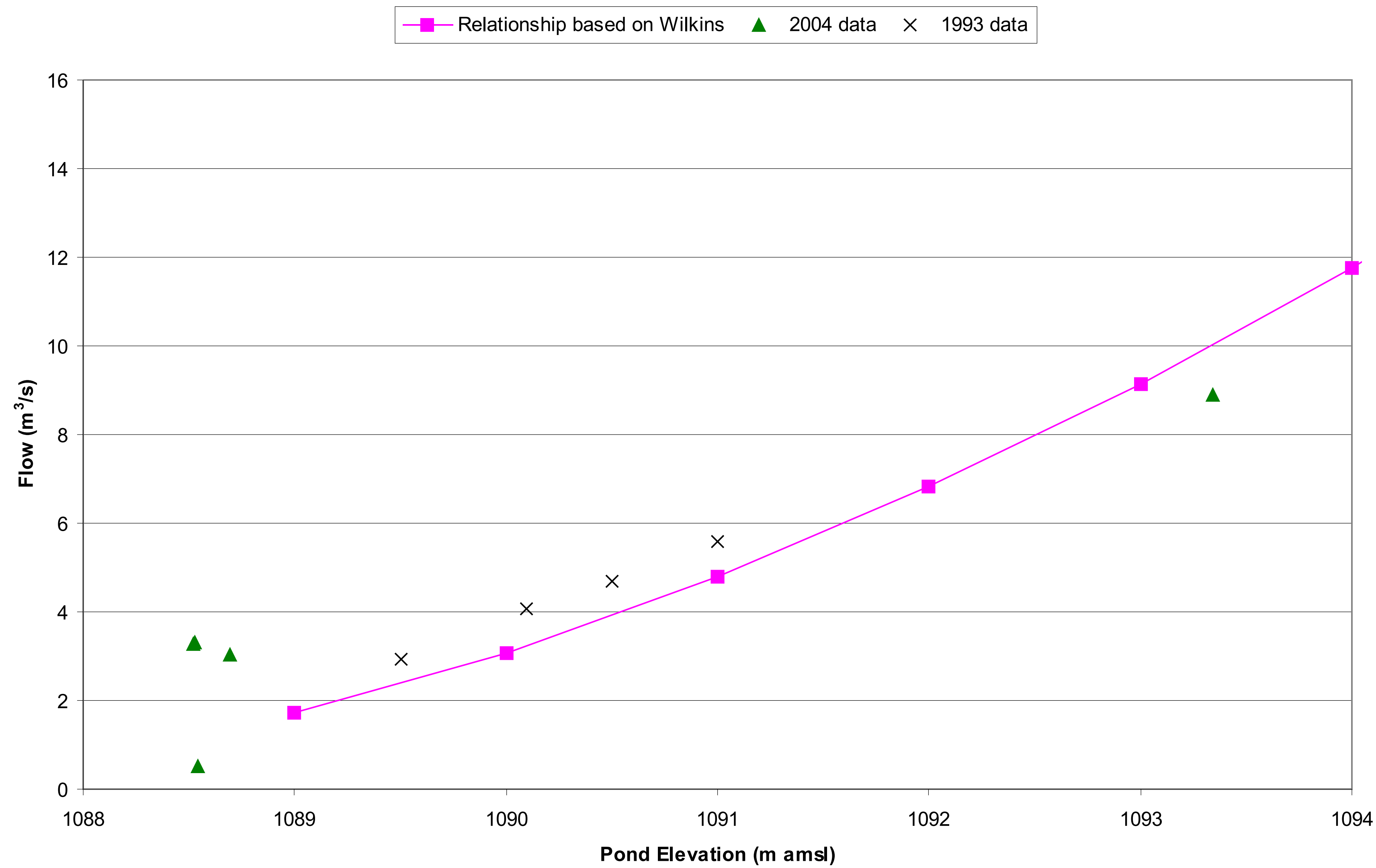

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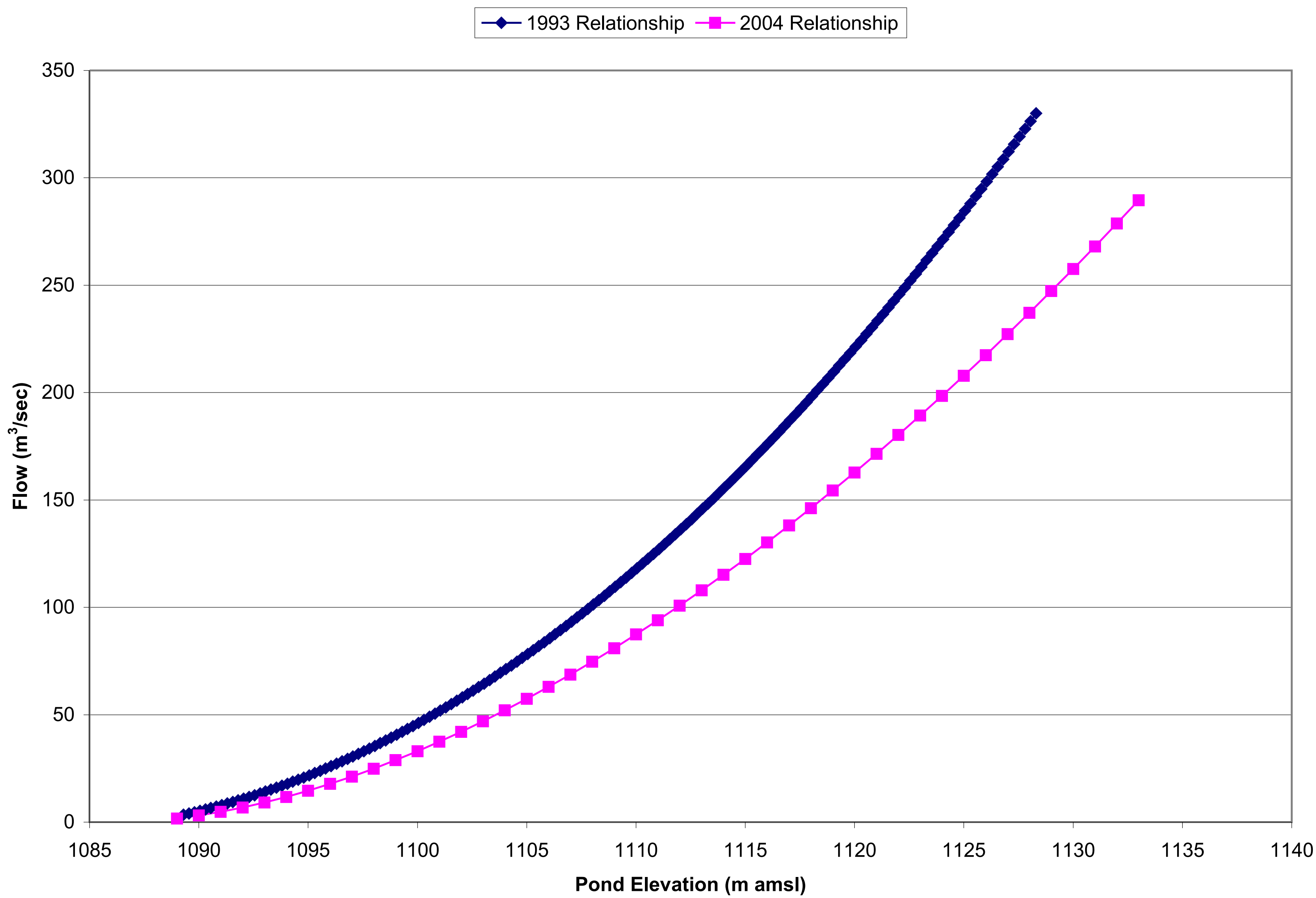

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Phone: (403) 250-5185

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PROJECT NORTH FORK ROCK DRAIN EVALUATION

TITLE COMPARISON OF DEVELOPED RELATIONSHIPS

PROJECT No. 0257-023-03

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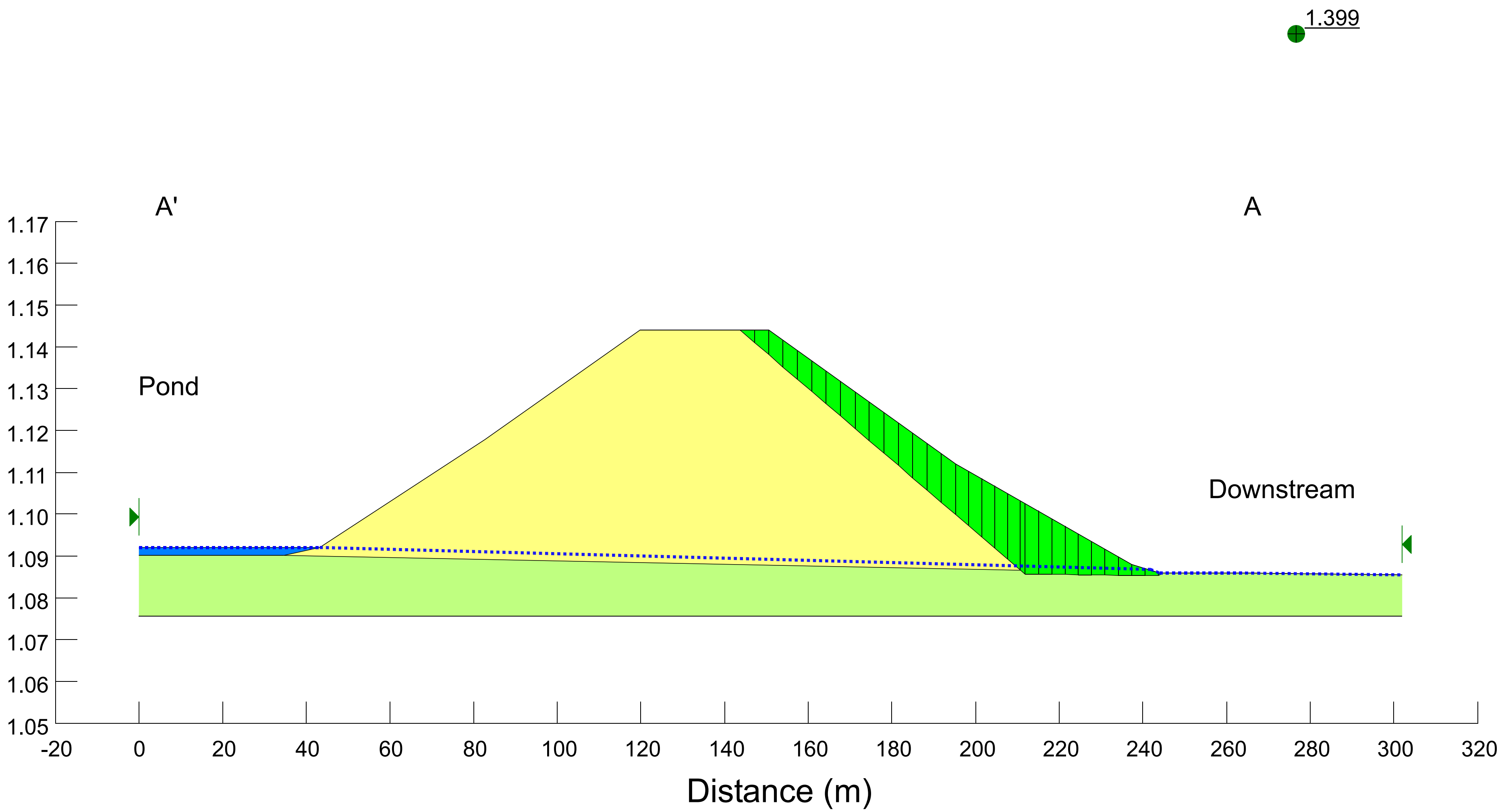

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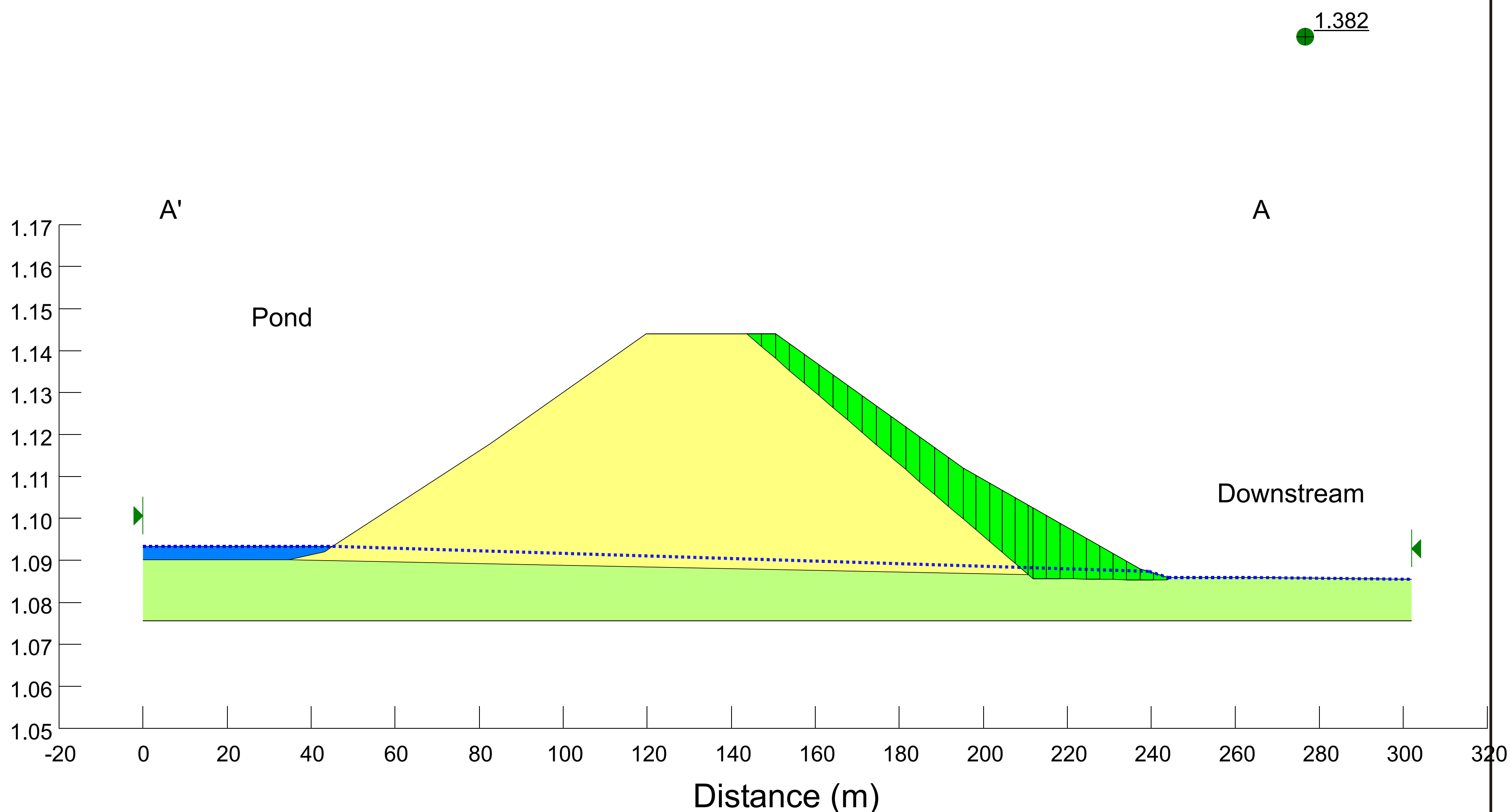

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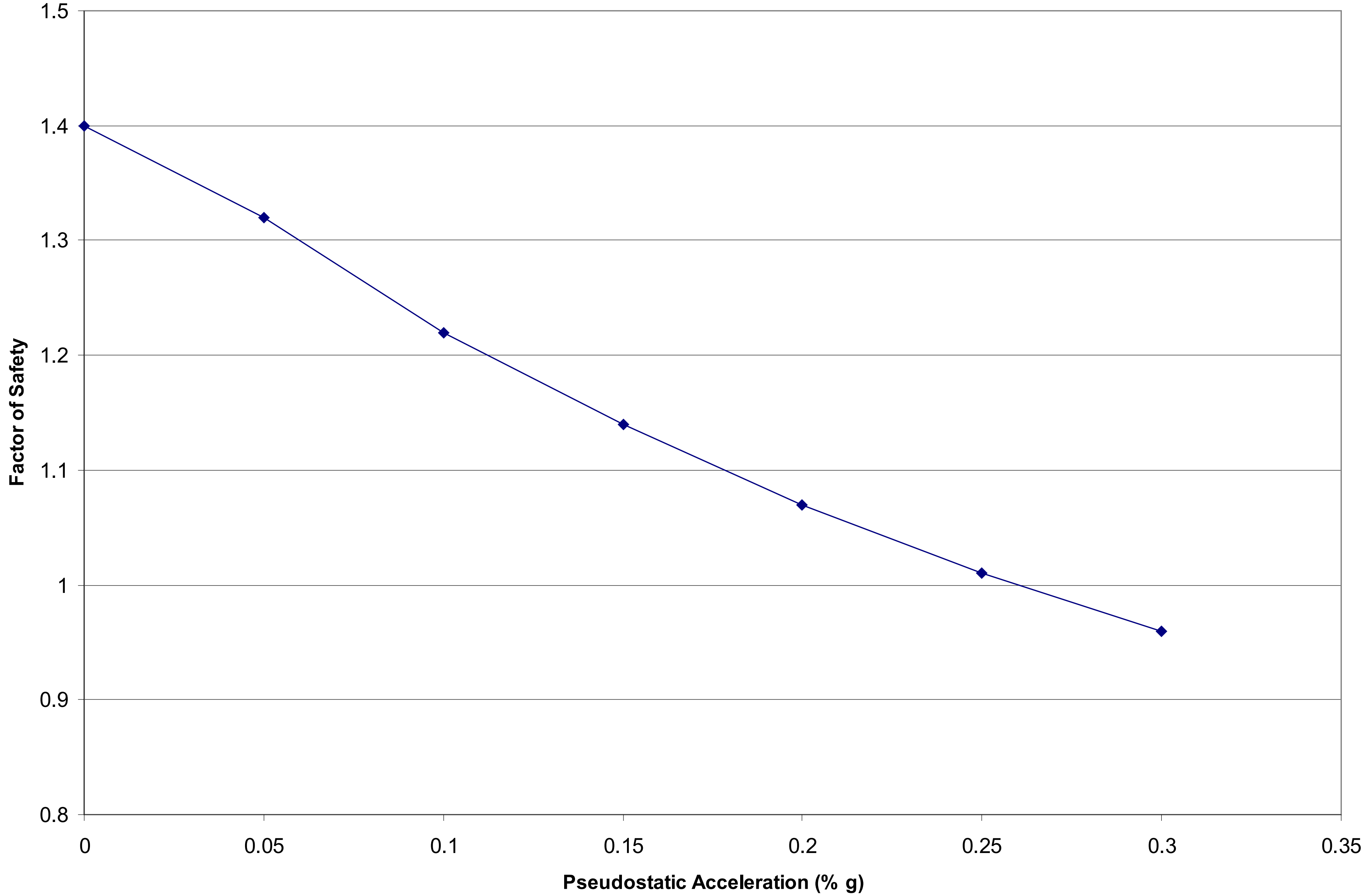
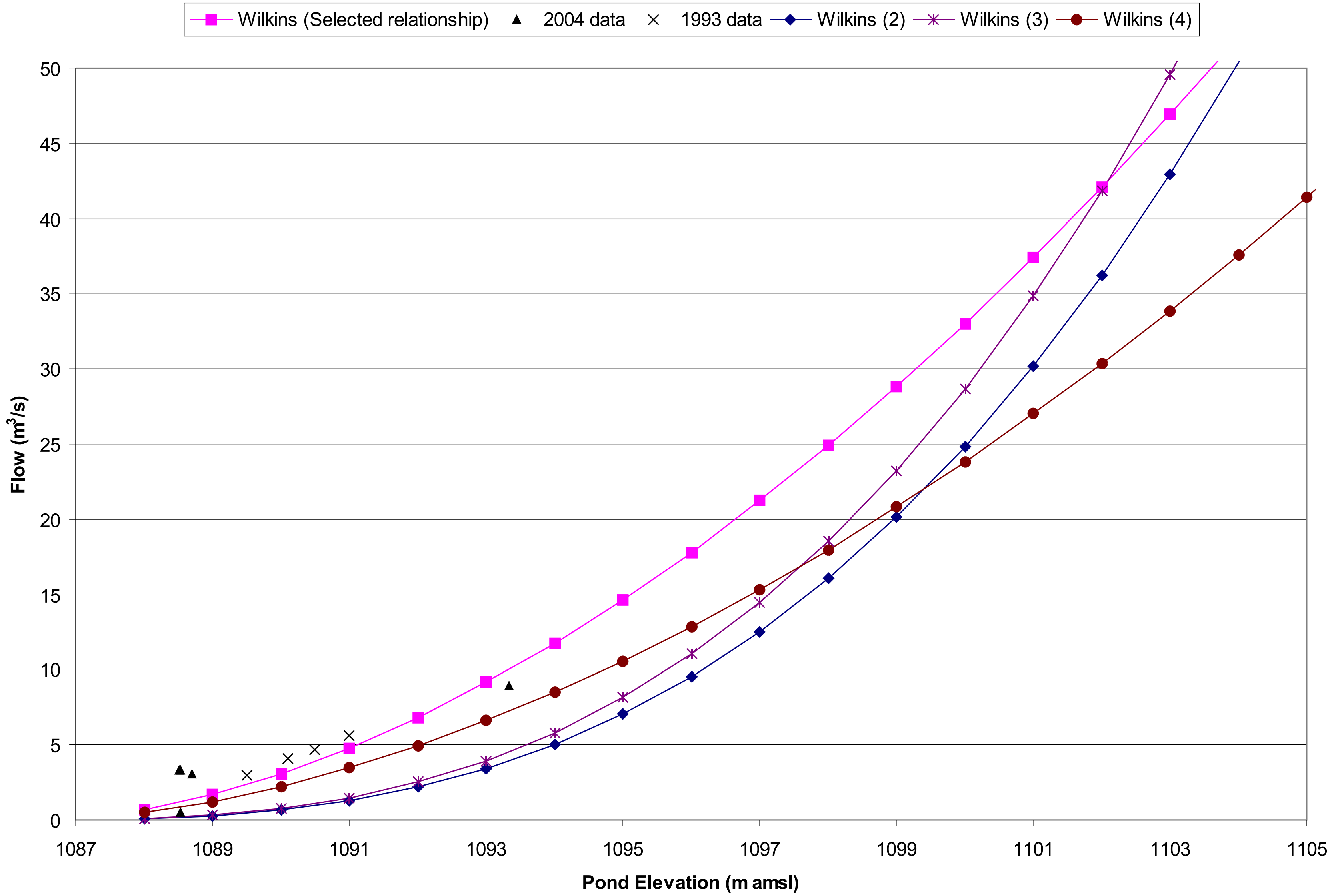


DATE: OCT 2004	DRAWN SLF	BGC ENGINEERING INC. AN APPLIED EARTH SCIENCES COMPANY Calgary, Alberta Phone: (403) 250-5185	PROJECT NORTH FORK ROCK DRAIN EVALUATION
REFERENCED DRAWING DESCRIPTION			TITLE PSEUDOSTATIC STABILITY ANALYSIS
		CLIENT Deloitte & Touche	PROJECT No. 0257-023-03 / DWG. No. 12 / REV. 0



DATE: OCT 2004	DRAWN SLF	BGC ENGINEERING INC. AN APPLIED EARTH SCIENCES COMPANY Calgary, Alberta Phone: (403) 250-5185	PROJECT NORTH FORK ROCK DRAIN EVALUATION	
REFERENCED DRAWING DESCRIPTION			TITLE SENSITIVITY OF FLOW THROUGH RELATIONSHIP	
		CLIENT Deloitte & Touche	PROJECT No. 0257-023-03	DWG. No. 13 / REV. 0

PHOTOGRAPHS

Photo 1 monitoring station X2.

Photo 2 looking upstream at monitoring station X2.

Photo 3 a panoramic view of the upstream side of the rock drain. As can be seen some sloughing of fine grained material from the upper edge of the haul road surface has occurred. Note location of the woody debris on the lower surface of the rock, this marks the previous high water level for the drain since installation.

Photo 4 looking upstream at the pond from the haul road. Not the blue barrels marking the initial position of the pond data logger. The second position of the data logger is at the left hand side of the pond, in this photo, near the boulders.

Photo 5 a panoramic of the downstream face of the drain. Note the sloughing of fine grained material from the haul road surface.

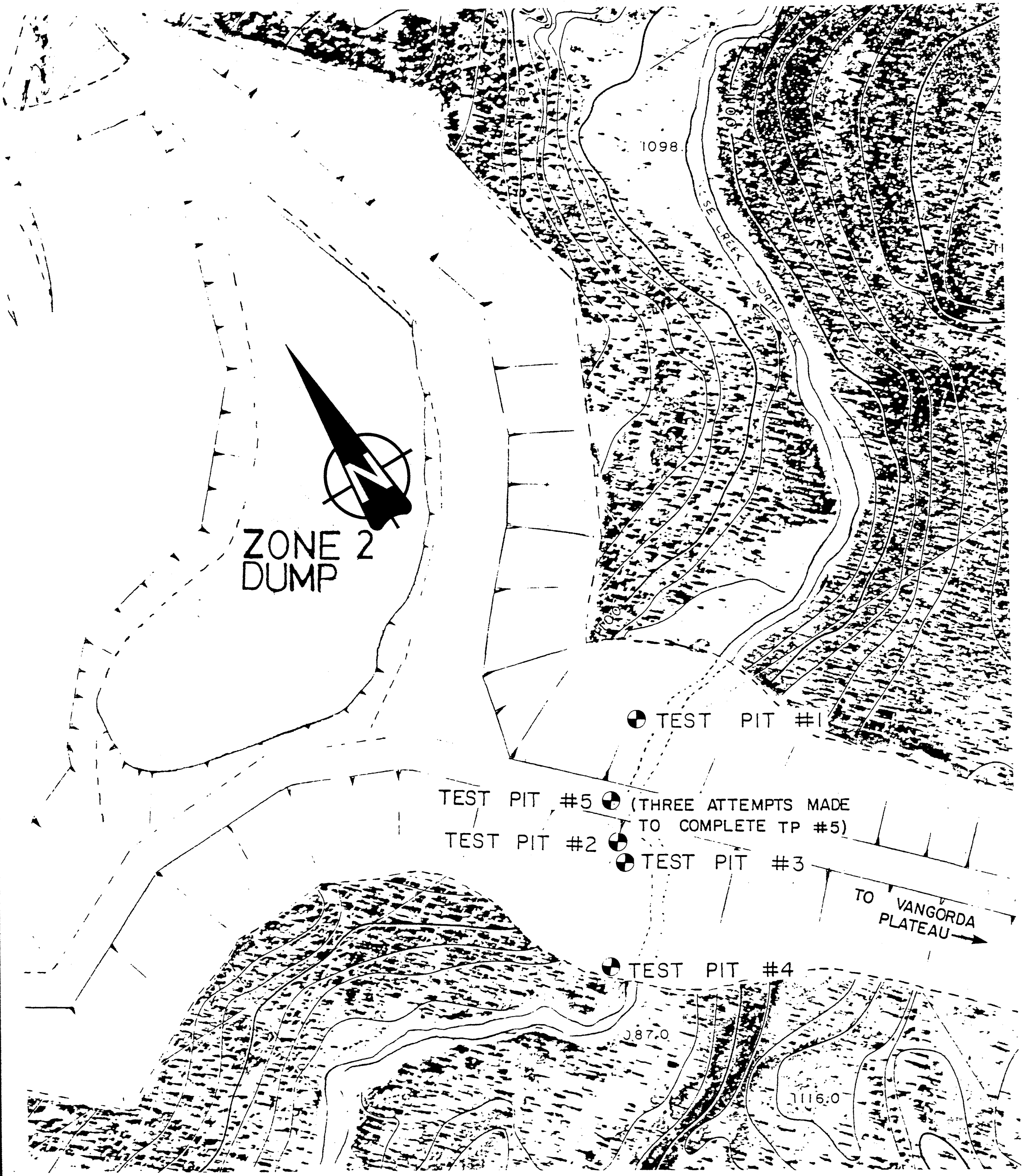
Photo 6 (118) shows the size of the boulders located at the downstream toe of the drain, near the eastern limit of the wetted area. This photo was taken at GPS waypoint 51 (See Figure 4) and includes the two red balls. These balls are 10 inch diameter and were used to estimate the grain size of the rock.

Photo 7 (122) is another view of the boulders at the downstream toe of the drain. This is GPS way station 52 (Figure 4).

Photo 8 a panoramic showing the toe of the rock drain taken from the downstream crest edge.

Photo 9 is a view of the water discharging the downstream toe.

APPENDIX I
1987 FOUNDATION INVESTIGATION

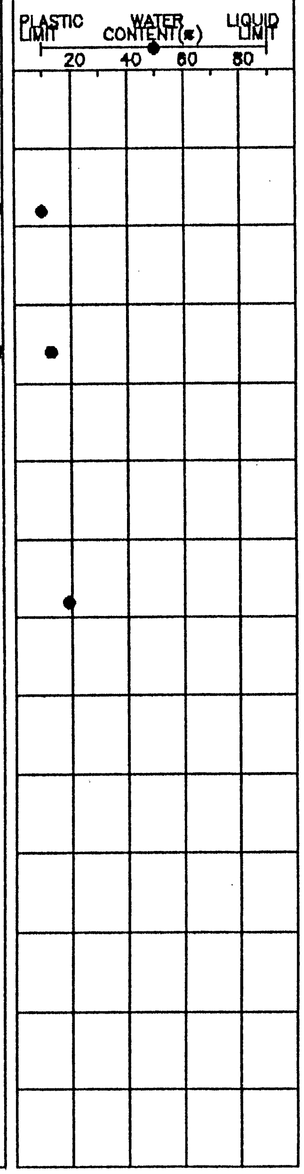


EBA Engineering Consultants Ltd.	PROJECT ROCKFILL CAUSEWAY - PERMAFROST EVALUATION
CLIENT CURRAGH RESOURCES LTD.	TITLE SITE PLAN & APPROXIMATE TEST PIT LOCATIONS
DATE 1987-04-22 / DWN WAS / CHKD	DWG NO. 4661-A-1 / 0201-4661

PROJECT NUMBER: 0201-4661 | COMPLETION DEPTH: 5.5 m | LOGGED BY: MAV

CLIENT: CURRAGH RESOURCES LTD. | EQUIPMENT USED: CAT 245 BACKHOE

DEPTH (m)	SAMPLE TYP	SAMPLE NO.	Q	SOIL DESCRIPTION	GROUND ICE DESCRIPTION	TEMP C
0				PEAT(Pt)–moss,rootlets and organic silt; frozen due to seasonal frost; very dark brown to black	Frozen	
				SAND TILL(SM)–gravelly,some silt,trace clay;with occasional cobbles and boulders throughout;quartz and schist fragments;iron oxide staining;subrounded to subangular gravel –silt content increasing with depth		
1	GRAB	1				–0.7
2	GRAB	2		–with randomly orientated ice formations (Vr Vc 5–10%) –seasonal frost to 2.0 m –ground water infiltrating side-wall of test pit	Vr Vc 5–10%	–0.1
				PEAT(Pt) AND ORGANIC SILT–with decayed organic matter;strong organic odour;black		
3				SAND(SM)–silty;coarse to fine-grained sand;wet;nonplastic;orange		
	GRAB	3		SILT(ML)–trace of fine-grained sand; with mica platlets disseminated throughout;uniform;homogeneous; wet;nonplastic;dark grey		
4				GRAVEL(GM)–sandy,some silt;with occasional cobbles/boulders throughout; rounded to subrounded,smooth gravel;wet;orange		
5.5				END OF TEST PIT 5.5 m TEST PIT TERMINATED DUE TO SLOUGHING SOILS		



SPECIAL TESTS | UNIT

DEPTH (ft): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22

ROCKFILL DRAIN–PERMAFROST EVALUATION
NORTH FORK ROSE CREEK
FARO, YUKON
DATE EXCAVATED: 1987-04-10

SAMPLE TYPE: SHELBY, NO RECOVERY, SPT, GRAB, CRREL, CORE

EBA ENGINEERING CONSULTANTS LTD.
WHITEHORSE YUKON
DRAWING NUMBER 4661-A-2

BOREHOLE NUMBER 4661-1

BOREHOLE LOG AND LABORATORY TEST RESULTS

PROJECT NUMBER: 0201-4661 COMPLETION DEPTH: 1.4 LOGGED BY: MAV

CLIENT: CURRAGH RESOURCES LTD. EQUIPMENT USED: CAT 245 BACKHOE

SAMPLE TYP	NO.	Q	SOIL DESCRIPTION	GROUND ICE DESCRIPTION	TEMP	PLASTIC LIMIT / WATER CONTENT (%) / LIQUID LIMIT (20 40 60 80)		SPECIAL TESTS	UNIT
			PEAT(Pt)—moss,rootlets,and organic silt; with occasional cobbles throughout;with stratified and randomly orientated ice formations (Vr Vs 30%), ice lenses to 20 mm thick;dark brown	Frozen					
			VOLCANIC ASH—with stratified and randomly orientated clear ice formations, ice lenses to 3 mm thick;white —boulders encountered						
			PEAT(Pt)—and fibrous organics;dark brown —boulders encountered,unable to penetrate past 1.4 m						
			END OF TEST PIT 1.4 m TEST PIT TERMINATED DUE TO COBBLES AND BOULDERS.						

DEPTH (m): 0 1 2 3 4 5 6 7

DEPTH (ft): 0 2 4 6 8 10 12 14 16 18 20 22

ROCKFILL DRAIN—PERMAFROST EVALUATION
NORTH FORK ROSE CREEK
FARO, YUKON
DATE EXCAVATED: 1987-04-10

SAMPLE TYPE: SHELBY, NO RECOVERY, SPT, GRAB, CRREL, CORE

EBA ENGINEERING CONSULTANTS LTD.
WHITEHORSE YUKON

DRAWING NUMBER
4661-A-3

BOREHOLE NUMBER
4661-2

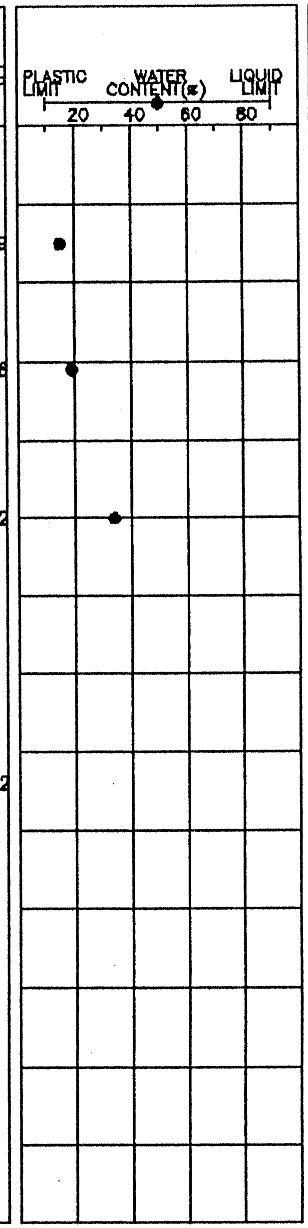
BOREHOLE LOG AND LABORATORY TEST RESULTS

PROJECT NUMBER: 0201-4661 COMPLETION DEPTH: 6.0 m LOGGED BY: MAV

CLIENT: CURRAGH RESOURCES LTD. EQUIPMENT USED: CAT 245 BACKHOE

SAMPLE TYP	NO.	Q	SOIL DESCRIPTION	GROUND ICE DESCRIPTION	TEMP C
			PEAT(Pt)–moss and rootlets at surface; with fibrous organics and organic silt;brown –with occasional cobbles –with occasional lenses of volcanic ash	Frozen	
GRAB	1			Vr 5–10%	–0.8
			SAND TILL(SM) AND SILT–trace gravel,trace clay;with occasional cobbles throughout;subrounded to angular gravel;with randomly orientated ice formations (Vr 5–10%) to 2 mm thick;iron oxide staining;low plastic;olive grey		
GRAB	2			Vc Vs 5%	–0.6
			–with stratified ice formations (Vs 40%) to 10 mm thick –gravelly,trace organics;with stratified ice formations (Vs Vc 5%)		
GRAB	3		–cobbles –with stratified and randomly orientated clear ice formations (Vr Vs 15–20%);olive	Vr Vs 15–20%	–0.2
			–easy digging with backhoe;soil is pliable with clear ice lenses		
GRAB	4		–ice lenses to 5 mm thick (Vr Vs 15%)	Vr Vs 15%	–0.2
			END OF TEST PIT 6.0 m TEST PIT TERMINATED DUE TO SLOUGHING SOIL AT 2.9 m. GROUND WATER INFILTRATING TEST PIT.		

DEPTH (m): 0–7; DEPTH (ft): 0–22



SPECIAL TESTS	UNIT

ROCKFILL DRAIN–PERMAFROST EVALUATION
NORTH FORK ROSE CREEK
FARO, YUKON
DATE EXCAVATED: 1987-04-11

SAMPLE TYPE: SHELBY, NO RECOVERY, SPT, GRAB, CRREL, CORE

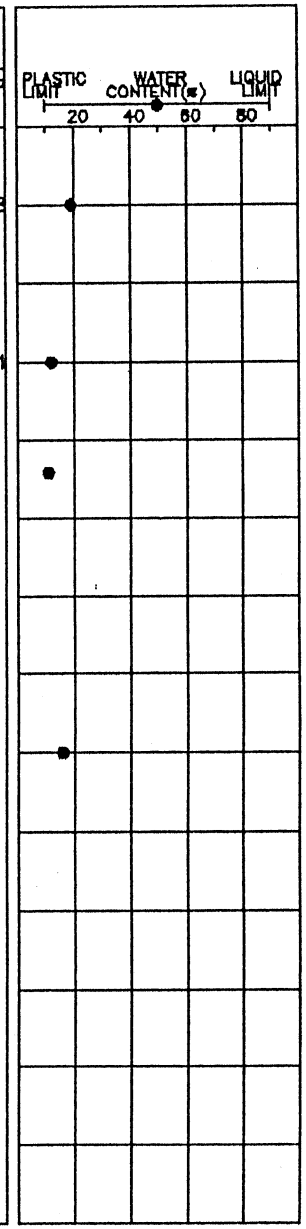
EBA ENGINEERING CONSULTANTS LTD.
WHITEHORSE YUKON
DRAWING NUMBER 4661-A-4

BOREHOLE NUMBER 4661-3

BOREHOLE LOG AND LABORATORY TEST RESULTS

PROJECT NUMBER: 0201-4661
CLIENT: CURRAGH RESOURCES LTD.
COMPLETION DEPTH: 6.0 m LOGGED BY: MAV
EQUIPMENT USED: CAT 245 BACKHOE

SAMPLE TYP	NO.	Q	SOIL DESCRIPTION	GROUND ICE DESCRIPTION	TEMP C
			PEAT(Pt)–root mat,moss and organics	Frozen	
GRAB	1		SAND TILL(SM) AND SILT–some gravel, some clay,trace coal fragments; with occasional cobbles;sub–rounded to angular gravel;iron oxide staining;low to nonplastic; olive brown	Vr Vs 10%	–0.8
			–with stratified and randomly orientated ice formations (Vr Vs 10%)		
GRAB	2		–easy digging	Vr Vs 5%	–0.1
			PEAT(Pt)–and organics,black		
GRAB	3		SAND TILL(SM) AND SILT–some gravel,some clay,trace organics;rounded to subrounded,smooth gravel;with iron oxide staining;wet;soft to firm(estimated);low plastic; olive		0
GRAB	4				
			END OF TEST PIT 6.0 m TEST PIT TERMINATED DUE TO SLOUGHING SOIL AT 2.0 m. GROUND WATER INFILTRATING TEST PIT.		



SPECIAL TESTS | UNIT

DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7
DEPTH (ft): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22

ROCKFILL DRAIN–PERMAFROST EVALUATION
NORTH FORK ROSE CREEK
FARO, YUKON
DATE EXCAVATED: 1987-04-11

SAMPLE TYPE
SHELBY | GRAB
NO RECOVERY | CRREL
SPT | CORE

EBA ENGINEERING CONSULTANTS LTD.
WHITEHORSE YUKON
DRAWING NUMBER
4661-A-5

BOREHOLE NUMBER
4661-4

BOREHOLE LOG AND LABORATORY TEST RESULTS

PROJECT NUMBER: 0201–4661
CLIENT: CURRAGH RESOURCES LTD.

COMPLETION DEPTH: 0.7 m LOGGED BY: MAV
EQUIPMENT USED: CAT 245 BACKHOE

SAMPLE TYP	SAMPLE NO.	SAMPLE Q	SOIL DESCRIPTION	GROUND ICE DESCRIPTION	TEMP
			PEAT(Pt)–moss,rootlets and organic silt; dark brown to black –volcanic ash	Frozen	
			SAND TILL(SM) AND SILT–some gravel,trace clay;with occasional cobbles throughout;subrounded to sub–angular gravel;nonplastic; olive brown –hard digging due to seasonal frost and cobbles –unable to penetrate past 0.7 m due to cobbles/boulders.		

END OF TEST PIT 0.7 m
TEST PIT TERMINATED DUE TO COBBLES AND BOULDERS.

NOTE: Three attempts made at different locations to complete Test Pit#5. Backhoe unable to penetrate past cobbles/boulders encountered.

DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7

PLASTIC LIMIT — WATER CONTENT(%) — LIQUID LIMIT: 20, 40, 60, 80

SPECIAL TESTS | UNIT

DEPTH (ft): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22

ROCKFILL DRAIN–PERMAFROST EVALUATION
NORTH FORK ROSE CREEK
FARO, YUKON
DATE EXCAVATED: 1987–04–10

SAMPLE TYPE
SHELBY | NO RECOVERY | SPT | GRAB | CRREL | CORE

EBA ENGINEERING CONSULTANTS LTD.
WHITEHORSE YUKON
DRAWING NUMBER
4661–A–6

BOREHOLE NUMBER
4661–5

BOREHOLE LOG AND LABORATORY TEST RESULTS

EBA Engineering Consultants Ltd.

PARTICLE - SIZE ANALYSIS OF SOILS

Project: Rock Causeway - Permafrost Evaluation
North Fork Rose Creek, Faro, Yukon

Project Number: 0201-4661

Date Tested: 1987-04-15

Borehole Number: TEST PIT #1

Depth: 0.8 - 1.0 m

Soil Description: SAND TILL (SM) - gravelly, some silt

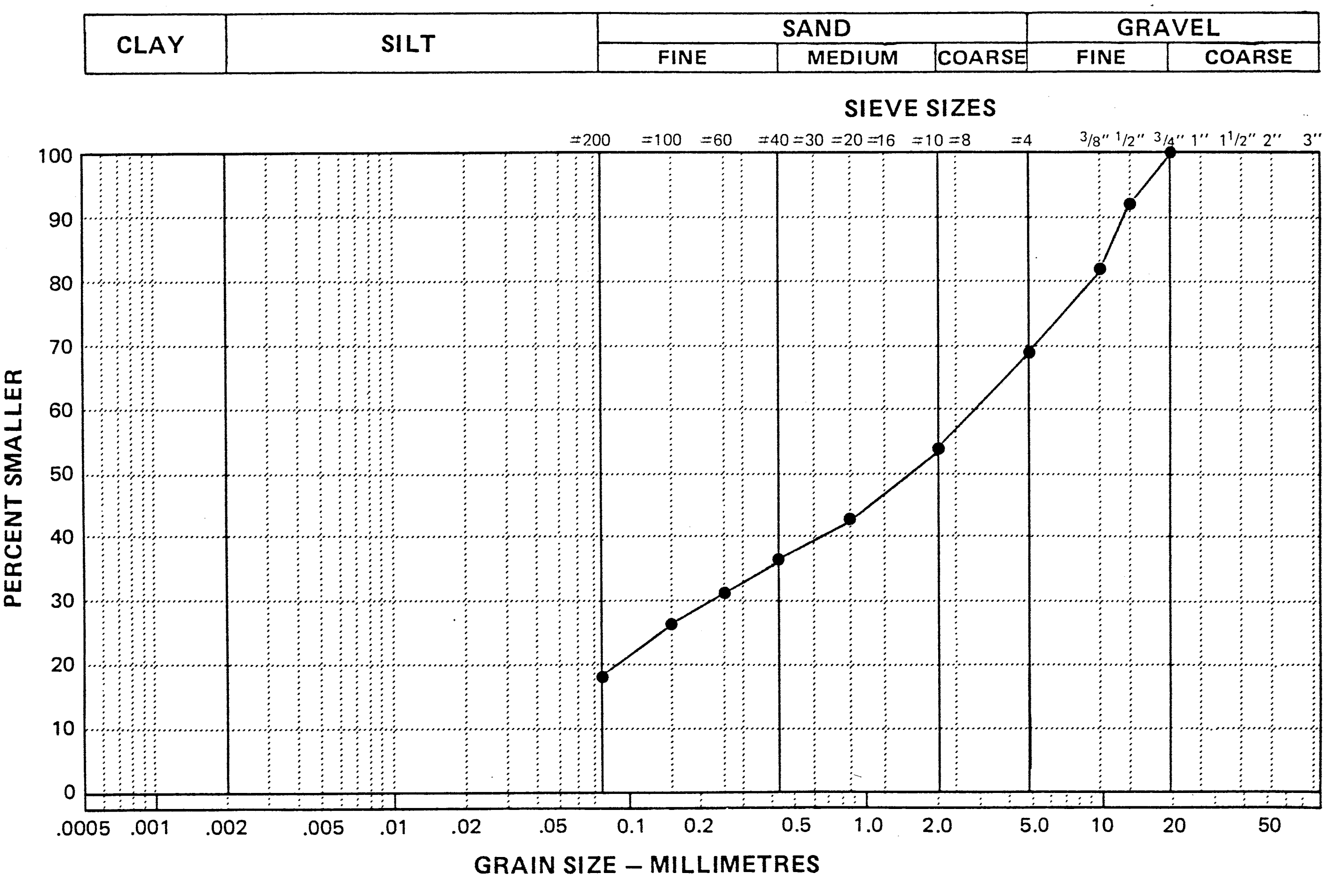
Cu:

Cc:

Natural Moisture Content: 9.7 %

Remarks:

SIEVE	PERCENTAGE PASSING
3"	
1 1/2"	
1"	
3/4"	100
1/2"	92
3/8"	82
No. 4	69
No. 10	54
No. 20	43
No. 40	36
No. 60	31
No. 100	26
No. 200	18



Tested in accordance with ASTM D422 unless otherwise noted.

EBA Engineering Consultants Ltd.

PARTICLE - SIZE ANALYSIS OF SOILS

Project: Rock Causeway - Permafrost Evaluation
North Fork Rose Creek - Faro, Yukon

Project Number: 0201-4661

Date Tested: 1987-04-15

Borehole Number: TEST PIT #3

Depth: 0.5 - 1.0 m

Soil Description: SAND TILL(SM) AND SILT - trace gravel, trace clay

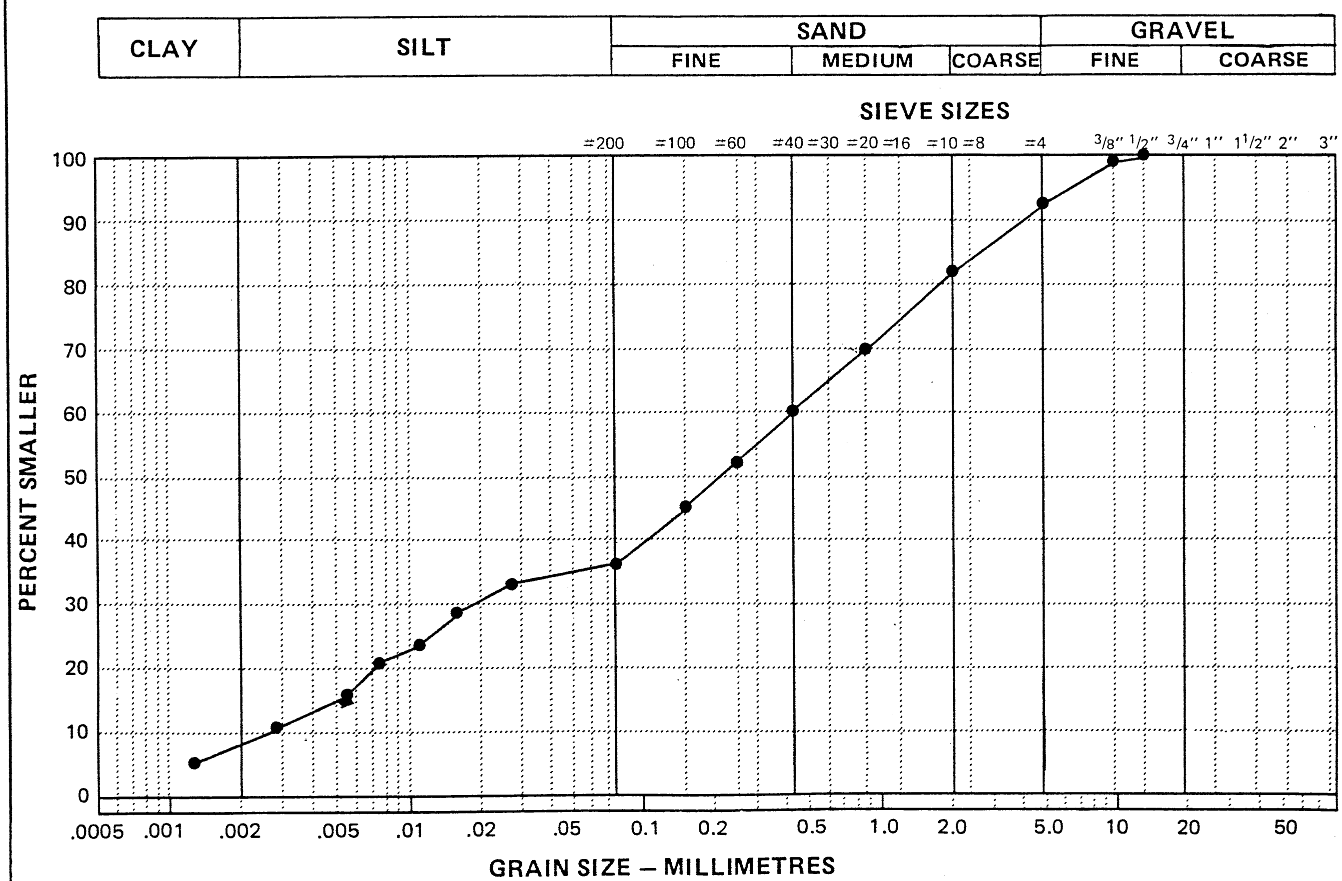
Cu:

Cc:

Natural Moisture Content: 15.2 %

Remarks:

SIEVE	PERCENTAGE PASSING
3"	
1 1/2"	
1"	
3/4"	
1/2"	100
3/8"	99
No. 4	93
No. 10	82
No. 20	70
No. 40	60
No. 60	52
No. 100	45
No. 200	36



Tested in accordance with ASTM D422 unless otherwise noted.

EBA Engineering Consultants Ltd.

eba

PARTICLE - SIZE ANALYSIS OF SOILS

Project: Rôck Causeway, Permafrost Evaluation
North Fork Rose Creek - Faro, Yukon

Project Number: 0201-4661

Date Tested: 1987-04-15

Borehole Number: TEST PIT #4

Depth: 3.9 - 4.1 m

Soil Description: SAND TILL (SM) AND SILT - some gravel, some clay

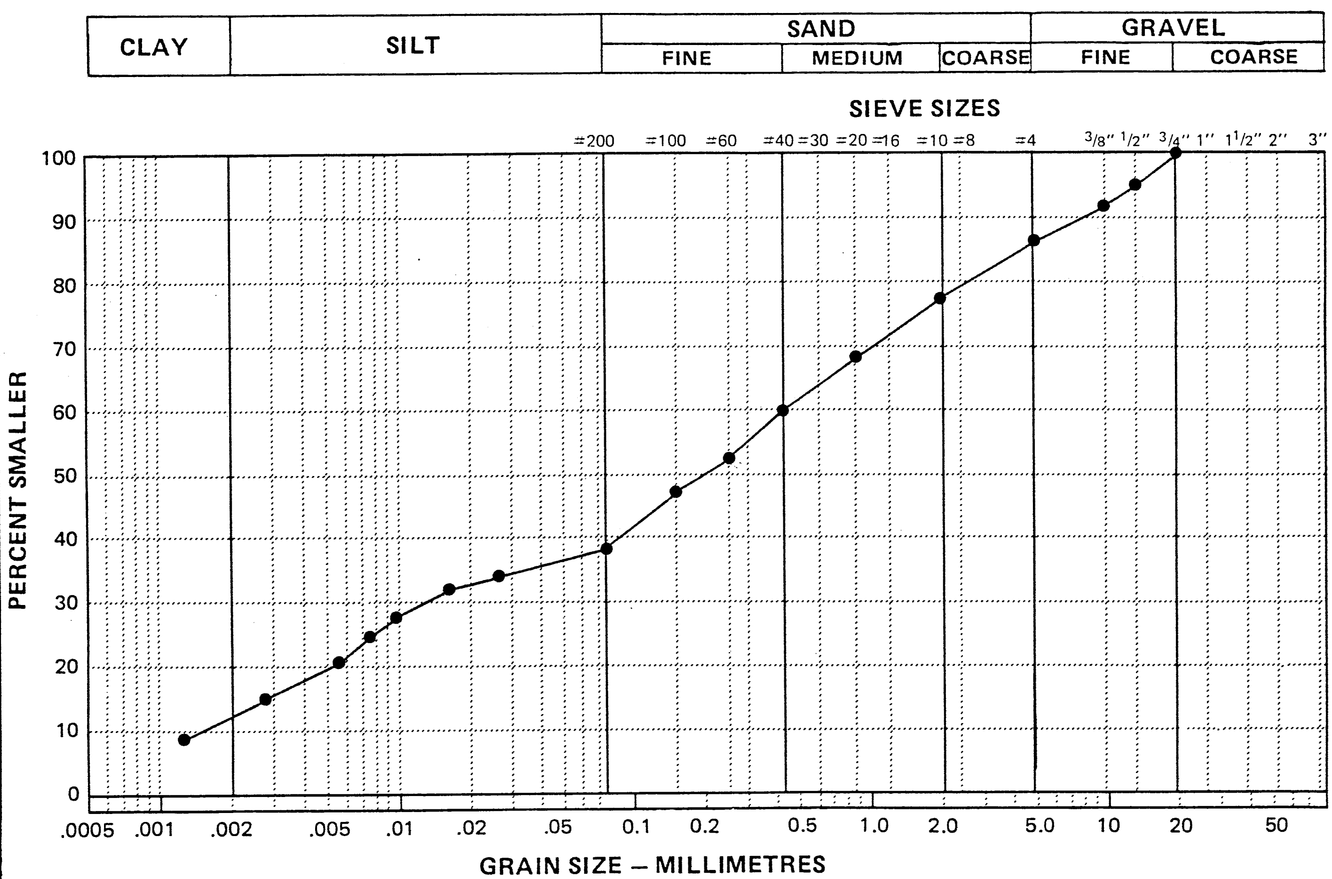
Cu:

Cc:

Natural Moisture Content: 15.9 %

Remarks:

SIEVE	PERCENTAGE PASSING
3"	
1 1/2"	
1"	
3/4"	100
1/2"	95
3/8"	92
No. 4	86
No. 10	77
No. 20	68
No. 40	60
No. 60	53
No. 100	47
No. 200	38



Tested in accordance with ASTM D422 unless otherwise noted.

APPENDIX II
PHOTOGRAPHS TAKEN DURING 1987 CONSTRUCTION OF NFRD

PHOTO 1

VIEW FROM THE CREST OF THE ROCK FILL THAT IS BEING ADVANCED ACROSS THE NORTH FORK OF ROSE CREEK, AS PART OF THE ROAD ACCESS TO THE VAGNORDA ORE DEPOSIT. THE SEGMENT OF THE FILL SHOWN WILL FORM PART OF THE NORTH FORK ROCK DRAIN, AND THE MATERIAL CONSISTS OF CALCIUM SILICATE (CaSi) BRECCIA.
SOME LARGE ROCK FRAGMENTS AT THE TOE OF THE FILL HAVE BEEN SHIFTED BY BULLDOZER INTO INTO THE NORTH FORK DRAINAGE CHANNEL. THE SIZE OF THE ROCK FRAGMENTS THAT SEPARATE ON THE ADVACING FACE OF THE FILL AND ROLL TO THE TOE, ARE OF SUFFICIENT SIZE THAT PLACEMENT OF THE LARGEST OF THESE FRAGMENTS BY BULLDOZER IS NOT NECESSARY. BULLDOZER PLACEMENT OF ROCK IN THE CHANNEL IS TO BE DISCONTINUED.

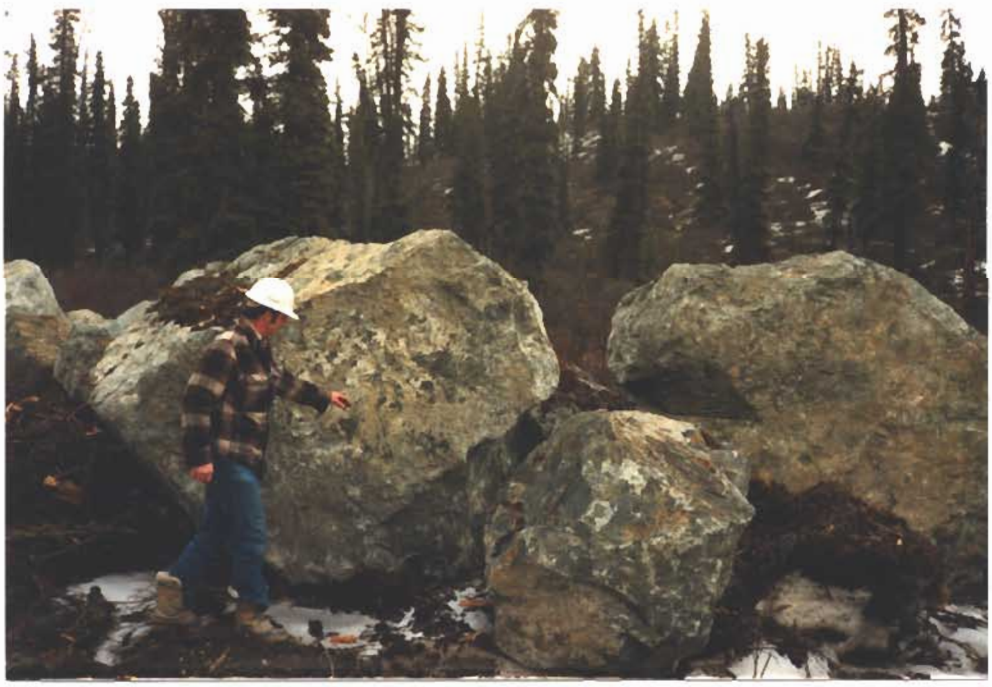


PHOTO 2

AN EXAMPLE OF THE SIZE OF CaSi ROCK FRAGMENTS THAT HAVE BEEN PLACED IN THE NORTH FORK DRAINAGE CHANNEL.

PHOTO 3

ILLUSTRATING THE SIZE OF CaSi ROCK FRAGMENTS THAT HAVE SEPARATED ON THE FACE OF THE ROCK FILL, AND HAVE COME TO REST AT THE TOE.

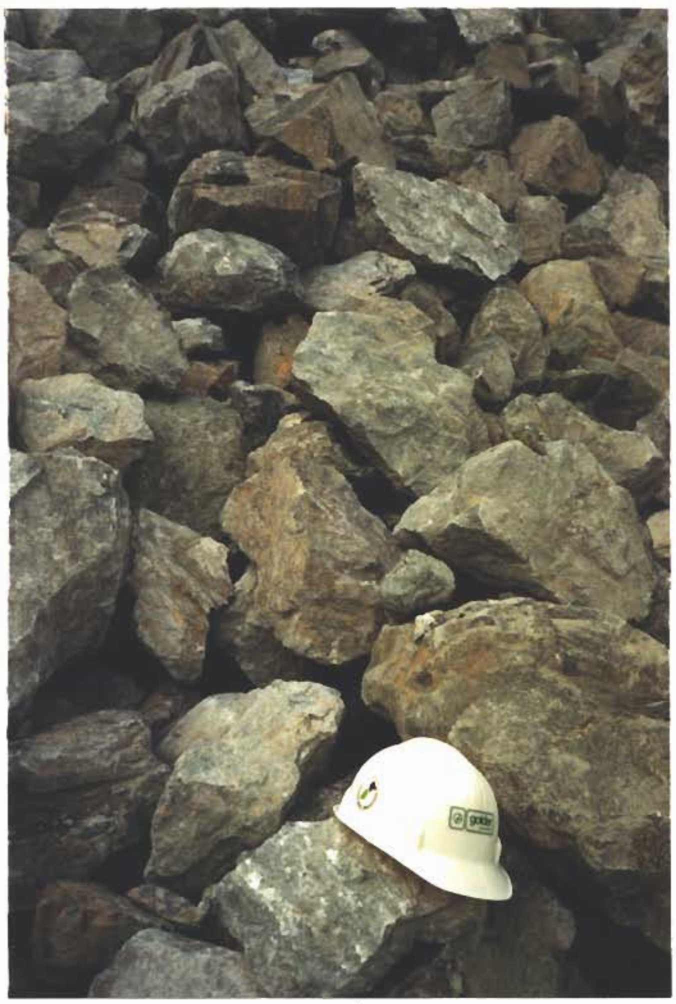


PHOTO 4

ILLUSTRATING THE SIZE OF THE ROCK FRAGMENTS ON THE FACE OF CaSi FILL, AT A HEIGHT OF APPROXIMATELY 7m ABOVE THE TOE OF THE FILL AS AT 7 MAY 87. THE 65% SIZE IS ESTIMATED TO BE 0.5m, AND THE 50% SIZE IS ESTIMATED TO BE APROXIMATELY 0.3m.

PHOTO 5

ILLUSTRATING THE SIZE OF THE CaSi ROCK FRAGMENTS ON THE FACE OF THE FILL AT A HEIGHT APPROXIMATELY 10m ABOVE THE TOE, AS AT 7 MAY 87. AT THIS PARTICULAR SEGMENT ON THE FACE, THE AVERAGE SIZE OF THE ROCK FRAGMENTS IS SLIGHTLY LARGER THAN THE SIZE OF THE FRAGMENTS AT A HEIGHT 7m ABOVE THE TOE.

PHOTO 6

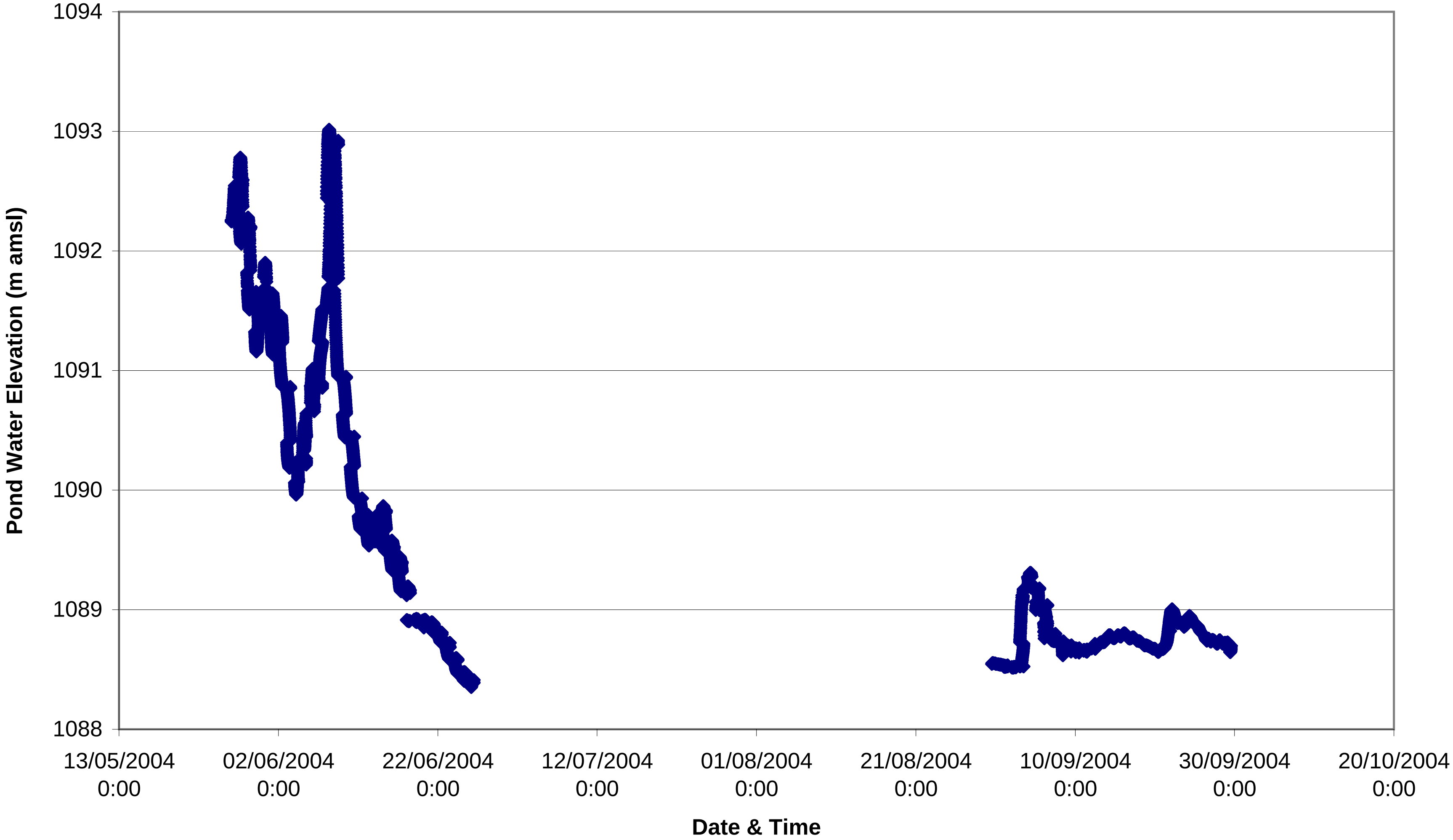
SHOWING THE ROCK FRAGMENTS ON THE FACE OF THE ADVANCING ROADWAY FILL AT AN ESTIMATED HEIGHT OF 17m ABOVE THE TOE, AS AT 7 MAY 87. THE 65% SIZE IS ESTIMATED TO BE 0.5m, THE 50% SIZE IS ESTIMATED TO BE 150mm. NOTE THE 75 TO 150mm SIZE FRAGMENTS ON THE FACE, SIGHTLY ABOVE AND TO THE LEFT OF THE HARD HAT. THE SMALLEST SIZE OF THESE FRAGMENTS IS APPROXIMATELY 40mm.

PHOTO 7

ILLUSTRATING THE CaSi ROCK ON THE FACE OF THE VANGORDA ROADWAY FILL AT A HEIGHT ESTIMATED AT 55m ABOVE THE TOE OF THE FILL AS AT 7 MAY 67. AS IS APPARENT IN THE PHOTO, THE ROCK AT THIS LEVEL IS DEVOID OF FINES. FINES WERE EVIDENT ONLY WITHIN THE UPPER 10 METERS ON THE FACE, ie BETWEEN THE CREST OF THE FILL AT 3900 Ft AND ELEVATION 3770 Ft APPROXIMATELY.

APPENDIX III
2004 MONITORING RESULTS

Figure III-1
2004 Pond Elevations



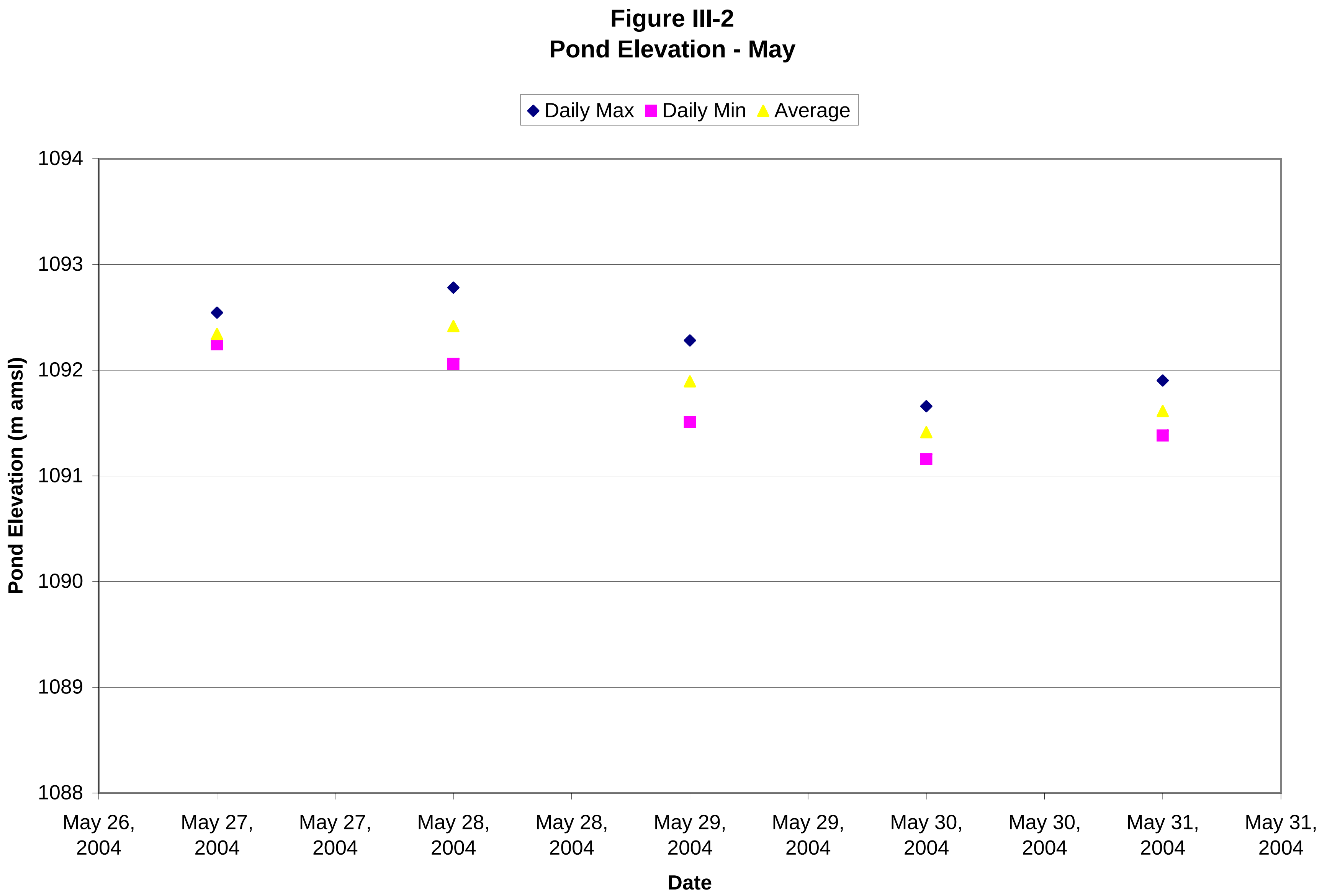


Figure III-2
Pond Elevation - May
Daily Max
Daily Min
Average
Pond Elevation (m amsl)
1094
1093
1092
1091
1090
1089
1088
May 26, 2004
May 27, 2004
May 27, 2004
May 28, 2004
May 28, 2004
May 29, 2004
May 29, 2004
May 30, 2004
May 30, 2004
May 31, 2004
May 31, 2004
Date

Figure III-3
Pond Elevation - June

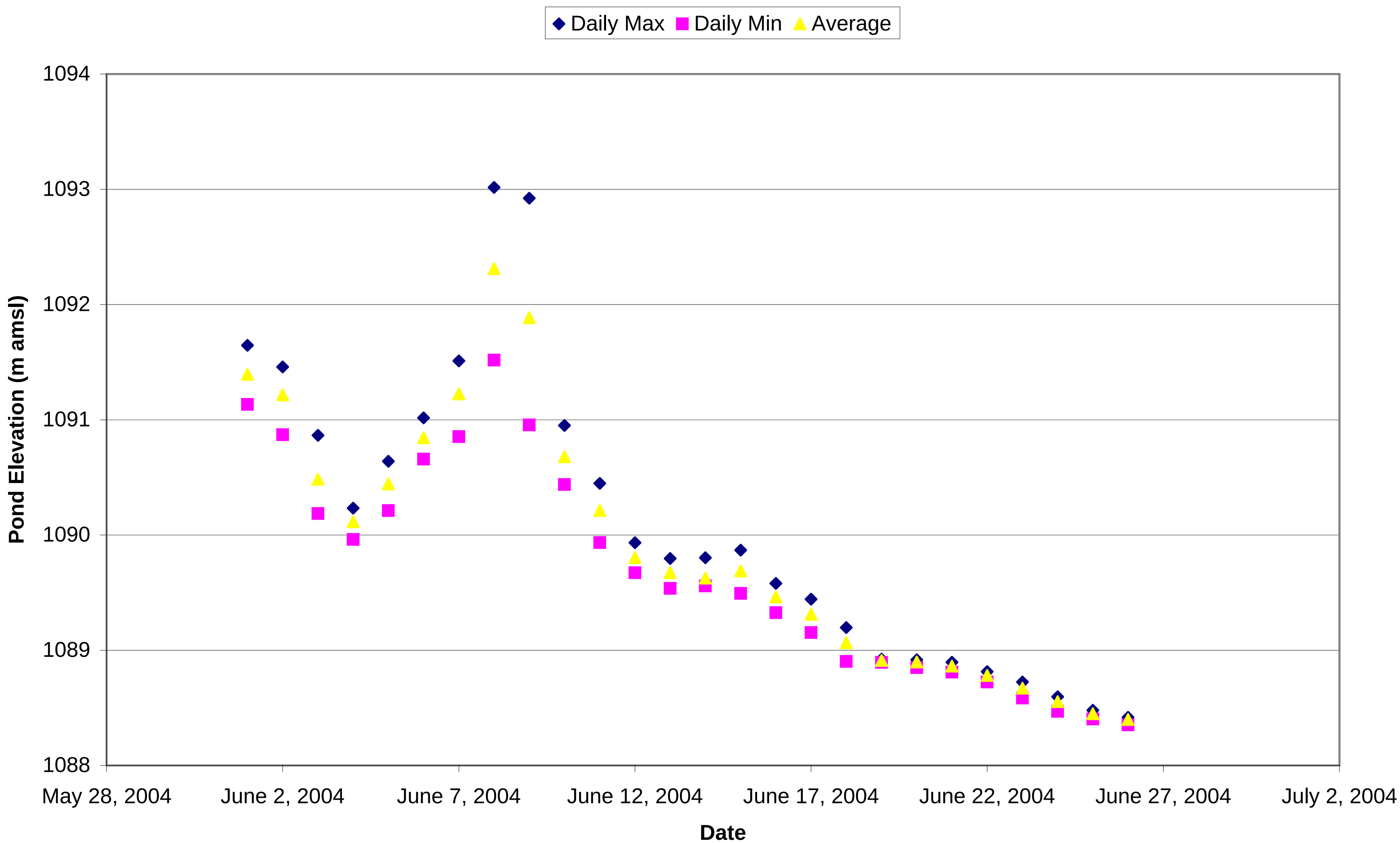


Figure III-4
Pond Elevation - August

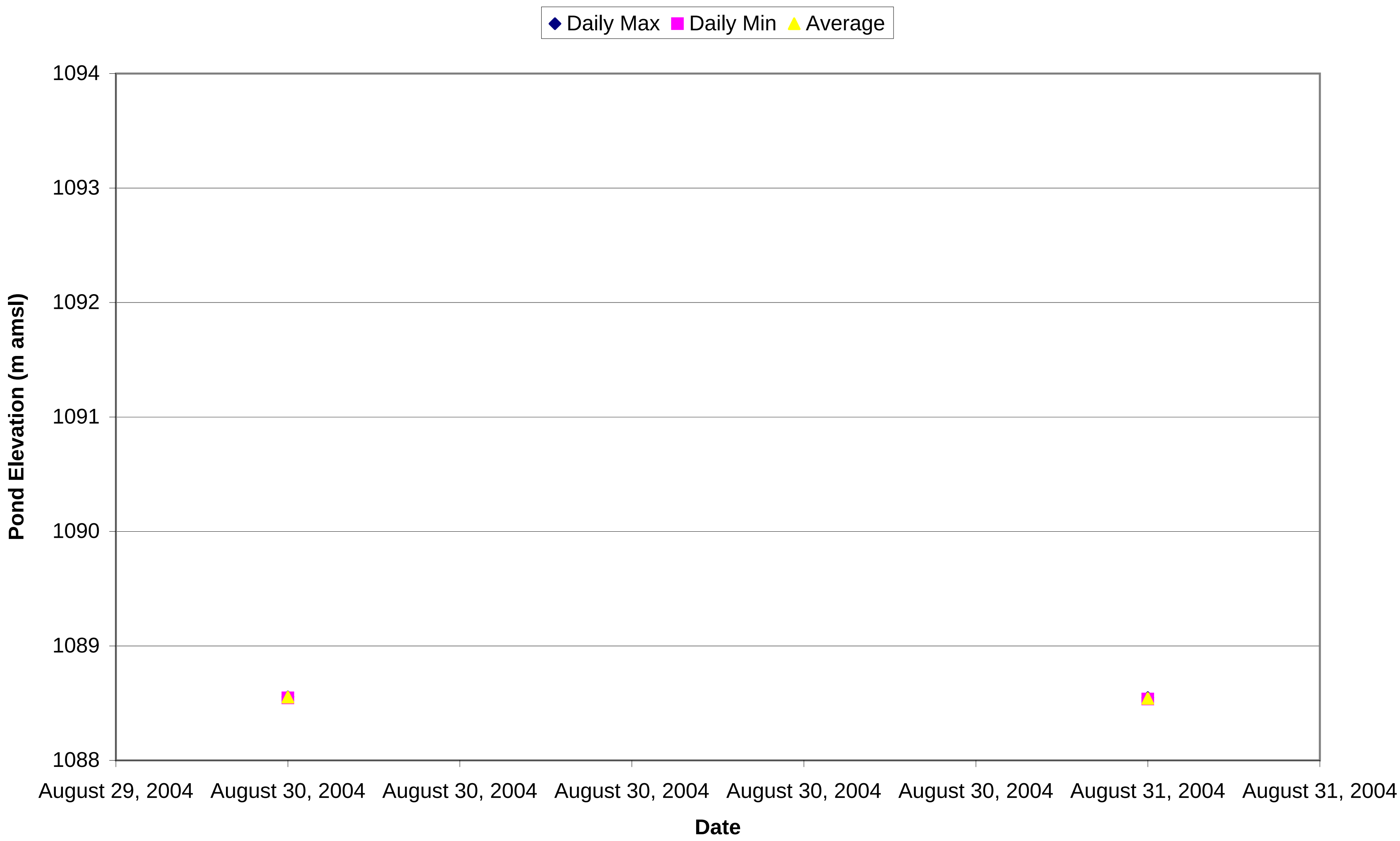


Figure III-5
Pond Elevation - September

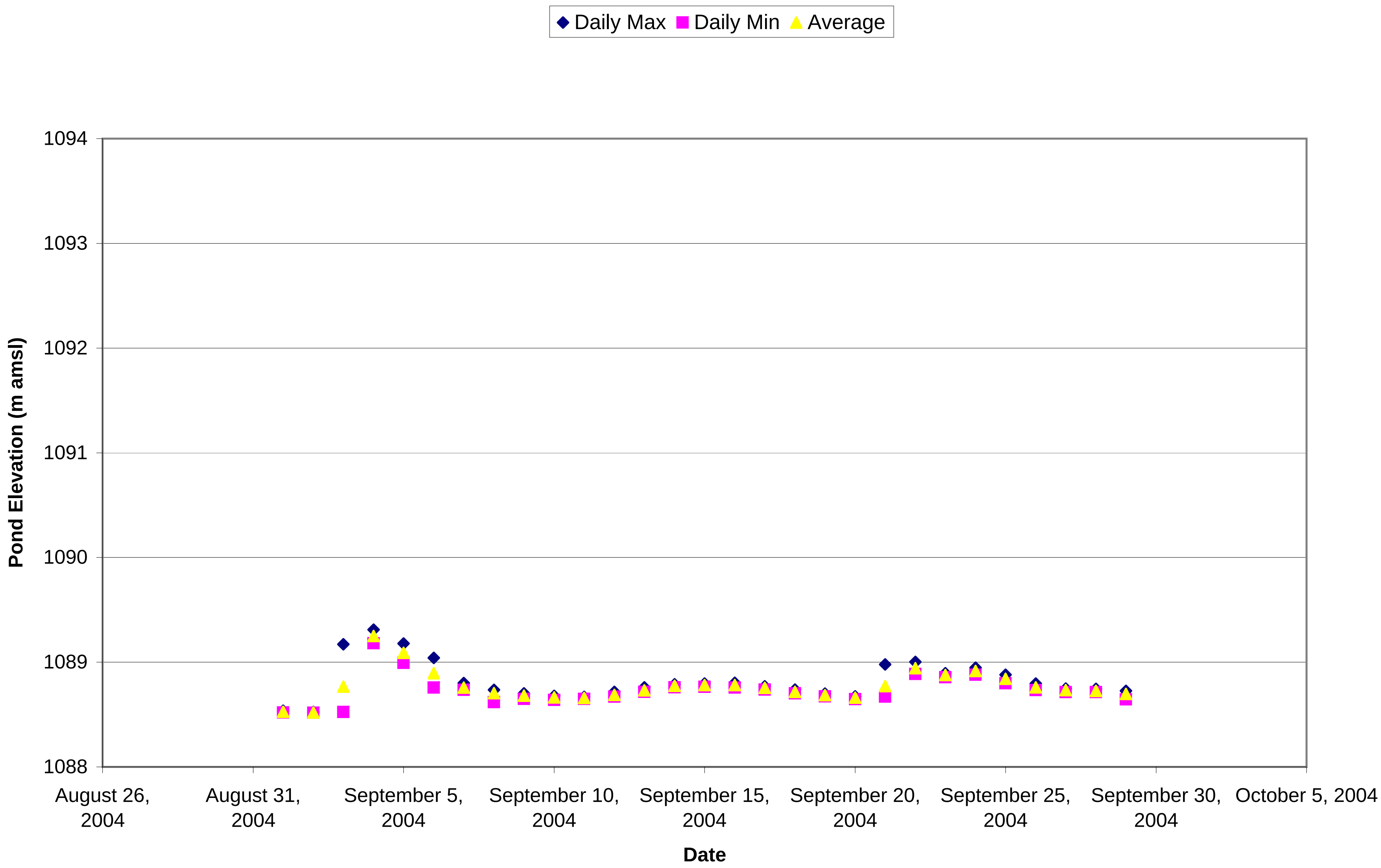
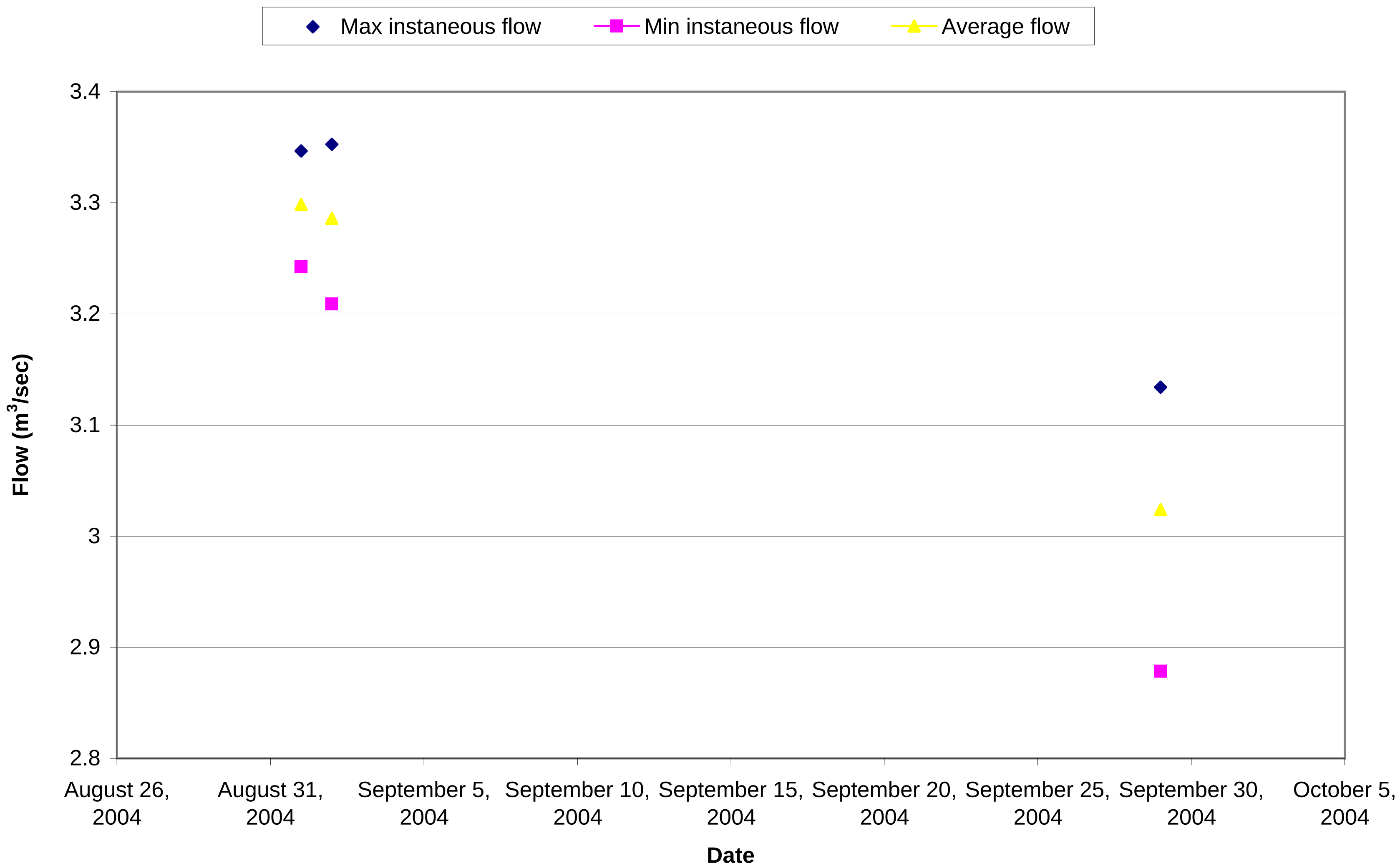


Figure III-6
August Flow at Station X2

Figure III-7
September Flow at Station X2



APPENDIX IV
HYDRAULIC MODELLING