UPPER CHASE DAM SEISMIC ASSESSMENT
NANAIMO, BC

Submitted To:

CITY OF NANAIMO
NANAIMO, BC

Prepared by:

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1.0 INTRODUCTION

1.1 General

The City of Nanaimo (the City) owns and operates a series of dams on the Upper Chase River. All of these dams have been the subject of a recent Dam Safety Review completed by Golder Associates Limited n 2004. All of these dams, including the Upper Chase River Dam, are currently classified as High Consequence dams.

One of the recommendations in the Dam Safety Review report was that a seismic analysis of the Upper Chase Dam should be conducted. Consequently, the City issued a request for proposal titled "Upper Chase River Dam – Seismic Analysis" dated June 2004. This project is solely focused on a geotechnical and structual assessment of the integrity of the Upper Chase Dam under seismic loading. Hydraulic design issues associated with the spillway capacity or flood routing were excluded from this study. EBA Engineering Consultants Ltd. (EBA) was awarded this work in early August 2004.

This report is subject to the General Terms and Conditions presented in Appendix A.

1.2 Site Description

The Upper Chase River Dam is the upper most of the Chase River Dams. It's impoundment is known as Reservoir No. 2 which is immediately upstream of Reservoir No. 1. It is believed to have been constructed about 70 years ago.

This dam is generally described as an earthfill dam which serves as the road embankment for the Nanaimo Lakes Road. An upstream concrete retaining wall is located immediately adjacent to the road for the majority of the dam length. The concrete retaining wall appears to have been constructed to act as a retaining wall for the road fill. The earthfill-concrete wall dam section is approximately 64 m long, 5 to 6 m high, generally 20 m wide and impounds about 4 m of water. The concrete wall has a series of buttresses on the upstream face over about the north half of the dam length.

A second section of the dam, consisting of earthfill only, starts at the left end of the existing concrete wall, extends over the spillway culverts and terminates approximately 5 m to the left of the concrete spillway channel. In this area the dam is approximately 2 to 3 m high, 8 to 10 m wide at the crest and does not impound water at normal reservoir levels. The spillway consists of a concrete channel that leads to two corrugated steel pipe culverts that pass beneath Nanaimo Lakes Road, located at the crest of the dam.

Discharge from the spillway passes into the Chase River, bypassing Reservoir No. 1. The flow from the Upper Chase spillway is impounded by the Middle Chase Dam. Reservoir No. 1 is impounded by a concrete gravity dam founded on bedrock. Two other dams are located downstream of the Upper Chase River Dam on the Chase River. They are the Middle Chase River Dam and the Lower Chase River Dam.

A low level conduit (450 mm diameter) was installed through the Upper Chase Dam in 1998 to provide the capacity to divert water during an emergency from Reservoir No. 2 into Reservoir No. 1. To date, it is our understanding that it has not been used.

A location plan for the Upper Chase Dam is presented in Figure 1. A detailed plan and sections of this dam based on survey data obtained from the City of Nanaimo are presented in Figure 2.

2.0 SCOPE OF WORK

The scope of work for this assignment consisted of the following activities:

- Review of background information;
- Conduct an inspection of the dam, its foundations and ancillary structures, including a limited program of concrete coring and soil testing. This work also included an underwater inspection by a diver;
- Determine the seismic criteria to be used in the analysis;
- Determine the seismic return period for the dam (the earthquake that would cause the dam to fail) and determine the appropriate maximum design earthquake (MDE); and
- If rehabilitation is required, prepare a list of recommendations and costs to upgrade the structure.

The results of each activity are presented in the following sections. The work presented herein was conducted, when appropriate, in accordance with the suggested guidelines presented in the Canadian Dam Association – Dam Safety Guidelines (CDA Guidelines,

1999) and the British Columbia Dam Safety Regulation (BCDSR) of the British Columbia Provincial Water Act.

3.0 REVIEW OF BACKGROUND INFORMATION

3.1 General

EBA reviewed the following information at the outset of this project:

- Upper Chase River Data Book, prepared by EBA Engineering Consultants Ltd., September 1992;
- Upper Chase River Dam –2003 Dam Safety Review, prepared by Golder Associates Ltd. (Golder), March 2004; and
- Operation, Maintenance and Surveillance (OMS) Manual for Chase River Dams, prepared by Golder, April, 2004.

Additionally, EBA discussed various aspects of the operation of the dam and any recent modifications with Scott Pamminger, A.Sc.T. of the City's Engineering Department. A brief discussion with Mr. George Hrabowych, P.Eng., of Herold Engineering Ltd. (Herold) was held regarding the 1998 construction of the low level conduit from Reservoir No. 2 to Reservoir No. 1 as he was with the design firm responsible for that work.

General results from the review of background information are presented in the following section. Conclusions of the likely modes of failure of the dam presented in the Dam Safety Review report are presented in a section following.

3.2 Results of Background Information Review

Key information obtained from the background information review is presented as follows:

- Reservoir No. 2 was initially a small lake that did not discharge into Reservoir No. 1 in 1911;
- Upper Chase Dam was likely constructed as a road embankment initially rather than a dam, with the concrete retaining wall being later added to buttress the road fill at the edge of the lake;

- The dam is a massive dam from the perspective of the ratio of its crest width to its height and the flat side slopes;
- Bedrock was encountered beneath the Upper Chase Dam downstream slope during construction of the low level conduit in 1998. Furthermore, the side slopes of the excavation were noted to be stable at relatively steep angles, qualitatively indicating that the fill was not excessively weak due to a loose, saturated state;
- There are water mains, storm sewers and other appurtenances present within the dam fill below the Nanaimo Lakes Road and the downstream slope of the dam. Two water mains, 200 mm and 760 mm in diameter, are present beneath the road at the dam crest. It is understood that the 760 mm diameter water main is used to supply water to all of North Nanaimo and that the 200 mm diameter watermain has been decommissioned via the installation of a shutoff valve and is no longer in use;
- Seepage has never been observed at the toe of the dam or at the abutments. Furthermore, the nature of the structure (i.e. concrete wall on bedrock or till) is such that seepage through the structure would be minimal; and
- The Golder 2003 Dam Safety Review report has recommended that the consequence classification for the Upper Chase River dam be revised from High to Low based on the incremental consequences of failure of this dam.

<u>Revision of Consequence Classification</u>

Golder's have proposed that the consequence classification of the Upper Chase Dam be revised due to the following reasons:

- The volume of water retained by the Upper Chase Dam is small (60,000 m^3);
- Structural failure of the concrete wall – earthfill section of the dam would result in release of water into Reservoir No. 1 with limited risk of overtopping;
- Rapid release of water under seismic loading or due to liquefaction is not credible;
- Seepage erosion causing failure does not appear credible; and
- Although overtopping during the design storm event appears to be credible, this storm would likely result in overtopping of the Middle and Lower Chase River Dams as well. Therefore, the incremental consequences of failure are not heightened by failure of the Upper Chase River Dam.

Incremental consequences of failure (i.e. loss of life or damage) are the consequences that would be associated solely with dam failure, excluding the loss of life or damage that would have occurred if the dam didn't fail.

An additional consequence of failure of the Upper Chase Dam is loss of, or damage to the 760 mm diameter waterline.

Implicit in this argument is that failure of the earthfill dam to the left of the concrete wall section would only be possible during an overtopping event. This is a reasonable assessment given the width and height of this section of dam and the observation that it doesn't impound water when the reservoir is at normal operating elevation.

EBA discussed the consequence classification of the Upper Chase River Dam with City engineering staff to assess whether Land and Water British Columbia (LWBC) had accepted Golder's recommendation. As a result of this conversation, EBA was directed to proceed with this work as if LWBC had accepted this recommendation and reclassified the structure as a Low Consequence dam.

Presence of Bedrock

Construction of the low level conduit in 1998 encountered bedrock extensively beneath the downstream slope of the dam. This is consistent with Golder's interpretation that the dam was initially constructed as road embankment. As the original lake did not appear to drain towards the current location of Reservoir No. 1 in 1911, it is reasonable to conclude that there must have been a ridge of higher ground between the two areas. In retrospect, it is also reasonable to conclude that the original road would have been built at least partially on this ridge with subsequent construction of the concrete retaining wall at the edge of the road fill.

Lack of Observed Seepage

Seepage has not been reported to be present at the toe of the dam at any time during its operating history. The Golder's site inspection, nor any previous inspections by City staff or EBA in 1992, did not encounter any existing, or indication of historic, seepage. The relatively un-cracked nature of the concrete wall combined with it being founded on till or bedrock would result in negligible seepage occurring through or beneath the concrete wall.

4.0 SITE INSPECTION

The site inspection methodology employed by EBA incorporated the following activities:

- A preliminary site inspection and kick off meeting held on August 24, 2004 between Mssrs Chris Gräpel, P.Eng. and Bob Patrick, P.Eng. of EBA and Mssrs Wayne Hansen, A.Sc.T. and Scott Pamminger, A.Sc.T. of the City Engineering department;
- Detailed site inspection by Mr. Gräpel on September 7, 2004 which included a brief inspection by Mr. Mike Herold, P.Eng. of Herold Engineering Ltd. (Herold), the structural engineering firm selected to assess the structural stability of the concrete retaining wall;
- A limited concrete coring and hand excavated test pitting program conducted on September 9 and 17, 2004 to assess the condition of the concrete in the retaining wall and to confirm the geometry of the concrete wall; and
- A diving inspection conducted on September 16, 2004 to assess the condition of the wall below the reservoir level.

The observations made during the detailed site inspection conducted on September 7, 2004 are presented in Table B.1 in Appendix B. The results of the limited concrete coring and hand excavated test pitting program are presented in a memo presented in Appendix B.

Key observations from the site reconnaissance are presented as follows:

- There was approximately 2.3 m of freeboard below the top of the concrete wall;
- A beaver dam was present at the inlet to the spillway and had raised the No. 2 reservoir level by approximately 0.3 m at the time of our inspection. Thus, the available freeboard during normal operating conditions would be about 2.6 m upon regular removal of beaver dams at this location;
- The concrete wall appears to be in good condition with the exception of some limited surficial deterioration of the concrete;

- Buttresses are only present on the upstream face of the concrete wall and then only in the highest section of the wall between 39 m from the right abutment and 10 m from the left abutment;
- The buttresses extend about 1 to 1.3 m past the upstream face of the dam;
- One crack is present between two buttresses at the right abutment;
- The wall does not appear to have a footing that extends past the upstream wall face;
- The wall appears to be founded on till; however, due to sediment cover, only probing with a mallet and hammer by our diver could be conducted;
- Seepage was not observed at the downstream toe or abutment areas;
- There were no signs of instability, settlement or any other movement that would indicate shallow or deep-seated movements had occurred in the past or were currently underway; and
- There is a large tree growing at the right abutment of the dam. This tree should be removed.

The site inspection included a brief visit to the Reservoir No. 1 dam. There was approximately 1.5 m of freeboard at this time.

The Upper Chase Dam and appurtenances and Reservoir No. 1 and Dam at the time of our inspections are depicted in Photos 1 through 12.

5.0 SEISMIC CRITERIA

The seismic loading criteria for a dam are a function of its consequence classification. The consequence classification of a dam is based on the incremental consequences of failure. As previously discussed in Section 3.0, the work presented herein has been completed as if the Upper Chase Dam was re-classified as a Low Consequence dam.

A Low Consequence dam must be capable of resisting the destabilizing forces that are applied during an earthquake that has a probabilistically determined return period varying between 1:100 years and 1:1,000 years.

EBA commissioned the preparation of a probabilistic assessment of earthquake ground motions in the vicinity of the Upper Chase River Dam by the Pacific Geoscience Centre in Sydney, BC as part of this study. The results of this work are attached in Appendix C. The results of this assessment indicated that 1:100 and 1:1,000 seismic events would have peak particle accelerations of 0.092 g and 0.311 g respectively.

A reasonable design earthquake for the Upper Chase River Dam would be an earthquake that had a return period of about 1:300, approximately mid-way (on a logarithmic scale) between the two extremes. The peak particle accelerations for an earthquake of this return period would be approximately 0.16 g, based on linear interpolation between the data provided by the Pacific Geoscience Centre.

6.0 STABILITY ANALYSIS

6.1 General

Stability analyses were conducted on the following portions of the Upper Chase Dam:

- Upstream concrete retaining wall;
- Downstream slope of earthfill – concrete wall section of dam; and
- Upstream face/slope of earthfill – concrete wall section of dam.

The earthfill section in the vicinity of the spillway does not normally impound water and is low and wide. As such, the only geotechnical issue facing this structure is settlement during a seismic event due to consolidation of loose fill.

Each phase of stability analysis is discussed in the following sections.

6.2 Retaining Wall Stability Analysis

EBA commissioned Herold to conduct a structural stability analysis of the concrete retaining wall during static conditions and a seismic event. The results of Herold's assessment were that the concrete wall is barely stable when water is impounded against the upstream face of the wall to normal operating level. Additionally, the analysis conducted by Herold indicates that the wall will topple during a seismic event. The historic stability of the wall requires additional consideration of this analysis as the performance of the wall indicates it has at least been marginally stable for the past 70 years, including original construction when reservoir levels were likely at or below the base of the wall. Herold's letter is attached in Appendix D.

It is not uncommon in geotechnical engineering to have to reconsider a stability analysis of a stable or failed slope when the analysis produces counter-intuitive results (i.e. a stable result for a failing slope). The stability of a slope generally depends on three factors which are listed as follows:

- Geometry;
- Shear strength of soils; and
- Pore pressures within the soils

In this case, the greatest unknown with regards to the stability of the wall is the geometry.

The wall was stable during the 1946 earthquake which is reported in the Dam Safety Review report to have applied horizontal accelerations of about 0.03 g to the wall. Although these are a fraction of the design accelerations (0.16 g) associated with a 1:300 year event, the wall would have experienced some damage due to flexure or movement during the earthquake if it was marginally stable with regards to overturning or sliding under static loading conditions. Finally, it is understood that the reservoir level was partially lowered to permit construction of the low level conduit in 1998. There are no reports of the wall exhibiting any instability or movement at that time either.

Static analyses conducted by EBA using Rankine earth pressure theory modified with the effects of soil-wall friction confirms that the wall has a factor of safety of 1.0 to 1.1 against overturning for the "just constructed" case when no water is present on the upstream face of the wall. A sliding resistance calculation for this same condition indicates that the wall is unstable with a factor of safety of 0.7. The historic performance of the wall clearly indicates that it is at least marginally stable with regards to sliding; therefore, there must be an additional shear resistance other than friction on the base of the wall and buttresses. However, a seismic analysis conducted by EBA using the Mononobe-Okabe method to estimate the effect of seismicity on earth pressures indicates that the wall is unstable during a 1:300 year seismic event.

The results of the analysis conducted by EBA indicates that it is likely that there is some other structural feature of the concrete retaining wall that adds to it's stability. This could be a structural connection to bedrock through anchor bolts or dowels, a cantilever section behind the wall, or a shear key. However, in the absence of information to confirm the existence of these measures and the recognized uncertainties that would still remain upon completion of an investigation of them, it is our opinion that the wall should be considered marginally stable (factor of safety of 1.0 to 1.1) in it's current configuration for static loading conditions and unstable during seismic loading from a 1:300 year seismic event. These factors of safety do not meet the CDA or BCDSR requirements for factors of safety against instability for a dam. Intuitively, it is reasonable to conclude that the wall is marginally stable under static loading conditions.

Failure of the concrete retaining wall during a seismic event does not mean concurrent loss of reservoir containment as the asphalt surfacing of the Nanaimo Lakes Road is approximately 1.3 m above the observed reservoir level at the time of our inspection. The zone of soil behind the wall that would fail as a result of retaining wall failure would be theoretically bounded by the active angle (60° from horizontal) extending from the downstream toe of the wall. However, localized saturation of soil behind the wall due to limited seepage through the wall and foundation before the seismic event or continued ground shaking after wall failure may result in deformation of fill beyond this theoretical limit.

Loss of the concrete wall as a seepage barrier would lead to increased seepage with corresponding decrease in factor of safety against slope failure for the upstream and downstream sides of the dam. It would not lead to instability of the downstream slope unless there was a subsequent second seismic event that triggered liquefaction of the fill in the dam after steady state seepage had been initiated upon toppling of the wall. This could result in additional ground movement that could further limit access across the dam and possibly loss of backfill support for the 760 mm waterline if not addressed after the seismic event. However, given that the 200 mm water line has been decommissioned and will not pose a threat the remainder of the embankment upon rupture, the probability of the 760 mm diameter water line being impacted is considered low

6.3 Stability Analysis of Earthfill Section of Dam

6.3.1 General

The stability of the earthfill portion of the Upper Chase Dam was assessed for the earthfill-concrete wall portion of the dam. We carefully considered the information obtained from the Dam Safety Review report and our site inspection before starting the stability analysis of the dam. Key considerations in our assessment of the structure included the potential presence of bedrock below the downstream shell and the lack of observed seepage at the downstream toe of the Upper Chase Dam since before 1992. Each are discussed further in the following sections.

6.3.2 Soil Conditions and Strength Parameters

The review of background information discussed in Section 3.0 indicates that a ridge of bedrock and/or soil likely exists between Reservoir No. 1 and Reservoir No. 2. The presence of such a ridge would serve to stabilize the entire dam from the perspective of deep-seated downstream slope instability. However, the extent of the bedrock is only known in the vicinity of the low level conduit as presented on Figure 2. It is not prudent to assume that this bedrock ridge is continuous across the downstream slope of the dam given the paucity of subsurface information for this site; therefore, EBA has assumed that bedrock only present at the base of the dam for the purposes of this analysis. This is a potentially conservative assumption as the confirmed presence of this ridge would negate any currently perceived need to analyze the stability of the downstream slope of the dam. Furthermore, the bedrock ridge, if continuously present beneath the downstream shell of the dam at the elevation encountered in 1998, would act as a natural dam for almost all of the water impounded in Reservoir No. 2 under normal operating conditions. This also would account for the lack of observed seepage at the downstream toe.

The density and type of fill that is present within the Upper Chase Dam is unknown. We have assumed that the fill placed during the original construction of the dam was predominantly granular in nature. Furthermore, based on our experience with aging dams on Vancouver Island, it is not unreasonable to assume that the fill originally placed within the road fill about 70 years ago received only nominal compaction. This assessment is based on our experience in assessing the strength of fill in aging earth dams of similar vintage on Vancouver Island.

The backfill placed around the water mains and storm sewer present within the dam fill would have received some compaction. However, it is our understanding records of compaction testing either do not exist or are not available for our review. Furthermore, the waterline backfill does not appear to extend to the base of the dam so it will provide limited benefit in providing additional strength with respect to the original, underlying loose fill.

Based on the preceding assessment, for the purposes of this study, the fill within the Upper Chase Dam was judged to have a friction angle of 30° and an in-situ density of 20 kN/m^3.

6.3.3 Water Table and Seepage

The background information reviewed at the outset of this work has indicated that seepage has never been observed at the toe or abutments of the Upper Chase River Dam. Inspection of the upstream concrete retaining wall has indicated that it is likely founded on till and that there appears to be only one crack in the wall. The integrity and massive nature of the concrete wall indicate that it is relatively impermeable. As such, minimal seepage is anticipated through the concrete wall into the fill. Some limited seepage could occur through the till, but only if it was predominantly granular with limited fines content. Therefore, it is unlikely that there is a water table within the structure of sufficient magnitude to permit liquefaction during or after a seismic event. The seepage that does occur is likely discontinuous in the areas of the crack observed during our inspection and the low level conduit constructed in 1998.

6.3.4 Method of Slope Stability Analysis

Slope stability analysis was conducted using limit equilibrium analysis methods. Limit equilibrium analysis uses the principles of statics to evaluate the stability of a dam or slope. The stability of a dam is represented by a factor of safety which is the ratio of the stabilizing forces to the destabilizing forces for any given failure surface geometry. Limit equilibrium analysis uses a highly iterative process through which many candidate slip surfaces are analyzed to identify the slip surface geometry with the lowest factor of safety. Limit equilibrium analysis can be used to provide a seismic assessment for dams by applying a peak horizontal acceleration within the slope stability model. The use of a constant horizontal acceleration to simulate earthquake loading is known as the pseudo-static method of seismic analysis. EBA used the commercially available slope stability modelling software SLOPE/W to assess the stability of the dam under static and seismic loading conditions.

A seismic analysis using the pseudo-static method of analysis typically entails the following activities:

- Analysis of the dam during the earthquake – this is conducted by applying a peak horizontal acceleration to the dam; and
- Analysis of the dam after the earthquake – this is conducted by using liquefied soil strengths where appropriate.

All information available to EBA for this study indicates that there is a very low likelihood of seepage being present in sufficient quantities to cause liquefaction of the dam fill. As such, analysis to assess the effects of liquefaction were judged to be inappropriate for this structure.

A summary of the soil groundwater and seismic input parameters and information used in the stability analysis is presented in Table 1.

Table 1 – Input Parameters for Stability Analysis

Unit Weight	20 kN/m^3
Friction Angle	30°
Groundwater	Dry
Liquified Strength	Not applicable
Design Seismic Event	1:300 years
Peak horizontal acceleration during design seismic event	0.16 g

6.3.5 Results of Earthfill Section Stability Analysis

The stability analysis of the earthfill portion of the Upper Chase Dam was conducted in two phases which are as follows:

- Downstream slope; and
- Upstream face after the retaining wall has toppled during a seismic event.

Each phase of analysis is discussed in the following paragraphs. A summary discussion of the stability analyses presented herein is also included in this section.

Downstream Slope

The results of the downstream slope stability analysis are presented in Figure 4 and in Table 2.

Table 2 – Results of Downstream Slope Stability Analysis

Run #	Factor of Safety	Required Factor of Safety (CDA Guidelines, 1999)	Comment
1	4.8	1.5	Static, no seepage
2	2.2	1.1*	During seismic, no seepage

*A Factor of Safety of 1.1 for seismic and post seismic stability is judged to be appropriate based on EBA's experience in western Canada.

All of the calculated factors of safety exceed the requirements of the CDA Guidelines for slope stability. The downstream failure geometries presented in Figure 4 were selected because they impacted the road surface and the 760 mm diameter pipeline.

Based on a pseudo-static back analysis, the seismic event that could cause the downstream failure surface presented herein to reach the point of failure (i.e. a factor of safety of 0.95) would be one that applied a peak horizontal acceleration of 0.5 g to the dam. This event corresponds to a 1:2,200 (approximately) event according to the information provided to us by the Pacific Geoscience Centre. It is important to note that this failure will not cause an uncontrolled discharge of water from the No. 2 Reservoir.

Upstream Slope (Post Retaining Wall Failure)

The results of the upstream slope stability analysis after toppling of the retaining wall are presented in Figure 4 and Table 3.

Table 3 – Results of Upstream Slope Stability Analysis

Run #	Factor of Safety	Required Factor of Safety (CDA Guidelines, 1999)	Comment
3	0.8	1.5	Static, no seepage
4	2.4	1.5	Static, no seepage, slip surface extends to location of 760 mm waterline
5	1.6	1.5	Seismic, no seepage, slip surface extends to location of 760 mm waterline

Failure of the concrete retaining wall will not immediately affect the 760 mm diameter waterline as illustrated in Figure 4. However, saturation of the dam fill upon failure of the concrete wall will result additional movement under static conditions and liquefaction with additional ground movement if further seismic activity (i.e. aftershocks) persists after the initial event. This combination of post seismic events could result in the 760 mm diameter waterline being impacted by ground movement related to retaining wall failure. However, the probability of this is judged to be relatively low.

Summary Discussion

Based on the results of the stability analyses presented herein, EBA concludes that the downstream shell of the dam is stable under static and seismic loading conditions. Although the analyses indicates that the upstream retaining wall will be unstable during a seismic event and that upon its failure, soil behind the wall will likely move, there is no indication that an uncontrolled discharge of reservoir water will be initiated by a 1:300 year seismic event.

The greatest concern with regards to the dam's integrity as a water retention structure is the potential damage that could occur to the 760 mm diameter waterlines that are located beneath the Nanaimo Lakes Road at the crest of the dam. There is a significant possibility that the 200 mm diameter water line will be impacted by failure of the wall, but the result of this is inconsequential aside from pipe damage as it has been decommissioned. There is less concern for the integrity of the 760 mm diameter waterline. However, if ruptured, this waterline could cause complete erosion of dam fill and cause an uncontrolled discharge of Reservoir No. 2 due to the erosion potential associated with the anticipated volume and pressure of water that could be released.

6.3.6 Anticipated Seismic Settlement

Judgment based estimates of settlement are discussed in this section as there is very little data available with which to compute settlements. Seismic settlements of loose dry fill such as that anticipated to be present within the Upper Chase Dam are anticipated to be in the within the range of 0 to 5% of the fill height. This corresponds to settlements varying from about 0.0 to 0.4 m for the earthfill-concrete wall section (height 7.1 m) and about 0.0 to 0.15 m for the earthfill only section (height 2.5 m) in the vicinity of the spillway. These ranges may lead to significant differential settlement of the 760 mm diameter waterlines, even without failure of the upstream concrete retaining wall. As such, the integrity of the 760 mm diameter waterline should be tested immediately after a seismic event to prevent damage or washout of the dam and road fill. The 200 mm diameter waterline need not be tested as it has been decommissioned. However, the settlements the waterlines will experience will be less than that at the crest of the structure.

7.0 RECOMMENDATIONS

Based the results of the seismic assessment of Upper Chase Dam presented herein, EBA concludes that the structural integrity of the dam is sufficient to maintain impoundment of the No. 2 reservoir under static and seismic loading conditions. However, a seismic event will likely result in toppling of the majority of the concrete retaining wall on the upstream side of the dam with corresponding lateral movement of the soil immediately behind the wall. These soil deformations would have a low probability of impacting the 760 mm diameter waterline.

EBA recommends that the City consider one of the following courses of action

- Adopt a risk management approach based on the massive nature and Low Consequences of failure of the Upper Chase Dam and accept that repair work will be required immediately after the significant seismic event; or
- Buttress the concrete retaining wall with a rock fill berm to improve its stability to meet the requirements of the CDA Guidelines and BCDSR under static and seismic loading conditions.

Additionally, the City could investigate the extent of bedrock beneath the downstream shell to assess whether or not the dam can be declassified from dam status. This work would relieve the City of having to rehabilitate the concrete wall to the requirements of the BCDSR and the CDA Guidelines. However, this does not alleviate the concerns relating to the post seismic event integrity of the 760 mm diameter waterline within the dam.

Each course of action is discussed further in the following paragraphs.

Conduct Seismic Rehabilitation on the Concrete Retaining Wall

The Upper Chase Dam will require rehabilitative work to improve the stability of the concrete retaining wall on upstream side of the structure. The purpose of this work would be primarily to stabilize the fill behind the wall and keep the road open after the design seismic event. The most economical method of rehabilitation is to buttress the wall with a rock fill berm founded on till. This is the most appropriate and economical method that uses readily available material and locally available resources. Other measures, such as anchoring, were considered but were not favoured due to the reliance on resources/services not readily available in Nanaimo. Another option to stabilize the wall with a smaller environmental impact than the rockfill berm would be to drive battered steel piles into the reservoir bed and connect the pile heads to the concrete wall. However, this approach would require knowledge of soil conditions and depth to bedrock.

The concrete retaining wall should be buttressed with coarse, angular, free draining rockfill founded on till to improve the static and seismic stability of the retaining wall. The rockfill buttress would have a 1:1 slope with a 1.5 m wide crest width at or just below normal reservoir elevation. The proposed geometry would result in a static factor of safety of 1.5 and a seismic factor of safety of 1.1.

The rockfill should have a maximum particle size of 0.3 m. The rockfill buttress could be constructed around the intake for the low level conduit. The foundation of the berm would need to be prepared through removal of sediments to expose dense till or bedrock. This may involve lowering the reservoir level. The rockfill buttress should be placed in lifts not exceeding 0.5 m and be compacted with a vibratory plate tamper attached to a large excavator before placement of the next lift of rockfill.

A conceptual design of the proposed rockfill berm is presented in Figure 5. The estimated cost of construction of this berm is approximately $20,000, excluding any environmental work that the City may require. A rough order of magnitude for environmental work that may be required would be between $7,000 to $15,000, depending on whether a fish habitat study would be required or not.

The proposed rockfill berm will not eliminate the need to check the post-seismic event integrity of the waterlines due to the potential for seismically induced settlements.

<u>Adopt a Risk Management Approach With Regards to Wall Failure</u>

The City could chose to adopt a risk management strategy with regards to toppling of the concrete wall. As this will not immediately cause failure of the rest of the structure, the City could chose to mobilize repair personnel to site to shut down both waterlines, assess if their integrity has been compromised, make the necessary repairs and recommission them. Failure of the concrete wall would require that another seepage barrier be constructed on the upstream side of the dam. Candidate seepage barriers would include either an upstream clay core, a reinforced concrete cantilever retaining wall or a massive fillcrete slurry wall encased in till, all founded on dense, impermeable till or bedrock.

It will be important to complete the post-seismic event repairs to the Upper Chase Dam in a timely manner as seepage will result in the development of a water table within the remaining dam fill. Saturated loose fill in the dam would be susceptible to liquefaction during any subsequent seismic events or after shocks. This would have the potential to cause downstream slope deformation and additional upstream slope deformations that could further compromise the integrity of the waterlines.

The City will need to evaluate what other potential emergencies it may have to address in the aftermath of a significant seismic event and compare them to the anticipated repair work required at Upper Chase Dam. Through this comparison the City will be able to assess whether a risk management approach is appropriate of not.

Investigate if the Dam Can be Declassified from Dam Status

The bedrock outcrop ridge encountered during the 1998 low level conduit construction may extend across the entire downstream shell from the left abutment to the right abutment. The elevation of the top of this outcrop was approximately the same as the normal operating level of the No. 2 reservoir. Therefore, if this ridge was continuous near this elevation across the length of the dam, the volume of water impounded by the road fill would become insignificant and the dam could become a candidate for declassification.

The presence of bedrock could be confirmed through a drilling program using a locally available air track drill rig. A private utility location survey would be necessary to avoid the buried pipes (sprinkler system, low level conduit and others) that are present across the downstream slope of the dam. A line of approximate 20 boreholes spaced 3 m apart could be drilled to depths of about 3 to 4 m in about one eight hour day. All boreholes would be grout backfilled. This program would confirm if bedrock is present below the downstream shell of the dam and would be the first indication of whether there is merit in pursuing the declassification option.

The estimated cost of this investigation (including drill rig, monitoring and report) is $5,000 excluding GST. The City, with guidance and assistance from EBA, could lead discussions with LWBC on the mechanism for declassifying the dam if the drilling indicates the bedrock ridge is present beneath the downstream shell at an elevation similar to that encountered near the low level conduit.

An alternate, non-intrusive means of investigation would be a ground penetrating radar survey. The use of geophysical methods to investigate for bedrock depth would eliminate the possibility of damaging any buried services located beneath the downstream shell. EBA could conduct the ground penetrating radar survey using personnel from our Edmonton office for a cost of $7,000, excluding GST.

<u>Summary List of Recommendations</u>

A summary of the recommendations made herein are listed as follows:

- Remove large tree on the right abutment of the dam concrete wall;
- Conduct an integrity test on the 760 mm diameter waterline after a seismic event to assess if it has been damaged and leaking by ground motions; and
- Consider one of the following courses of action with regard to the stability of the retaining wall:
    - Adopt a risk management strategy and accept that repair work will be required immediately after a significant seismic event; or
    - Conduct seismic rehabilitation/stabilization works on the wall; or
    - Investigations of the dam could be declassified from dam status with LWBC.

8.0 CLOSURE

We trust that this report meets with your current requirements. Please do not hesitate to contact the undersigned should you have any questions or comments.

EBA ENGINEERING CONSULTANTS LTD.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA
C. K. GRAPEL
26096
May 30/2005

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA
ROBERT A. PATRICK
June 03/05

Chris Gräpel, M.Eng., P.Eng.
Senior Geotechnical Engineer
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Bob Patrick, M.Sc., P.Eng.
Principal Engineer – Geotechnical Practice
Phone #: (250) 756-2256
E-mail: bpatrick@eba.ca

FIGURES

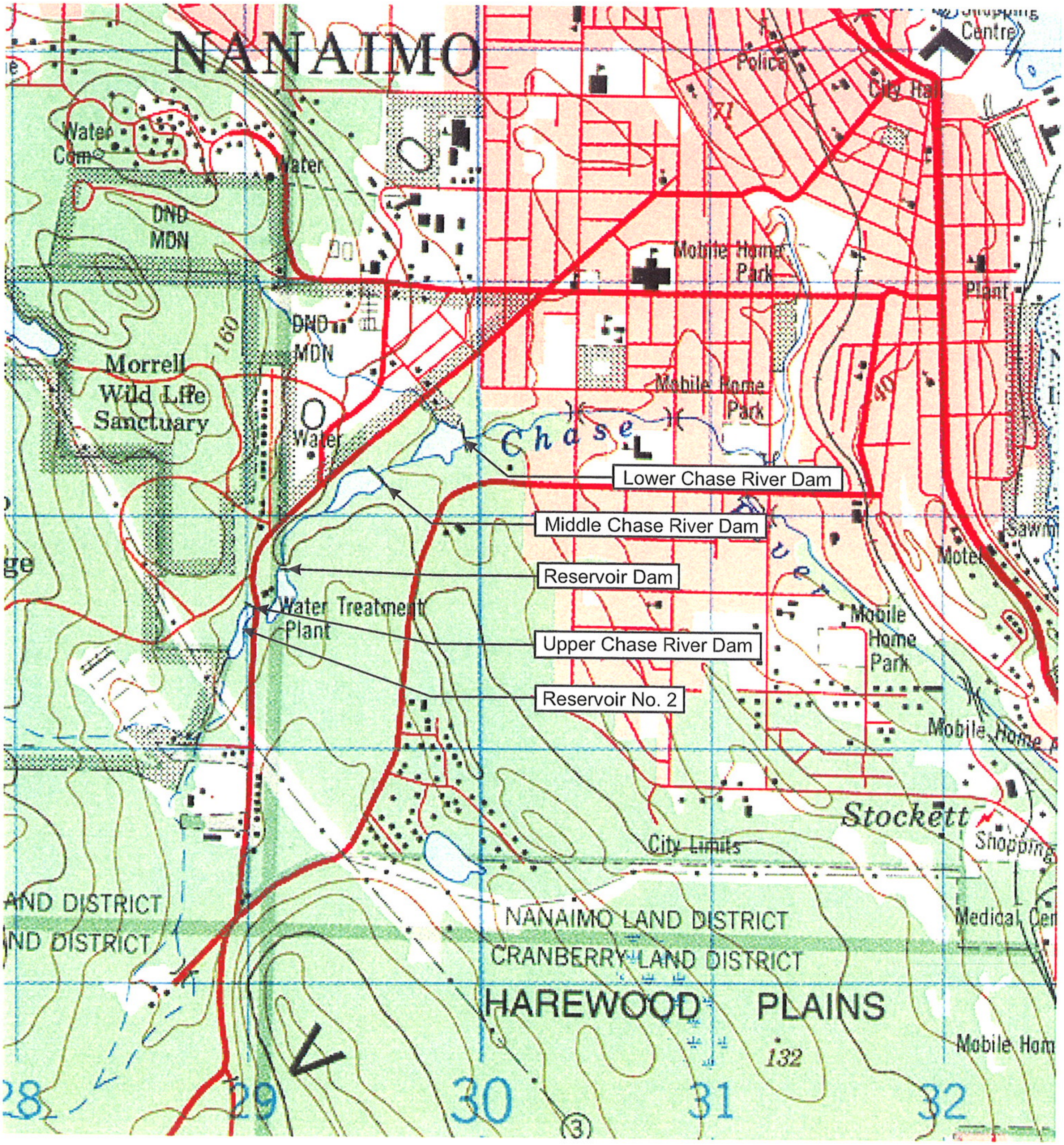


Figure 1

Site Location Plan

2800097q03b.cdr

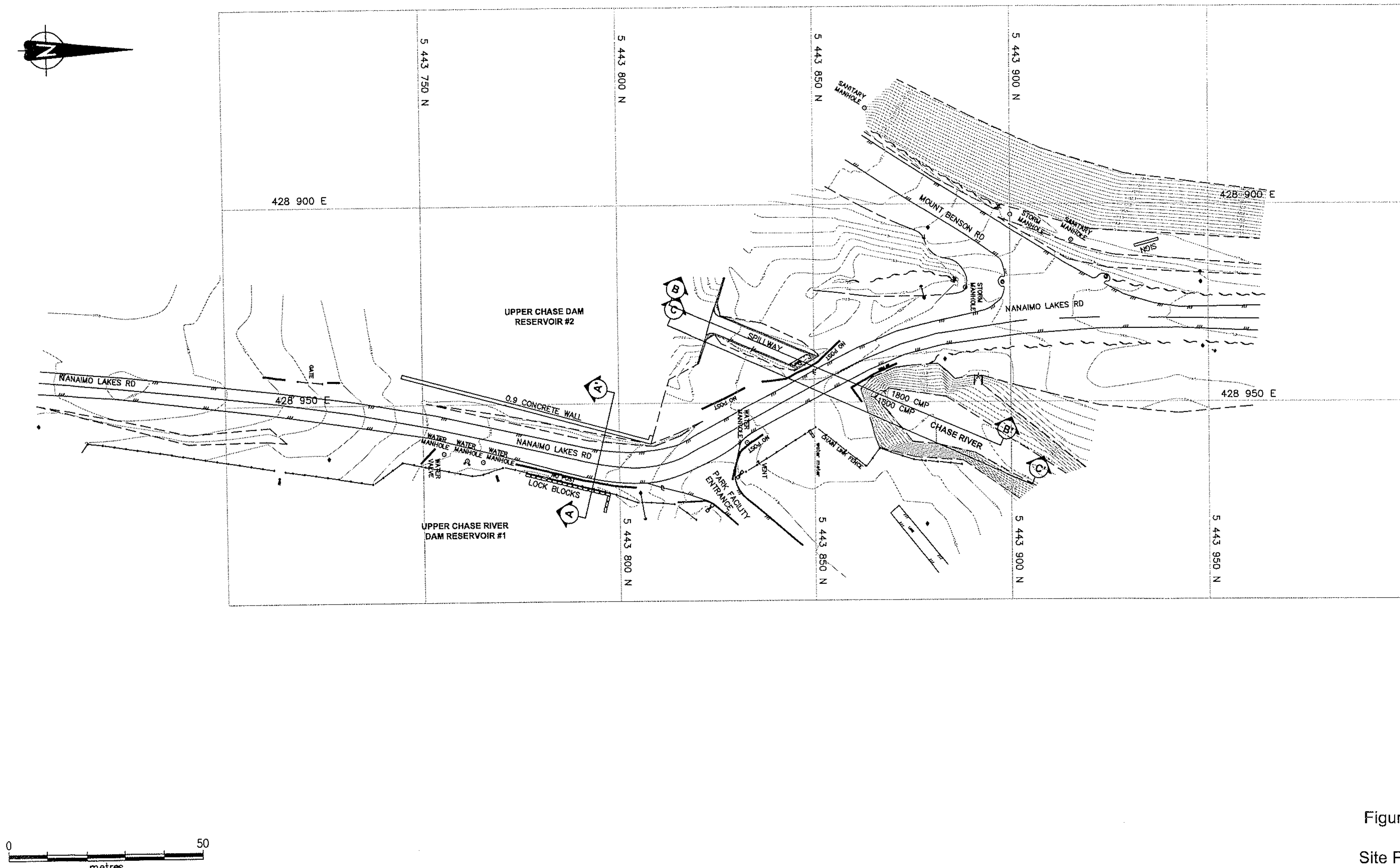
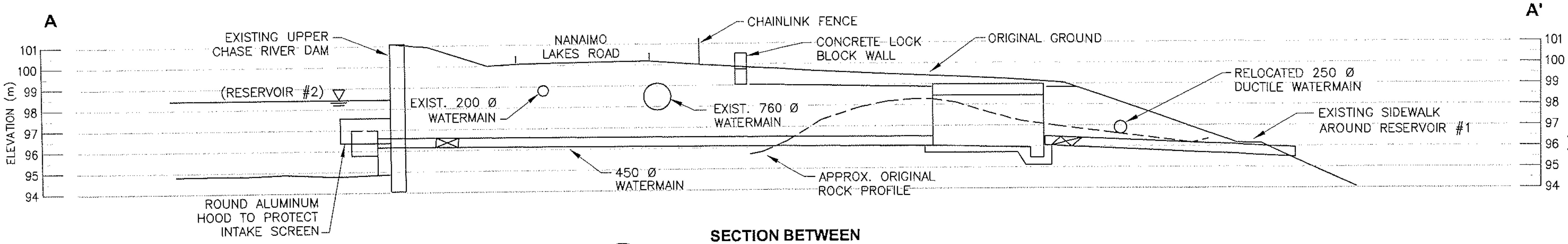


Figure 2

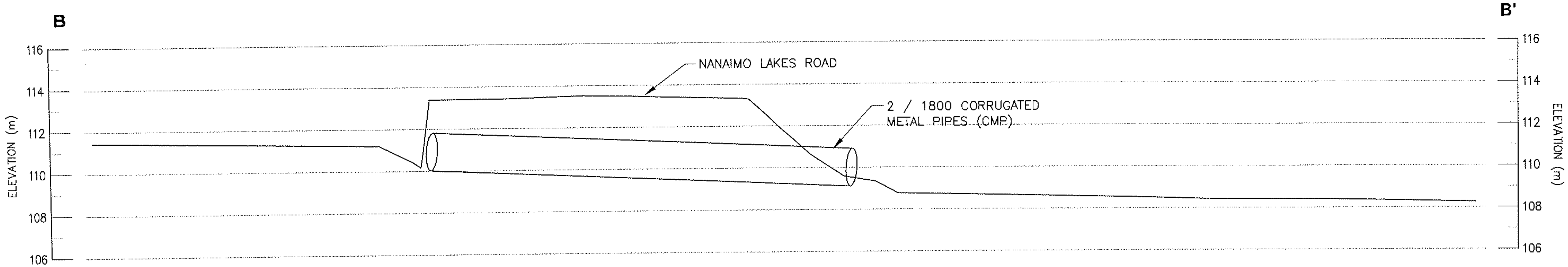
Site Plan

2800097Q01c.dwg



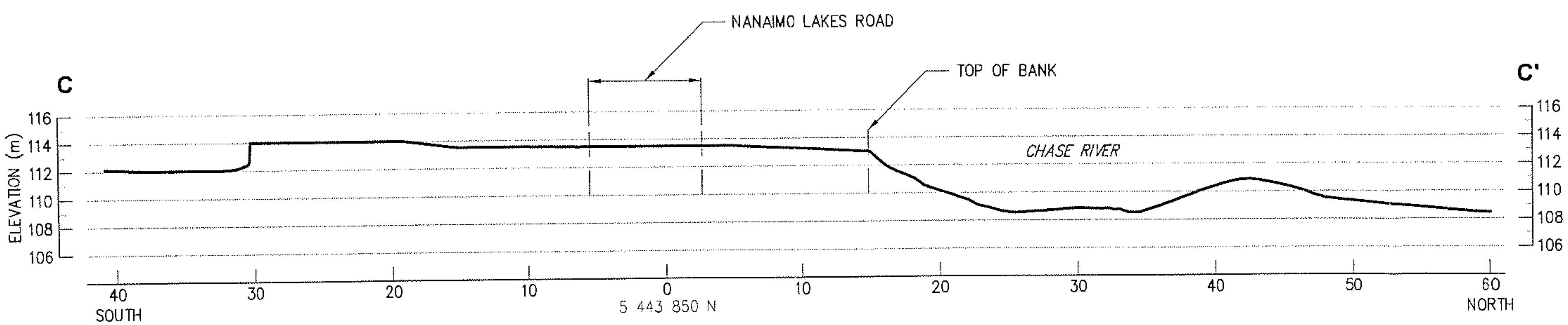
A SECTION BETWEEN RESERVOIR #1 AND RESERVOIR #2

0 2 4 6 8 10
Scale in Meters



B SPILLWAY PROFILE

0 2 4 6 8 10
Scale in Meters



C SPILLWAY PROFILE

0 4 8 12 16 20
Scale in Meters

Figure 3

Cross-Sections

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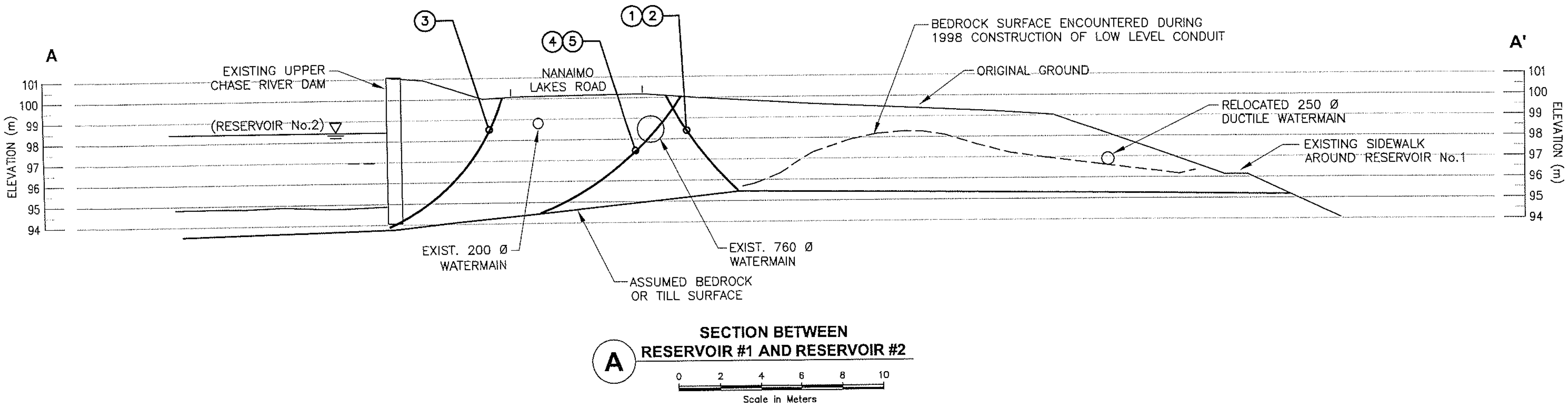


Figure 4

Stability Analysis

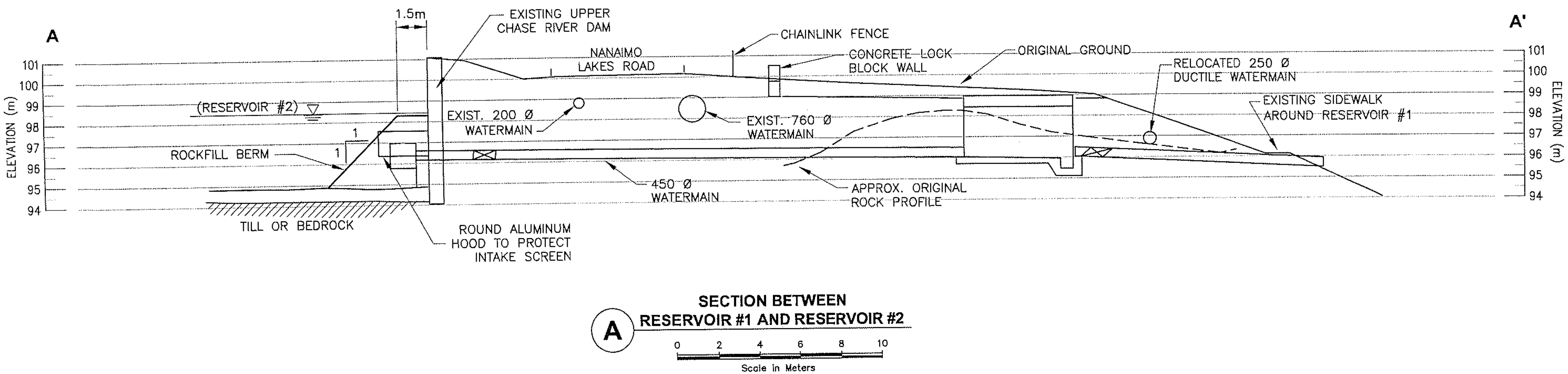


Figure 5

Proposed Rockfill Berm

2800097d03b.dwg

PHOTOS

Photo 1
Upper Chase River Dam, facing south.

Photo 2
Upper Chase River Dam, facing north. Reservoir No. 1 in rear right of photo.

Photo 3
Concrete wall on upstream of Upper Chase River Dam, facing south. Note buttresses on upstream side.

Photo 4
Concrete wall on upstream side of Upper Chase River Dam. Spillway inlet at far left of photo (see yellow arrow), facing north.

Photo 5
Downstream slope of Upper Chase River Dam, facing north.

Photo 6
Downstream slope of Upper Chase River Dam, facing south. Structure in foreground is a concrete banker founded on bedrock at approximate location of bedrock ridge encountered in 1998. Edge of Reservoir No. 1 at bottom left corner of photo.

Photo 7
Downstream toe of Upper Chase River Dam at edge of Reservoir No. 1 , facing northeast. Note no evidence of seepage.

Photo 8
Inlet to spillway, facing southwest. Note beaver dam at inlet.

Photo 9
Spillway channel to twin culverts under Nanaimo Lakes Road, facing northeast.

Photo 10
Downstream slope of Upper Chase Dam from control building for Reservoir No. 1, facing southwest. Water jets are from pipeline from South Fork Dam.

Photo 11
Upstream face of Reservoir No. 1 dam, facing east.

Photo 12
Reservoir No.1 dam, facing south.

APPENDIX A

GENERAL CONDITIONS

This report incorporates and is subject to these "General Conditions".

1.0 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site, a specific development and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for types of development other than that to which it refers. Any variation from the site or development would necessitate a supplementary geotechnical assessment.

This report and the recommendations contained in it are intended for the sole use of EBA's client. EBA does not accept any responsibility for the accuracy of any of the data, the analyses or the recommendations contained or referenced in the report when the report is used or relied upon by any party other than EBA's client unless otherwise authorized in writing by EBA. Any unauthorized use of the report is at the sole risk of the user.

This report is subject to copyright and shall not be reproduced either wholly or in part without the prior, written permission of EBA. Additional copies of the report, if required, may be obtained upon request.

2.0 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

3.0 LOGS OF TEST HOLES

The test hole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

4.0 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historic environment. EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional investigation and review may be necessary.

5.0 SURFACE WATER AND GROUNDWATER CONDITIONS

Surface and groundwater conditions mentioned in this report are those observed at the times recorded in the report. These conditions vary with geological detail between observation sites; annual, seasonal and special meteorologic conditions; and with development activity. Interpretation of water conditions from observations and records is judgmental and constitutes an evaluation of circumstances as influenced by geology, meteorology and development activity. Deviations from these observations may occur during the course of development activities.

6.0 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

7.0 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

8.0 INFLUENCE OF CONSTRUCTION ACTIVITY

There is a direct correlation between construction activity and structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques are known.

9.0 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, as well as the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

10.0 DRAINAGE SYSTEMS

Where temporary or permanent drainage systems are installed within or around a structure, the systems which will be installed must protect the structure from loss of ground due to internal erosion and must be designed so as to assure continued performance of the drains. Specific design detail of such systems should be developed or reviewed by the geotechnical engineer. Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function.

11.0 BEARING CAPACITY

Design bearing capacities, loads and allowable stresses quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition assumed. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions assumed in this report in fact exist at the site.

12.0 SAMPLES

EBA will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the client's expense upon written request, otherwise samples will be discarded.

13.0 STANDARD OF CARE

Services performed by EBA for this report have been conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practising under similar conditions in the jurisdiction in which the services are provided. Engineering judgement has been applied in developing the conclusions and/or recommendations provided in this report. No warranty or guarantee, express or implied, is made concerning the test results, comments, recommendations, or any other portion of this report.

14.0 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, EBA has not been retained to investigate, address or consider and has not investigated, addressed or considered any environmental or regulatory issues associated with development on the subject site.

15.0 ALTERNATE REPORT FORMAT

Where EBA submits both electronic file and hard copy versions of reports, drawings and other project-related documents and deliverables (collectively termed EBA's instruments of professional service), the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding. The hard copy versions submitted by EBA shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancies, the hard copy versions shall govern over the electronic versions. Furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed version archived by EBA shall be deemed to be the overall original for the Project.

The Client agrees that both electronic file and hard copy versions of EBA's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except EBA. The Client warrants that EBA's instruments of professional service will be used only and exactly as submitted by EBA.

The Client recognizes and agrees that electronic files submitted by EBA have been prepared and submitted using specific software and hardware systems. EBA makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.

APPENDIX B

SITE INSPECTION OBSERVATIONS

TABLE B.1
UPPER CHASE RIVER DAM - DESCRIPTION AND SITE INSPECTION OBSERVATIONS

GENERAL DESCRIPTION

Location:	Nanaimo Lakes Road, Chase River Crossing, between Reservoir #1 and Reservior #2		
Length:	66 m (concrete wall)		
Max. Height:	~ 5.6 m to mudline	**Downstream slope angle:**	varies
Crest width:	Concrete wall 3' wide	**Upstream slope angle:**	vertical
Appurtenances:	Low level conduit (not observed on supstream), wasterlines on crest, sterm server Water valve line and storm sewer gate at ~ centre line Water level 2.31 m below crest of wall		

DAM #1 OBSERVATIONS

Location	Observation	
Concrete wall, upstream face	spalling concrete, deterioration at right abutment	
Adjacent to concrete wall, downstream	waterline valve and storm sewer gate, replaced for ~ 6 m width centred on valve	
Downstream crest, ~10 m from right abutment	water valve cap	
~12 m from right abutment	GNWD manhole	2 manholes downstream of these
~19 m from right abutment	GNWD manhole, with 2 vents	on downstream side of fence
~24 m from right abutment	GNWD manhole	
Downstream crest, ~ centre dam	roadfill supported by concrete lock block concrete chamber and inspection ports pipe discharge into #1 reservoir	
Concrete wall, upstream	Buttresses only present at centre to left abutment, 39 m from right abutment, 4 m spacing on centre, last one 10m from left abutment, on centre	
Buttresses 0.6 m thick, sloped 0.32 m H: 1.7 m		
Spillway channel	beaver dam (2 high) at inlet	waterline valve observed at edge of pavement to left of dam
Lock block wall	~ 1.5 m high (2 blocks), ~ 1.3 m soil retained	

DAM #1 OBSERVATIONS

Location	Observation
Slope into reservoir	30% right end of lock block wall, 45% between left jets and trees on right abutment Depth of reservoir unknown slope drains (approx. 150 mm diameter) persent at reservoir edge
Downslope adjacent to reservoir	gravel on geotextile, ivy growing
Downslope of fence, adjacent to green vents	3 waterline valve caps, 2 manholes round, 1 square manhole
Downslope of fence, adjacent to edge of lock block wall	lawn watering sprinkler, 3 noted near downslope of slope of dam
Concrete bunker/chamber	no cracks, looks to be in good shape
Left abutment between dam and spillway	2 pipes that apparently act as emergency overflow conduit if spillway is overblocked at inlet inlets visible from outlets, clogged with blackberries

APPENDIX C

PROBABLISTIC SEISMIC ASSESSMENT CONDUCTED BY PACIFIC GEOSCIENCE CENTRE

NATURAL RESOURCES CANADA
GEOLOGICAL SURVEY OF CANADA

RESSOURCES NATURELLES CANADA
COMMISSION GEOLOGIQUE DU CANADA

SEISMIC RISK CALCULATION * / CALCUL DE RISQUE SEISMIQUE *

REQUESTED BY/ DEMANDE PAR: Chris Grapel, EBA Engineering

SITE: Nanaimo, B.C.

LOCATED AT/ SITUE AU: 49.17 NORTH/NORD 123.93 WEST/OUEST

PROBABILITY OF EXCEEDENCE PER ANNUM/ PROBABILITE DE DEPASSEMENT PAR ANNEE	0.010	0.005	0.0021	0.001
PROBABILITY OF EXCEEDENCE IN 50 YEARS/ PROBABILITE DE DEPASSEMENT EN 50 ANS	40 %	22 %	10 %	5 %
PEAK HORIZONTAL GROUND ACCELERATION (G) / ACCELERATION HORIZONTALE MAXIMALE DU SOL (G)	0.092	0.136	0.219	0.311
PEAK HORIZONTAL GROUND VELOCITY (M/SEC) / VITESSE HORIZONTALE MAXIMALE DU SOL (M/SEC)	0.082	0.127	0.218	0.323

* REFERENCES

1. NEW PROBABILISTIC STRONG SEISMIC GROUND MOTION MAPS OF CANADA: A COMPILATION OF EARTHQUAKE SOURCE ZONES, METHODS AND RESULTS. P.W. BASHAM, D.H. WEICHERT, F.M. ANGLIN, AND M.J. BERRY EARTH PHYSICS BRANCH OPEN FILE NUMBER 82-33, OTTAWA, CANADA 1982.

2. ENGINEERING APPLICATIONS OF NEW PROBABILISTIC SEISMIC GROUND-MOTION MAPS OF CANADA. A.C. HEIDEBRECHT, P.W. BASHAM, J.H. RAINER, AND M.J. BERRY CANADIAN JOURNAL OF CIVIL ENGINEERING, VOL. 10, NO. 4, P. 670-680, 1983.

3. NEW PROBABILISTIC STRONG GROUND MOTION MAPS OF CANADA. P.W. BASHAM, D.H. WEICHERT, F.M. ANGLIN, AND M.J. BERRY, BULLETIN OF THE SEISMOLOGICAL SOCIETY OF AMERICA, VOL. 75, NO. 2, P. 563-595, 1985.

4A. SUPPLEMENT TO THE NATIONAL BUILDING CODE OF CANADA 1990, NRCC NO. 30629. CHAPTER 1: CLIMATIC INFORMATION FOR BUILDING DESIGN IN CANADA. CHAPTER 4: COMMENTARY J: EFFECTS OF EARTHQUAKES.

4B. SUPPLEMENT DU CODE NATIONAL DU BATIMENT DU CANADA 1990, CNRC NO 30629F. CHAPITRE 1: DONNEES CLIMATIQUES POUR LE CALCUL DES BATIMENTS AU CANADA. CHAPITRE 4: COMMENTAIRE J: EFFETS DES SEISMES.

Nanaimo, B.C.

ZONING FOR ABOVE SITE/ ZONAGE DU SITE CI-DESSUS

1990 NBCC/CNBC: ZA = 4; ZV = 4; V = 0.20 M/S

ACCELERATION ZONE/ ZONE D'ACCELERATION ZA=4
ZONAL ACCELERATION/ ACCELERATION ZONALE 0.20 G

VELOCITY ZONE/ ZONE DE VITESSE ZV=4
ZONAL VELOCITY/ VITESSE ZONALE 0.20 M/S

1990 NBCC/CNBC **
SEISMIC ZONING MAPS/ CARTES DU ZONAGE SEISMIQUE

PROBABILITY LEVEL: 10% IN 50 YEARS
NIVEAU DE PROBABILITE: 10% EN 50 ANNEES

G OR M/S	ZONE	ZONAL VALUE/ VALEUR ZONALE
0.00		
	0	0.00
0.04		
	1	0.05
0.08		
	2	0.10
0.11		
	3	0.15
0.16		
	4	0.20
0.23		
	5	0.30
0.32		
	6*	0.40

* ZONE 6: NOMINAL VALUE/ VALEUR NOMINALE 0.40;
SITE-SPECIFIC STUDIES SUGGESTED FOR IMPORTANT PROJECTS/
ETUDES COMPLEMENTAIRES SUGGEREES POUR DES PROJETS D'IMPORTANCE.

** FOR NBCC APPLICATIONS, CALCULATED ZONE VALUES AT A SITE SHOULD BE REPLACED BY EFFECTIVE ZONE VALUES [ZA(EFF) OR ZV(EFF)] AS SHOWN BELOW/
POUR APPLICATIONS SELON LE CNBC, ON DOIT REMPLACER LES VALEURS ZONALES CALCULEES POUR UN SITE PAR LES VALEURS EFFECTIVES [ZA(EFF) OU ZV(EFF)] COMME MONTRE CI-DESSOUS:

1. IF/SI (ZA - ZV) > 1, ===> ZA(EFF) = ZV + 1.

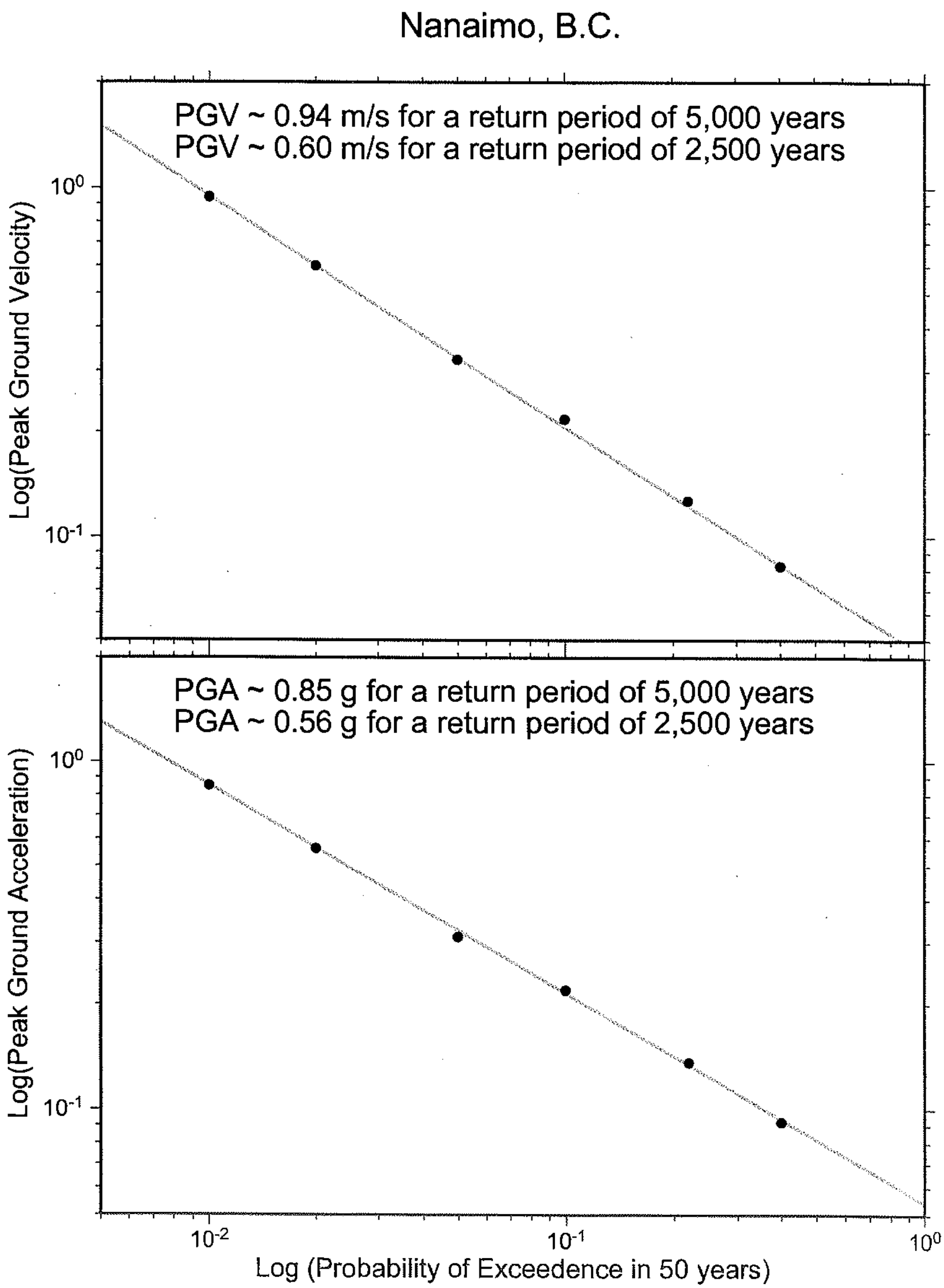
OR/OU

2. IF/SI (ZA - ZV) < 1, ===> ZA(EFF) = ZV - 1.

OR/OU

3. IF/SI ZV=0 AND/ET ZA > 0, ===> ZV(EFF) = 1.

(SEE REFERENCE 2 CITED ABOVE, PAGE 677)
(VOIR PAGE 677 DE LA REFERENCE 2 CI-DESSUS) Aug 25 2004 13:27



Nanaimo, B.C.
PGV ~ 0.94 m/s for a return period of 5,000 years
PGV ~ 0.60 m/s for a return period of 2,500 years
Log(Peak Ground Velocity)
10^0
10^{-1}
PGA ~ 0.85 g for a return period of 5,000 years
PGA ~ 0.56 g for a return period of 2,500 years
Log(Peak Ground Acceleration)
10^0
10^{-1}
10^{-2}
10^{-1}
10^0
Log (Probability of Exceedence in 50 years)

APPENDIX D

STRUCTURAL RETAINING WALL STABILITY ANALYSIS
LETTER FROM HEROLD ENGINEERING LTD.
DATED NOVEMBER 4, 2004

HEROLD
ENGINEERING LIMITED
Consulting Engineers

NOV 15 2004

November 4, 2004

868-006/02

Fax: 780-454-5688

EBA Engineering Consultants Ltd.
14940 -- 123 Avenue
Edmonton, Alberta
T5V 1B4

Attn: Chris Grapel P. Eng.

Re: City of Nanaimo
Upper Chase Dam
Seismic Assessment

Dear Chris:

At your request, Herold Engineering Limited completed a static and seismic assessment of the Upper Chase Dam in Nanaimo, BC to determine the ability of the structure to withstand current seismic loading.

We based our analysis on the information regarding the dam given to us by your office and as summarized below:

- No original construction drawings were made available
- Date of construction unknown
- Length of dam - 68 meters
- Thickness of dam – 914mm – no taper
- 605mm wide buttresses at 4.0m on centre on upstream face starting at middle of dam and going north
- Buttresses sloped at 0.32m horizontal to 1.7m vertical
- No footings on buttresses or vertical wall
- Maximum height of wall to mudline 5.8m
- Maximum depth from mudline to underside of dam 1.6m
- Height of water at time of assessment – 2.3m below top of dam
- Concrete compressive strengths of 31.6 MPa – 43.1MPa
- Seismic peak horizontal ground acceleration
  - .092g for 1% chance of exceedance per annum
  - .311g for 0.1% chance of exceedance per annum
- Backfill to full height of wall on road side

3701 Shenton Road, Nanaimo, BC V9T 2H1 Telephone: (250) 751-8558 Facsimile: (250) 751-8559
e-mail: mail@heroldengineering.com

Gravity Wall Designs

The wall of the dam is subject to both static pressure on the back of the wall due to horizontal pressures exerted by soil behind the wall and dynamic pressure from seismic forces. Stability of the wall against overturning for both static and dynamic loading is due to the self weight of the wall and the passive pressure on the water side of the wall from water and soil below the mudline.

There are buttresses on the upstream face of the dam wall at 4.0 metres on centre starting at the mid-length of the wall and going north. There are no buttresses south of the mid-length of the wall.

Where there are buttresses, the wall would tend to overturn about the toe of the buttress due to static and dynamic forces. Where there are no buttresses the wall would overturn about the toe of the wall. Buttresses therefore, if designed properly, add stability to the wall against overturning from static and dynamic forces.

Design Calculations

Design calculations were completed for both static loading and dynamic loading. Static loading is due to soil pressures exerted at the back of the wall. Dynamic loading is due to ground motion due to a seismic event and include a component for the wall itself and a component for the moving soil behind the wall.

Our design calculations indicated that the wall, assuming that buttresses are being used to help stability, had a factor of safety significantly less than one if there was no water on the upstream side of the dam and that you would require a minimum height of water of 3.75 meters below the top of the wall to have a factor of safety equal to one. (ie. No factor of safety against overturning).

If seismic forces were taken into account, they would be additive to the static force from the backfill and the total overturning moment assuming a buttressed wall would be 18% higher than the restoring force for a 1% chance of exceedance per annum and 230% higher for a 0.1% chance of exceedance per annum.

It is noted that the current seismic design forces for a building in the Nanaimo area are based on a peak horizontal ground acceleration of 0.2g which has a corresponding chance of approximately exceedance per annum of 0.2%.

Conclusion

We have provided design calculations based on wall parameters as noted previously. Probing for footings on the upstream side of the dam was completed by EBA Engineering Consultants Ltd. staff and there were no signs of footings for the buttresses or the wall. The wall however was not excavated on the soil side to determine if there was a footing on this side for stability. In the writer's opinion, it would be prudent to check to see if in fact a concrete footing did project into the backfill on the east side of the wall but in our opinion it would not be expected.

What does all this mean?

In simple terms, theoretically the wall is not stable under static loading alone unless the water level is at a minimum height of 3.75m below the top of the wall.

Under dynamic loading the wall in all likelihood would fail at seismic design loads recommended in the 1998 British Columbia Building Code.

We trust the foregoing is the information you require at this time. Please contact the undersigned if you require any further information.

Yours truly,

HEROLD ENGINEERING LIMITED

PROFESSIONAL
PROVINCE OF
M.G. HEROLD
BRITISH COLUMBIA
ENGINEER

Mike Herold, P.Eng.

Nov 4 /04

Mh/tt

August 17, 2005 EBA File: 1940008

Engineering Department
City of Nanaimo
200 Franklyn Street
Nanaimo, BC V9R 5J6

Attention: Scott Pamminger, A.Sc.T.

Subject: Upper Chase Dam, Geophysical Survey, Nanaimo, BC

1.0 INTRODUCTION

1.1 GENERAL

The City of Nanaimo (the City) owns and operates a series of dams on the Chase River. These dams were the subject of a recent Dam Safety Review completed by Golder Associates Limited in 2004. As a result of this review, a seismic analysis was requested by the City on the Upper Chase Dam and was carried out by EBA Engineering Consultants Ltd. (EBA) in August and September 2004. The full results of this study are contained in a report entitled 'Upper Chase River Dam Seismic Assessment' which was submitted to the City in May 2005 (EBA Project #0802-2800097).

One of the issues raised by the EBA seismic analysis was whether the main portion of the Upper Chase dam structure between Reservoir No. 2 and No. 1 is primarily an earth structure, or whether it was a road embankment constructed on a bedrock outcrop. An additional consideration, if the latter is the case, is how extensive the bedrock outcrop is. To address these questions, it was proposed to conduct a geophysical survey using Ground Penetrating Radar (GPR) over the main portion of the Upper Chase dam structure to obtain the bedrock profile.

The earthfill portion of the Upper Chase Dam over the spillway was not surveyed.

1.2 SITE DESCRIPTION

The Upper Chase River Dam is the upper most of the Chase River Dams. Its impoundment is known as Reservoir No. 2, which is immediately upstream of Reservoir No. 1. It is believed to have been constructed about 70 years ago.

The dam is generally described as an earthfill dam which serves as the road embankment for Nanaimo Lakes Road. An upstream concrete retaining wall is located immediately adjacent to the road for the majority of the dam length. The concrete retaining wall appears to have been constructed to act as a retaining wall for the road fill. The earthfill-concrete wall dam section is approximately 64 m long, 5 to 6 m high, generally 20 m wide and impounds

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about 4 m of water. The concrete wall has a series of buttresses on the upstream face over the central portion of the dam length.

A second section of the dam consisting of earthfill only, is located in the vicinity of the spillway structure. In this area the dam is approximately 2 to 3 m high, 8 to 10 m wide at the crest and does not impound water at normal reservoir levels. The spillway consists of a concrete channel that leads to two corrugated steel pipe culverts that pass beneath Nanaimo Lakes Road, which is located on the crest of the dam.

Discharge from the spillway passes into the Chase River, bypassing Reservoir No.1. The flow from the Upper Chase spillway is impounded by the Middle Chase Dam. Reservoir No. 1 is impounded by a concrete gravity dam founded on bedrock. Two other dams are located downstream of the Upper Chase River Dam on the Chase River. They are the Middle Chase River Dam and the Lower Chase River Dam.

A low level conduit (450 mm diameter) was installed through the Upper Chase Dam in 1998 to provide capacity to divert water during an emergency from Reservoir No. 2 into Reservoir No. 1. This diversion conduit was installed approximately 3 to 3.5 m below the crest of the dam. This conduit is situated on the north side of the main dam structure and terminates in a concrete box structure built into the downstream face. Shallow bedrock was encountered and an unknown quantity of bedrock was removed to allow for construction of the concrete box structure.

A detailed plan of the Upper Chase Dam based on survey data obtained from the City of Nanaimo is presented in Figure 1. The locations of the geophysical profiles collected are shown on this figure.

2.0 SCOPE OF WORK

A geophysical survey was carried out using Ground Penetrating Radar (GPR) to investigate the depth to bedrock below the crest and downstream slope of the dam. A series of profiles were collected both along the length of the main portion of the Dam and from the crest of the Dam down the downstream face. The lines were positioned to adequately cover the narrowest portion of the dam between Reservoir No. 1. and Reservoir No. 2. and also to extend the mapping of the bedrock encountered during construction of the diversion conduit and concrete box in 1998.

2.1 SURVEY METHODOLOGY

Ground penetrating radar is a non-destructive geophysical technique capable of delineating boundaries between complex stratigraphies due to changes in bulk electrical properties of the subsurface lithology, mineralogy or the character of the interface between layers (Davis and Annan, 1989). Operationally, GPR systems transmit a short duration electromagnetic (EM) pulse into the ground generating a downward propagating wavefront. At each stratigraphic interface, a portion of the wavefront energy is reflected back to the surface. A radar receiver, located at the surface, detects (and typically digitally samples and records) the reflected EM pulse. The detected pulse amplitude and delay time are a function of the subsurface electrical properties. The strength of the reflected signal is approximately proportional to the difference in dielectric contrasts at the reflecting interface. The pulse transmit/receive delay time is inversely proportional to tbe EM propagation velocity (determined by the bulk electrical properties), and proportional to the distance from the receiver at the surface to the reflecting stratigraphic interface (Davis and Annan, 1989). Changes in dielectric constants and electrical conductivity also affect signal attenuation. High conductivities, as found in fine-grained materials such as silts and clays, can increase signal attenuation and limit signal propagation to a few metres or less. Conversely, in areas not affected by excessive signal attenuation, interfaces deeper than 50 m can be detected. It becomes apparent that the correct subsurface structure interpretation, based on reflected pulses detected in the radar return signals, requires extensive knowledge and experience with radar pulse propagation properties as well as with local material properties.

Table 1 shows typical electrical properties of various materials (SSI, 1989). However, these properties of the materials vary slightly from site to site. The properties can be determined site-specifically using ground truth information.

TABLE 1: ELECTRICAL PROPERTIES OF MATERIALS (SSI, 1989)

Material type	Dielectric constant	Velocity (m/ns)	Conductivity (mS/m)	Attenuation (dB/m)
Wet Sand	20-30	0.06	0.1-1	0.03-0.3
Silts	5-30	0.07	1-100	1-100
Granite	4-6	0.14	0.01-1	0.01-1
Fresh Water	80	0.033	0.5	0.1

Most bedrock falls within the range for granite quoted in Table 1. High moisture content rocks such as sandstone may deviate from this range.

By exploiting the sensitivity to variations in bulk material electrical properties, GPR is an established method for detecting subsurface anomalies, profiling complex geological stratigraphic components, and mapping natural phenomena.

Two separate GPR systems were used to minimize the uncertainty over the attenuation characteristics of the soil used for the dam fill. The first was a GSSI SIR 8 GPR system with a 500 MHz antenna and the second a Pulse Ekko IV GPR system with a pair of 200 MHz antennas. A total of 11 separate geophysical profiles were collected (see Figure 1); each profile was collected with both GPR systems.

The survey methodology for both systems was similar. Each consisted of moving the antennas along the geophysical profile with the GPR system collecting a vertical shot or trace at specific locations along the profile. The GSSI system with its higher repetition rate is moved at a slow walking pace resulting in approximately one trace per 5 cm of travel. The Pulse Ekko IV system was used at stationary shot points every 25 cm along the geophysical profile. Key locations along each geophysical profile were marked and located using a mapping grade Dumb Global Positioning System (DGPS) system. In addition, key features such as the fence line along the Reservoir No. 1. edge, the turning points on Geophysical Profile 01 and 02 and the start and end points of each profile line were logged. All pertinent survey information is located on the site drawing of the dam structure provided by the City of Nanaimo in Figure 1.

3.0 RESULTS

Data collected using both GPR systems (the GSSI SIR 8 and the Pulse Ekko IV) were reduced and analyzed.

GSSI SIR8 System

Good horizontal and vertical detail was seen in the 500 MHz data but typical penetration depths were only in the order of 1.5 to 2 metres and, therefore, insufficient information was obtained over the majority of the geophysical profiles collected to comment on the likely presence of bedrock. As a result, the GSSI 500 MHz data has not been used in this analysis and discussion.

Pulse Ekko IV System

The GPR data collected using the 200 MHz Pulse Ekko IV data had lower horizontal and vertical resolution than the 500 MHz data but typically identified reflections between 6 to 8 metres below grade on the majority of the profiles. This penetration depth was sufficient to identify any bedrock horizon of interest provided the bedrock contact was detectable (see Limitations Section 3.1). The Geophysical Profile data collected using the 200 MHz antennas are shown in Figures 2, 3 and 4.

Figures 2, 3, and 4 consist of two panels for each geophysical profile, the lower panel shows the processed GPR profile with reflectors that have been identified as significant selected, and the upper panel shows an interpreted panel with depth of the reflectors, their location along the profile line and some descriptive comments. In addition, significant tie points between lines are provided as well as the approximate water level in Reservoir No. 2.

It should be noted that all of the conclusions drawn from the GPR surveys are interpretations of the recorded signal reflections based on judgement and experience. Some of the reflected signals clearly indicated the presence of bedrock; however, there were instances where the depth to bedrock was ambiguous, likely due to weathered or infilled zones of bedrock. This is for the following reasons. A number of reflections have been identified in Figures 2, 3 and 4. Solid lines have been used for definite reflectors, dashed lines have been used for reflections that are more ambiguous and, therefore, have a higher level of uncertainty. Likewise, interpretive text with a question mark following the descriptor indicates that there is some uncertainty in the interpretation, but is considered the most likely one. These reflectors represent an approximate 70% confidence level in the interpretation process. A number of reflectors are generally seen around the one identified as the bedrock reflector. These are due to multipath reflections which are typically located around the concrete box structure as well as fractures and weathering at the bedrock surface. The reflector selected was the most likely bedrock reflector was usually one of the strongest and/or most contiguous seen. There are also a number of reflections under the existing roadway that are likely caused by buried utilities, the road structure and soil. Numerous pipe reflectors can be seen of varying sizes. Only the larger more obvious pipe reflections have been identified.

All GPR measurements directly measure the two way travel time between the ground surface and the various reflectors. These measurements must be converted to depth using a bulk propagation velocity. A bulk propagation velocity of 0.106 metres/nanosec (m/ns) corresponding to a soil dielectric constant of 8 was used in the interpreted profiles presented in Figures 2, 3 and 4. This velocity would be typical for a moist sandy/silty material with some gravel particles throughout.

3.1 LIMITATIONS

Two issues need to be discussed regarding the data limitations.

The first concerns the accuracy of the DGPS data. The DGPS system used was a mapping grade unit meaning that given suitable satellite coverage, x, y positions will be sub metre in accuracy. Elevation data has not been used as the accuracy with this type of system was insufficient (+/- 2 metres) for the data to be useful. In addition to the DGPS locations, the length of each geophysical profiles was chained. It is therefore expected that the positional accuracy of the geophysical profiles shown in Figure 1 is within one metre.

The second issue concerns the accuracy and reliability of the interpreted profiles derived from the geophysical data. Typical absolute accuracy in interpreted depth profiles is ±10 % of the actual depth. However in situations where a number of potential bedrock reflectors are present and the selection is ambiguous greater errors in the interpreted bedrock are possible. Areas where there is an increased risk of incorrect depth estimates have been indicated using dashed interpreted interface lines and question marks after descriptors.

It is also important to note that as with all remote sensing techniques in order for GPR to successfully map a physical contact such as top of bedrock, three issues must be satisfied:

- There must be a reasonably significant dielectric contrast present at the boundary to generate a reflection that can be measured.
- The layer overlying the boundary must be sufficiently resistive to allow the GPR signals to pass through and back to the surface.

- The geometry of the boundary must be sufficiently flat and of a geometric shape that enough signal is reflected back to the surface to be detected.

There are a number of specific locations along the GPR profiles collected where bedrock reflections have not been identified for one or more of the above reasons. However, it is felt, based on the information available, comparing the 500 MHz and 200 MHz GPR data and previous sitework, that the bedrock trends shown do summarize the overall bedrock trends.

Based on our judgement and experience, the depths to bedrock presented in the attached profiles represent a general trend and are, in the case of some of the less clearly defined interpretations, within 1.0 m of actual bedrock. The clearly defined interpretations are judged to be generally within 0.5 m of the actual location, based on judgement, experience and correlation of different profiles in this survey at intersection points.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the GPR survey presented herein, the results of our previous work (presented in our report Upper Chase River Dam Seismic Assessment), dated November 2004 and the Golder Associates Ltd. 2003 Dam Safety Review report dated June 2004, EBA has made the following conclusions:

- Upper Chase Dam appears to impound water greater than 1.0 m deep under normal operating reservoir levels over a length of up to 50 m; and
- The left (i.e. north) abutment appears to be bedrock with an elevation within 0.5 to 1.0 m of normal operating level of Reservoir #2 to a distance of about 10 m to the right (i.e. South) of Section A-A.

At the right abutment of the dam, the bedrock is at approximately 2.5 m below normal operating level. However, it is at this location that the dam fill is the thinnest and where the upstream concrete wall terminates. Therefore, it is our opinion that the majority of the soil overlying bedrock at this location is natural soils, possibly till.

Even if the greatest depth of fill (or combination of fill and till) overlying bedrock below the normal operating water level was 2.5 m, based on our seismic assessment work presented in our November 2004 report, it is doubtful that any failure due solely to a seismic event could result in immediate failure of the dam due to the width of the structure near the right abutment.

However, as indicated in our November 2004 report, failure of the upstream concrete wall could result in loss of support and possihle rupture of the 200 mm diameter waterline. It is our understanding this waterline has been decommissioned and capped. Therefore, the potential rupture of the 760 mm diameter waterline under the centreline of the Nanaimo Lakes Road will likely not occur. It is important to recognize that there are lawn watering sprinkler pipes, a 760 mm diameter water line, a storm sewer line, an emergency conduit, and a decommissioned 200 mm diameter waterline within this dam. After a significant seismic event, checks on all waterlines and piping should be conducted to ensure that leakage does not saturate the dam fill and exacerbate any instabilities caused by the seismic event or initiate new deformations.

Based on these conclusions, EBA makes the following recommendations:

- As per our recommendation presented in our May 2005 report, a large tree present on the upstream side of the dam at the right abutment should be removed. Removal of this large tree should be accompanied by the excavation of a testpit which could be used to remove the majority of the central stump of the tree. EBA should attend this testpitting to observe the depth of fill (if possible) and nature of underlying native soils. This assessment would provide additional information with regard to depth to competent soil such as till. Till at a shallow depth could result in reduction if the height of water retained by the fill during normal operating conditions in Reservoir #2; and
- Based on the results of the testpitting discussed in the aforementioned paragraph, the City of Nanaimo should calculate the volume of water stored by the dam (i.e. above bedrock and till) and compare this result to the guidelines for definition of a dam presented in the British Columbia Dam Safety Regulation. This expansion may still hold open an avenue through which Upper Chase Dam could be declassified from being a dam from the prospective of Land and Water British Columbia.

A testpit at the right abutment would serve to supplement the GPR data and provide ground truthing of the geophysical survey data. Detailed assessment of depth to bedrock would require additional testpits or boreholes to confirm the depth to bedrock. However, the original purpose of this study was to investigate if bedrock was present within the embankment to an extent that the embankment fill did not retain water. As the GPR data presented herein indicates, this is not the case, but that tbe extent of the dam (height and length) is less than initially assumed in our May 2005 report, the degree of ground truthing through comparison to known bedrock depth at the concrete structure and, eventually, at the right abutment, should be sufficient at this time. EBA should review the data presented herein in comparison to the observations made during the testpitting to assess the consistency with the GPR interpretations with in situ conditions.

5.0 CLOSURE

Please do not hesitate to contact the undersigned should you have any questions or comments regarding this report.

Yours truly,
EBA Engineering Consultants Ltd.

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Attachments Figure 1: Site Map, GPR Profile Locations
Figures 2 to 4: Interpreted GPR Profiles

FIGURES

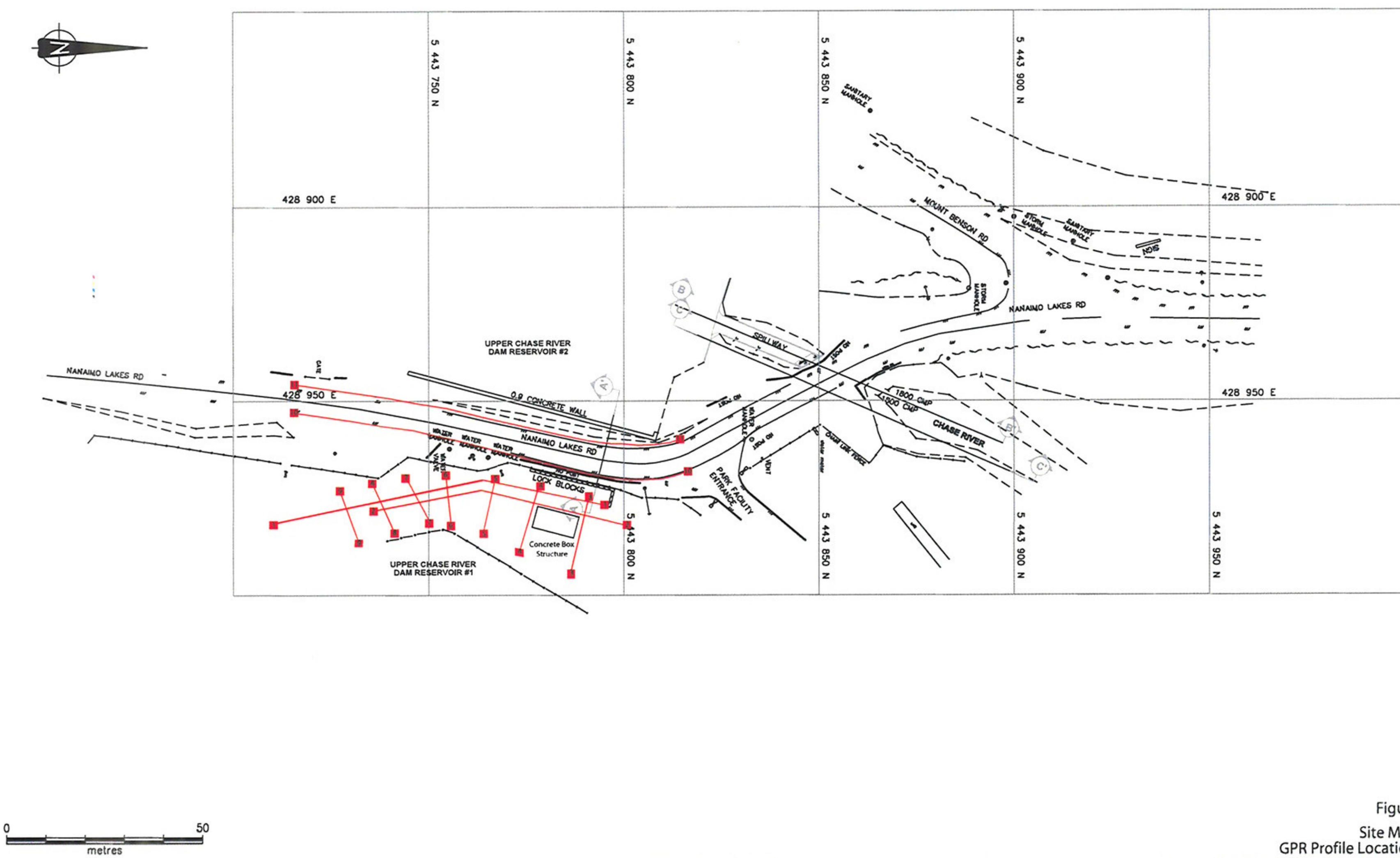

Figure 1
Site Map,
GPR Profile Locations

nsp1940008Figure1.ai

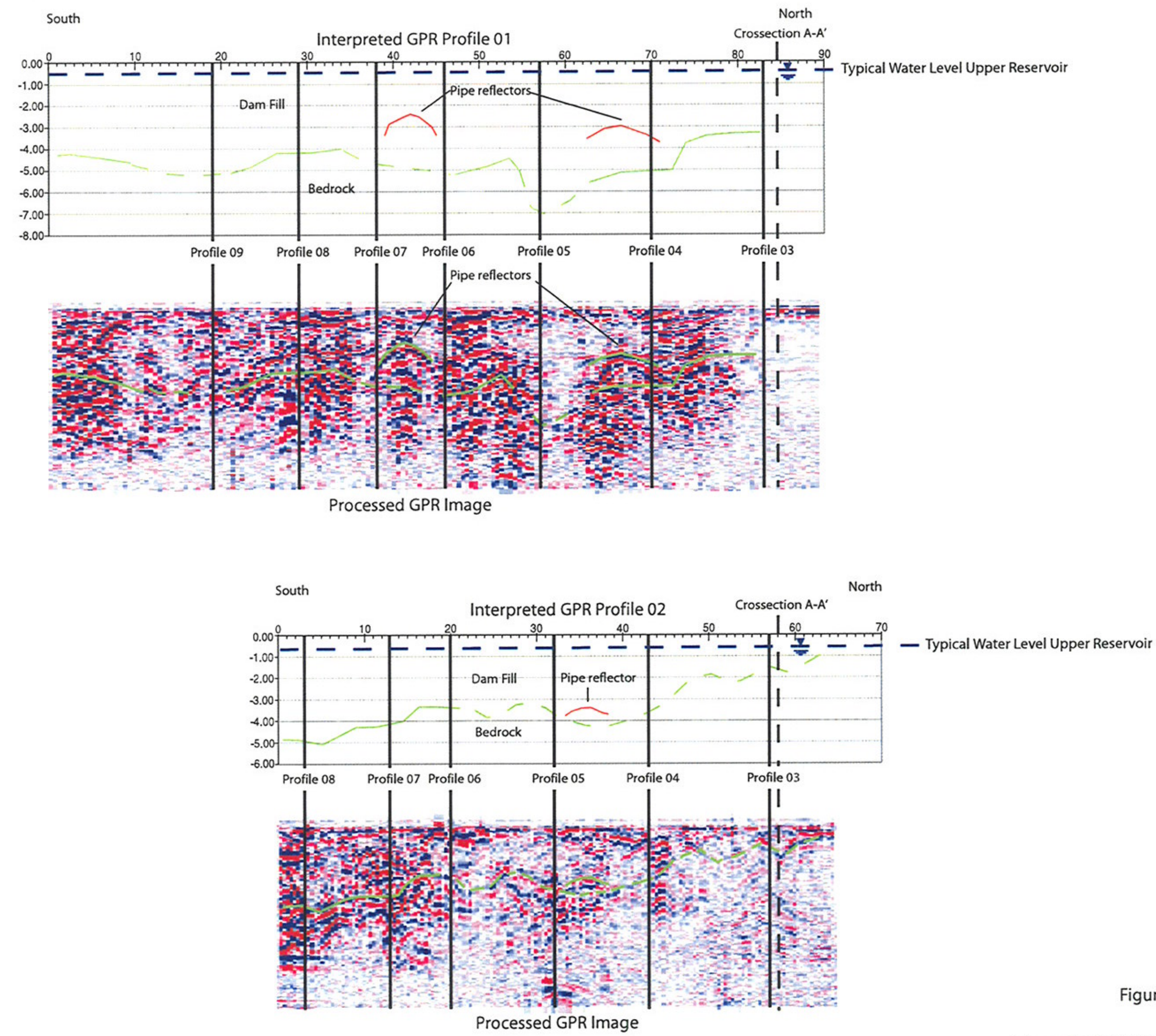

Figure 2

Interpreted GPR Profiles

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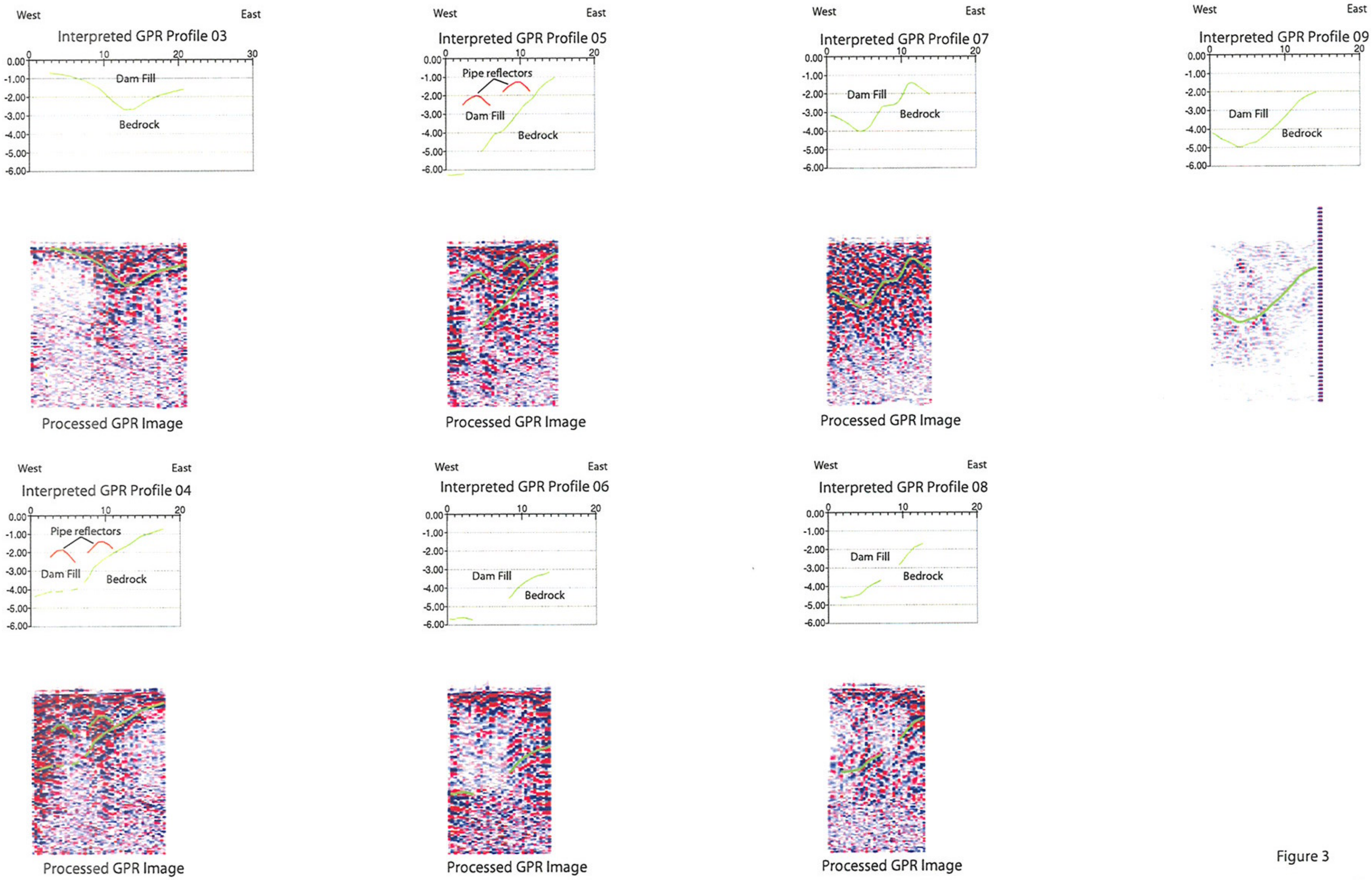

Figure 3

Interpreted GPR Profiles

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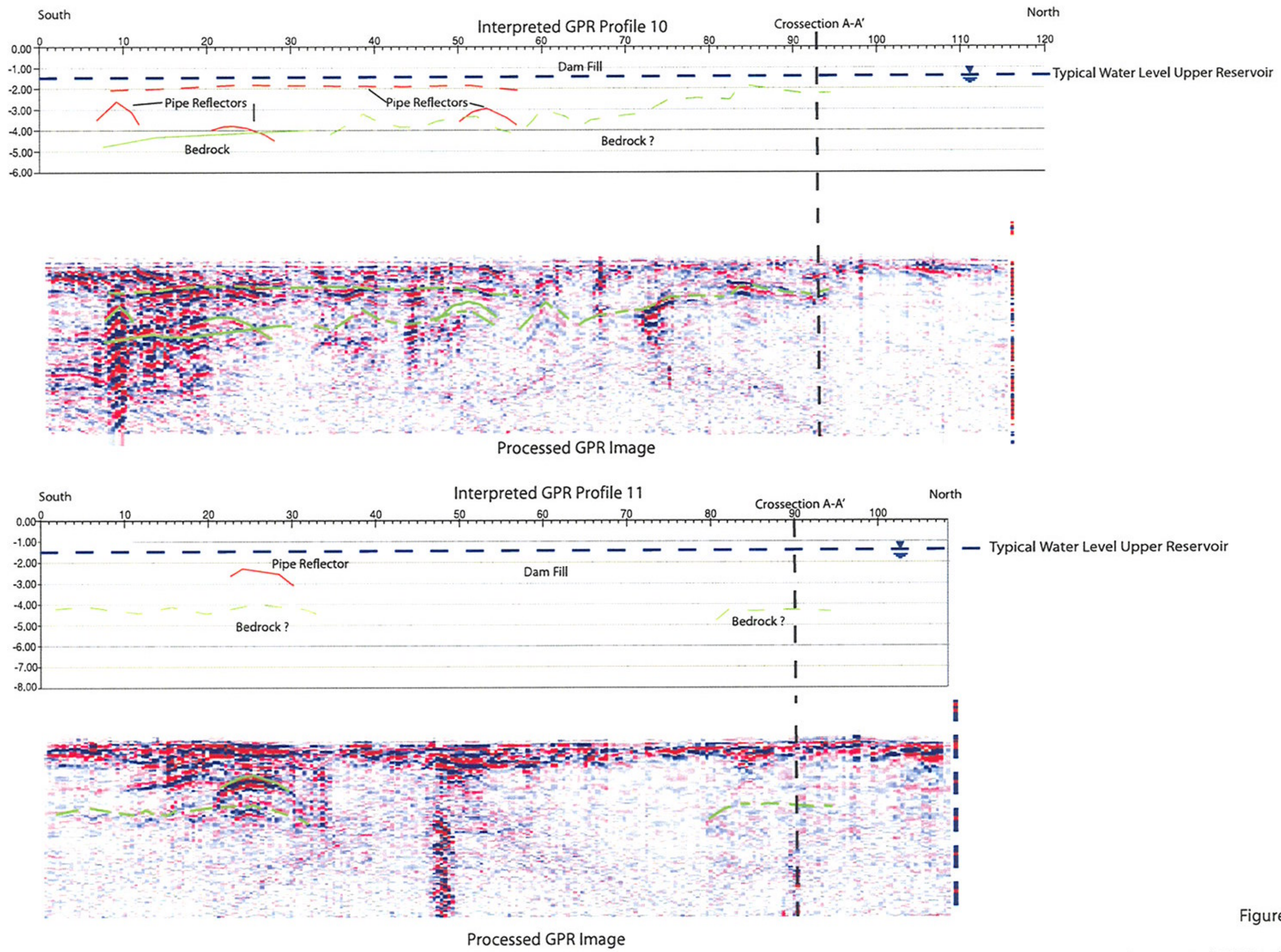

Figure 4

Interpreted GPR Profiles

nsp1940008Figure4.ai