WILLIAMS LAKE CONSERVATION COMPANY

WILLIAMS LAKE DAM EVALUATION

MARCH 2005

YMCL Engineering Limited

YMCL Engineering Limited

Suite 1404, CIBC Building, 1809 Barrington Street Halifax, Nova Scotia B3J 3K8
Telephone: (902) 429-3514 Facsimile: (902) 431-9622

File: WLC-6358
Date: May 16, 2005

Williams Lake Conservation Company
29 Wyndrock
Halifax, Nova Scotia
B3P 1R8

Attention: Mr. Andrew Ross, MD, FRCPC

Dear Mr. Ross:

Re: Williams Lake Dam Evaluation

YMCL Engineering Limited is pleased to provide two copies of the attached dam evaluation for the Williams Lake Dam. The report includes the findings for the following activities:

- Geotechnical Investigation
- Site Survey
- Condition Survey
- Flood Study
- Dam Evaluation
- Mitigation

The mitigation section of this report provides a preliminary design for a new dam construction at the present site of the existing dam. The preliminary design consists of a cost estimate and preliminary design sketches. Should the Williams Lake Conservation Company choose to proceed with the construction of a new dam the following course of actions are recommended:

- Submit applications to regulatory authorities including Nova Scotia Department of Environment and Labor, Navigable Waters and the Department of Fisheries.
- Proceed with the design drawings for the new dam construction.
- Invite Tenders from approximately four contractors.
- Upon receiving permits to construct the construction should proceed during the summer months preferably between July to September.

YMCL has completed engineering services for a number of similar projects including permit applications and we would be pleased to provide fees for these services upon your request.

We trust the above information and the enclosed dam evaluation report is to your satisfaction. YMCL is available should you have any questions.

Yours truly,

Thom. A Mosher

Thomas A. Mosher, P. Eng.

WILLIAMS LAKE CONSERVATION COMPANY

WILLIAMS LAKE DAM EVALUATION

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1.0 INTRODUCTION

1.1 General

As part of the ongoing preservation activities of the Williams Lake Conservation Company, YMCL was commissioned to complete an engineering evaluation of the Williams Lake Dam. Should this evaluation determine that repairs and/or strengthening of the dam are required, the construction would be co-ordinated with present schedules and budgets.

The following evaluation report was prepared by YMCL Engineering Limited. The report includes the findings of the site investigation and the evaluation of the existing dam. The existing dam was evaluated using the "Dam Safety Guidelines" prepared by the Canadian Dam Safety Association.

Structural drawings were not available for the existing dam; therefore drawings were prepared to indicate the existing geometry of the dam structure as well as details of the water control structures and spillway channels. These drawings are included in Appendix B of this report.

The dam evaluation report includes the findings from several investigations that are included in this report and form the overall Dam Evaluation. The following investigations have been completed as part of this Dam Evaluation:

- Geotechnical Investigation
- Flood Study Investigation
- Site Survey Investigation

1.2 Historical Information

The Williams Lake Dam is presently used to retain water from Williams Lake. Williams Lake is used for recreational purposes. Other uses could consist of an emergency water supply and fire fighting purposes.

The original dam at this location may have been constructed in the early 1900's and provided water for a sawmill and logging operation in the area.

2.0 DAM DESCRIPTION

2.1 General Description

The Williams Lake Dam consists of an earth/rock dam structure and an overflow channel. The dam has two functions; one to retain Williams Lake as a source of recreation for the houses surrounding the lake and two to provide a minimum flow to the stream below the dam. There are few users downstream that use the water in the stream.

The following paragraphs provide a general description for each area of the dam. Refer to Figure 1 and 2 of Appendix B that indicate the location plan and site plan for the dam and surrounding areas.

2.2 Earth Dam Structure

The earth/rock dam structure is approximately 200 feet long with ends running more or less into the original terrain at each end of the dam. Refer to Figure 2 of Appendix B indicating the general arrangement of the dam. Refer to Photograph 1 of Appendix A, that indicates a view along the top of the dam. The upstream face near the centre of the dam consists mostly of stone rubble, grout bags and cast-in-place concrete placed on top of the rocks. Refer to Photograph 2 of Appendix A, indicating the grout bags on the upstream face of the dam. The centre section of the dam is approximately 60 feet long. The dam on each side of this centre section appears to be constructed of a rock and gravel fill.

At the centre of the dam there is a 30 foot wide area that has been backfilled with large rocks that vary from 1 foot to 3 feet in diameter. This area of the dam is flanked by a gabion basket wall on either side. Refer to Photograph 3 of Appendix A, indicating a view of the rock backfill and the gabion baskets.

2.3 Overflow Channel

There is an overflow channel located approximately 25 feet south of the dam. The elevation of the overflow channel at Williams Lake is 60.37 feet which is slightly lower than the crest of the dam in front of the rock fill. Refer to Photograph 4 indicating the entrance to the overflow channel at Williams Lake. Refer to Figure 2 of Appendix A, indicating the location of the overflow channel.

2.4 Spillway Channel

The spillway channel is approximately 30 feet wide and starts at the toe of the dam. Refer to Photograph 5 indicating a view of the spillway channel from the crest of the dam. Refer to Figure 1 that indicates the location of the spillway channel and stream from Williams Lake to the Northwest Arm. The stream enters the Northwest Arm at the Royal Nova Scotia Yacht Squadron. Refer to Photograph 6 of Appendix B that indicates the box culvert under Purcell's Cove Road. Between Purcell's Cove Road and the Northwest Arm is a small pond that is on the Royal Nova Scotia Yacht Squadron property. There are three culverts at one end of the pond that empty into the Northwest Arm.

3.0 FIELD INVESTIGATION AND CONDITION SURVEY

3.1 General

A field investigation was completed on October 20, 2004 to obtain existing information for the dam facility as well as to determine the existing condition of the structure. The site investigation was completed by Tom Mosher, P. Eng. of YMCL Engineering Limited.

A Geotechnical investigation was completed on October 20, 2004 by Mitchelmore Engineering Inc. During this investigation three boreholes were drilled through the top of the dam. A copy of the geotechnical report is included in Appendix D.

A Site survey was completed on November 1, 2004 to determine elevations and general geometry of the various structures of the dam facility. The site survey was completed be Stephan Vaughan Surveys. A benchmark was established at the site.

The site investigation consisted of a walkover survey of the embankment dam. The structure was photographed to indicate the existing condition of the dam.

3.2 Earth/Rock Dam Structure

The general condition of the earth/rock dam is in an advanced state of deterioration and is prone to several factors which could cause overtopping. Overtopping of the dam is possible depending on the severity of wind, flood, ice conditions and water levels.

The dam is completely covered in vegetation on the crest and downstream sides of the dam. Refer to Photographs 1 and 7 of Appendix A indicating the severity of this vegetation. As indicated in these photographs this vegetation consists of small bushes to mature trees. Vegetation of this extent presents a potential danger to the integrity of the Dam. The root systems create a channel for the water to flow and will create piping through the dam. Due to the rock construction of the dam the root systems are usually very shallow and when the tree is damaged or felled by wind, the integrity of the dam structure will be compromised. The photographs indicate some damaged trees.

At the 30 foot wide spillway section, water from Williams Lake passes through and under the dam. Refer to Photograph 8 of Appendix A indicating the flow from the toe of the dam.

There is a major leak just to the north of the spillway area, refer to Photograph 9 of Appendix A that indicates the location of this leak.

There is evidence that the crest of the dam has been overtopped, refer to Photograph 10 of Appendix A that indicates a severely eroded area on one side of the spillway. There is also erosion on the upstream side of the dam. Refer to photograph 11 of Appendix A indicating this erosion. Refer to Figure 2 of Appendix B indicating the location of these two areas. Depending on the water levels and ice conditions over the next few years; dam failure at these locations is immanent.

3.3 Overflow Channel

The overflow channel located at the South End of the dam has not been properly engineered and the level of water that enters this channel appears to depend on the amount of debris collected at the inlet to the channel. Refer to Photograph 4 indicating the inlet to the channel. The inlet elevation at Williams Lake is 60.37 feet, this is approximately 10 inches lower than the elevation of the top of the dam at the lowest point. Therefore water from Williams Lake will flow into this overflow channel prior to overtopping the dam. Refer to Figure 2 of Appendix B indicating the location of the overflow channel.

3.4 Spillway Channel

The spillway channel appears to be in good condition and there does not appear to be any erosion to either side of the channel. The spillway channel appears to be the original riverbed from Colpitt Lake and Williams Lake to the Northwest Arm. Refer to Photograph 12 of Appendix A for a view of the spillway channel. As indicated in the Photograph the river flows naturally through the existing rocks, boulders and bedrock at the base of the small valley.

The sides of the river are heavily vegetated with trees and bushes. During heavy rainfall events the entire dam at Williams Lake is overtopped. During these events the river has inundated the valley on either sides of its banks. The vegetation on each side of the river is a potential hazard to structures downstream and should be removed.

3.5 Geotechnical Investigation

A geotechnical Investigation at the site was completed on October 20, 2004 by Mitchelmore Engineering Company Limited. Refer to the geotechnical report that is included in Appendix D. The locations of the boreholes are located on Figure 2 of Appendix B. The level of the bedrock is also located on the dam sections that are shown on Figure 3 and 4 of Appendix B.

Refer to Photograph 13 of Appendix A indicating the drill rig on the dam in the process of one of the borehole installations. Refer to Photograph 14 that shows a view of rock core that was retrieved from one of the boreholes.

The elevations recorded for the top of the bedrock vary substantially between the three boreholes. This surface of the bedrock is very similar to the bedrock outcrops that are visible around the lake.

At all three borehole locations there were no evidence of a core in the dam. Cores are usually constructed of impermeable fill material with a high clay content and till content.

4.0 EVALUATION

4.1 General

As indicated in the introduction the dam was evaluated using the Canadian Dam Association (CDA) "Dam Safety Guidelines".

The Williams Lake Dam ca be considered a high potential for failure based on the existing condition of the Dam and the inability of the dam to resist the Inflow Design Flood (IDF).

The following sections indicate the results of the flood study and the evaluation of the Williams Lake Dam.

4.2 Flood Study

A flood study was completed by Ekistics Planning and Design to determine the flows at the dam and water elevations of Williams Lake during heavy rainfall and storm events. A copy of the Flood study report is included in Appendix C of this dam evaluation report.

An inundation mapping study was not part of the scope of work for this dam evaluation. However, based on the review of the contours downstream from the dam and the site conditions it is safe to conclude that any floodwaters from storm events will be contained within the small valley downstream from the dam. The only property and structures within this valley are the following:

- One house located to the north of the river at the Purcell's Cove Road.
- Purcell's Cove Road.
- Royal Nova Scotia Yacht Squadron.

Based on the preliminary investigation the downstream the consequence of the Williams Lake Dam can be assumed a LOW consequence. A LOW consequence would result in no fatalities and some property damage should a dam failure occur. If we assume that the dam is a LOW Consequence the 1/1000 year storm event would then be selected as the Inflow Design Flood (IDF). The water elevation during the 1/1000 year storm event is elevation 63.9 feet.

Presently there is no control structure at the Williams Lake Dam. Water flows through and under the dam and provides flow for the small stream that leads from the dam to the Northwest Arm. As a result the water levels of Williams Lake cannot be controlled and as a result the lake is often dry during extreme dry months during the summer.

There are only are only two users that use water from the stream that leads to the Northwest Arm. The flow that is removed should be determined and a minimum flow would then be selected for the river.

4.3 Earth Dam Evaluation

The evaluation for the Williams Lake Dam includes a review of the dam for the following:

- Piping
- Slope Stability
- Free Board Allowance
- Discharge Capacity

The following paragraphs provide discussions on each of these items. This discussion is based on a limited knowledge of the construction of the dam.

4.3.1 Piping

A traditional assessment of piping potential for an embankment dam requires conjecture and judgement to qualify zoning in the dam, filters (if present), quality of construction, foundations and performance indicators.

Within the ranking criteria for susceptibility to piping, Williams Lake Dam is considered a risk for piping because of the assumed internal structure and visual evidence of seepage at the toe of the dam in two locations.

4.3.2 Slope Stability

The slopes at Williams Lake Dam appeared stable and did not illustrate any historic signs of instability.

4.3.3 Free Board Allowance

Freeboard is the vertical distance between the crest of the embankment and the lake water surface. The survey that was completed during the site visits to the Williams Lake Dam indicates that there is no freeboard available. The CDA Guidelines outline freeboard requirements as the following:

- Full Supply Level plus waves and set-up due to the 1/1000 AEP wind event (Normal Freeboard).
- Design Flood Level plus waves and set-up due to the 1/100 AEP wind event (Minimum Freeboard).

The recommended minimum and maximum freeboard for reservoirs with fetch of less than 1 mile is 3 feet and 4 feet, respectively. The values assume a design wind speeds of 100 mph and 50 mph for the normal and minimum situation and a crest height equal to 1.5 times the significant wave height.

For Williams Lake the water elevation at 1/1000 year event is elevation 63.9 feet. For an earth/rock dam constructed at this lake to crest elevation should be approximately 66.9 feet. The crest elevation of the Williams Lake Dam is presently at elevation 60.43. Therefore the dam is approximately 6 feet lower than required by the CDA guidelines.

4.3.4 Discharge Capacity

The Canadian Dam Association Guidelines indicate that discharge facilities must be capable of passing the Inflow Design Flood (IDF) without the reservoir level infringing on the freeboard for the dam.

Visual evidence confirms inadequate discharge capacity for the Williams Lake dam due to freeboard and discharge deficiencies. The dam is overtopped on a regular basis during heavy rainfalls and has caused extensive erosion of the dam.

5.0 MITIGATION

As indicated in the previous sections of this report the Williams Lake Dam does not satisfy the basic requirements for an earth dam in the following areas:

- Insufficient freeboard for all structures.
- Piping development at two locations.
- Potential for loss of dam integrity by toppling of trees on the crest and downstream slope.
- Insufficient discharge capacity to prevent overtopping.

To satisfy these basic requirements the following tasks should be undertaken as soon as possible and in the order of importance:

1. The reservoir water levels be lowered as a precaution.
2. Remove the existing structure and install a new dam structure that will control flow into the stream downstream from the dam.

The following paragraphs describe each of the above alternatives and provide estimated construction costs.

5.1 Lower Water Levels

As a matter of public and environmental safety, consideration should be give to lowering the water levels in the Williams Lake approximately two feet below the top of the dam. This elevation would be approximately 58.5 feet. This water level should be maintained until the full scope of mitigation items have been completed or addressed.

To maintain this water level a temporary pump and piping should be installed. A sedimentation structure should be installed at the outlet to prevent sediment from the pumping operation entering the stream. The pump would require a level indicator to activate the pump as the water rises above elevation 58.5 feet.

5.2 New Dam Construction

The existing dam does not satisfy the requirements of the Canadian Dam Association (CDA) "Dam Safety Guidelines". There is no method that could be developed to reinforce or modify the existing dam configuration. When the original dam was constructed the fill materials were placed on top of the existing grade and there does not appear to have been any attempt to create an impermeable layer inside the dam. There was also no attempt to create a key into the bedrock.

As indicated in the Geotechnical Report the elevation of the bedrock surface is relatively close to the top of the dam elevation. Therefore any successful dam constructed at this location must include a seal between the dam and the bedrock surface. This requirement will determine the type of structure that could be constructed at this location.

The second requirement for a dam constructed at this location will be to maintain a minimum flow to the downstream river and retain water levels in Williams Lake to ensure the full potential of this lake is maintained during dry summer months.

Therefore the following are the requirements that any new dam constructed at this location must meet or exceed:

- Create watertight seal to the bedrock.
- Control structure to provide minimum flow downstream and to regulate lake levels.
- The new dam should satisfy recreational uses around the dam.
- Satisfy safety criteria.
- The dam must be a low maintenance facility.
- Dam should be aesthetically pleasing and suit the environment.

A number of alternative dam designs and configurations were reviewed for this location. Most of these dams did not satisfy all the requirements as indicated above.

The dam configuration most suitable for this site and also the most economical, is a dam structure very similar to the existing dam. The existing dam presently operates as a broad crested weir, however it was never properly designed to resist the water flows during storm events. The existing dam also does have any control of the flow.

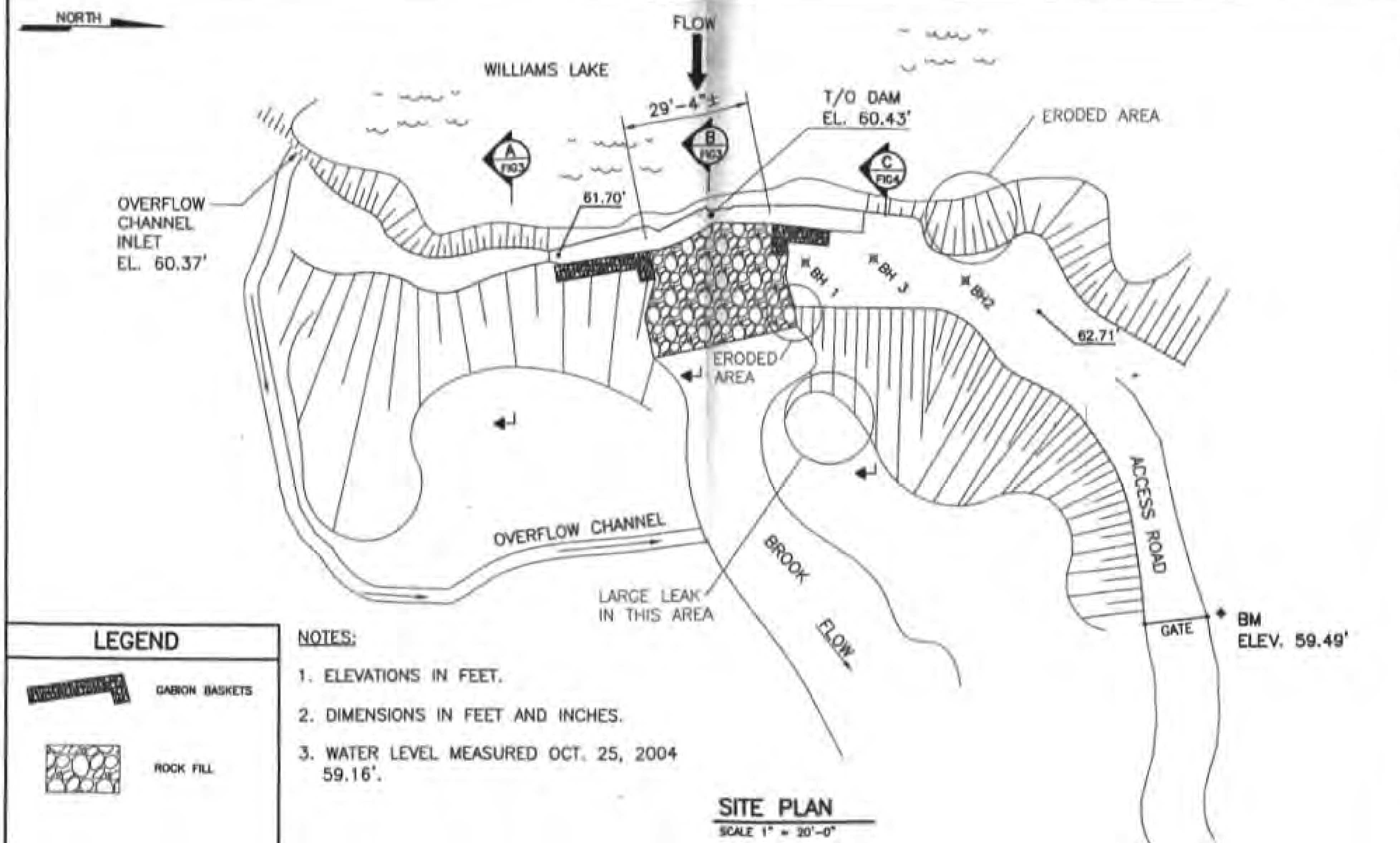
The intent would be to design a dam that would be stable during the IDF and prevent constant leaking through and under the dam which would ensure water was retained in Williams Lake. The dam would be overtopped during storm events such as 1:10, 1:100 and the 1:100 year.

Refer to Figure 5 of Appendix B that indicates a plan of the proposed dam. Sections through the dam are shown on Figure 6 of Appendix B. As indicated on these sections the broad crested weir dam would consist of a concrete core and rockfill on either side of the core. The concrete core was selected as the only feasible means to seal the dam to the bedrock. The rock placed on the downstream slope will be designed to resist the flows as a result of the IDF. A simple control structure consisting of three 8" x 8" timbers used as stop logs enable flows to be adjusted in the stream.

The construction cost estimate for this dam configuration would be approximately $ 275,000. This estimate includes demolition of the existing dam and environmental measures that would be required to prevent silt from entering the stream or bedrock during construction.

APPENDIX B
Drawings

LEGEND				
EXTENT OF OF WATERSHED	William's Lake Conservation Company Purcell's Cove, Nova Scotia, Canada	YMCL Engineering Limited	WILLIAMS LAKÉ DAM EVALUATION	MARCH 2005
			LOCATION PLAN AND APPROXIMATE WATERSHED AREA	Figure 1



LEGEND

GABION BASKETS

ROCK FILL

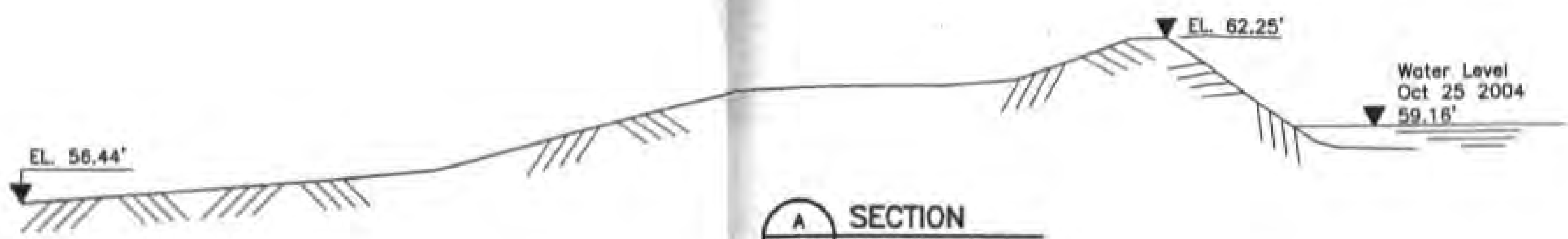
NOTES:

1. ELEVATIONS IN FEET.
2. DIMENSIONS IN FEET AND INCHES.
3. WATER LEVEL MEASURED OCT. 25, 2004
   59.16'.

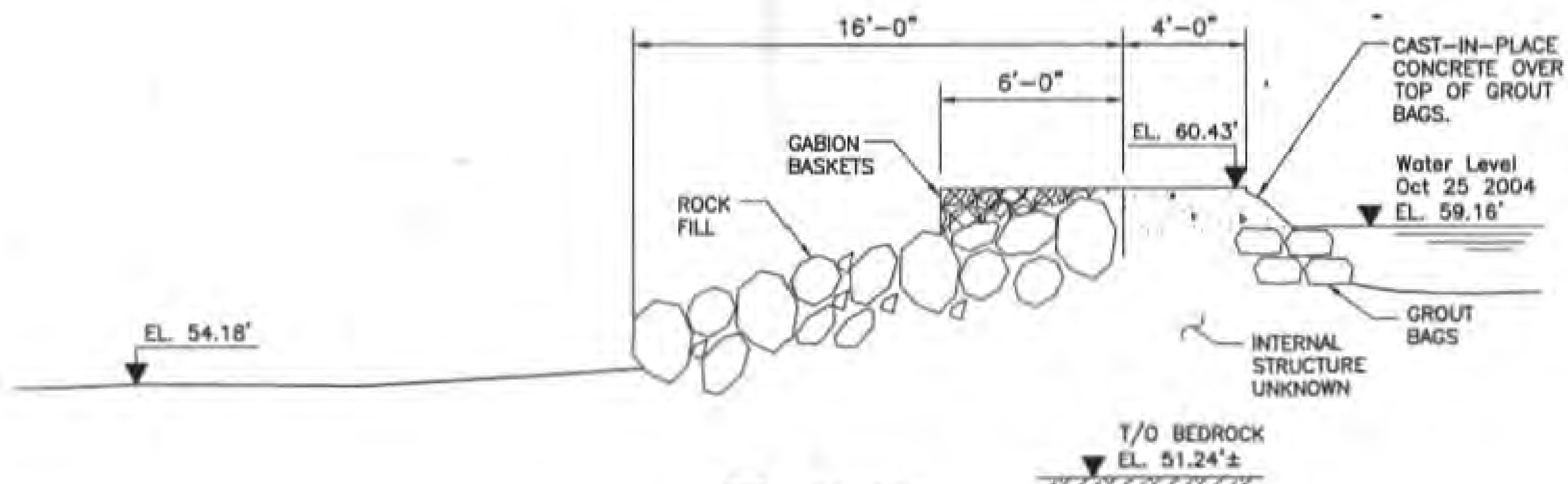
SITE PLAN

SCALE 1" = 20'-0"

William's Lake Conservation Company Purcell's Cove, Nova Scotia, Canada	YMCL Engineering Limited	WILLIAM'S LAKE DAM EVALUATION	NOVEMBER, 2004
		EXISTING DAM SITE PLAN	Figure 2

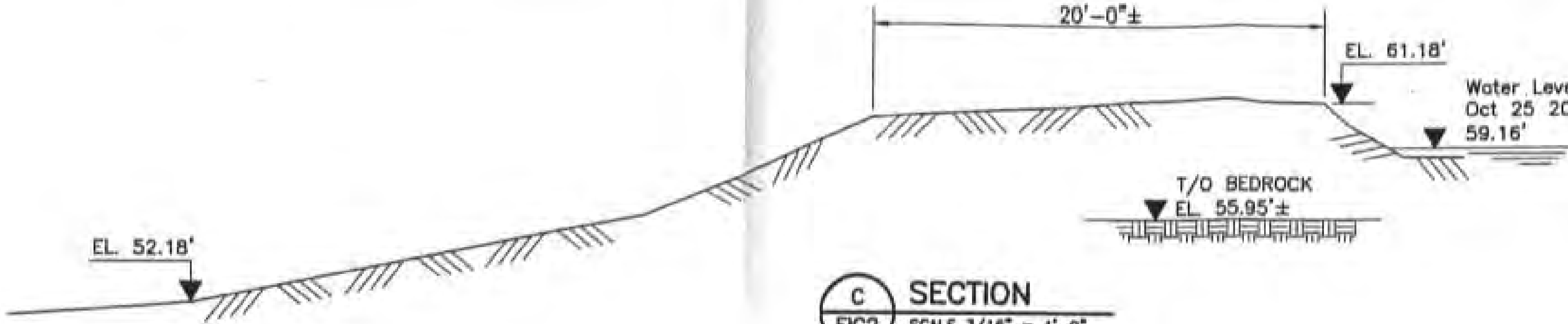


A
FIG2
SECTION
SCALE 1/4" = 1'-0"



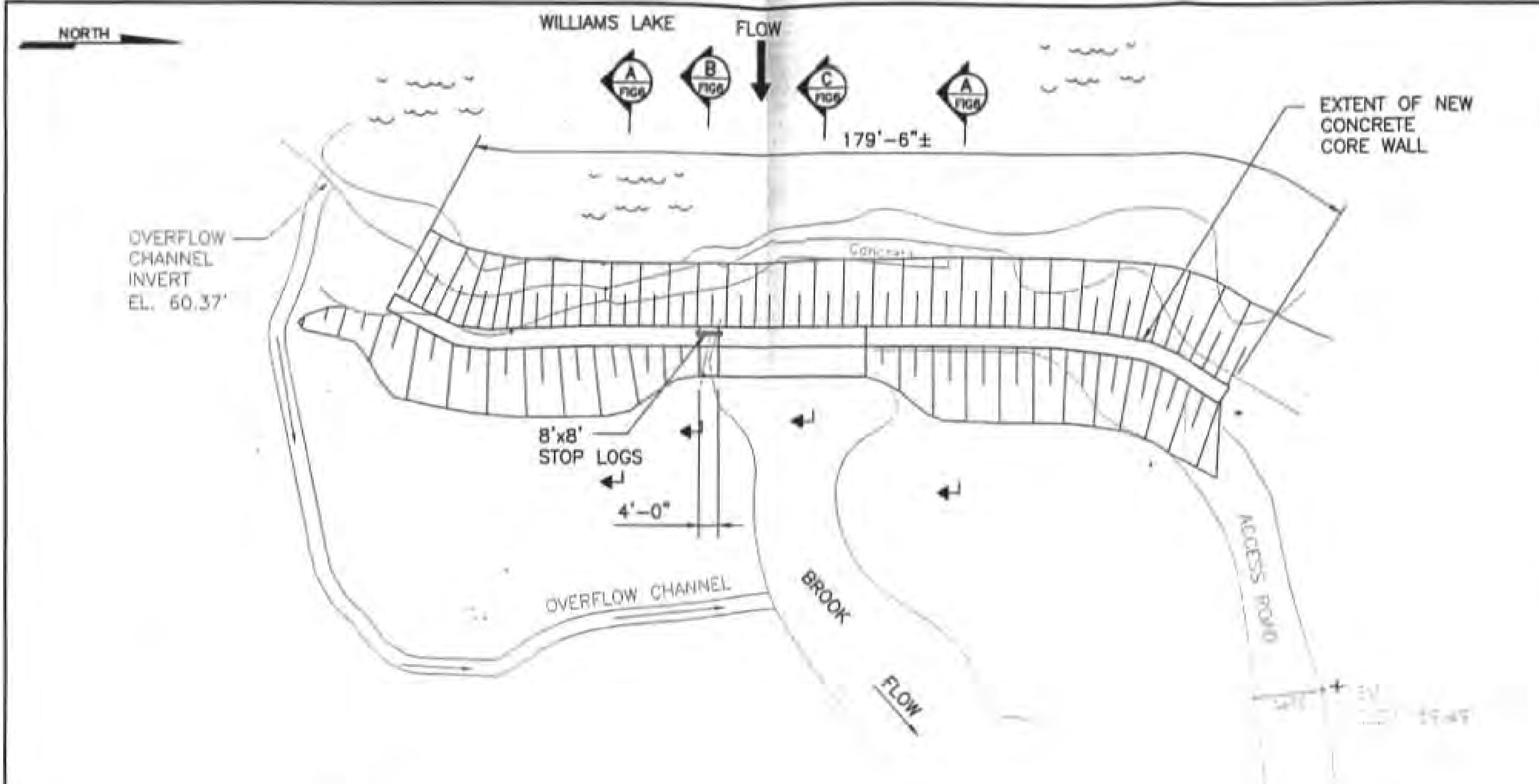
B
FIG2
SECTION
SCALE 1/4" = 1'-0"

William's Lake Conservation Company Purcell's Cove, Nova Scotia, Canada	YMCL Engineering Limited	Project Title: WILLIAM'S LAKE DAM EVALUATION Drawing Title: EXISTING DAM SECTIONS	Date: NOVEMBER, 2004 Figure 3



C
FIG2
SECTION
SCALE 3/16" = 1'-0"

William's Lake Conservation Company Purcell's Cove, Nova Scotia, Canada	YMCL Engineering Limited	Project Title: WILLIAM'S LAKE DAM EVALUATION	Date: NOVEMBER, 2004
		Drawing Title: EXISTING DAM SECTION	Figure 4

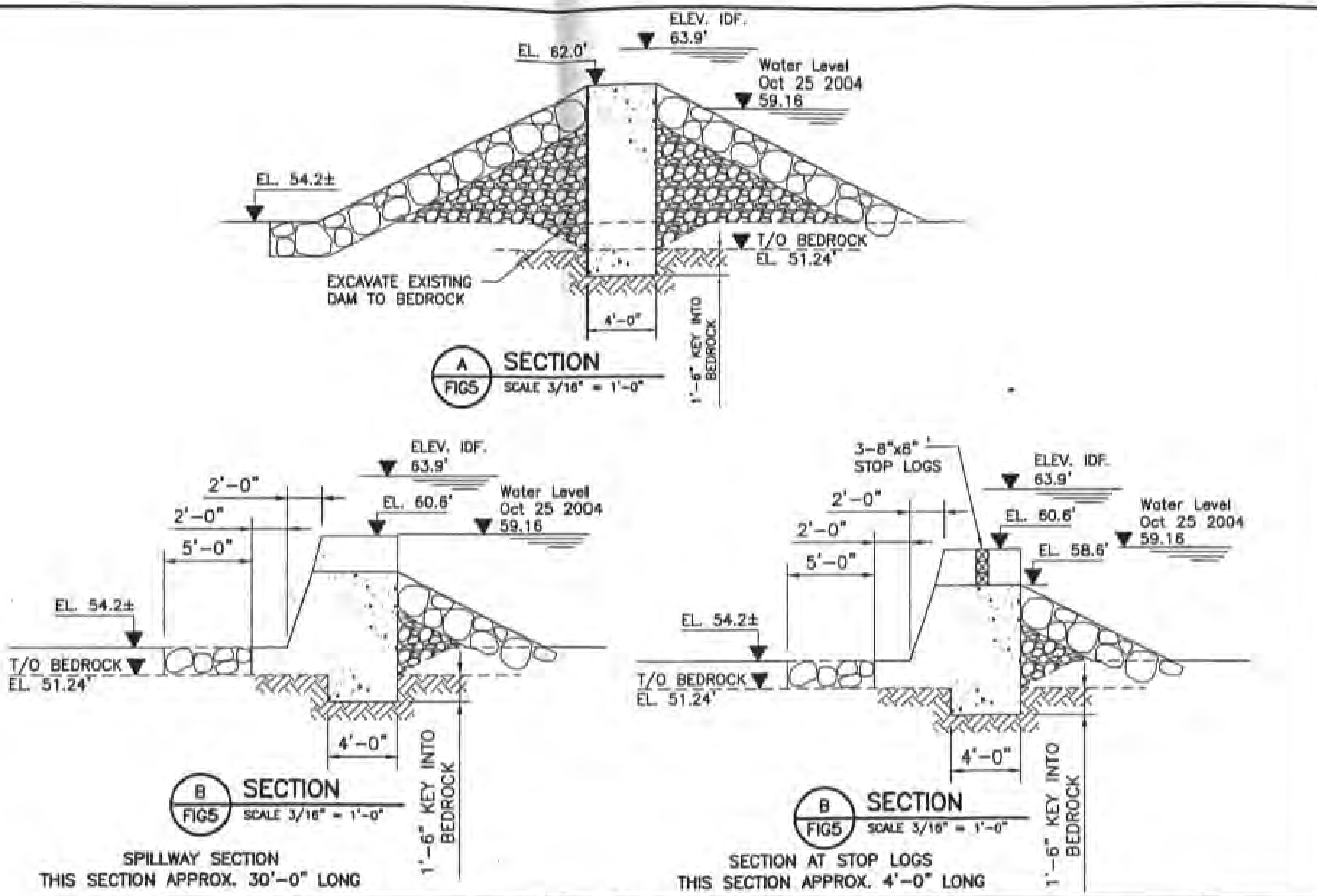


NOTES:

1. ELEVATIONS IN FEET.
2. DIMENSIONS IN FEET AND INCHES.

SITE PLAN
SCALE 1" = 20'-0"

William's Lake Conservation Company Purcell's Cove, Nova Scotia, Canada	YMCL Engineering Limited	Project title: WILLIAM'S LAKE DAM EVALUATION	Date: NOVEMBER, 2004
		Drawing title: SITE PLAN (POSSIBLE NEW DAM)	Figure 5



William's Lake Conservation Company
Purcell's Cove, Nova Scotia, Canada

YMCL
Engineering Limited

Project Title: WILLIAM'S LAKE DAM EVALUATION

Drawing Title: DAM SECTIONS (POSSIBLE NEW DAM)

Date: NOVEMBER, 2004

Figure 6

APPENDIX C
Flood Study

Williams Lake
Outlet Dam Flood Elevation Estimates

Williams Lake is located just outside the urbanized area of Halifax, near Purcell's Cove. The purpose of this work is to determine supportable estimates for flood levels in the Lake resulting from design storms at the 100 and 1000 return period intervals, for the purposes of allowing a dam safety evaluation to proceed, and to inform design of a replacement dam at a later date.

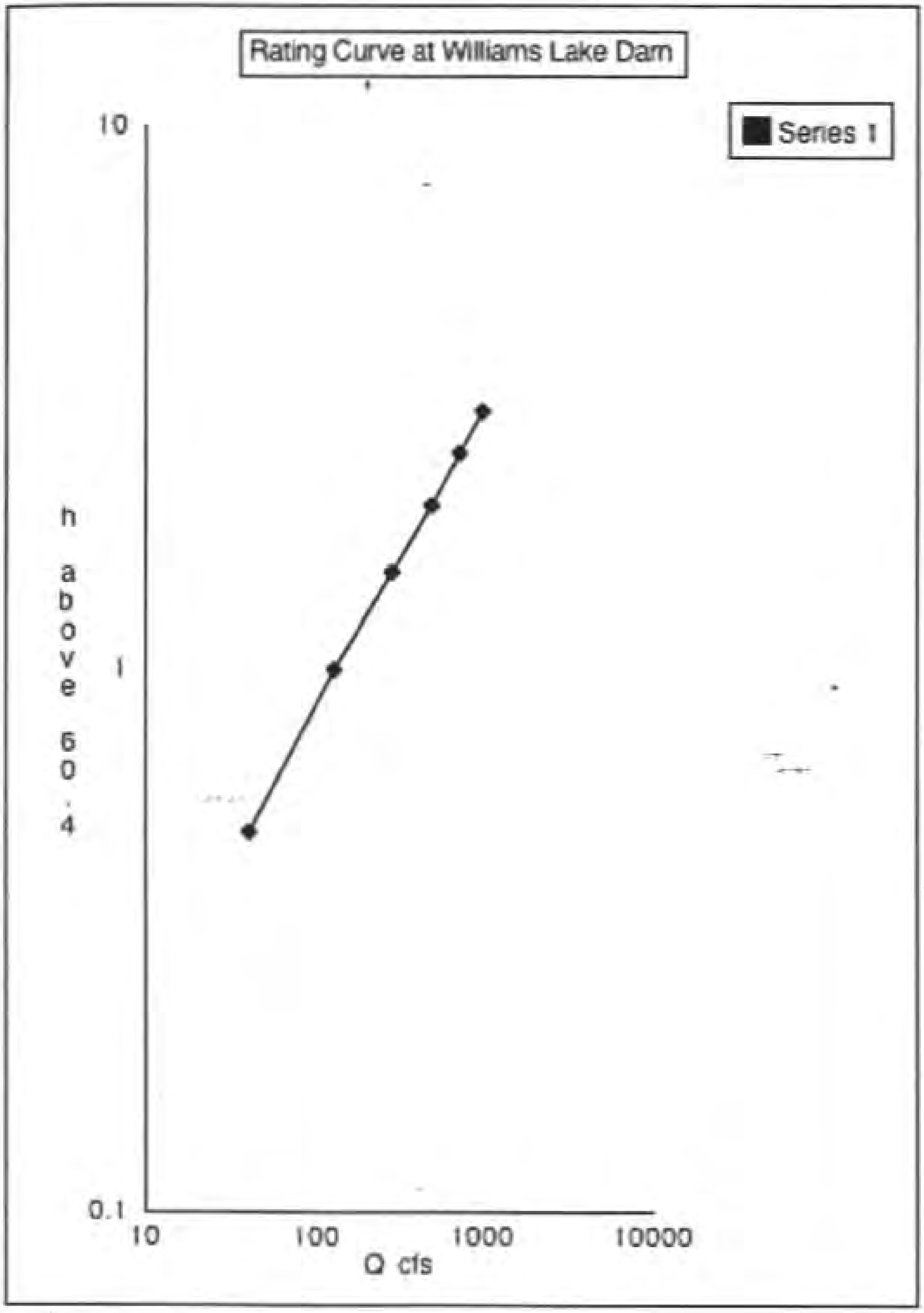
No flow measurement data was available for the watershed, although information on the outlet structure, and water level measurements made by a resident were available. The former was used in this work to develop a hydraulic rating curve for the dam. The latter serves an anecdotal support for the conclusions reached by this work.

The evaluation considered various means of accomplishing the desired objectives, and the following process was undertaken. The outlet structure and valley section was examined, and a stage – discharge relationship calculated for the cross section there, using an approximate model section from Measurement of Peak Discharge at Dams by Indirect Method, by Harry Hulsing, as published by The United States Geological Survey (USGS). This allows one to estimate the level the lake will rise to behind the structure to crate a given flow passing over it. The curve is referred to herein as a "Rating Curve".

The next step was to estimate theoretical peak flows that would be expected to be discharged from the Williams Lake watershed in response to different design storm events. Three methods were used, and compared to each other. The first used was the USGS "Techniques for Estimating Magnitude and Frequency of Floods in Rural Basins of Georgia" from a region where the geomorphology was similar to this – minimal soil cover, rocky terrain, and low infiltration. The second used was The Rational Method, using a SCS method for calculating the Time of Concentration in the watershed. Finally, after these theoretical methods yielded remarkably similar results, we compared those results with information from the Water Survey of Canada, in their Preliminary Examination of the Flood Following Hurricane Beth, a storm that occurred in 1971 commonly accepted as the closest recorded event we have to a 100 year storm event. As part of that work, we also used the Regional Design Flood Envelope Curves that have been in use in Nova Scotia since 1966.

Developing the Rating Curve

The rating curve developed for the outlet cross section flows:



This rating curve was calculated incrementally, assuming the same cross section through the dam/embankment, but adding the contribution of each rise in stage to the flow, using the formula:

$$Q= CbH^{3/2}$$

Where:

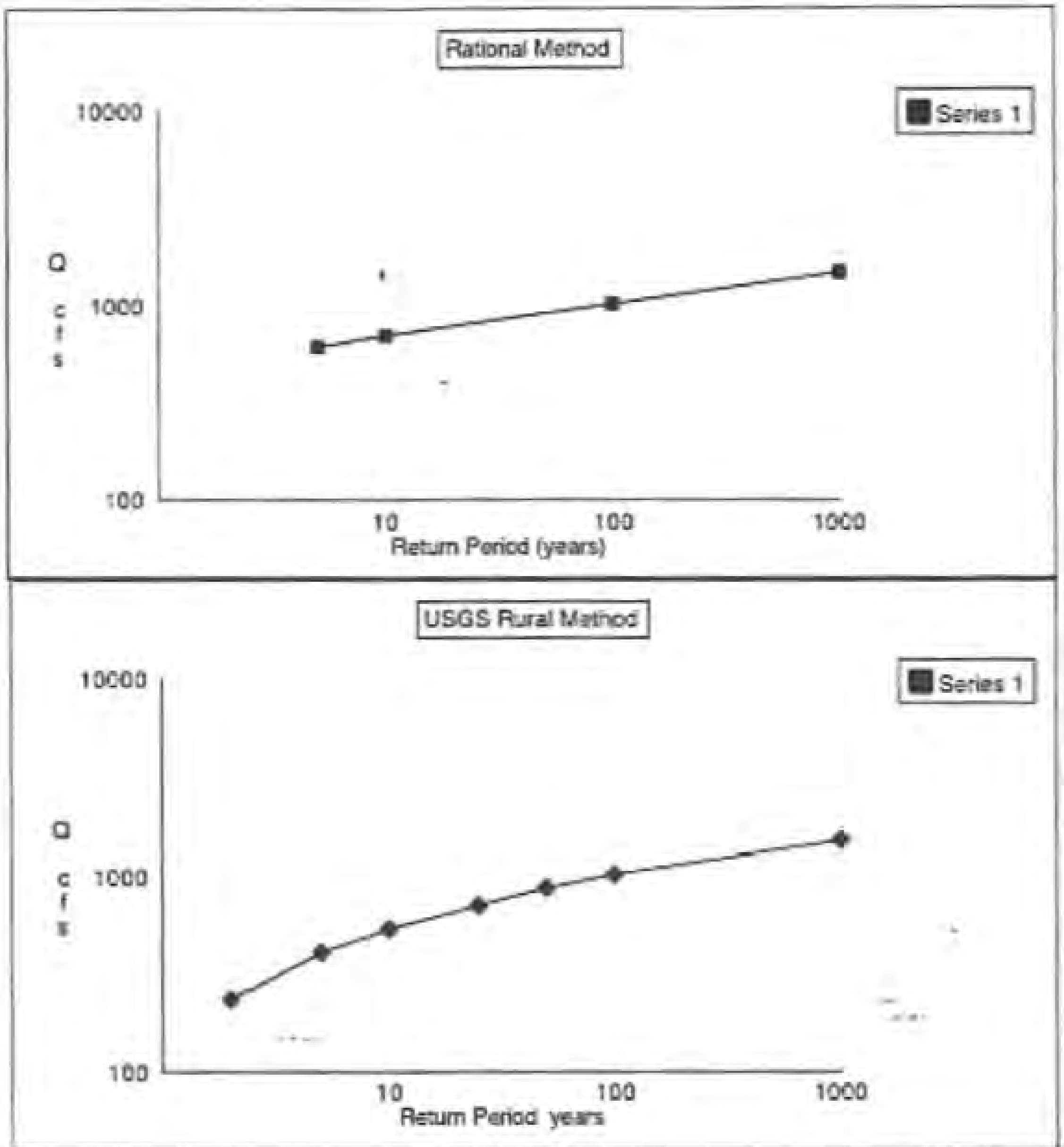
Q is the outflow in cubic feet per second (cfs)
B is the width of the opening in the structure
H is the height above the base elevation, and
C is a coefficient calculated for the cross section which varies with the depth of flow.

The current structure passes water as a fairly high rate (we estimated about 700 gallons per minute, or about 1.9 cfs). The elevation of the bottom of the spillway is 60.4 feet, which is close to the low point on the concrete dam crest. For the purposes of this evaluation, it was assumed that this would be the initial elevation of the lake, prior to a storm event.

Estimating Design Flows

The theoretical peak flows for the watershed form different design storm intervals were estimated using a rough method for predicting peak flows for different storms in the State of Georgia, and then those results are compared to the peak flows developed using the Rational Method and locally developed hydrological design storm information form the HRM Stormwater Design Manual.

The results of the two methods were strikingly similar, and are compared on the following charts.



The flows for the 1000 year return period storm have been graphically extended on these charts, and both arrive at 1500 cfs.

These flows are large. The 100 year peak flow form the Rational Method is 1020 cfs, or 382,000 gallons per minute. The flow from the USGS method is essentially identical.

Checking with Real Data

In 1971, Hurricane Beth devastated much of Nova Scotia. Almost 12 inches of rain fell in a little over two days. In a post storm assessment of the impact of Beth, the Water Survey of Canada reviewed flows form Beth in gauged watersheds along the southwestern and eastern shores, where much of the storm has hit.

Prior to the storm, in 1966, a consultants report for the WSC had provided the following Flood prediction equation for watersheds in South West Nova Scotia:

$$Q = 300A^{3/4}$$

Where:

Q is the Maximum Mean Daily Design Flood Flow in CFS
A is the Drainage Area in square miles

And where the equation is applicable for use when a reservoir is full prior to a storm.

For the watersheds in the Province that were gauged to measure flow during Beth, compared to the peak flow predicted by the above equation, only those which really got the major rain of close to one foot exceeded the predicted peak flow. The most they did so was by 28%.

To provide some comparison of this somewhat less theoretical assessment, we can calculate the peak flow the WSC would calculate for the Williams Lake Watershed as:

$$300 \times 1.504^{0.75} = 407 \text{ cfs}$$

Assuming that it would be exceeded at the highest observed variance form the WSC equations experienced with Beth, we can increase this flow by 1.28 and get approximately 520 cfs for a 100 year peak flow from the Williams Lake watershed. This is a little more than one half of the theoretical 100 year peak design flow from both earlier methods employed.

Predicting Lake Levels

Returning to the rating curve, with the different design flows available, we can now estimate the level to which water behind a competent dam would rise to allow these flows to pass through and over the structure. Flow is in cfs, and elevation is in feet.

Design Storm	USGS Method		Rational Method		Beth	
	Flow	Elevation	Flow	Elevation	Flow	Elevation
10	528	62.4	700	62.8	360*	62.1
100	1016	63.4	1020	63.4	520	62.4
1000	1500	63.9	1500	63.9	702**	62.8
PMF**	4480	66.9	4500	66.9	3095	65.6

* projected from the 100 year flow using the ratio of 10/100 from the rational method assessment

** projected from the 100 year flow using a 1000/100 ratio of 1.35 and a PMF/100 ratio of 4.41 (averaged from previous NS work)

Does this make sense?

The first thing we can observe is that the results from the theoretical methods appear to be somewhat conservative. The empirical method yields results about 50% of the flow, and 1 to 2 feet lower in lake elevation.

At the dam, an elevation of 63.9 would be well up on the banks on either side of the valley section. This is probably affected by the lack of information in the valley section as the flow tops the dam – our rating curve would assume vertical walls on either side of the section, 229 feet wide. But even the empirical model we have developed using Hurricane Beth would result in overtopping of the existing dam in a 100 year event.

The 100 year event differs between the theoretical and empirical methods by only a foot of water on the lake. At the 1000 return period event, the difference is still only about a foot.

It is in predicting the more extreme event, a Probable Maximum Flood, where more detailed modeling is required. We have shown a flow and stage from the Probable Maximum Flood using ratios to the 100 year runoff event to the PMF from other work done in Nova Scotia. As expected, the differences remain similar. The flows estimated for the PMF in this case also work out to a 1:1,000,000 year return period, another estimate for the PMF.

It is reasonable to assume that the larger numbers developed in this work should be used for the evaluation of the dam safety aspects regarding overtopping and structural stability. However, at the same time, we suggest that the 100 year design flow of 520 cfs, developed from the empirical method, the Hurricane Beth Analysis, may be used for the design of spillway and crestflow capacity.

Respectfully submitted,

Jeffrey A. Pinhey, M.A.Sc., P.Eng.

APPENDIX D
Geotechnical Investigation

MEC REPORT P3010

GEOTECHNICAL ENGINEERING INVESTIGATION (GEI)
WILLIAMS LAKE DAM
NOVA SCOTIA

NOVEMBER 2004

REGISTERED PROFESSIONAL ENGINEER
DATE 4-NOV-04
P. MITCHELMORE
7468
PROVINCE OF NOVA SCOTIA

©MEC REPORT P3108

TO

YMCL ENGINEERING LIMITED
1809 BARRINGTON STREET
SUITE 1404, CIBC BUILDING
HALIFAX, NS B3J 3K8

ON

GEOTECHNICAL ENGINEERING INVESTIGATION
WILLIAMS LAKE DAM
NOVA SCOTIA

BY

MITCHELMORE ENGINEERING COMPANY LTD
14 ROBERT SCOTT DRIVE
LANTZ, NOVA SCOTIA
B2S 2A3
TEL: (902) 883-1177
FAX: (902) 883-7591

NOVEMBER 5, 2004

Table of Contents

1. INTRODUCTION

Mitchelmore Engineering Company (MEC) Ltd. was contracted by **YMCL Engineering** (YMCL) to complete a Geotechnical Engineering Investigation (GEI) of the dam located at the southeast corner of Williams Lake.

The present structure consists of earth fill abutments with an uncontrolled concrete overflow spillway. The level of Williams Lake is controlled by leakage beneath the existing structure and overtopping of the dam during floods. Leaking was observed beneath the concrete weir and the left abutment. The rate of leakage has reportedly increased in recent years and created concern for the structure and preservation of lake levels at Williams Lake.

The purpose of the work is to develop a geotechnical model of the dam area, to assist and guide engineering decisions related to dam safety and remediation options. The deliverable for the project consists of a report outlining the procedure adopted, and geotechnical characterization of the soil and bedrock observed.

1.1. Scope of Work

The scope of work for the site included the following:

- Review of available literature and development of the regional geological setting.
- Reconnaissance geotechnical survey consisting of walkover at the site to identify potential concerns.
- Three (3) Boreholes for assessment of subsurface soils, bedrock and groundwater conditions.
- A geotechnical report presenting the findings of the investigation.

1.2. Methodology

All aspects of the work were completed under the direction of senior geotechnical staff. Ms. Robyn Homans, EIT, of MEC, assisted with field supervision, sample preparation, field logs and report writing. The available literature included the following references

Stea, R.R. and D. Hemworth (1979). "Pleistocene Geology – Central Nova Scotia", Department of Natural Resources, Government of Nova Scotia.

Lewis, C.F.M., B.B. Taylor, R.R. Stea, G.B.J Faber, R.J.Horne, S.G. MacNiel and J.G. Moore (1998)."Earth Science and Engineering: Urban Development in the Metropolitan Halifax Region," from *Urban Geology of Canadian Cities*, GAC Special Publication Paper 42.

We had intended to review stereo pairs of aerial photographs of the area but none were available. Only one aerial photograph of the dam site was available, number 32 of roll 02319, taken in July 2002 at a scale of 1:10,000.

On May 28th, MEC performed a site walk over with Tom Mosher, P.Eng of YMCL to observe site conditions and discuss investigation requirements. A subsequent borehole investigation was completed Oct 20th, 2004. All fieldwork was performed under an engineer from MEC and detailed logs and representative samples were maintained and collected. A total of three (3) boreholes were put down using a wheel mounted rotary drill rig provided by Logan Geotech Inc of Stewiake., Use of a tracked drill rig was considered impractical for this site because of the potential damage to neighboring properties and landscapes during access.

Each borehole was completed into bedrock or suspected bedrock. Samples were recovered using the SPT method at regular intervals, where possible. The "N" value was recorded for each SPT test and the recovered soil sample logged. The drill rig was equipped with an automatic trigger for performance of the SPT test. Bedrock was cored using a double tube corebarrel and "NQ" size core. Core samples were also collected from BH101 and BH102 and stored in coreboxes. Samples will be stored until February 20 2004 at which time they will be discarded unless instructions to the contrary are received. The topography and borehole locations were provided by YMCL and all elevation data presented by MEC corresponds to the survey results provided.

Due to access restrictions, all three boreholes were completed in the left (looking downstream) abutment of the dam. The initial intention was to install a borehole in the right abutment and/or the concrete weir of the dam but concern over the potential to rupture a hydraulic line in the drill rig and the consequent environmental effects negated the potential benefits.

2. GEOTECHNICAL MODEL

2.1. Geological Setting

The site is located at the southeasterly end of William's Lake where the lake exits into a small stream and eventually spills into Northwest Arm. The lake is located in Spryfield within the Halifax Regional Municipality (HRM). Access to the site is provided by a trail via Purcell's Cove Road. In order to access the trail, a small traverse of private property is required.

The terrain in the vicinity of the dam and for much of the Williams Lake area is bedrock controlled. The dam and stream appear to be near the bottom of the Point Pleasant Syncline giving the terrain a "half-barrel" shape. The underlying bedrock is the Halifax Formation, which consists of low grade metamorphic slates and sandstones. Bedrock controlled terrain is typically rougher, with more sharp edges than hummocky and undulating soil controlled terrain. This physiographic feature is evident at the Williams Lake dam. Bedrock controlled terrain is also typically associated with shallow, or near surface, bedrock. This is also evident at the Williams Lake dam and demonstrated during the borehole investigation.

The surface soils are generally believed to be glacial till soils of the more recent Mid-Late Wisconsinian age. They are characterized a silty sands with numerous cobble and boulder sized materials, typical of a Lawrencetown Till.

2.2. Stratigraphy

Each soil is given an engineering classification based on the Unified Soil Classification System (USCS) (Wagner, 1957). The USCS classifies soils based on physical properties and is commonly used in engineering practice. All soils are classified based on visual methods. No laboratory classification tests were performed as part of the scope of work.

2.2.1. Topsoil (OH)

Topsoil consisting of dark brown silty SAND (OH) interspersed with black organic material from vegetation was encountered in all BH locations. The thickness of the stratum varied from 0.61 m in the locations of BH101 & BH103 and 0.47 m in the location of BH 102. Based on the SPT N-values, the insitu relative density is classed as loose to compact.

2.2.2. Silty SAND with Gravel (SM)

A dark brown silty SAND with gravel (SM) was encountered in BH101 and BH103 where it was 0.61 m below ground surface with thicknesses varying from 0.61 m to 0.93 m. In BH103, the soil overlay bedrock whereas in BH101 it was underlain by another soil. The stratum is believed to be a fine grained fill soil likely used in construction of the abutment comprising the dam. Based on the SPT N-values, the insitu relative density is classed as compact to dense.

2.2.3. Boulder Till ??

A dense, boulder strewn soil unit was encountered in BH101 below the silty SAND with gravel described above. Despite several attempts at sampling with the SPT sampler, no samples were recovered for analysis. The soil unit is inferred as a typical till soil unit for the area with a high concentration of boulder and cobble sized particles. At BH101, the only location in which it was encountered, the unit was 1.22 m below ground surface, 1.55 m thick and overlying bedrock. The stratum is believed to be the natural foundation soil for the site and a typical till soil unit for the area, with a high concentration of boulder and cobble sized particles.

2.2.4. Bedrock (BR)

Based on observations during the investigation, bedrock depth was noted in all borehole locations. Depth to bedrock varied from 0.47 m to 2.79 m below ground surface at BH102 and BH101, respectively. Bedrock observed at both boreholes was a bluish gray, medium grained metasandstone of the Halifax Formation. It is qualitatively assessed as medium strong with close joint spacing.

2.3. Groundwater

Use of water during drilling in soil and rock will alter natural groundwater levels. For more precise water level readings, water level monitoring devices were installed in BH101 and BH102 to observe the steady state water levels.

Water levels were observed on October 27th, 2004. BH101, with the monitoring zone between 3.7 m and 1.2 m below ground surface, had a water level of 0.9 m below ground surface. BH102, with the monitoring zone between 2.79 m and 0.6 m below ground surface, had a water level of 1.5 m below ground surface. At the time of the water level readings, the lake level was approximately 0.4 m below the same ground surface.

Groundwater levels may fluctuate seasonally and in response to meteorological events. The levels reported above are only applicable to the time they were recorded.

APPENDIX A

Location Plan P3010-001

Synbols and Terms

Borehole Logs BH101, BH102, BH103

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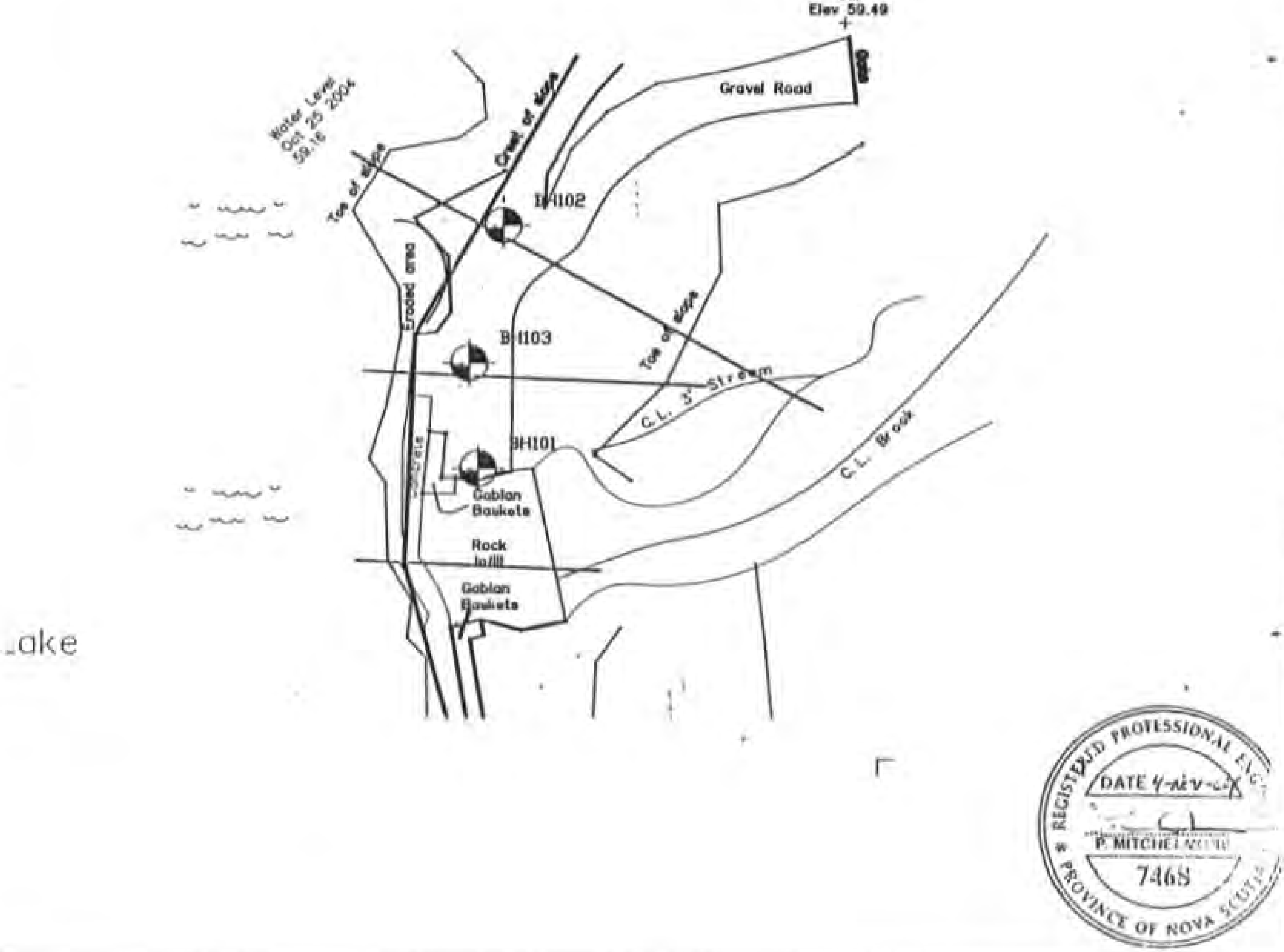
INNOVATION RELIABILITY SERVICE-ORIENTED



1	Final Report	4 Nov 04
REVISION	DESCRIPTION	DATE

CLIENT:

YMCL Engineering

PROJECT:

WILLIAMS LAKE DAM

TITLE:

LOCATION PLAN

DESIGNED BY: P.M.	DRAFTED BY: R.M.H
DATE: 04/11/04	SCALE: NTS
DRAWING No:	FIGURE 1

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

ENRAL DESCRIPTION

evations

fer to the Datum indicated on the Borehole or Test Pit record.

epth

l depths are given in meters measured from the ground surface unless otherwise noted.

mple Type

e first letter describes the sampling method and the second, the shipping container.

Sampling Method

A – Split Tube	E – Auger
B – Thin Wall Tube	F – Wash
C – Piston Sampler	G – Grab Sample
D – Core Barrel	H –

Shipping Container

N – Insert	S – Plastic Bag
O – Tube	U – Wooden Box
P – Water Content Tin	Y – Core Box
Q – Jar	Z – Discarded
R – Cloth Bag	

mple No.

mples are numbered consecutively in the order in which they were obtained from the borehole or test pit.

mple Size

mension is in millimeters and refers to the nominal diameter of the sample.

mple Retained

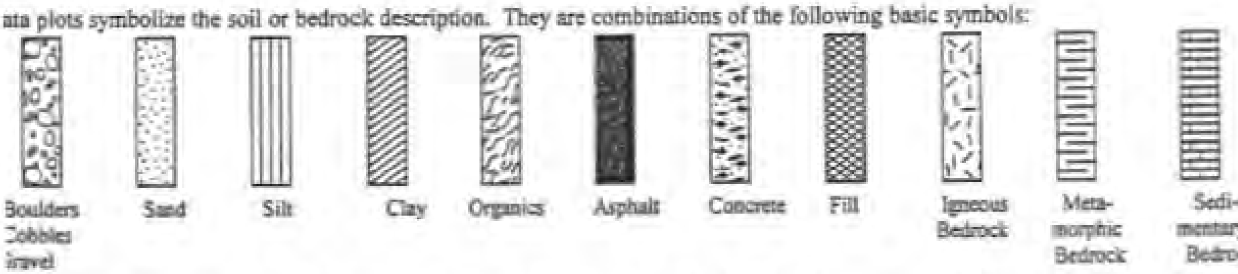
licates the length in millimeters of sample retained in the sampler.

breviations

A – Not applicable
E – Not Encountered
O – Not Observed

ata Plot

ata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



id lines between strata indicate the boundary between different strata. Dashed lines between strata indicate the boundary between strata is erred.

SOIL DESCRIPTION

Standard Penetration Test 'N-Value'

The performance of the Standard Penetration Test provides an 'N-value'; the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (51 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration is achieved and 'N' values cannot be determined, the number of blows is reported over sampler penetration in millimeters. The 'N-Value' reported, is as recorded in the field (i.e. non corrected).

Grain Size

Soil	Grain - Size
Clay	<0.002mm
Silt	0.002 – 0.075mm
Sand	0.075 – 4.75mm
Gravel	4.75 – 75mm
Cobbles	75 – 200mm
Boulders	>200mm

Soil Description

The terminology describing materials based on the visually estimated proportion of these materials present:

Trace, or occasional	< 10%
Some	10 – 20%
Adjective	20 – 35%
And	35 – 50%
Noun	> 50%

Relative Density of Cohesionless Soils

The standard terminology to describe cohesionless soils includes the compactness (formerly "relative density"), as determined by laboratory test or by the Standard Penetration Test 'N- value'.

Density	N-Value
Very Loose	< 4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	> 50

Consistency of Cohesive Soils

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, unconfined compression tests, or occasionally by standard penetration tests.

Consistency	Undrained Shear Strength		N-Value
	psf	kPa	
Very Soft	< 250	< 12.5	< 2
Soft	250 -500	12.5-25	2-4
Firm	500 - 1000	25-50	4-8
Stiff	1000 - 2000	50-100	8-15
Very Stiff	2000 – 4000	100-200	15-30
Hard	> 4000	> 200	> 30

asticity/ Compressibility

		Liquid Limit(%)
w plasticity clays	Low compressibility silts	<30%
edium plasticity clays	Medium compressibility silts	30-50%
gh plasticity clays	High compressibility silts	>50%

ROCK DESCRIPTION

Total Core Recovery (TCR)

Sum of the lengths of rock core recovered from a core run, divided by the length of the core run and expressed as a percentage.

Rock Quality Designation (RQD)

The classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be applied to NW core; however, it can be used on different core sizes if most of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures.

RQD (%)	Rock Quality		
90-100	Excellent	-	intact, very sound
75-90	Good	-	moderately jointed, massive, sound
50-75	Fair	-	fractured, blocky and seamy
25-50	Poor	-	severely fractured, shattered and very seamy or blocky
0-25	Very poor	-	very severely fractured, crushed

Terminology

Spacing (mm)	Bedding, Laminations, Bands	Discontinuity
2000-6000	Very Thick	Very wide
600-2000	Thick	Wide
200-600	Medium	Moderately close
60-200	Thin	Close
20-60	Very Thin	Very close
< 20	Laminated	Extremely close
< 6	Thinly Laminated	

Rock Strength

Grade	Classification	Unconfined Compressive Strength (MPa)
R0	Extremely Weak	< 1
R1	Very Weak	1 - 5
R2	Weak	5 - 25
R3	Medium Strong	25 - 50
R4	Strong	50 - 100
R5	Very Strong	100 - 250
R6	Extremely Strong	> 250

Weathering State

Symbol	Term	Description
W1	Fresh	No visible sign of rock weathering. Slight discoloration along major discontinuities
W2	Slightly Weathered	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored
W3	Moderately Weathered	Less than half the rock is decomposed and/or disintegrated into soil.
W4	Highly Weathered	More than half the rock is decomposed and/or disintegrated into soil.
W5	Completely Weathered	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

MITCHELMORE ENGINEERING COMPANY LTD.

PROJECT # P3010

YMCL Engineering Limited

Williams Lake Dam

Borehole Log

Borehole No. **BH 101**

Sheet 1 of 1

Site : Williams Lake Dam

Location : Inside L of Gabion

Reference Elevation : -

Datum : Geodetic

Contractor : Logan Geotech Inc.

Equipment : 16 Ton Rubber Tire Drill Rig

Date Started : 20-Oct.-04 Finished : 20-Oct.-04

Drilling Method: Soil : Drill

Rock : Drill

Drill Size:

	Depth	Elevation
Ground Surface :	0.0	18.41m
Bedrock Surface :	2.79m	15.62m
End of Hole :	4.34m	14.07m
Groundwater Level :	0.90M	NE

Date: 27/10/2004

Weather : Overcast with Sunny Periods, Dry

Vegetation : Maple and Spruce trees, shrubs and leaves covering the ground surface

Comments : T. Mosher on site for YMCL

Depth (Elev.) (m)	Soil Class	Description	No.	Type	Recovery (mm)	N-Value Or RQD(%)	Other Tests
0.61m	OH	Loose, dark brown silty SAND (OH) with black organic matter, some gravel, moist. – TOPSOIL	1	AS	90mm	8	
(17.80m) 1.22m	SM	Compact, dark brown, silty SAND (SM) with some gravel, moist.	2	AS	70mm	14	
(17.19m)		Large bolders encountered, casing drilled through, ground vibrating. Attempts at sample recovery using SPT method unsucessful. Soil type is inferred as a boulder strewn glacial til typical of the area.					
2.79m			3	AS	0	18	
(15.62m)	HFX FMT	Bluish gray, medium grained metasandstone, strong, of the Halifax Formation.	4	DY	180mm	–	
			5	DY	40mm	–	
			6	DY	150mm	–	
4.34m			7	DY	370mm	72.2%	
(14.07m)		End of Borehole Monitoring well installed from 1.1m to 4.1m.					

Water Content/ Atterberg Limits Standard Penetration Tests: 20 40 60 80

Sampling Method :
A – Split Tube
B – Thin Wall Tube
C – Piston Sampler
D – Core Barrel
E – Auger
F – Wash
G – Grab
H – Bulk

Shipping Container :
N – Insert
O – Tube
P – Water Content Tin
Q – Glass Jar
R – Cloth Bag
S – Plastic Bag
T – Metal Can
Y – Core Box
Z – Discarded

Graph Legend :
● Water Content (%)
Atterburg LimitS (%)
☐ Standard Penetration Test N-Value (Blows/0.3m)

Inspector : R. Hamans

Logged By : R. Hamans

Approved :

Date : 4-Nov-04

MITCHELMORE ENGINEERING COMPANY LTD.

PROJECT # P3010

YMCL Engineering Limited

Williams Lake Dam

Borehole Log

Borehole No.

BH 102

Sheet 1 of 1

Site : Williams Lake Dam

Location : Leftside Earth Embankment

Reference Elevation : .

Datum : Geodetic

	Depth	Elevation
Ground Surface :	0.0	18.69m
Bedrock Surface :	0.47m	18.22m
End of Hole :	2.79m	15.90m
Groundwater Level :	1.5m	NE

Date: 27/10/2004

Contractor : Logan Geotech Inc.

Equipment : 16 Ton Rubber Tire Drill Rig

Date Started : 20-Oct.-04 Finished : 20-Oct.-04

Drilling Method: Soil : Drill

Rock : Drill

Drill Size:

Weather : Overcast with Sunny Periods, Dry

Vegetation : Maple and Spruce trees, shrubs and leaves covering the ground surface

Comments : T. Mosher on site for YMCL

Depth (m)	Depth (Elev.) (m)	Graphic Log	Soil Class.	Description	Sample Data No.	Type	Recovery (mm)	N-Value Or RQD(%)	Other	Water Content/ Atterberg Limits Standard Penetration Tests 20 40 60 80
	0.47m		OH	Compact, dark brown silty SAND (OH) with black organic matter, some gravel, moist. – TOPSOIL	1	AS	400mm	23		□
	(18.22m)		HFX FMT	Bluish gray, medium grained, Metasandstone, strong of the Halifax Formation.	2	DY	265mm	–		
1					3	DY	220mm	–		
					4	DY	210mm	–		
2					5	DY	495mm	–		
					6	DY	180mm	–	HB	
	2.79m				7	DY	320mm	57.8%		
3	(15.90m)			End of Borehole Monitoring well installed from 0.6m to 2.79m.						
4										
5										
6										

Sampling Method :
A – Split Tube
B – Thin Wall Tube
C – Piston Sampler
D – Core Barrel
E – Auger
F – Wash
G – Grab
H – Bulk

Shipping Container :
N – Insert
O – Tube
P – Water Content Tin
Q – Glass Jar
R – Cloth Bag
S – Plastic Bag
T – Metal Can
Y – Core Box
Z – Discarded

Graph Legend :
● Water Content (%)
Atterburg LimitS (%)
□ Standard Penetration Test N-Value (Blows/0.3m)

Inspector : R. Homans

Logged By : R. Homans

Approved :

Date : 4-Nov-04

MITCHELMORE ENGINEERING COMPANY LTD.

PROJECT # P3010

YMCL Engineering Limited

Williams Lake Dam

Borehole Log

Borehole No. **BH 103**

Sheet 1 of 1

Site : Williams Lake Dam

Location : Leftside Earth Embankment

Reference Elevation : -

Datum : Geodetic

	Depth	Elevation
Ground Surface :	0.0	18.61m
Bedrock Surface :	1.55m	17.06m
End of Hole :	1.55m	17.06m
Groundwater Level :	NE	NE

Date: 27/10/2004

Contractor : Logan Geotech Inc.

Equipment : 16 Ton Rubber Tire Drill Rig

Date Started : 20-Oct.-04 Finished : 20-Oct.-04

Drilling Method: Soil : Drill

Rock : Drill

Drill Size:

Weather : Overcast with Sunny Periods, Dry

Vegetation : Maple and Spruce trees, shrubs and leaves covering the ground surface

Comments : T. Mosher on site for YMCL

Depth (m) (Elev.) (m)	Graphic Log	Soil Class.	Description	No.	Type	Recovery (mm)	N-Value Or RQD(%)	Other Tests	Water Content/ Atterberg Limits Standard Penetration Tests (20 40 60 80)
0.62m (17.99m)		OH	Compact, dark brown silty SAND (OH) with black organic matter, some gravel, moist. – TOPSOIL	1	AS	250mm	16		
1.55m (17.06m)		SM	Dense, dark brown, silty SAND (SM) with some gravel, moist.	2	AS	130mm	37		
				3	AS	160mm	17/ 150		
			End of Borehole Bedrock inferred by bouncing of SPT sampler. Bedrock was not proven by coring.						

Sampling Method :
A – Split Tube
B – Thin Wall Tube
C – Piston Sampler
D – Core Barrel
E – Auger
F – Wash
G – Grab
H – Bulk

Shipping Container :
N – Insert
O – Tube
P – Water Content Tin
Q – Glass Jar
R – Cloth Bag
S – Plastic Bag
T – Metal Can
Y – Core Box
Z – Discarded

Graph Legend :
● Water Content (%)
Atterburg Limits (%)
☐ Standard Penetration Test N-Value (Blows/0.3m)

Inspector : R. Homans

Logged By : R. Homans

Approved : [signature]

Date : 4-Nov-04