Q. Please provide details or reports of any programs Hydro has undertaken to assess the potential for future Supply Side Enhancements related to its hydraulic generating stations, including re-runnering, rewinds, existing plant refurbishments or water management projects. If this has not been examined, is this type of review expected to be a component of Hydro's consideration of supply side resources to meet supply constraints in the next 5 years?

A. Hydro has identified two possible runner projects for its hydraulic units. Attachments 1, 2 and 3 are reports referring to these two projects. The runner projects involve Hinds Lake and Bay d'Espoir Unit 7. Initial analysis indicated capacity improvements could be achieved. The net energy gains due to increased efficiency would be difficult to quantify as they are estimated as being less than 1% or within the error band of most transducers that would be used to calculate such gains. Hydro has performed a preliminary economic analysis of these projects and determined that at this time, to proceed would not be feasible. Hydro will perform an updated review when increased capacity is required to see if the projects economics have changed or if operational factors indicate a requirement for capital investment.

Hydro has identified four possible water management projects. Attachments 4, 5, 6, 7 and 8 are reports referring to these four projects. The projects involve an upstream regulating structure at Paradise River, a dyke at the outlet of Spruce Pond into Burnt Pond, a diversion of Kitty's Brook into Hinds Lake and a diversion of Lloyds River into Victoria Lake. The Paradise River project was reviewed in 1994, however, due to increased capital costs, environmental costs and lower fuel forecasts that were not included in the original report the project was not considered to be economically feasible at that time. The dyke at Spruce Pond was presented as an alternative to other works to increase flood handling at Burnt Pond. Hydro elected not to proceed with this alternative. The Kitty's Brook diversion project was reviewed in early 2006. Environmental impact studies and mitigation efforts were not included in the original report and their costs are believed to be significant. Due to the high capital and environmental costs this project remains uneconomical at this time. The Lloyds River diversion project has presented many environmental issues that have rendered the project not feasible.

FILE: 113.80.20/46.00

HYDRO
THE POWER OF COMMITMENT

HINDS LAKE GENERATING STATION RUNNER REPLACEMENT

REGISTERED PROFESSIONAL ENGINEER
J. E. MALLAM
SIGNATURE
PROVINCE OF NEWFOUNDLAND

	Mechanical
	Electrical
	N.A. Civil
Approved:	N.A. P&C

Generation Engineering
2004-03-31

SUMMARY

This report summarizes a review of the technical and financial aspects associated with replacing the existing Hinds Lake runner. Since installation, this runner has experienced serious cavitation damage and cracking. Discussions concerning alternate runner designs were initiated with American Hydro, a manufacturer which has established a reputation for supplying successful replacement runners in a wide range of hydro plants throughout North America. In addition to providing a new runner which would be free of cracking and considerably reduce, or eliminate, cavitation damage, American Hydro can increase the capacity of the plant by 16 MW. There would be a minor increase in annual energy production, resulting from a slight increase in runner hydraulic efficiency.

This report quantifies the benefits which would result from replacing the runner and identifies technical issues which must be investigated. It does not contain recommendations pertaining to the viability of the project, as this will be determined by System Planning.

TABLE OF CONTENTS

INTRODUCTION

Discussions with American Hydro concerning the Hinds Lake runner were initiated in 1999, with the intent of reducing the cavitation and eliminating the cracking problem. A detailed review of the drawings and computer analysis of the unit hydraulics indicated that a new runner could be designed which would achieve these two goals and provide increased capacity. Discussions proceeded over the following years and culminated in the receipt of three proposals from American Hydro. The most recent proposal was solicited following an internal Hydro review. Hydro was concerned that the transmission network could not accommodate the increased power offered by American Hydro. System Planning reviewed the capability of the network and presented its findings in a report titled “Transmission System Analysis – Hinds Lake Runner Upgrade”, dated 2003-05-15. The report concluded that the maximum capacity of the Hinds Lake generator should not exceed 91.6 MW. American Hydro was requested to revise its proposed design based on the following goals:

Maximum turbine output: 93 MW (corresponds to a generator output of 91.6 MW)

Range of best efficiency: 60-90 MW

Efficiency peak: approximately 75 MW

American Hydro responded with a proposal dated 2003-09-04. This report contains the American Hydro proposals, with estimates of the cost to modify the unit and an analysis of the benefits these modifications will provide.

All costs presented in this report are in January 2004 Canadian dollars.

SCOPE

In total, American Hydro submitted three proposals, all of which are contained in Appendix I. The last proposal was solicited following a review by System Planning of the capabilities of the transmission system in the region of Hinds Lake.

The capabilities of the plant electrical equipment (generator, isolated phase bus, current transformers, exciter, rectifier transformer, generator breaker, main transformers) were reviewed and found to be adequate for the increased power output.

The performance curve for the proposed runner is shown in Figure 1, in which “Model” indicates the predicted efficiency of the original runner based on the model test (an absolute efficiency test has not been performed at Hinds Lake) and “AH-3” indicates the design proposed by American Hydro. This figure indicates an increase in maximum efficiency of 0.7% and an increase in capacity of 10 MW. To achieve the increase in capacity and efficiency while reducing cavitation, a completely new runner will be provided and modifications will be made to the bottom ring and upper sections of the draft tube. This modification will require that these components be shipped to American Hydro's facility in Pennsylvania. Hydro forces will dismantle and reassemble the unit.

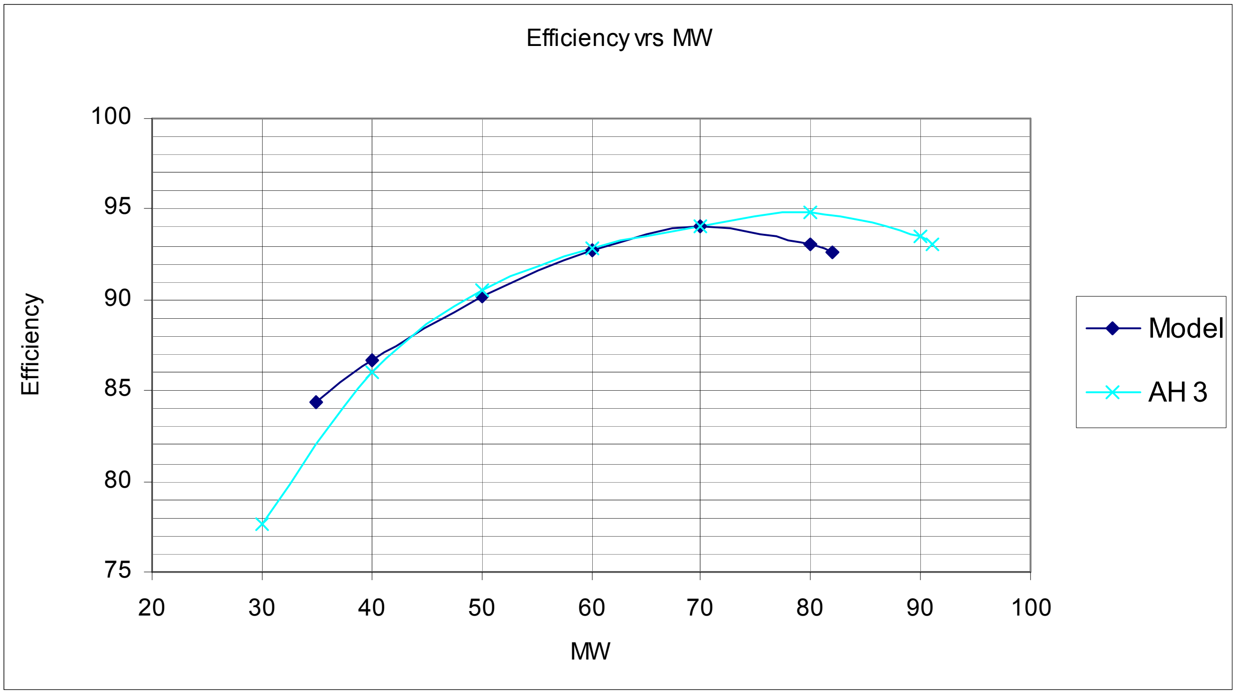


Figure 1

ENERGY PRODUCTION

Figure 1 indicates that there is an increase in capacity and an increase of efficiency over the range of 70 to 80 MW. It was stated earlier that System Operations indicated that the plant could be operated in this range, where the increased efficiency occurs. To verify this assertion, the operating records for 1997 (a near average water year) were reviewed to determine how successful Operations were at operating the existing runner within its range of best efficiency (60 to 80 MW), given the many and varied demands which must be addressed while operating the electrical system. The review indicated that Hinds Lake was operated within that band 88% of the time. It can be stated that the efficiency of the new runner, if operated within its range of best efficiency, will be 0.7% better than the existing runner and that the efficiency of the two runners is essentially identical over the rest of the operating range. Therefore the efficiency increase which will be realized from the new runner is 88% of 0.7%, or 0.62% and the annual increase in energy production will be 0.62%.

MAINTENANCE COSTS

The Hinds Lake runner has required more maintenance than is consistent with a runner of its vintage. It has suffered cavitation damage requiring extensive repairs at the inlet and discharge of blades at frequent intervals. In addition it has been prone to cracking. A review of maintenance records indicates that the average annual cost of runner repairs for the period 1984 to 2003 is $20,000. (See Appendix IV for a runner maintenance history. The costs presented have probably been underestimated, as we do not have an accurate record of repairs and associated costs.) Based on our experience with modern runners (Bay D'Espoir runner replacement), it can be expected that a new runner would be essentially cavitation free and that the annual cavitation repair cost would be negligible.

TECHNICAL ISSUES

The increased power capability of the proposed runner requires consideration of several technical aspects of the Hinds Lake development.

1. At maximum output, the new runner will consume approximately 10% more water than the original runner. This increase in flow implies a higher water velocity in the penstock. A hydraulic review will be required to determine if the gouvernor times will require adjustment, and the effect such adjustment may have on system regulation.

2. The effect of increased flow on the head loss in both the power canal and penstock requires investigation. It is probable that the effects would be very small, but they must be quantified to ensure that the small increase in efficiency derived from a modern design will not be significantly reduced by increased head losses.

3. The effect of increased flow on erosion of the power canal liner must be investigated. It is probable that the effect would be negligible or nil, but this must be quantified.

4. The design proposed by American Hydro, or by any other manufacturer, will be sensitive to tail water elevation. For the purposes of this study American Hydro was provided with Grand Lake elevations for a two year period. A more thorough review of tail water elevations over a longer period would be required to ensure that the design provides adequate cavitation protection.

5. The cavitation warranty offered by American Hydro as part of their standard conditions should be considered as an initial offer as it is only equivalent to the IEC code stipulation, which is not an aggressive standard. If Hydro is to commit to the considerable expense and effort required to change the Hinds Lake runner, American Hydro, or any other manufacturer, should be prepared to offer a more favourable warranty.

6. The duration of the outage required to effect the modifications is approximately 4 ½

months. The probability of spilling water will have to be investigated and a plan developed to avoid such an event.

CAPITAL COST

The capital cost estimate is summarized below, in January 2004 Canadian dollars.

Item	Capital Cost
Runner, bottom ring and draft tube modifications	840,000
Engineering, project management, testing and commissioning	325,000
Hydro forces	55,000
Environment	0
Contingency	122,000
Allowance for Funds During Construction (AFUDC)	Not Included
Corporate Overheads (6%)	80,500
Escalation	Not Included
Total	1,422,500

This is a prefeasibility class estimate and has an accuracy of + or - 15%. The project cash flow is presented in Appendix II

SCHEDULE

The project can be completed in 21 months. See Appendix III for a detailed schedule.

DISCUSSION

Installation of a new runner having higher capacity will require a slight change in the mode of operation of the Hinds Lake plant. The peak efficiency of the new runner occurs at a higher power output, therefore the plant will have to be operated at a higher output for fewer hours, to produce the same energy at slightly higher efficiency. This was discussed with ECC staff, who foresee no difficulty with this requirement. This requirement should be revisited with ECC should we decide to proceed.

Substantiating some of the benefits which will be realized as a result of changing the runner would be easy, while others would be difficult. The elimination of cracking and reduction or elimination of cavitation will be obvious after a year or two of operation. The increase in capacity will become obvious during the commissioning of the new runner. Quantifying the efficiency gain is quite a different matter. A gain of 0.7% in efficiency will challenge the ability of test instrumentation and procedures to verify and it must be stated that it is effectively impossible to prove. This is due to the fact that the accuracy of an absolute efficiency test employing the most accurate instrumentation, most rigorous test procedure and recording numerous data sets at each test point to minimize the effects of random errors, is unlikely to result in a test accuracy of better than 0.7%. Thus, the uncertainty band of the pre-modification test will encompass the absolute curve of the post-modification test and vice versa. In short, for all practical purposes, it will be impossible to demonstrate that the expected efficiency gain has been realized and the gain must be accepted on faith. Given the sterling reputation of the proponent, it is reasonable to accept the predicted gain in efficiency.

GREENHOUSE GAS EMISSIONS

Installation of a new runner will result in higher efficiency, which can be converted into an equivalent reduction of fuel consumption at the Holyrood Thermal Generating Station. Hydro may be able to take advantage of these reductions as carbon credits and the value of these credits should be included in the evaluation of this project.

CONCLUSIONS

1. The project is technically feasible

2. It is possible to design, procure and install a runner having a higher output and better cavitation resistance than the existing design and which would not be subject to cracking.

3. The proposed new runner would be more efficient than the existing design, but the increase in efficiency, although real, is too small to measure using existing field test methods.

4. The new runner would reduce annual maintenance costs by approximately $20,000.

5. The new runner would increase the annual energy production by 0.62%

6. The new runner would increase the installed capacity of Hinds Lake by 10 MW.

7. There are several technical concerns associated with increasing the plant capacity, which must be investigated prior to negotiating a contract for turbine modifications. These include the effects of higher water velocity on the control structure, canal liner, trashrack, intake gate and gate hoist, penstock, gouvernor, etc. It is expected that the effects on all components would be negligible, but they must be thoroughly investigated before proceeding with the installation of a new runner.

RECOMMENDATIONS

1. System Planning should review the financial viability of this project.

2. If the project is financially viable, the technical concerns identified in this report should be investigated before proceeding with the project.

APPENDIX I

AMERICAN HYDRO PROPOSALS

AMERICAN HYDRO CORPORATION

"The Service Company"

135 STONEWOOD ROAD, P.O. BOX 3628, YORK, PA 17402-0136
(717) 755-5300 • FAX (717) 755-5522

September 4, 2003

Mr. John Mallam
Newfoundland and Labrador Hydro
P. O. Box 12400
St. John's, Newfoundland A1B 4K7
CANADA

SUBJECT: Hinds Lake Upgrade
American Hydro Inquiry Number 2175

Dear Mr. Mallam:

American Hydro is pleased to provide this revision to our proposal of 14 June, 2002. Based on the preferred operating characteristics you forwarded on 6 June, 2003 we have designed a runner that optimizes at a lower capacity while maintaining good high power characteristics. Our emphasis has been to achieve high efficiency with excellent cavitation resistance.

In order to achieve the best cavitation free range, the new design utilizes the modified bottom ring/upper draft tube which provides a larger runner discharge diameter. While is it not possible to achieve 93 MW for full load while providing best efficiency operation at 75MW we have come close. The attached performance curves show our expected efficiency. Our updated guarantees and pricing are:

Guaranteed peak efficiency (in accordance with IEC 41)	94.0%
Guaranteed Full load Turbine Output	87.5 MW at 84.75 m T.W.E 90.5 MW at 88.3 m T.W.E.
Price for one stainless steel runner plus new nose cone and bolts, F.O.B., York, PA	$560,000 (U.S.)
Standard delivery time for one runner	8 months
Price for bottom ring/upper draft tube modification	$82,360 (U.S.)
Standard shop time for the modification	8 weeks

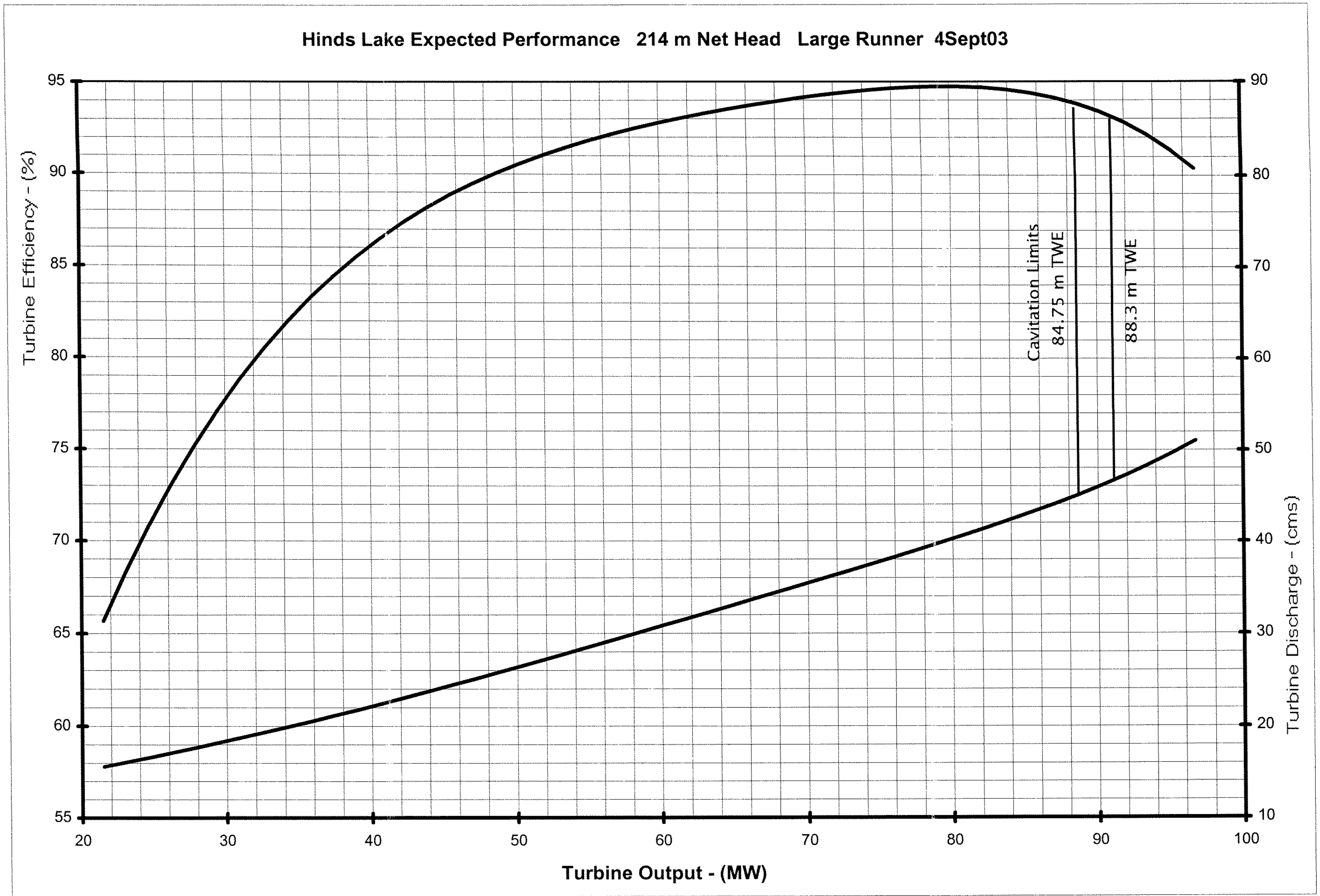
All other terms and conditions are in accordance with our 14 June, 2002 proposal. We would be pleased to answer any questions you may have and look forward to working with you on the Hinds Lake upgrade.

Very truly yours,

William H. Colwill

William H. Colwill
Vice President
Marketing

WHC/dfh


Hinds Lake Expected Performance 214 m Net Head Large Runner 4Sept03
Turbine Efficiency - (%)
95
90
85
80
75
70
65
60
55
Turbine Discharge - (cms)
90
80
70
60
50
40
30
20
10
20
30
40
50
60
70
80
90
100
Turbine Output - (MW)
Cavitation Limits
84.75 m TWE
88.3 m TWE

AMERICAN HYDRO CORPORATION

"The Service Company"

135 STONEWOOD ROAD, P.O. BOX 3628, YORK, PA 17402-0136
(717) 755-5300 • FAX (717) 755-5522

2002-06-18

June 14, 2002

Mr. John Mallam
Newfoundland and Labrador Hydro
P. O. Box 12400
St. John's, Newfoundland A1B 4K7
CANADA

SUBJECT: Hinds Lake Upgrade
American Hydro Inquiry Number 2175

Dear Mr. Mallam:

Based on the additional data and drawings you forwarded, American Hydro has completed a detailed evaluation of the Hinds Lake turbine. We believe that this unit is an excellent candidate for upgrade and we have made a detailed assessment of all the turbine components. By replacing the runner with a runner of modern custom design, the best efficiency operation can be shifted from 71 megawatts to about 87 megawatts. For this design the important parameters are:

Net Head:	214 m
Speed:	360 rpm
Flow:	44 cms
Minimum Tailwater:	85.2 m
Turbine Centerline Elevation:	82.9 m

The turbine component analysis has shown the following:

Spiral Case – Is large and well proportioned. It will provide excellent performance.
Stay Vanes – Are well shaped but do present a slight incidence angle with the oncoming flow. An additional loss of 0.1% is calculated.
Wicket Gates - Are well designed.
Runner – The new runner will provide state-of-the-art performance.
Draft Tube – The draft tube shape is not ideal. However, for this high head unit, draft tube performance is not critical. The additional loss is estimated to be 0.65%.

A modern model for Hinds Lake will have a model efficiency near 93.8%. Accounting for the losses given above, the Hinds Lake model efficiency with a new runner is expected to be 93.0%. Using the IEC step-up, the expected prototype peak efficiency is 94.74%.

American Hydro has developed two new runner designs for Hinds Lake. The first design utilizes the existing wheelcase with no modifications. Design "A" shows good performance and excellent cavitation resistance up to 45.5 cms. The second design requires a modification to the bottom ring/upper draft tube (this piece is removable). Runner "B" has a larger discharge diameter and will perform very well up to 48 cms. The attached expected performance curve is valid for either runner with the full load capacities as noted on the curve.

For Runner "B" we would anticipate modifying the bottom ring/upper draft tube in our shop. This work includes installation of a new seal ring and reboring the gate stems to eliminate any movement caused by the welding. (The gate stem holes would be numerically located prior to modifying the piece.)

American Hydro is pleased to present this proposal for either Runner "A" or "B" with performance guarantees and pricing (in U.S. $) as follows:

	Runner "A"	**Runner "B"**
Guaranteed peak efficiency (in accordance with IEC 41)	94.0%	94.0%
Guaranteed Full load Turbine Output	87.5 mW	94.0 mW
Price for one stainless steel runner plus new nose cone and bolts, F.O.B., York, PA	$548,300	$560,000
Standard delivery time for one runner	8 months	8 months
Price for bottom ring/upper draft tube modification	N/A	$82,360
Standard shop time for the modification		8 weeks

Either runner is guaranteed against excessive metal removal by cavitation for 8000 hours or 1 ½ years of operation (whichever comes first) provided the unit is not run for more than 100 hours at outputs greater than the expected full load or for more than 400 hours at outputs less than 60% of the full load. Excessive cavitation is defined by the middle of the range for volume or area from IEC 609.

This proposal is made in accordance with American Hydro's Standard Conditions of Sale and is valid for 60 days. A milestone payments schedule would apply.

We hope you find this proposal attractive. Either runner will provide an excellent upgrade at Hinds Lake. Please let us know if you have any questions.

Very truly yours,

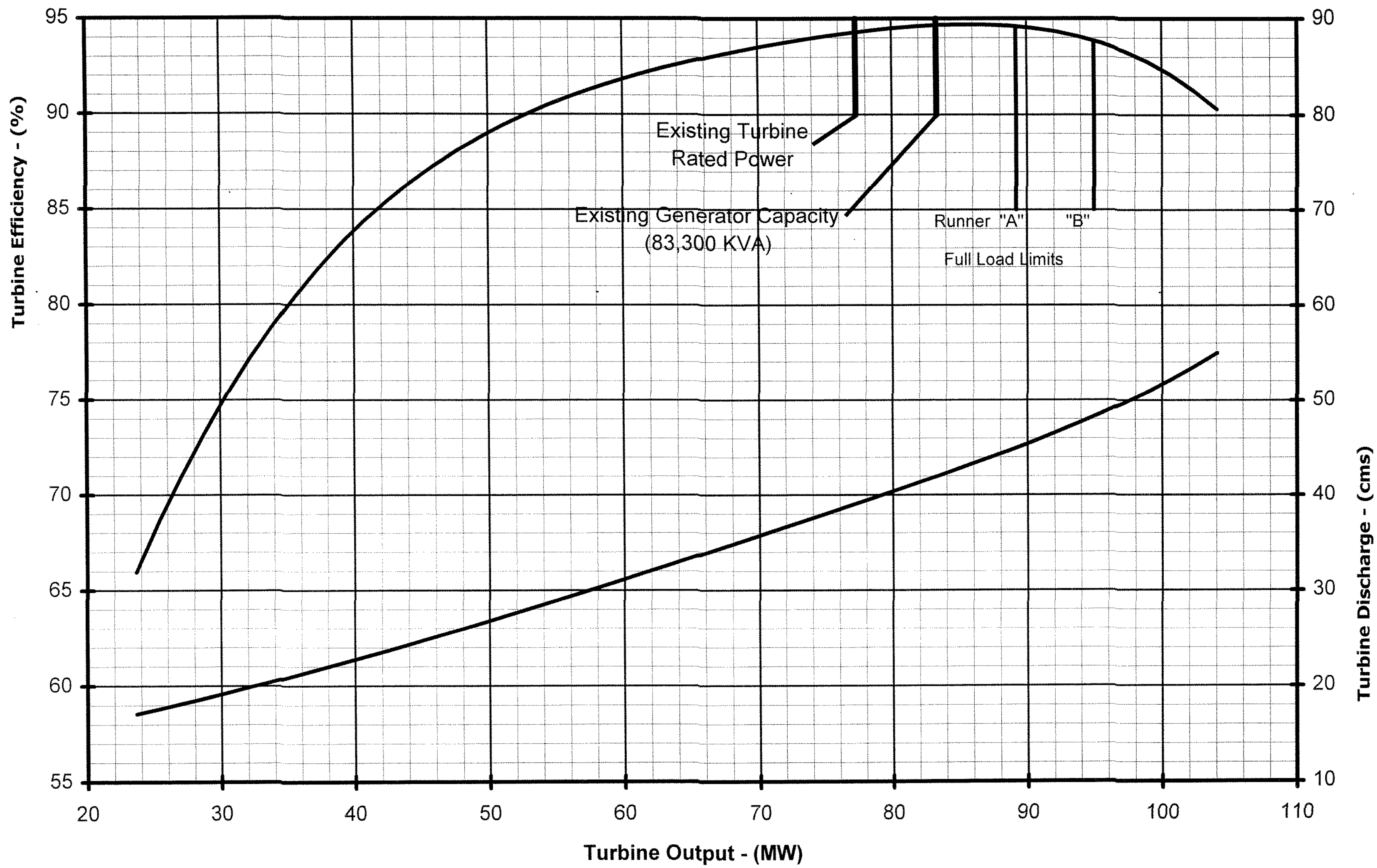
William H. Colwill

William H. Colwill
Vice President
Marketing

WHC/dfh

cc: Mr. George Agami

Expected Hinds Lake Upgrade Performance at 214 m Net Head



Upgrade Analysis for Hinds Lake and Churchill Falls for Newfoundland and Labrador Hydro

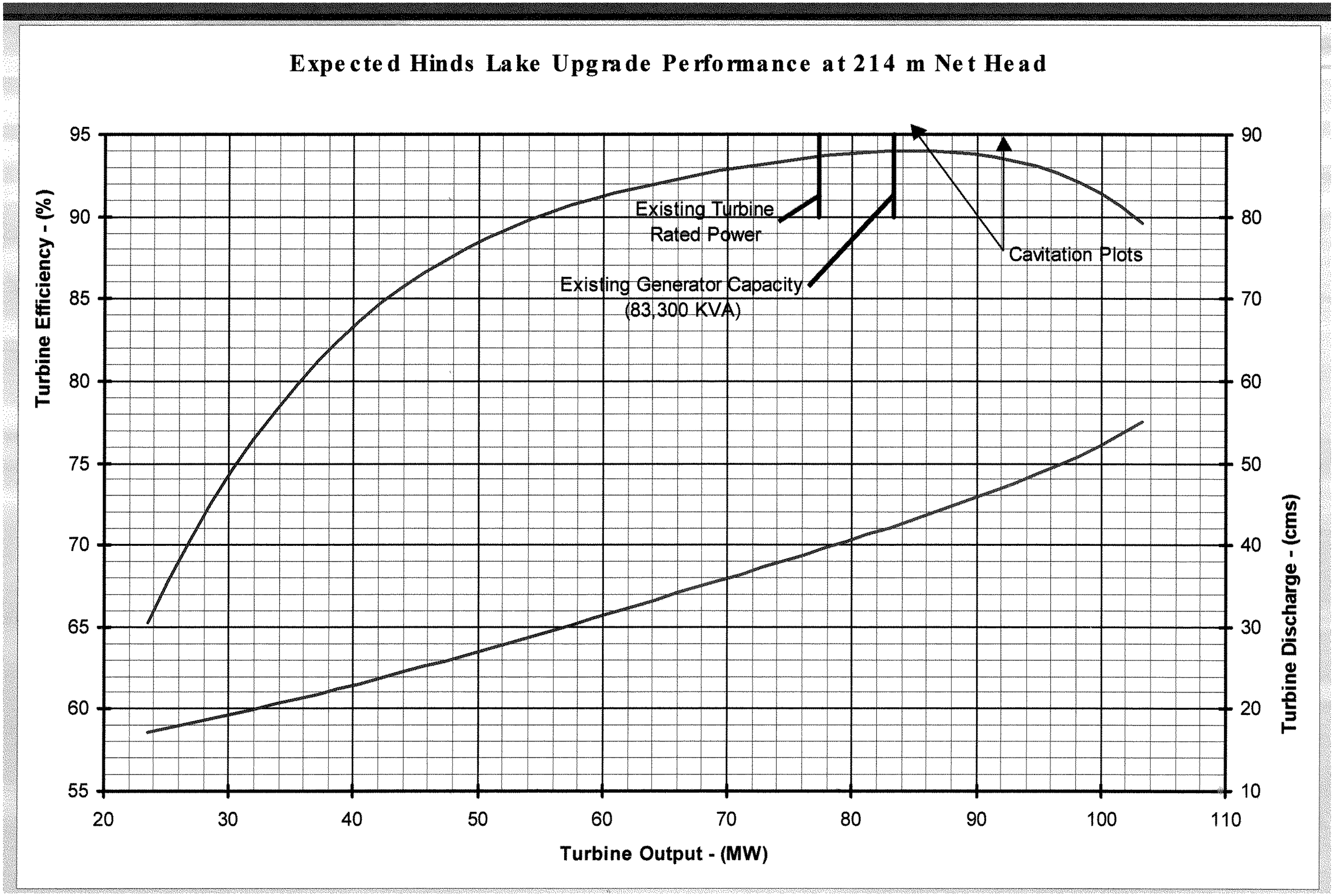
By: ***American Hydro Corporation***

Date: **February 22, 2001**

American Hydro has completed preliminary upgrade analyses for the Hinds Lake and Churchill Falls power plants. For both projects a new runner design was developed and analyzed. These studies demonstrate that new runners for these projects can improve both the efficiency and the capacity of the turbines. Furthermore, the cavitation resistance for both new designs is excellent.

The attached figures present the expected hydraulic performance and pressure distributions. We would recommend runners fabricated from 100% stainless steel. The budgetary prices and deliveries for these runners are:

Hinds Lake: One Runner - $550,000 (U.S.)
Delivery within eight months


Expected Hinds Lake Upgrade Performance at 214 m Net Head
Turbine Efficiency - (%)
Turbine Discharge - (cms)
Turbine Output - (MW)
Existing Turbine
Rated Power
Existing Generator Capacity
(83,300 KVA)
Cavitation Plots
95
90
85
80
75
70
65
60
55
90
80
70
60
50
40
30
20
10
20
30
40
50
60
70
80
90
100
110

APPENDIX II

PROJECT CASH FLOW

Prepared by: J. Mallam

CAPITAL BUDGET PROPOSAL

Capital Cost Estimate & Cash Flow Requirements
2003 Fiscal Year : Prepared: 2004-01-05

Hinds Lake Runner Replacement
In-Service: Year 2

AFUDC= Annual Monthly Quarterly

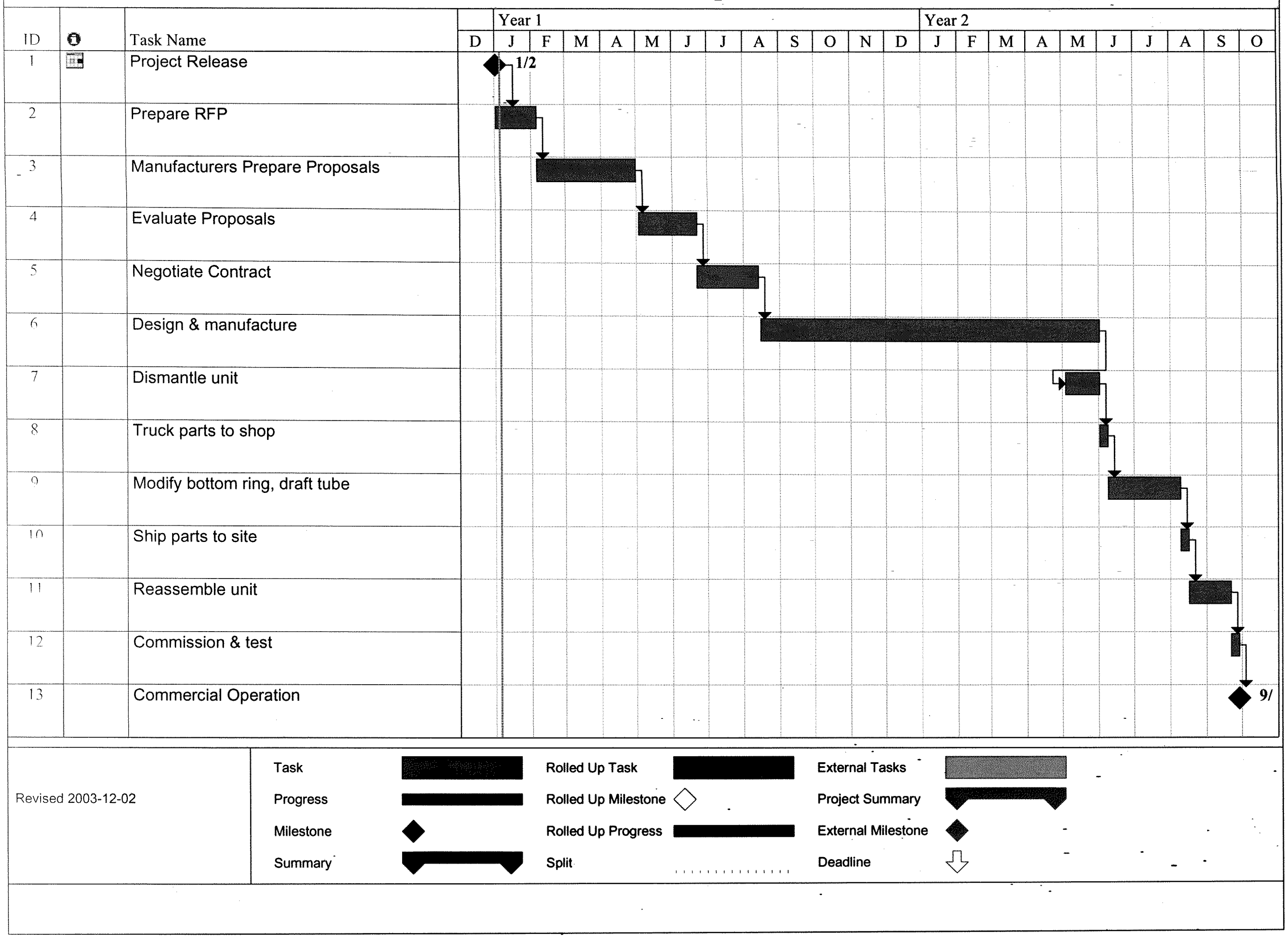
Escalation

Period		Constr. Servcs	Equip. Purch.	Matrls Purch.	Constr. Internal	Land & Survey	External Eng.	Environ-ment	Eng.& Mgmt.	Proj./Constr Mgmt.	Inspection & Comm.	O/H @ 6.00%	Cont @ 10%	Sub Total	Escln	AFUDC	Total Project	Cash Flow (Excl AFUDC)
Year 1	Jan								10.0			0.6		10.6			10.6	10.6
	Feb								10.0			0.6		10.6			10.6	10.6
	Mar								10.0			0.6		10.6			10.6	10.6
	Apr								10.0			0.6		10.6			10.6	10.6
	May								10.0			0.6		10.6			10.6	10.6
	Jun		150.0						10.0			9.6		169.6			169.6	10.6
	Jul								10.0			0.6		10.6			10.6	169.6
	Aug								10.0			0.6		10.6			10.6	10.6
	Sep								20.0			1.2		21.2			21.2	21.2
	Oct								10.0			0.6		10.6			10.6	10.6
	Nov		150.0						10.0			9.6		169.6			169.6	10.6
	Dec								10.0			0.6		10.6			10.6	169.6
Total	Year 1	0.0	300.0	0.0	0.0	0.0	0.0	0.0	130.0	0.0	0.0	25.8	0.0	455.8			455.8	455.8
Year 2	Jan								10.0			0.6		10.6			10.6	10.6
	Feb								20.0			1.2		21.2			21.2	21.2
	Mar								10.0			0.6		10.6			10.6	10.6
	Apr		200.0						20.0			13.2		233.2			233.2	21.2
	May								10.0			0.6		10.6			10.6	222.6
	Jun				25.0				20.0			2.7		47.7			47.7	47.7
	Jul								20.0			1.2		21.2			21.2	21.2
	Aug		140.0						20.0			9.6		169.6			169.6	21.2
	Sep				30.0				20.0		25.0	11.8	122.0	208.8			208.8	227.9
	Oct		200.0						20.0			13.2		233.2			233.2	150.5
	Nov											0.0		0.0			0.0	212.0
	Dec											0.0		0.0			0.0	0.0
Total	Year 2	0.0	540.0	0.0	55.0	0.0	0.0	0.0	170.0	0.0	25.0	54.7	122.0	966.7			966.7	966.7
Beyond												0.0	0.0	0.0			0.0	0.0
Total Proj.		0.0	840.0	0.0	55.0	0.0	0.0	0.0	300.0	0.0	25.0	80.5	122.0	1,422.5			**1,422.5**	1,422.5

APPENDIX III

PROJECT SCHEDULE

Hinds Lake Runner Replacement



APPENDIX IV

RUNNER MAINTENANCE HISTORY

Hinds Lake Runner History

c:\projects\hinds lake\runner history.xls

Year	Mon/day	Hours	Comments	Estimated Cost ($ x 1,000)
1982	Jan 21	8,044	warrenty inspection. Inlet cavitation noted. Total material loss less than warrented.	
1982	Dec		P.O. to Nohab for templates, dwgs, instructions to modify runner. $5,100. Dwg 12000207	
1983	Mar		Descriptive instructions from Nohab of how to modify runner at	
1983	Sept	17,067	Inlet cavitation 13 mm deep (??!!) in places	
1984	Sep/Oct	22,904	Modified inlet edge "...changing the blade near the inlet fillet (approx. 100 mm up the blade) to have them conform to the blades which experienced less cavitation"	60
1986			Ferralium 255 with Certanium 227 overlay on inlet	30
1986	Dec 05		Some cracks in weld repaired areas. 359 hours since repairs	
1987	Mar 31		Usual cavitation at inlet	
1987	Oct 05	35,323	Inlet cavitation repaired with Ferralium and Certanium	30
1988	Nov 8-10	41,237	1/8 to 3/8 deep pits at inlet. Gouged and repaired with Duratough elastomer	30
1989	Mar 7 or 27 ?	42,697	Inspected by NLH and Kvaerner. Usual cavitation	
1989			Kvaerner performed model test ??????	
1989	Oct 04	44,620	3383 hours since repairs. Cavitation evident at inlet	
1990	Oct/Nov	49,168	Serious cavitation at inlet. Cracks in previously repaired areas. Voith Hydro on site to take inlet profiles.	
1991			Requested proposals for runner mods. Kvearner and RSW	
1992	April		Kvaerner dwgs for runner and wicket gate mods	
1992	fall	58,942	Wicket gates modified (blade to trunnion radius) and four blades (7, 8, 15, 16) were modified by adding larger preformed metal fillets at inlet (lp side)	100
1993	Jan	60,306	Frosting at inlet after 1364 hours since repairs. "Fillet mods to 7, 8, 15, 16 have not caused a noticable reduction in cavitation as compared to unmodified blades.	
1993	Apr 13	61,650	2708 hours since mods - cavitation evident	
1993	Sept 17	64,255	5314 hours since mods - no improvement in blade cavitation	
1994	Mar 22	67,011	8068 hours since mods fall 1992 - no improvements resulting from blade to band fillet mods and wicket gate blade to trunion mods in 1992	
1995			Three blades repaired on discharge side; n situ.	15
1997	Sept		Repairs at inlet (using Cavitec) but NOT discharge	60
1999	Aug 19		Usual cavitation at inlet	
2000	Nov 15		Usual cavitation at inlet	

TRANSMISSION SYSTEM ANALYSIS
OF
PROPOSED TURBINE RUNNER UPGRADE
AT
HINDS LAKE GENERATING STATION

Prepared By:
System Planning Department
Newfoundland and Labrador Hydro

May 8, 2003

TABLE OF CONTENTS

INTRODUCTION

The Hinds Lake runner (77.3 MW at 214 m net head) has experienced cavitation damage and cracking since commissioning in December of 1980. Newfoundland and Labrador Hydro (NLH) Generation Engineering has been exploring runner replacement with American Hydro (AH). AH has proposed a runner with rated and ultimate capacities of 96 and 104 MW respectively. The new runner would contribute capacity only, as there would be no increase in energy production from the plant (i.e. no addition water diversion into the reservoir). NLH Generation Engineering has completed a check of the capacity of plant electrical equipment in light of the capacity increase proposed by AH. The limiting components include the electrical generator, which is rated at 100 MVA, and the 13.8 kV isolated phase bus, which is rated at 95.6 MVA (4000 Amps).

The purpose of this transmission system analysis is to investigate the impact of increasing the Hinds Lake Generating Station capacity will have on the surrounding transmission system and to identify transmission system limitations and constraints. The analysis is completed using Power Technologies Inc. software package PSS/E. The analysis does not deal with water management issues related to the upgrade of the turbine runner.

THE TRANSMISSION SYSTEM

Hinds Lake Generating Station is connected to the Island Interconnected Transmission System at Howley Terminal Station via the 138 kV transmission line TL243, which consists of 559.5 MCM, 19 strand, AASC “DARIEN”. The Howley Terminal Station is one of three stations situated on the 211.3 km long Deer Lake to Stony Brook 138 kV loop. The predominant conductor on the loop portion of the transmission system is 266.8 MCM, 26/7, ACSR “PARTRIDGE”, with 78.5 km of 266.8 MCM, 6/7, ACSR “OWL” originating at Howley and extending eastward 21 km beyond Indian River Terminal Station. The line ratings for the 138 kV loop are provided in the following table:

Table 1
Transmission Line Ratings
Deer Lake to Stony Brook 138 kV Loop

TL #	From	To	MVA Rating for Ambient Temp of			
			30 C	25 C	15 C	0 C
222	Stony Brook	Springdale	63.3	73.9	91.4	112.4
223	Springdale	Indian River	52.2	60.9	75.4	92.7
224	Indian River	Howley	52.2	60.9	75.4	92.7
245	Howley	Deer Lake	63.3	73.9	91.4	112.4
243	Hinds Lake	Howley	89.1	104.9	130.8	161.7

At Howley Terminal Station, a 138/69/4.16 kV, 7.5/10/12.5 MVA power transformer supplies the town of Howley and the White Bay 69 kV transmission system. The 69 kV transmission system supplies Hampden, Jackson’s Arm and Coney Arm Terminal Stations as well as interconnecting the Rattle Brook Hydro Plant.

At the Deer Lake end, the 138 kV loop terminates on a 138 kV ring bus that also contains the 138 kV transmission line TL239 supplying the Great Northern Peninsula and a 138/66 kV, 25/33.3/41.7 MVA power transformer connecting the Deer Lake Power Plant to the system. The 138 kV ring bus is connected to the 230 kV grid via a single 230/138 kV, 45/60/75 MVA power transformer.

At the Stony Brook end, the 138 kV loop terminates on a 138 kV load bus along with three 138 kV transmission lines supplying the Stony Brook to Sunnyside 138 kV loop. The 138 kV bus is connected to the 230 kV grid via two 230/138 kV, 75/100/125 MVA power transformers.

Figure 1 provides a simplified single line diagram of the transmission system described above.

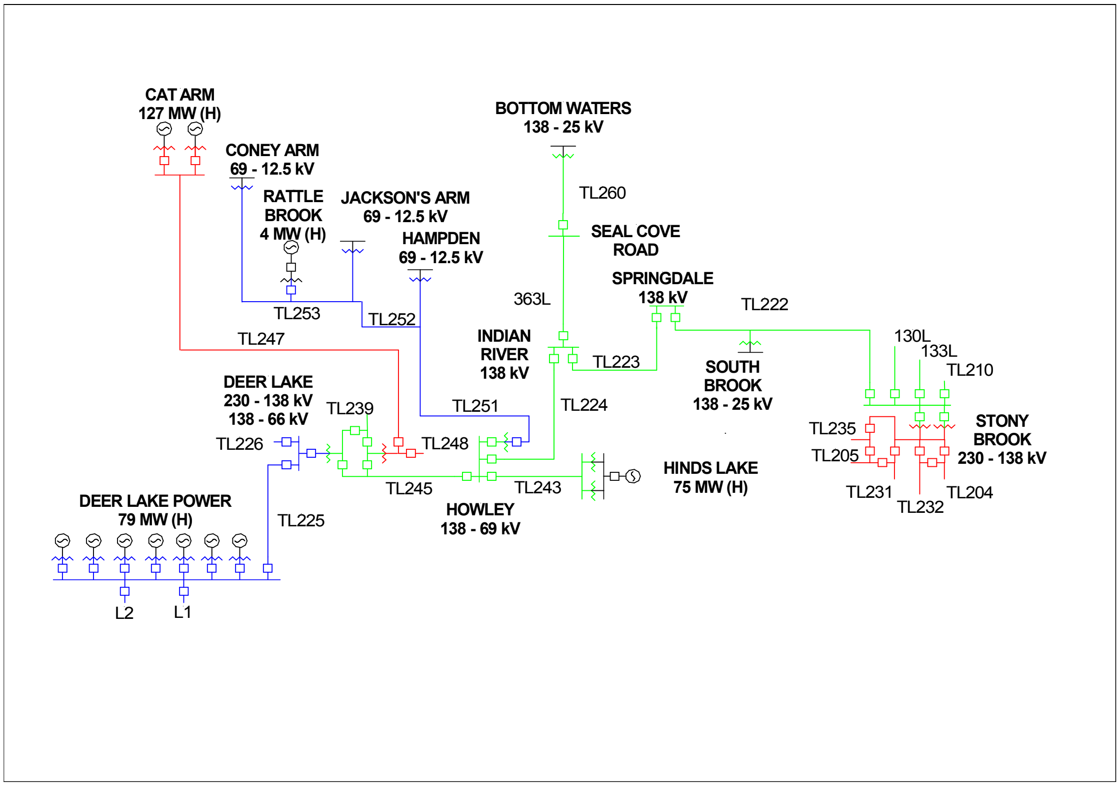


Figure 1 – Deer Lake to Stony Brook 138 kV Loop

GENERATOR CAPABILITY

Figure 2 provides a sketch of the Hinds Lake generator capability curve. Figure 2 is based upon the manufacturer's capability curve given in drawing M-1453-092-100-5. From the capability curve one notes that the generator is rated at 83.3 MVA based upon stator and rotor temperatures of 60 ^{o}C and 99.96 MVA (83.3 MVA x 1.2 p.u.) based upon temperatures of 80 ^{o}C. The dashed line provides an estimate of the unit's capability curve for operation at 95.6 MVA, which is the capability of the 13.8 kV isolated phase bus or bus duct (i.e. 4000 A).

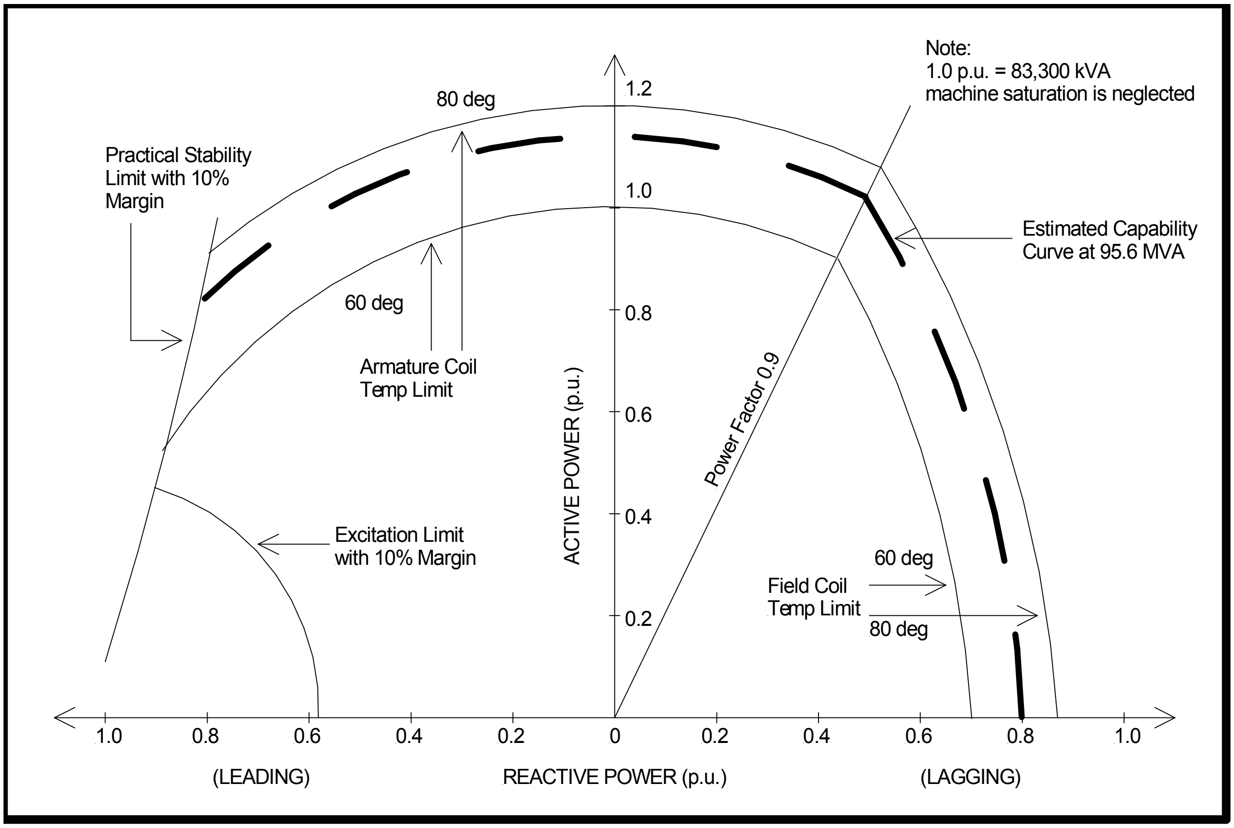


Figure 2 – Hinds Lake Generator Capability Curve

The VAR capabilities of the unit for a 95.6 MVA rating are summarized in Table 2.

Table 2 **Hinds Lake MVAR Capability** **For Operation at 95.6 MVA**		
MW	MVAR Lagging (out of machine)	MVAR Leading (into machine)
95.6	0	0
91.6	25	25
86.0	41.6	40.8
0	66.6	48.0

Given that the isolated phase bus is rated for 95.6 MVA, it makes little sense to have the replacement runner sized any larger than 95.6 MW. Further, from a voltage control perspective on the 138 kV loop, injection of 95.6 MW from Hinds Lake will undoubtedly require MVAR from/into the machine depending upon system conditions. With 95.6 MVA set as the ultimate transfer limit of the 13.8 kV bus, further reductions in the MW output of the replacement runner may be justified.

Typically, the capacity, or MW, of a hydro plant is determined based upon physical design parameters such has head and flow and economic factors such as the value of capacity and energy. Transmission system analysis is in turn used to determine the MVAR requirement, or machine power factor, based upon a series of operating requirements including system contingencies. However, in this particular case the total MVA is presented as the limiting factor and transmission system analysis is to be used to determine the split between MW capacity and MVAR under contingency.

LOAD FLOW ANALYSIS

Figure 3 provides a load flow plot of the Deer Lake – Stony Brook 138 kV loop with Hinds Lake at 95.0 MW during summer loading conditions and 25 ^{o}C line ratings. For the light load case NLH generation equals 512 MW. Total utility load is set equal to 35% of peak and the industrials are set at peak for a total Island load of 710 MW net NP generation. The light load case assumes one unit on at Cat Arm generating 35 MW.

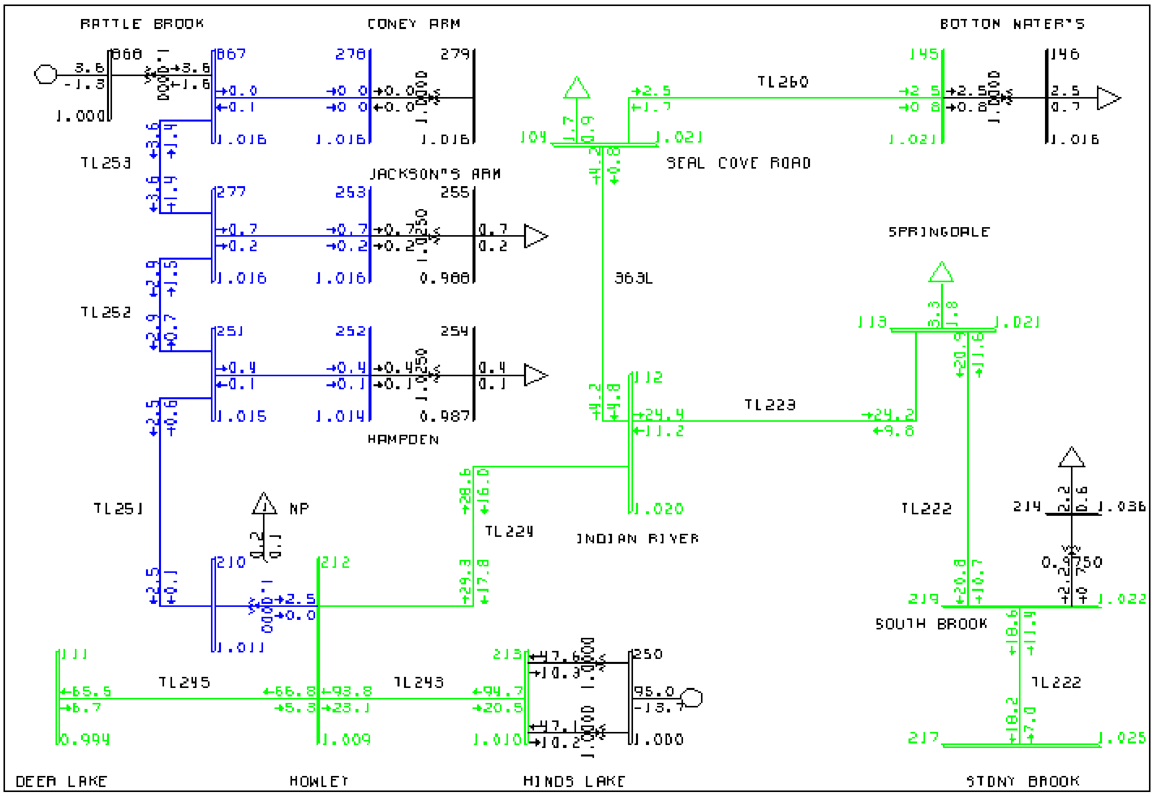


Figure 3 – Light Load Base Case – Hinds Lake at 95.0 MW

In order to hold 1.00 p.u. voltage on the terminals of the Hinds Lake generator, the machine would be required to absorb 13.7 MVAR. The total loading on the machine equals 95.98 MVA, which is beyond the 95.6 MVA limit. Clearly voltage control would be problematic for Hinds Lake operation at 95 MW during lighter load conditions.

Based upon values shown in Figure 3 and the transmission line ratings provided in Table 1, it is quite obvious that the existing transmission system would not be capable of carrying the output of an uprated Hinds Lake at ambient temperatures of 25 ^{o}C and above during maintenance or forced outages to TL245 or TL224. Operation of Hinds Lake at 95.6 MVA during the summer months would require upgrading of the 138 kV loop to ensure adequate ground clearances during line out contingencies or reductions in plant output during line out contingencies. For example, with TL224 out during the summer months, Hinds Lake output would have to be limited to approximately 70 MW to avoid overloading of TL245. Similarly, an outage to TL245 would require the output of Hinds Lake to be limited to approximately 60 MW to avoid overloading of TL224. Further, the potential exists for capacity constraints on Deer Lake T2 for line outages to TL222, TL223 or TL224 during the summer months requiring either a 230/138 kV transformer addition at Deer Lake or reductions in plant output during line out contingencies.

Assuming that the 138 kV loop would not be upgraded, nor a second 230/138 kV power transformer added at Deer Lake, there is little benefit in increasing the runner output during the summer months. This leads to the possibility of a dual plant rating (i.e. summer and winter). It must be noted that dual zone operation of a Francis turbine can have a significant impact on water management due to the traditional variances in turbine efficiency over the operating range of the turbine. Clearly, this is beyond the scope of the transmission system analysis, but nonetheless requires careful consideration and detailed analysis prior to a final decision to proceed with a turbine upgrade.

In order to address the application of a dual plant rating it becomes prudent to assess the impact of the increased output of Hinds Lake during the 15 ^{o}C and 0 ^{o}C loading conditions.

15 ^{o}C Ambient Temperature

Economic Analysis has determined that the coincident peak load for the Island Interconnected utility load would be in the 500 – 600 MW range for a 15 ^{0}C ambient temperature in the Deer Lake region. The analysis completed by Economic Analysis also indicated that the highest utility peaks at this temperature level were most likely to occur during the lunch time hours. As a result, the total utility load was scaled to 600 MW in the load flow model for the 15 ^{o}C ambient temperature analysis, while the industrial loads were kept at their coincident peak loads. Figure 4 provides the load flow plot of the Deer Lake – Stony Brook loop with the utility load at 600 MW, all lines in service, Hinds Lake at 95.0 MW, two units on at Cat Arm for 35 MW each and 15 ^{o}C line ratings.

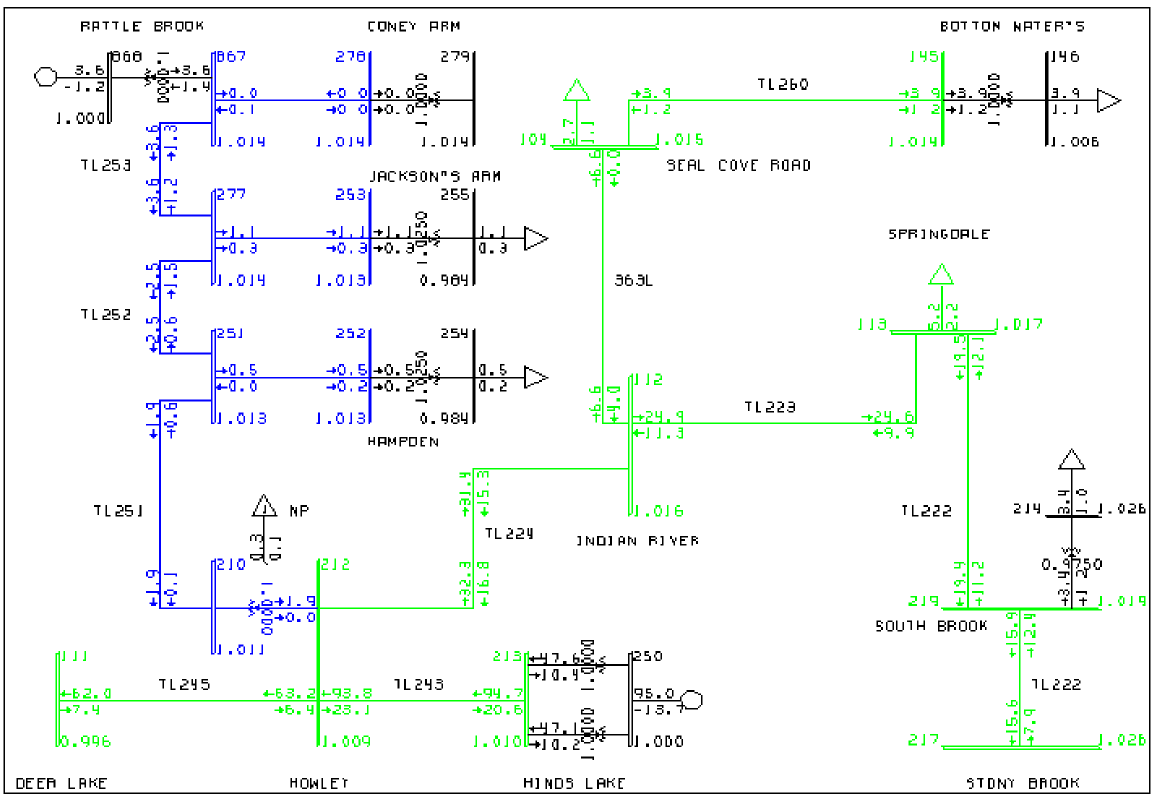


Figure 4 - 15 ^{o}C Day – Hinds Lake at 95.0 MW

With all lines in service, operation of Hinds Lake at 95.0 MW during 15 ^{o}C ambient temperatures will not be a problem with respect to transmission line loading. However, the reactive power loading of 13.7 MVAR on the Hinds Lake generator places the total load on the machine at 95.98 MVA, or 100.3% of rating. Similarly, the Hinds Lake unit would exceed the 95.6 MVA limit for outages to TL222, TL223 and TL224. With Hinds Lake at 95 MW, unit loadings are 95.81 MVA, 95.89 MVA and 95.74 MVA for TL222, TL223 and TL224 line outages respectively.

Clearly, a reduction in the MW loading of the Hinds Lake generator is required to provide sufficient MVAR capability for the machine to provide effective voltage control on the 138 kV loop. Note that the original design provided +/- 36 MVAR of reactive capability for a 75 MW loading. A review of the MW and MVAR output at Hinds Lake for the period 2000 to 2002 was conducted using EMS data. Plots of MW versus MVAR loadings for each of the three years is provided in Appendix A. Based upon the data, the MVAR output has varied from –25 MVAR to +15 MVAR for unit loads in the 60 to 75 MW range. Using the estimated capability curve, reducing the MW loading to 91.6 MW will provide +/- 25 MVAR of reactive capability.

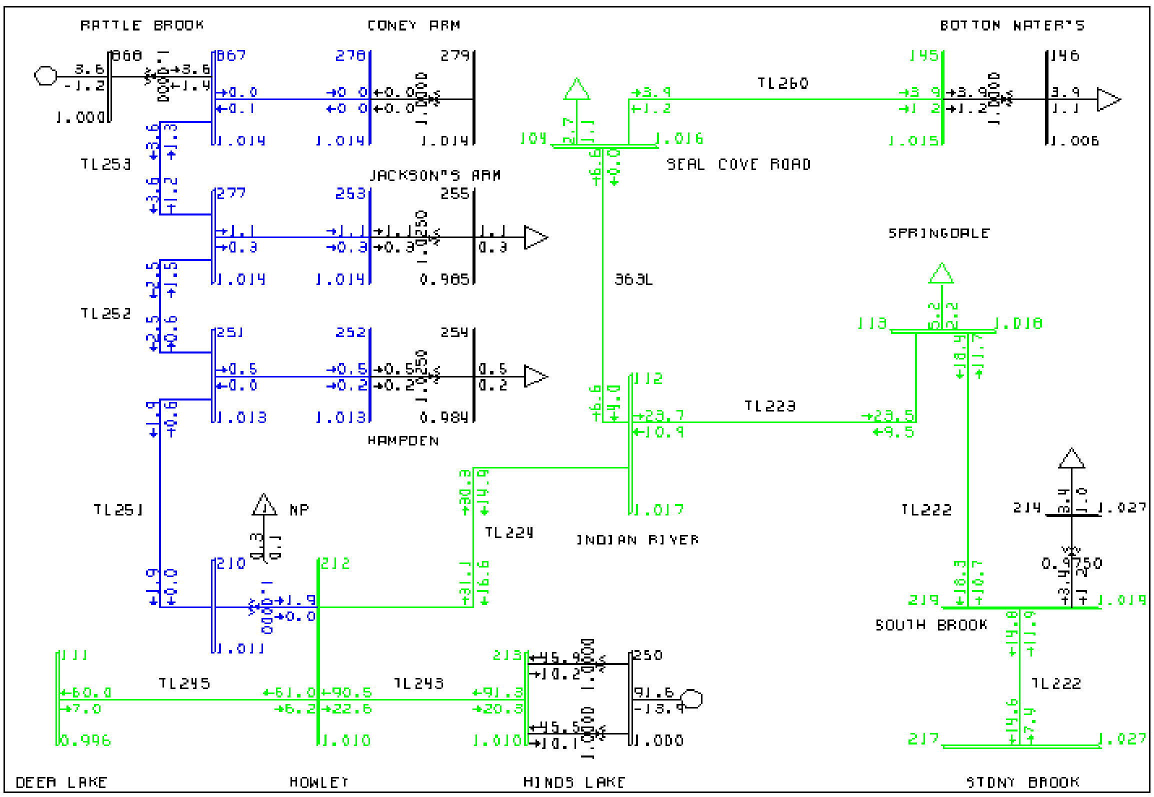


Figure 5 - 15 ^{o}C Day – Hinds Lake at 91.6 MW

Figure 5 provides the load flow plot for the 15 ^{o}C day with Hinds Lake at 91.6 MW and all transmission lines in service. The 13.9 MVAR loading on the machine results in a total load of 92.6 MVA, or 96.9% of the isolated phase bus rating. The Deer Lake T2 transformer is expected to be loaded to approximately 40 MVA given a 138 kV GNP loading of 17 MW.

Figure 6 and 7 provide the load flow plots for the 15 ^{o}C day with Hinds Lake at 91.6 MW and TL 222 and TL223 out respectively. In each case the Hinds Lake generator is within the 95.6 MVA limit. Deer Lake T2 loadings are estimated as 50 MVA and 57 MVA for outages to TL222 and TL223 respectively.

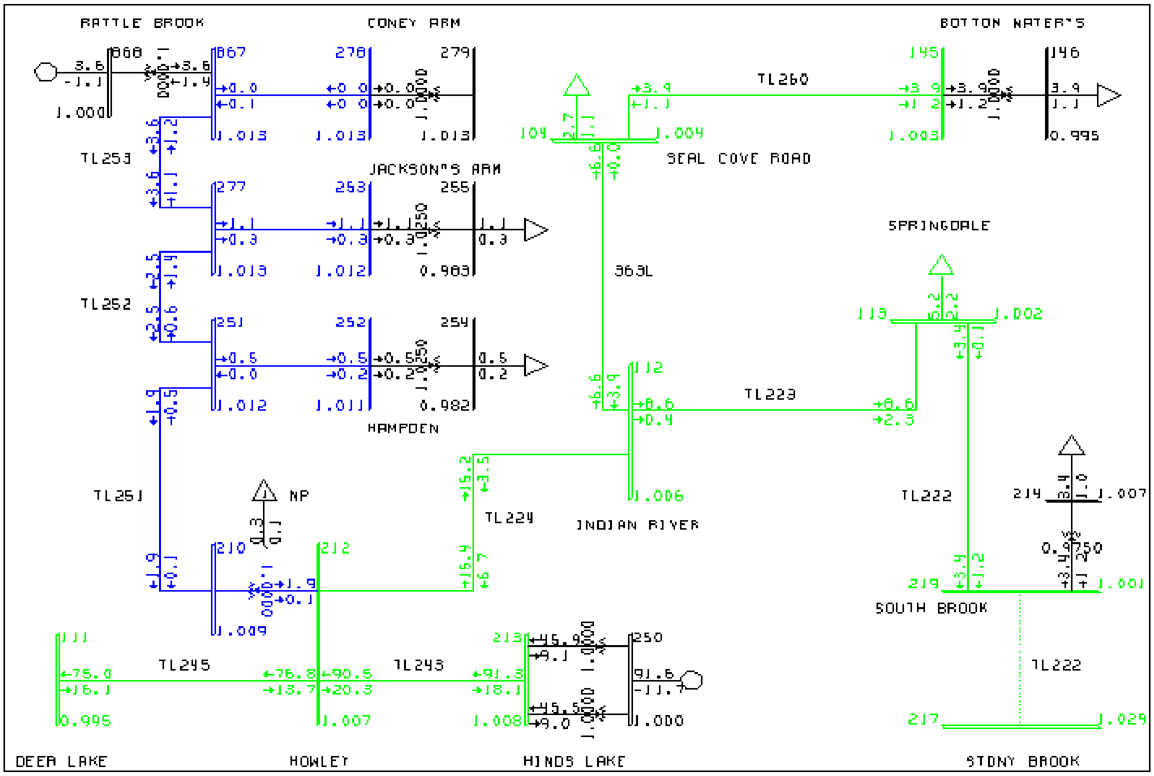


Figure 6 - 15 ^{o}C Day – Hinds Lake at 91.6 MW - TL222 STB to SOK Out

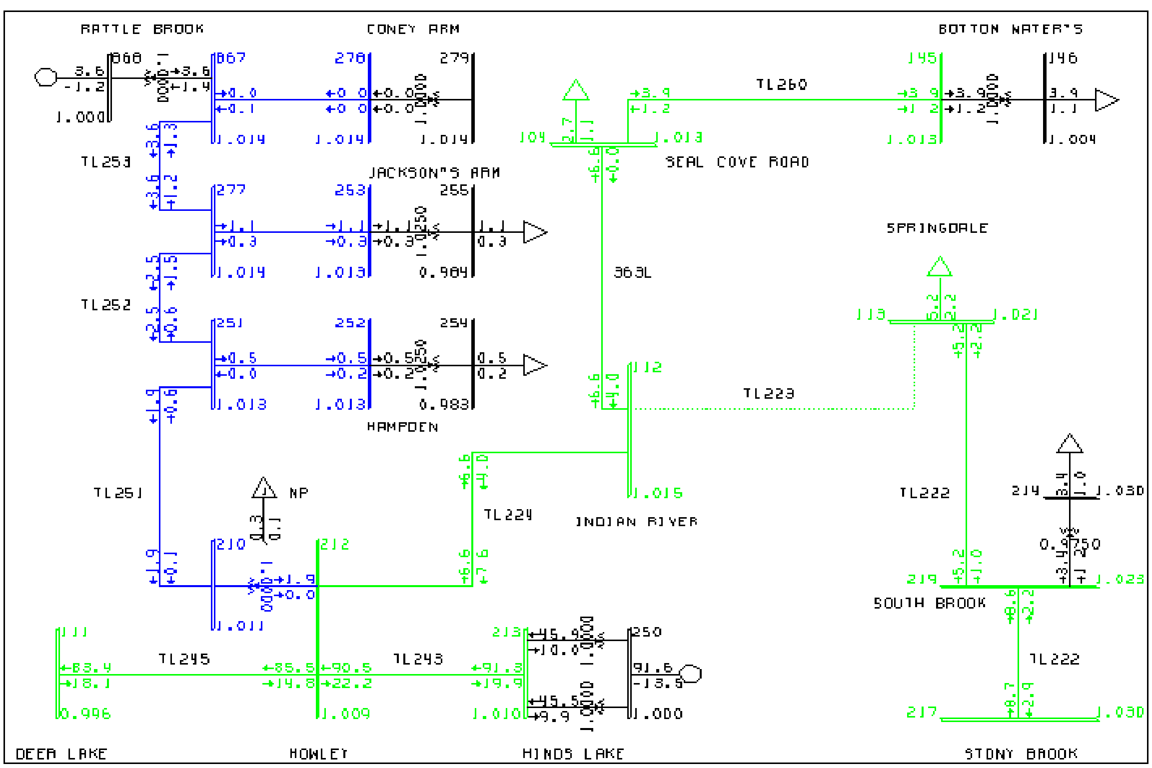


Figure 7 - 15 ^{o}C Day – Hinds Lake at 91.6 MW - TL222 IRV to SPL Out

For loss of TL224 (HLY – IRV) an overload condition is expected on TL245 (DLK – HLY) with Hinds Lake operating at 91.6 MW and Rattle Brook at 3.6 MW during 15 ^{o}C ambient temperatures. The Deer Lake T2 loading is expected to reach approximately 63 MVA. Figure 8 provides the load flow plot. The total load on the Hinds Lake generator equals 92.2 MVA. The line loading on TL245 during the outage to TL224 equates to 391.9 A, which is 102.4% of the 382.3 A (91.4 MVA) TL245 15 ^{o}C ambient rating. In order to avoid overloading of TL245, a marginal reduction (approximately 3 MW) in the output of Hinds Lake would be required for the 15 ^{o}C day.

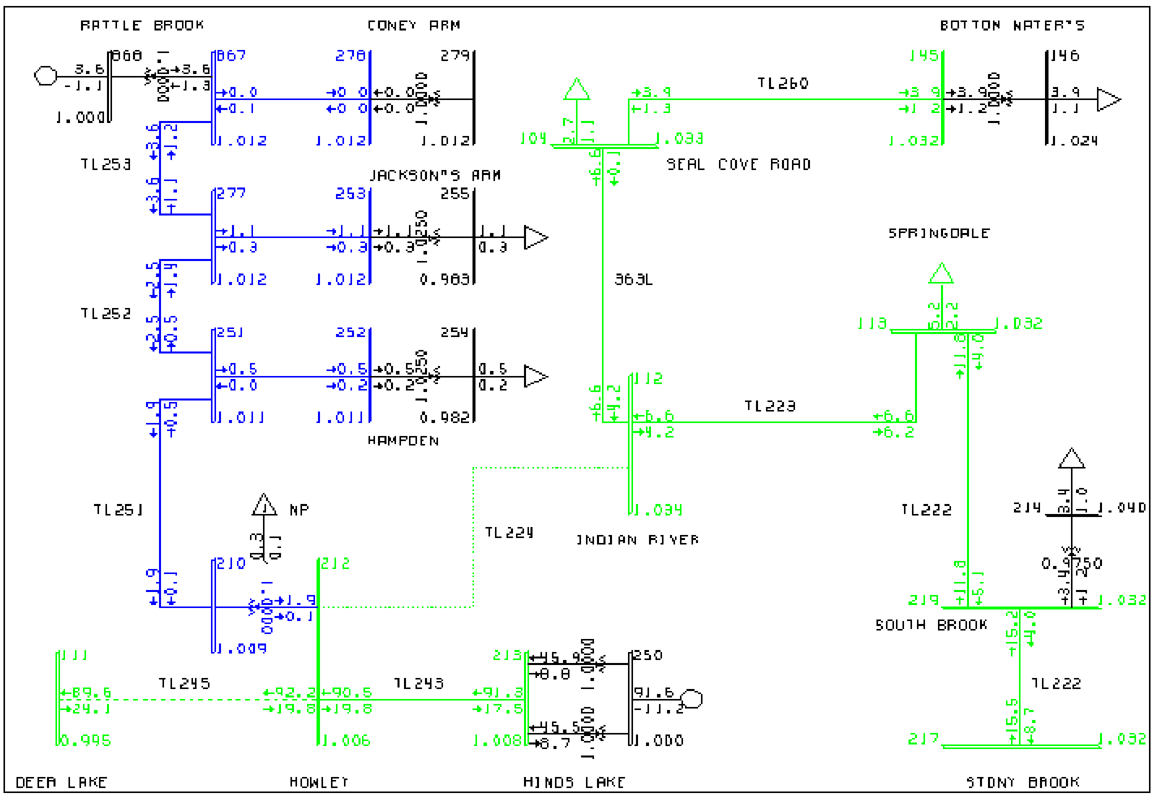


Figure 8 - 15 ^{o}C Day – Hinds Lake at 91.6 MW – TL224 HLY to IRV Out

Conductor ampacity calculations were completed to determine the maximum ambient temperature to avoid conductor sag violations on TL245 with TL224 out and Hinds Lake at 91.6 MW. Analysis indicates that the 266.8 MCM ACSR PARTRIDGE conductor on TL245 would have a rating of 395.4 A (94.5 MVA) for a 13 ^{o}C ambient temperature and 2 ft/sec wind. Therefore, reductions in the output of Hinds Lake are warranted for ambient temperatures above 13 ^{o}C in order to avoid thermal overloads of the 138 kV loop during loss of transmission line TL224. Figure 9 provides a plot of the maximum daily ambient temperature at Hinds Lake Generating Station as recorded by the EMS for the period 1999 to 2002. From Figure 9 one finds that the maximum daily ambient temperature to be below 13 ^{o}C from November 1st to May 1st based upon the three years of data.

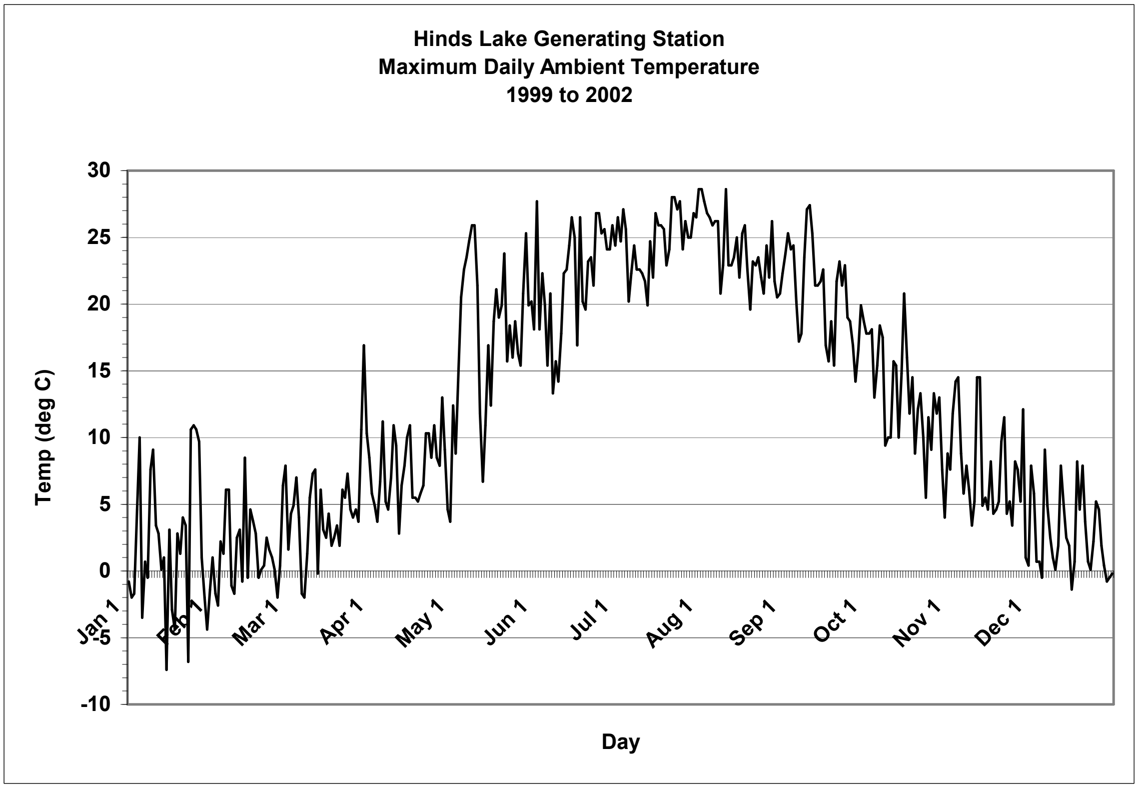
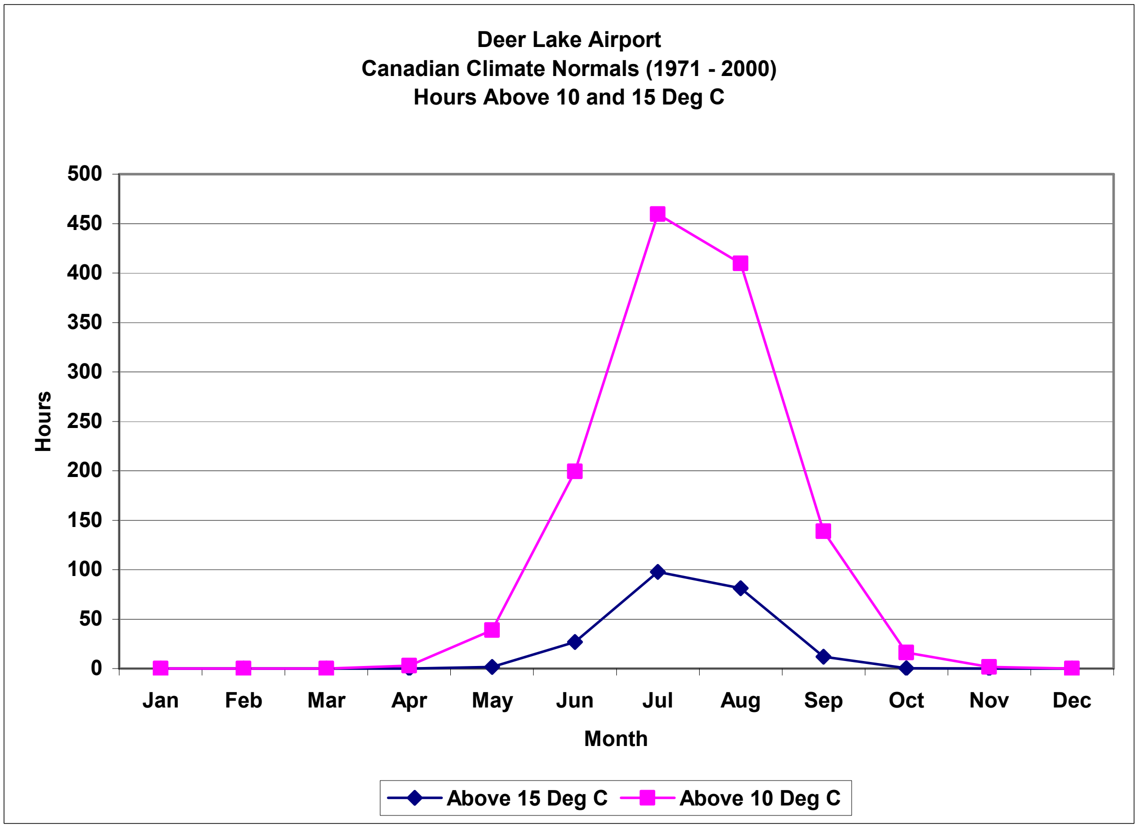


Figure 9

Further, the Environment Canada website (www.msc.ec.gc.ca) provides climate normals for the period 1971 to 2000 on a monthly basis. The data available includes degree days above 10 ^{o}C and degree days above 15 ^{o}C. The monthly values can be easily converted to the number of hours in each month that the temperature will be above 10 ^{o}C and 15 ^{o}C. Figure 10 provides a graph of the number of hours each month that the ambient temperature at Deer Lake Airport is expected to be above 10 ^{o}C and above 15 ^{o}C. It is expected that the number of hours that the ambient temperature is above 13 ^{o}C will fall between the 10 ^{o}C and 15 ^{o}C lines on Figure 10. Based upon Figure 10, it is clear that the number of hours where the ambient temperature is above 13 ^{o}C is insignificant for the period November 1st to May 1st. Consequently, overloading of TL245 for the loss of TL224 with Hinds Lake at 91.6 MW does not appear to be a concern for the November to May time frame. Note this is the time period when the maximum capacity of Hinds Lake would be required for the system.


Deer Lake Airport
Canadian Climate Normals (1971 - 2000)
Hours Above 10 and 15 Deg C
Hours
500
450
400
350
300
250
200
150
100
50
0
Jan
Feb
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Nov
Dec
Month
Above 15 Deg C
Above 10 Deg C

Figure 10

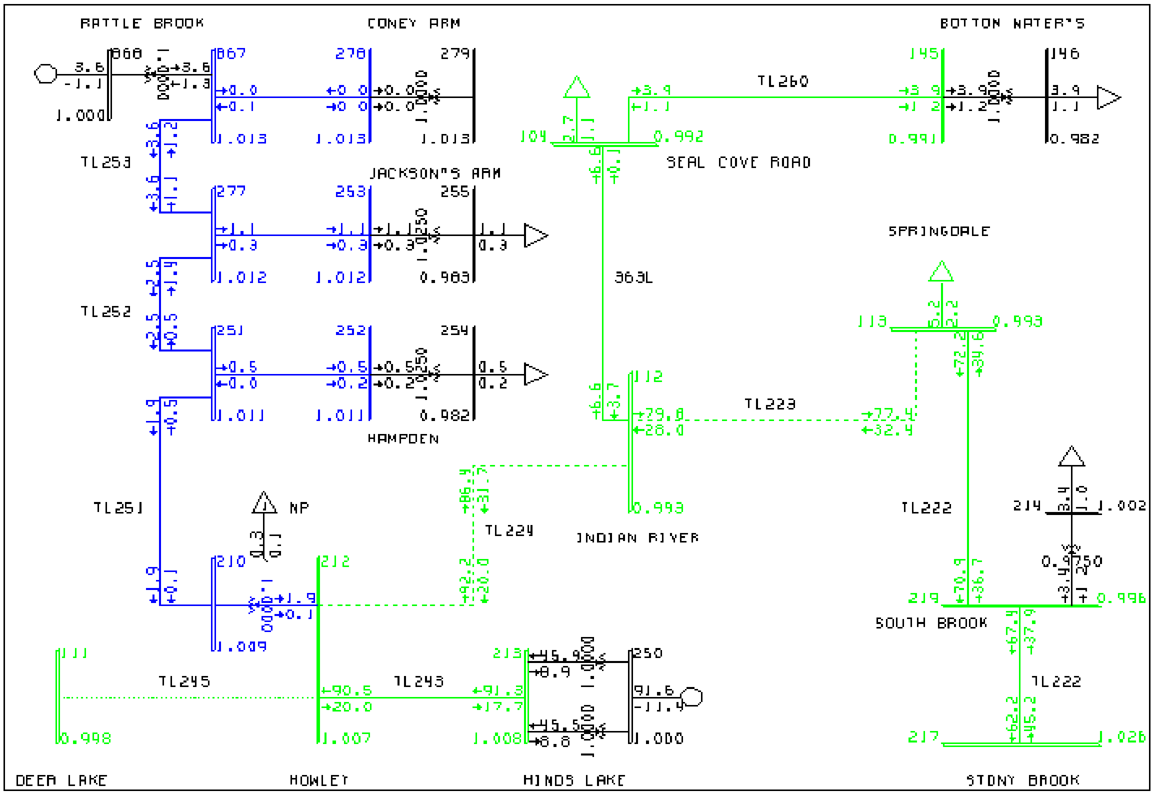


Figure 11 - 15 ^{o}C Day – Hinds Lake at 91.6 MW – TL245 DLK to HLY Out

Figure 11 provides the load flow plot with TL245 out of service and Hinds Lake at 91.6 MVA. The results indicate that TL224 will be loaded to 390.9 A (93.4 MVA), which is a line loading of 123.9% of rating. In addition, TL223 is loaded to 355.8 A (85 MVA), which is a line loading of 112.8% of rating. One will recall from Table 1 that the 0 ^{o}C ambient rating of TL224 and TL223 is 92.7 MVA. Therefore, for ambient temperatures above 0 ^{o}C, reduction in to output of Hinds Lake will be required to eliminate overloading and subsequent conductor ground clearance violations on TL224 and TL223 for an outage to TL245. Reducing the Hinds Lake output to 72 MW will result in a TL224 line loading of 314.4 A (75.1 MVA), which is 99.6% of the 15 ^{o}C ambient rating. Figure 12 provides the load flow plot with Hinds Lake at 72 MW and TL245 out.

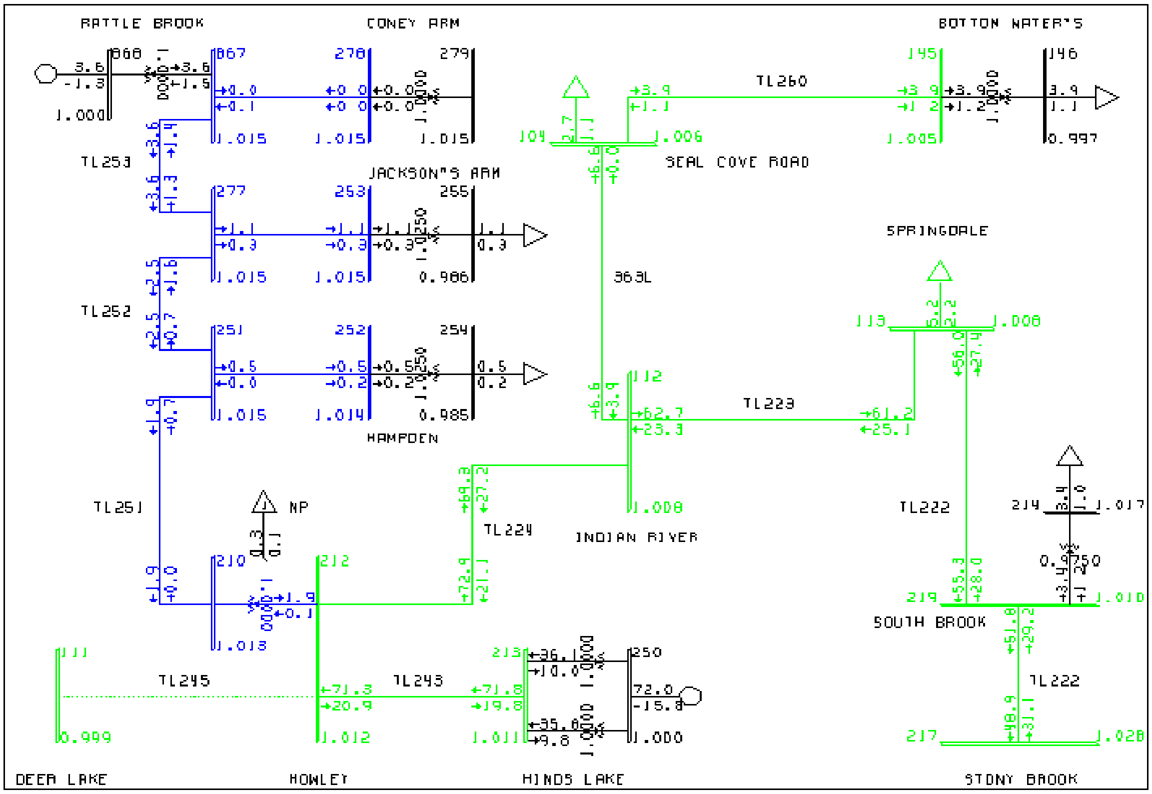


Figure 12 - 15 ^{o}C Day – Hinds Lake at 72.0 MW – TL245 DLK to HLY Out

For loss of Deer Lake T2, overloading of the 138 kV loop is not expected with Hinds Lake at 95.6 MVA. However, an overload of Deer Lake Power lines L1 and L2 (120% and 112% respectively) are expected due to the 138/66 kV T1 at Deer Lake Terminal Station and TL225 connection to Deer Lake Power. This is not a new issue, but one that exists at present. With Hinds Lake at 75 MW and Deer Lake T2 out, Deer Lake Power lines L1 and L2 are expected to be loaded to 111% and 104% respectively for an ambient temperature of 15 ^{o}C. Reducing the Hinds Lake output to 53 MW eliminates the overloads on Deer Lake Power lines L1 and L2 with Deer Lake T2 out for the 15 ^{o}C day.

The loss of one of the 230/138 kV power transformers at Stony Brook Terminal Station is expected to have little impact on the loading of the Hinds Lake generator or the Deer Lake to Stony Brook 138 kV loop. Figure 13 provides the load flow plot of the 138 kV loop with Stony Brook T1 out of service. Comparing Figure 13 with Figure 5 one notes a 0.1 MVAR difference in the output of Hinds Lake.

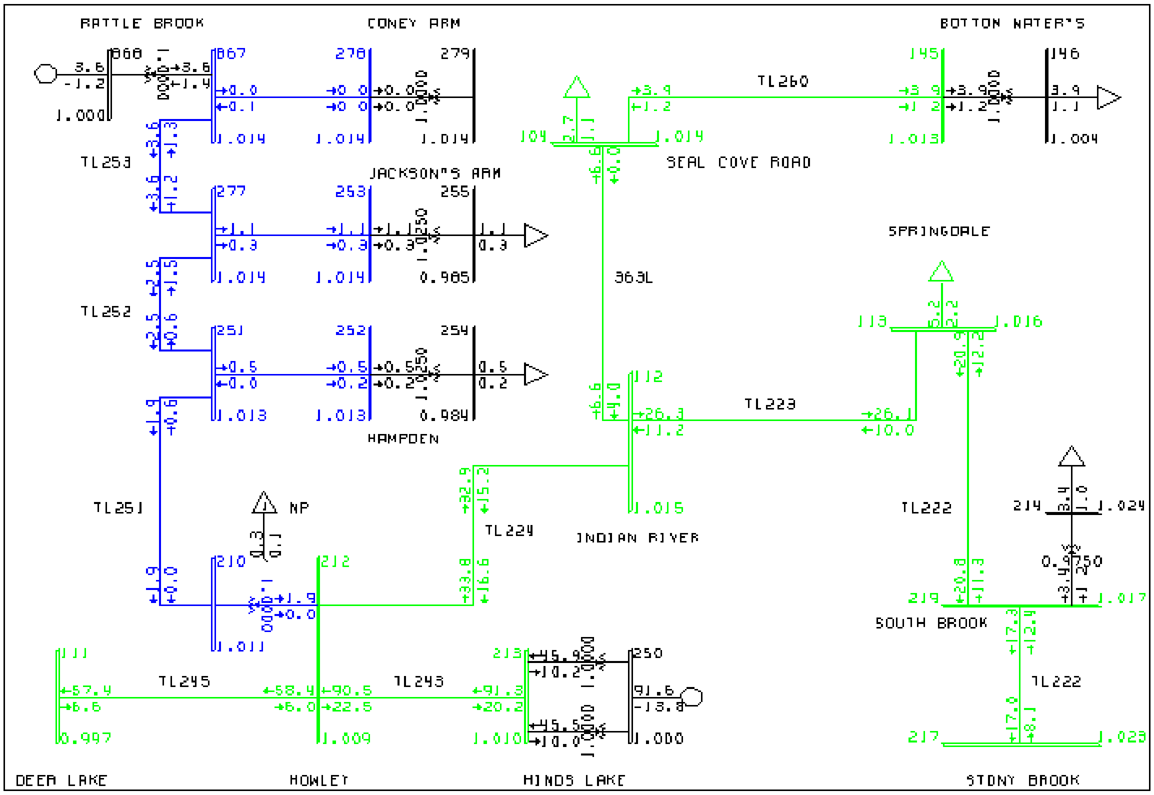


Figure 13 - 15 ^{o}C Day – Hinds Lake at 91.6 MW – STB T1 Out

An outage to Cat Arm including TL247 and TL248 would require that Hinds Lake provide the voltage regulation for the Deer Lake 138 kV bus. During ambient temperatures of 15 ^{o}C, load conditions would be such that MVAR would be flowing into the Deer Lake 138 kV bus from the GNP given the lightly loaded 138 kV radial system. As a result, Hinds Lake would be expected to absorb the additional MVAR. Figure 14 provides the load flow plot with Cat Arm and TL247/248 out of service. From the figure one finds that Hinds Lake would be required to absorb 18.8 MVAR during this contingency. The total generator loading would be 93.5 MVA or 97.8% of rating.

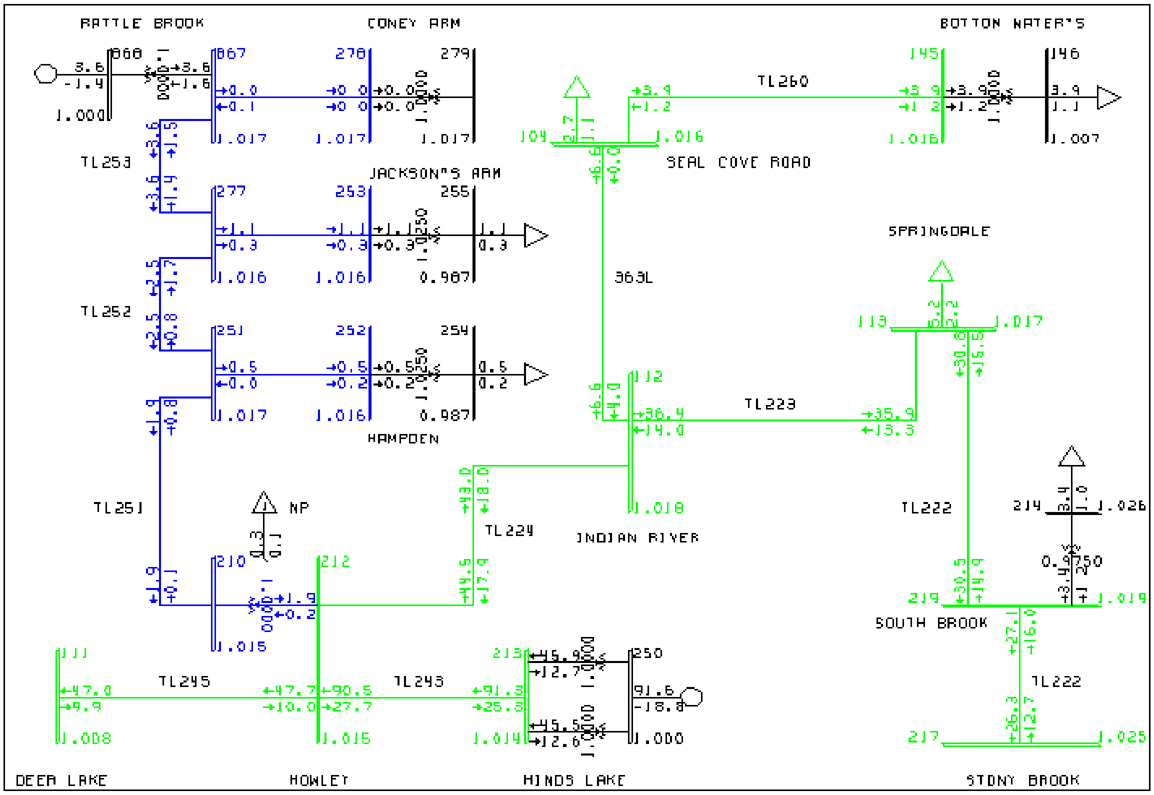


Figure 14 - 15 ^{o}C Day – Hinds Lake at 91.6 MW – Cat Arm Out

From the analysis completed, it is apparent that further increases in the MVAR loading of the Hinds Lake generator would require multiple contingencies during operation at 91.6 MW. One such contingency would be simultaneous outages to Cat Arm, TL247/248 and Abitibi Consolidated – Stephenville Division. In this scenario, Hinds Lake was found to absorb 20.7 MVAR for a total unit loading of 93.9 MVA or 98.2% of rating. Figure 15 provides the load flow plot.

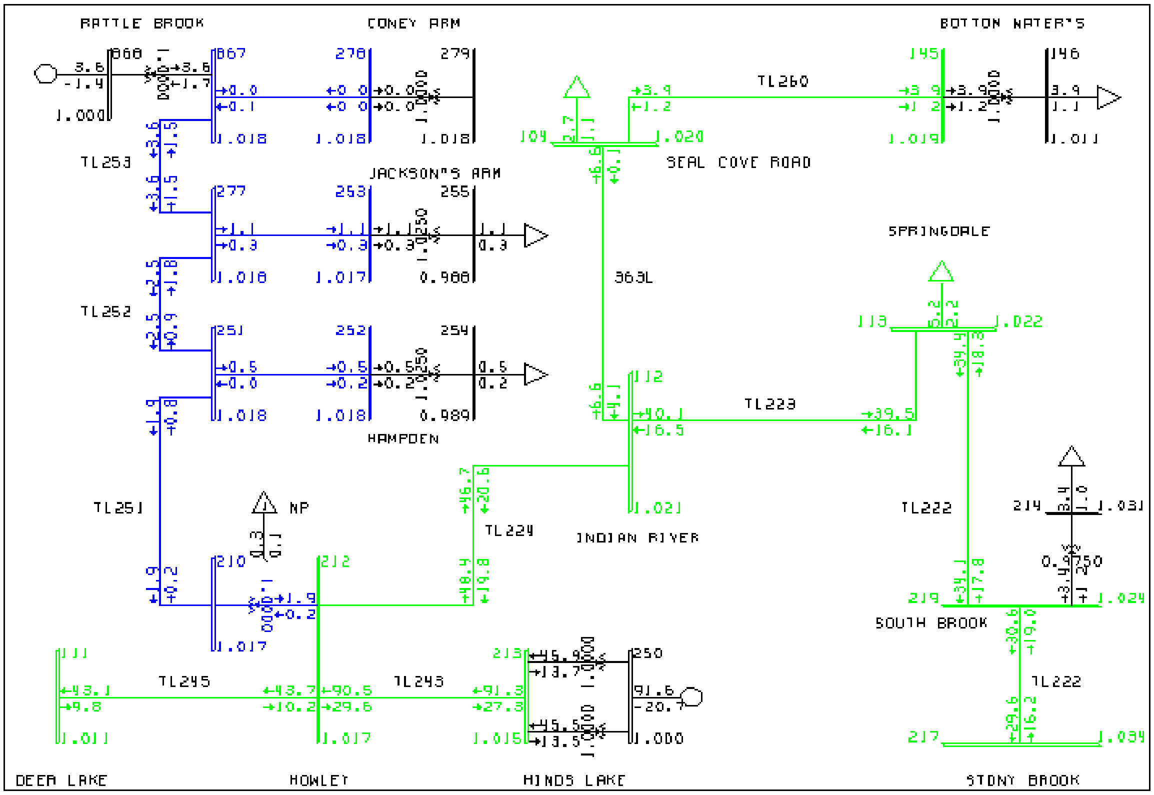


Figure 15 - 15 ^{o}C Day – Hinds Lake at 91.6 MW – Cat Arm & ACI-SVL Out

0 °C Ambient Temperature – System Peak

The 2005 peak load base case and the 0 °C thermal ratings of the transmission system are used to evaluate the impact of operating Hinds Lake at 95.6 MVA during system peak load conditions. The total Island load is set at approximately 1420 MW net NP generation and Cat Arm is generating 127 MW. Deer Lake T2 is in tap position 5 (nominal) to hold the Deer Lake 138 kV bus voltage to 1.00 p.u. Figure 16 provides the load flow plot of the peak load case with all lines in service. The analysis indicates that the Hinds Lake generator will be loaded to 92.8 MVA or 97.1% of rating.

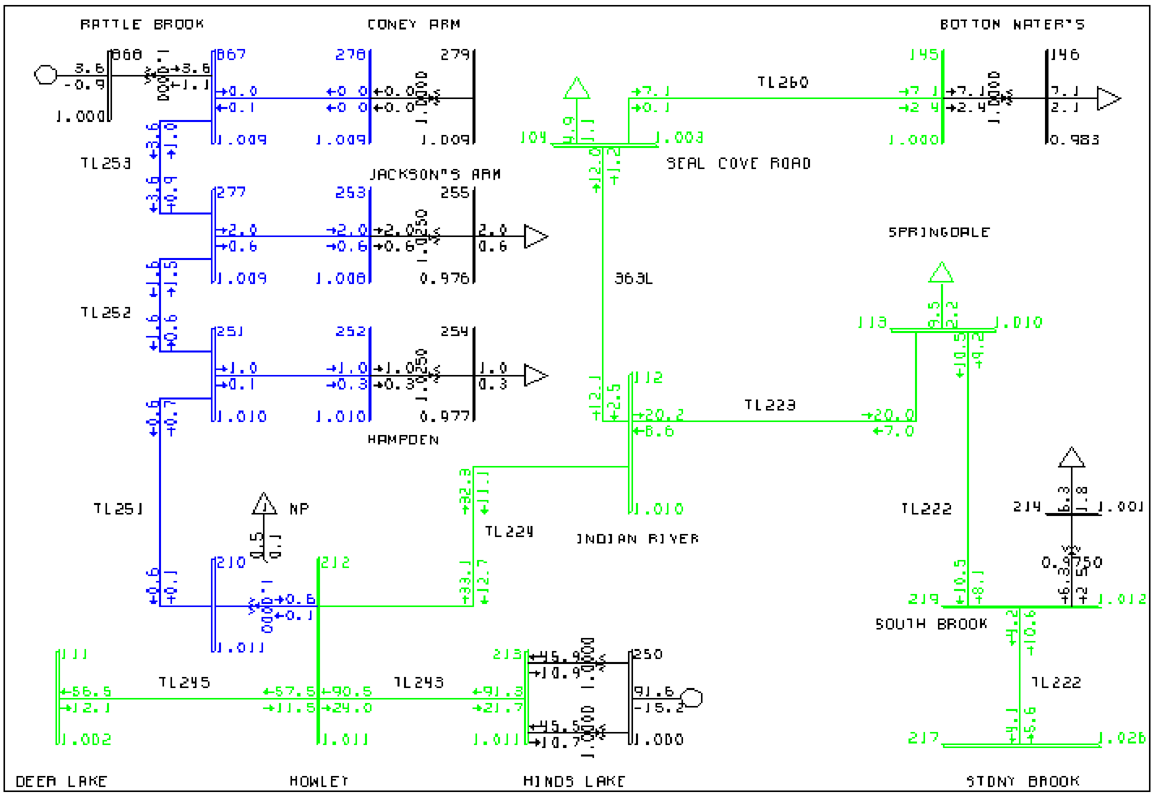


Figure 16 - Peak Day – Hinds Lake at 91.6 MW

The analysis for the 15 °C day indicated that outages to TL245 and TL224 were the most critical from a transmission constraint perspective. Figure 17 provides the load flow plot for the outage to TL245. With TL245 out and Hinds Lake at 91.6 MW, TL224 is loaded to 384.7 A (91.96 MVA) or 99.2% of the 0 °C ambient rating of the transmission line. The 138 kV bus voltages at Indian River and Springdale are within acceptable contingency limits at 98.6%. Increasing the terminal voltage at Hinds Lake would serve to increase the 138 kV bus voltages on the loop to unity, and, at the same time, reduce the loading on TL224.

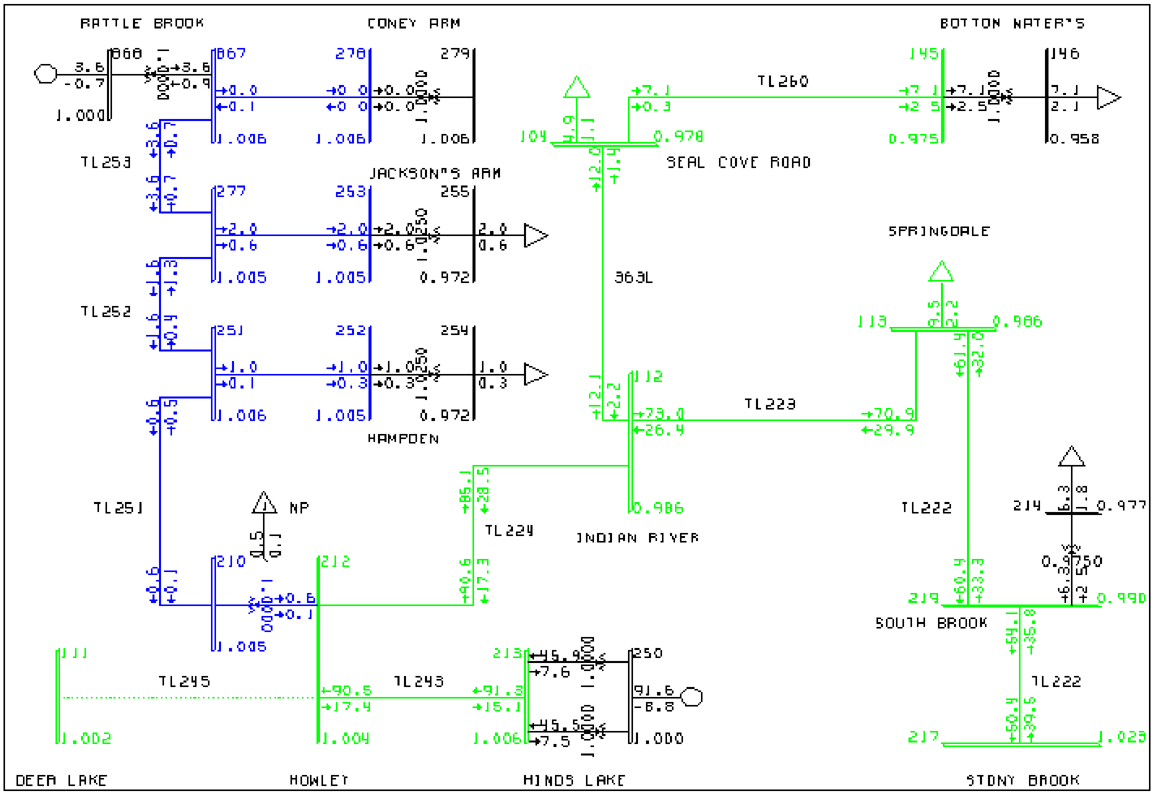


Figure 17 - Peak Day – Hinds Lake at 91.6 MW – TL245 DLK to HLY Out

Figure 18 provides the load flow plot for the TL224 outage during 0 ^oC ambient temperature conditions. With 15.5 MVAR being absorbed, the total load on Hinds Lake is 91.8 MVA or 97.1% of rating. TL245 is loaded to 388 A (93.8 MVA at 139.5 kV), which is 82.4% of the line rating.

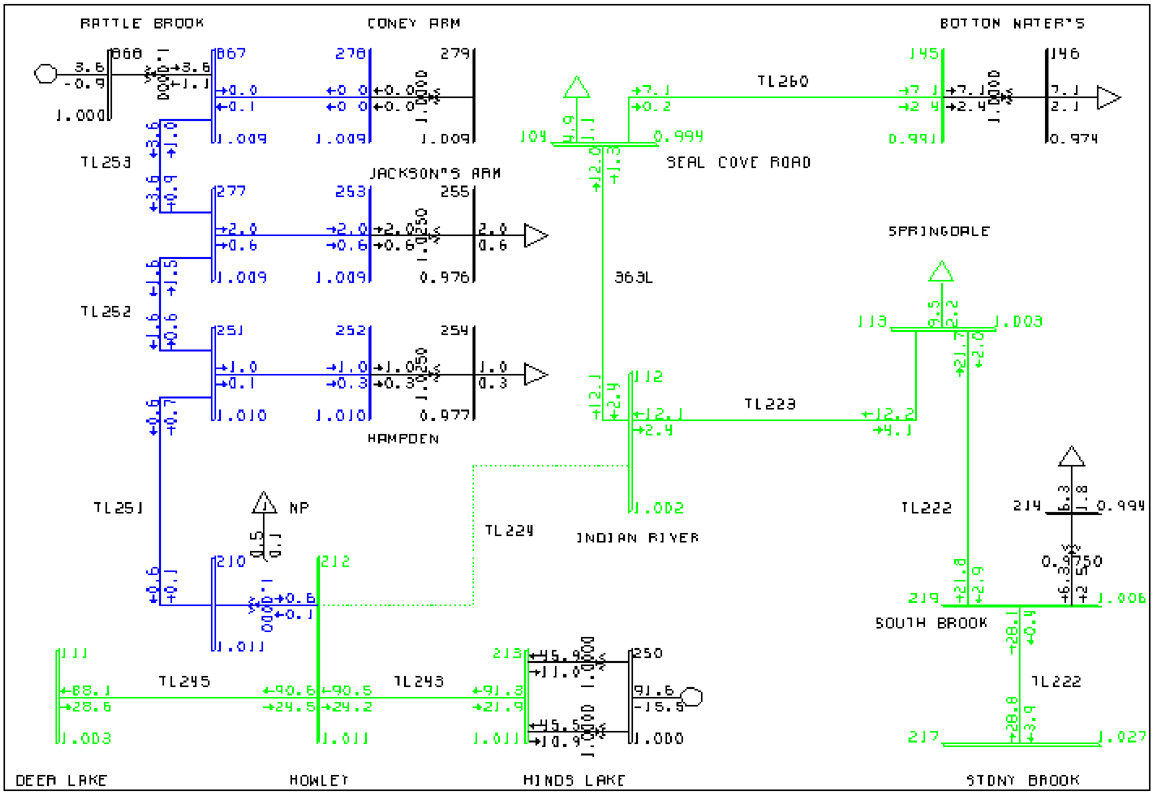


Figure 18 - Peak Day – Hinds Lake at 91.6 MW – TL224 HLY to IRV Out

STABILITY ANALYSIS

Increasing the capacity of the Hinds Lake plant from 75 MW to 91.6 MW has the potential to affect the frequency response of the system for loss of generation. The peak and light load base cases are used in a preliminary stability analysis to assess the impact of Hinds Lake capacity increase on system frequency response. The generation dispatches for the peak and light load cases are summarized in Table 3.

Table 3
Generation Dispatches

Unit	Peak Load MW	Light Load MW
Hinds Lake	75.0 / 91.6	75.0 / 91.6
Bay D'Espoir 1	64.4	51.6 / 49.8
Bay D'Espoir 2	64.8	Off
Bay D'Espoir 3	64.8	52.1 / 50.3
Bay D'Espoir 4	64.8	Off
Bay D'Espoir 5	64.8	52.1 / 50.3
Bay D'Espoir 6	64.8	Off
Bay D'Espoir 7	135.0	135.0 / 125.0
Holyrood 1	142.5 (net)	Off
Holyrood 2	142.5 (net)	Off
Holyrood 3	142.5 (net)	S.C.
Cat Arm 1	55.0 / 47.0	35.0
Cat Arm 2	55.0 / 47.0	Off
Upper Salmon	73.0	73.0
Granite Canal	23.0	23.0
Paradise River	8.0	8.0
Hardwoods GT	Off	Off
Stephenville GT	Off	Off
Deer Lake Power	79.1	79.1
CBP&P Steam	18.0	18.0
CBK FRC	18.0	18.0
ACI GFL G4	25	25
ACI GFL G5	4.5	4.5
ACI GFL G6	4.5	4.5
ACI GFL G7	4.5	4.5
ACI GFL G8	4.5	4.5
ACI GFL G9 (Beeton)	27	27
ACI Bishop's Falls	18.0	18.0
Star Lake	17.9	17.9
Rattle Brook	3.1	3.1
Rose Blanche Brook	6.1	6.1

Loss of 75 MW versus 91.6 MW Hinds Lake

The first issue under investigation is the impact that loss of a 91.6 MW Hinds Lake will have on system frequency when compared to the loss of a 75 MW Hinds Lake. The under frequency load shedding schedule is provided in Table 4.

Table 4
Under Frequency Load Shedding Schedule

	59.0 15 sec	58.8	58.8 6 sec	58.6	58.6 6 sec	58.4	58.4 6 sec	58.2	58.1	58.0	Total
NP	33.7	14		27		34		49.7	74.5	173.6	406.5
NLH		19		6		6					31
CBP&P		15									15
ACI-GF		6	14.5		14.5	6	14.5				55.5
ACI-SV		12		12		12		12		12	60
Total	33.7	66	14.5	45	14.5	58	14.5	61.7	74.5	185.6	568

Figure 19 provides the system frequency plot for the loss of Hinds Lake during system peak load conditions. The case with Hinds Lake at 75 MW is shown in red, while the 91.6 MW case is shown in green.

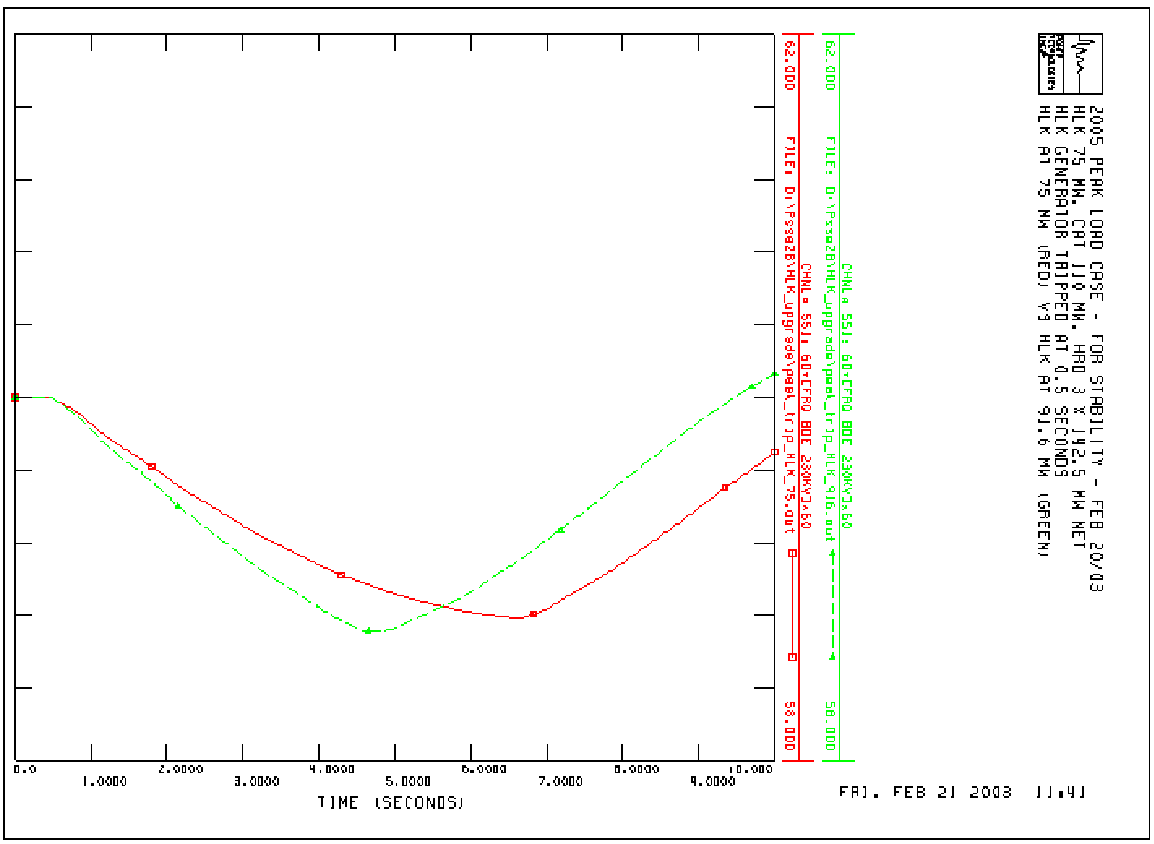


Figure 19 – System Frequency – Trip HLK - 75 MW vs 91.6 MW

As expected, the rate of change of frequency is greater for the 91.5 MW Hinds Lake case. As well, the minimum frequency is lower for the 91.6 MW Hinds Lake case. For loss of a 75 MW Hinds Lake during peak load conditions, the system frequency is expected to fall to 58.78 Hz compared to 58.71 Hz for the 91.6 MW Hinds Lake case. Based upon the under frequency load shedding schedule, the 16.6 MW increase in Hinds Lake capacity is not expected to increase the amount of load shed for loss of Hinds Lake over peak.

Figure 20 provides the system frequency plot for loss of Hinds Lake during light load conditions. Once again, the 75 MW Hinds Lake case is shown in red, while the 91.6 MW case is shown in green.

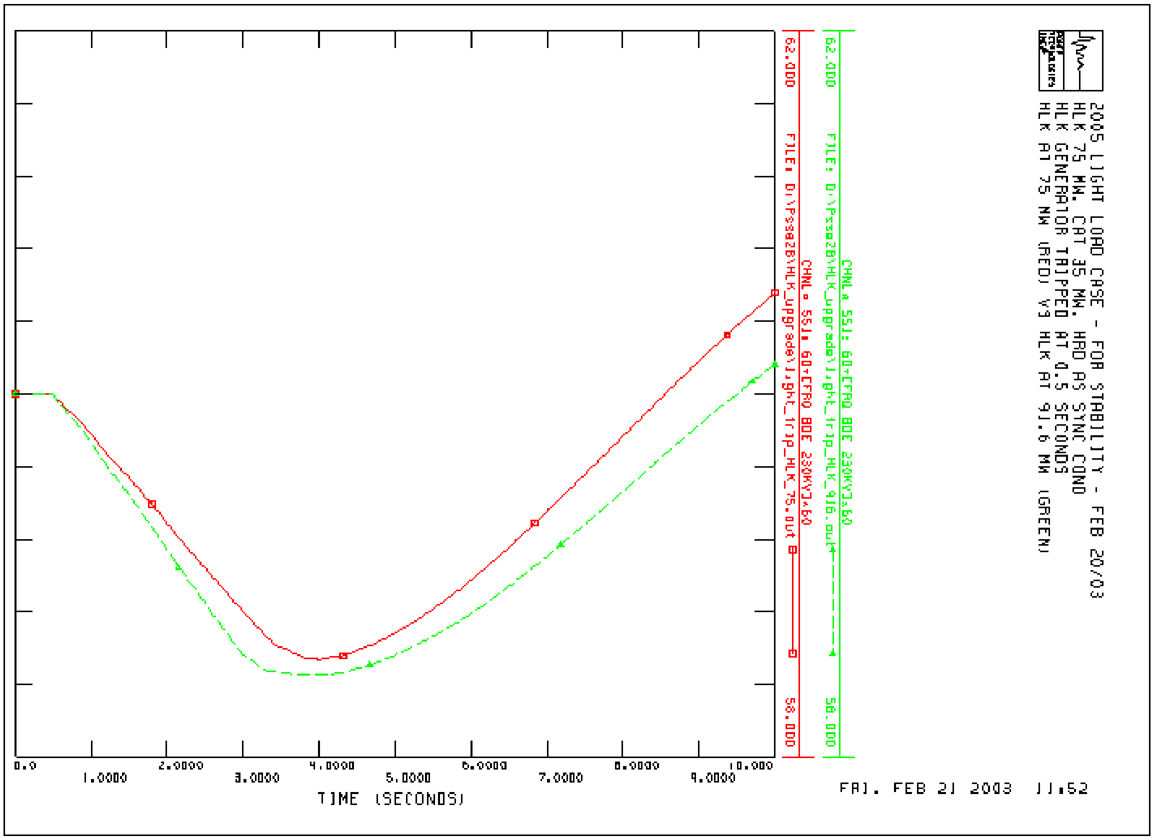


Figure 20 - System Frequency – Trip HLK - 75 MW vs 91.6 MW

For the light load case the frequency is expected to fall to 58.53 Hz for loss of a 75 MW Hinds Lake. Trip of a 91.6 MW Hinds Lake is expected to result in a minimum frequency of 58.45 Hz for the same light load case. Based upon the under frequency load shedding schedule, the increase in Hinds Lake capacity would not result in increased load shedding during light load conditions.

Frequency Response of Hinds Lake

The second issue under investigation is the response capability of an uprated Hinds Lake unit during loss of generation elsewhere on the system. The load flow analysis demonstrates that the operation of Hinds Lake at 91.6 MW during the summer months presents the risk of transmission line overloads for loss of transmission lines in the 138 kV loop between Deer Lake and Stony Brook. One option available is to operate Hinds Lake at 91.6 MW during winter peak conditions and limit the unit to a loading of 67 MW during the summer. This presents the potential for the system to have 24.6 MW of spinning reserve during the summer months compared to 10.3 MW for the existing runner. The counter argument to the potential benefit of addition spinning reserve lies within the mechanical design requirements. Given that the plant head, penstock diameter and scroll case will remain unchanged, increasing the capacity of the Hinds Lake turbine from 77.3 MW to 91.6 MW will require an increase in water flow. With the increase in flow, one would expect a marginal increase in penstock losses. However, for a runner upgrade to be effective, there must be at least a marginal increase in turbine efficiency, which, in turn, compensates for the increase in penstock losses. In addition to flow, one must consider the impact a runner replacement will have on penstock pressures and water hammer. The existing turbine has the wicket gate opening time set at 15 seconds and the closing time set at 23 seconds to ensure pressure drops on opening and pressure increases on closing are maintained within the design limitations of the penstock. As a result, the pick up rate of the turbine is limited to 77.3 MW in 15 seconds or 5.15 MW/s. For the uprated turbine, the load pick up rate will be held constant to avoid penstock damage. At a rate of 5.15 MW/s, the wicket gate opening time becomes 17.77 seconds for a turbine rated 91.6 MW (i.e. 91.6/5.15 = 17.77). In essence, the wicket gate velocity will be slower for the 91.6 MW runner when compared to the existing runner (i.e. 0 to 100% in 17.77 seconds versus 0 to 100% in 15 seconds).

Preliminary stability analysis is used to assess the relative impacts of increased spinning reserve and slower wicket gate velocities on Hinds Lake's response to loss of system generation during light loads. For this analysis the light load base case is used. In each case a trip of Upper Salmon at 73 MW is simulated.

Figure 21 provides the frequency response of the system for the loss of Upper Salmon during the light load condition. The system frequency is shown in red for the existing 83.3 MVA Hinds Lake unit with 10.3 MW of spinning reserve, while the 95.6 MVA Hinds Lake unit with 24.6 MW of spinning reserve is shown in green. The simulation demonstrates that an identical response can be expected for the first 3 seconds following loss of Upper Salmon. This is attributed to the fact that there has been no change to the water start time or governor time constants on the system. The analysis indicates that there will be very little difference in the minimum frequency observed (i.e. 58.82 Hz for the 83.3 MVA machine and 58.587 Hz for the 95.6 MVA machine) and, as a result, it is expected that the same amount of load will be shed in each case.

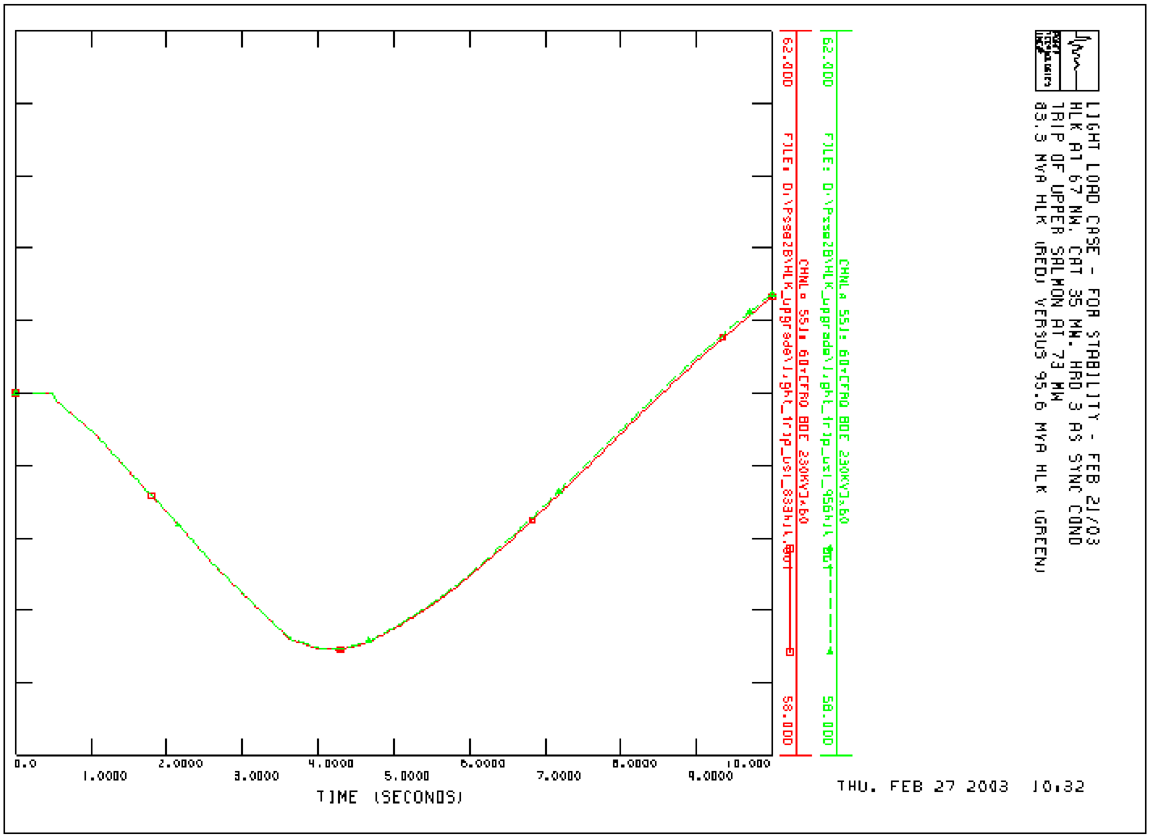


Figure 21 – Trip of USL at 73 MW – 83.3 MVA HLK vs 95.6 MVA HLK

With respect to frequency recovery, there is a very marginal improvement provided by the 95.6 MVA machine with 24.6 MW of spinning reserve following under frequency load shedding. This is attributed to a slight gain in frequency response of the 95.6 MVA unit based upon the trade off between increased spinning reserve and lower wicket gate velocity. The gain is also evident in the plot of Hinds Lake MW provided in Figure 22. Figure 22 provides a plot of the Hinds Lake MW for loss of Upper Salmon at 75 MW. The 83.3 MVA machine response is shown in red and the 95.6 MVA machine response is shown in black. One notices that the 95.6 MVA machine picks up approximately one additional MW during the disturbance.

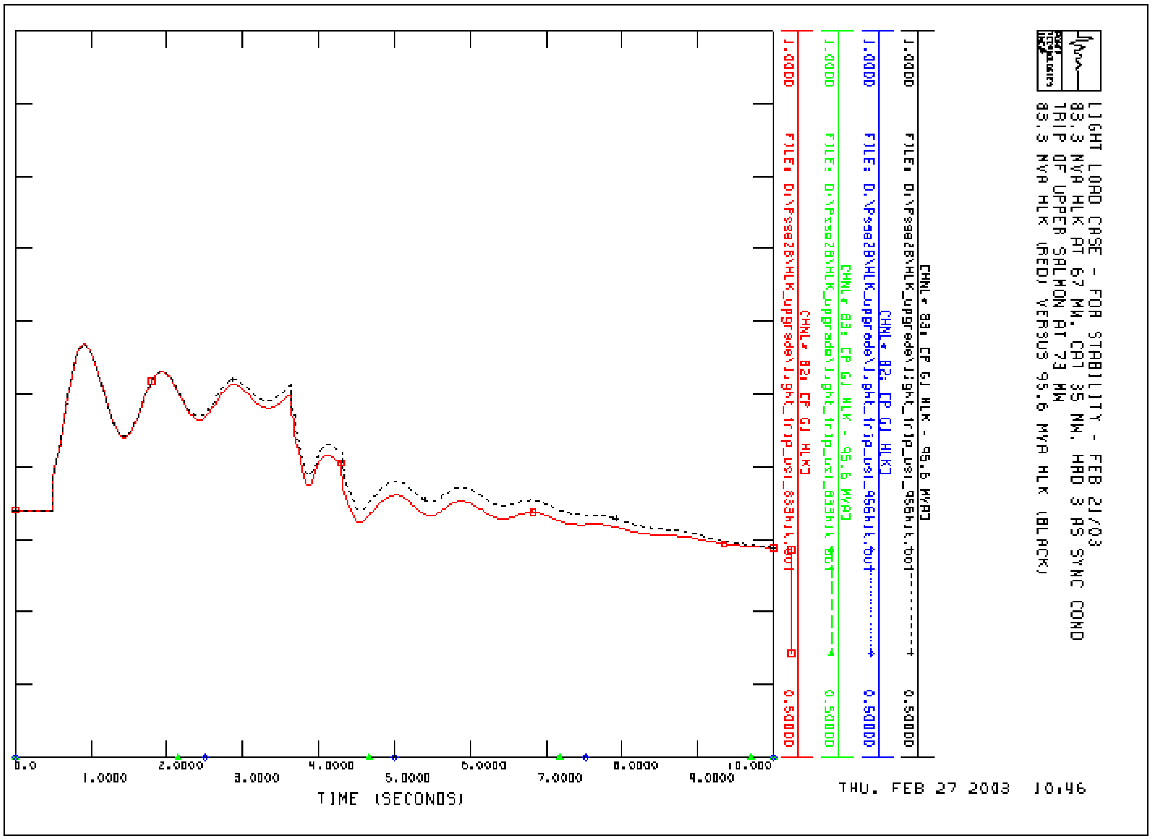


Figure 22 – Trip of USL at 73 MW - Hinds Lake MW

CONCLUSIONS

The load flow analysis indicates that operation of Hinds Lake at 90 to 95 MW during the summer months presents the risk of transmission line overloads in the range of 120 to 150%, as well as overloading of Deer Lake T2, for loss of transmission lines in the 138 kV loop between Deer Lake and Stony Brook.

Operation of Hinds Lake at 91.6 MW appears to provide adequate MVAR capacity from the machine for voltage support during line outages on the Deer Lake to Stony Brook 138 kV loop.

Operation at 91.6 MW during 15 ^{o}C ambient temperature conditions will result in a TL224 loading of 123.9% for loss of TL245 and a TL245 loading of 102.4% for loss of TL224. There are no apparent transmission line overloads for operation of Hinds Lake at 91.6 MW during 13 ^{o}C ambient temperature conditions.

For ambient temperatures below 0 ^{o}C, there are no apparent transmission line constraints.

Without upgrades to the 138 kV transmission loop two possible operating scenarios exist for a 91.6 MW Hinds Lake. First, NLH may choose to have dual ratings for the plant. In this scenario Hinds Lake is rated and operated at 91.6 MW during winter, or peak load conditions from November 1st to May 1st. During the summer months, or May 1st to November 1st, Hinds Lake is rated and operated as a 75 MW plant. The second scenario would have the plant rated and operated at 91.6 MW year round with operating procedures and/or special protection schemes that would effect plant output reduction to eliminate transmission system overloading for line out contingencies on the 138 kV loop.

Dual rating of an upgraded Hinds Lake turbine runner will require detailed analysis of turbine efficiencies and overall impact on water management.

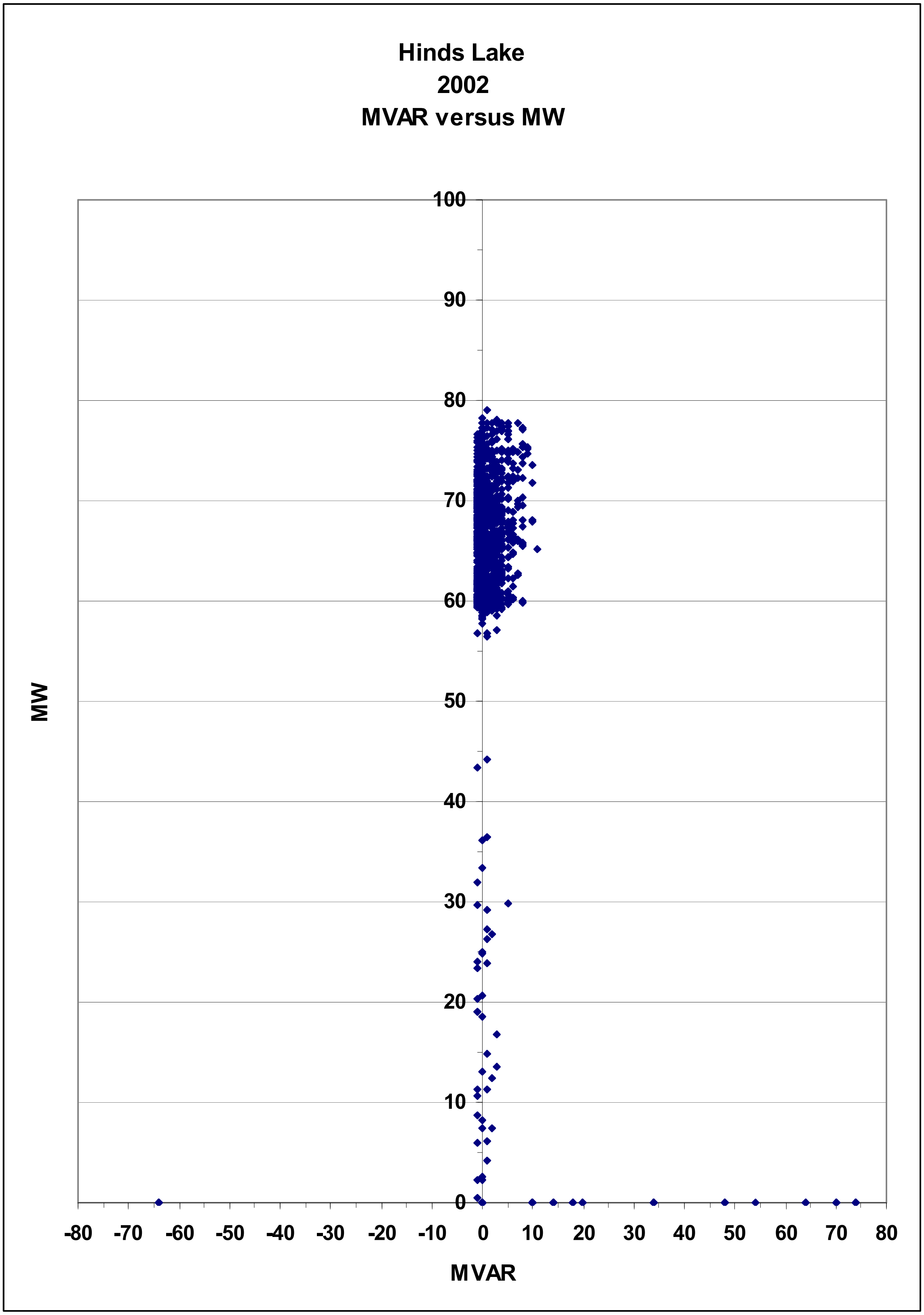
Preliminary stability analysis indicates that an loss of Hinds Lake during operation at 91.6 MW will result in a 0.07 Hz increase in the frequency deviation when compared to loss of a 75 MW Hinds Lake based upon the existing under frequency load shedding schedule.

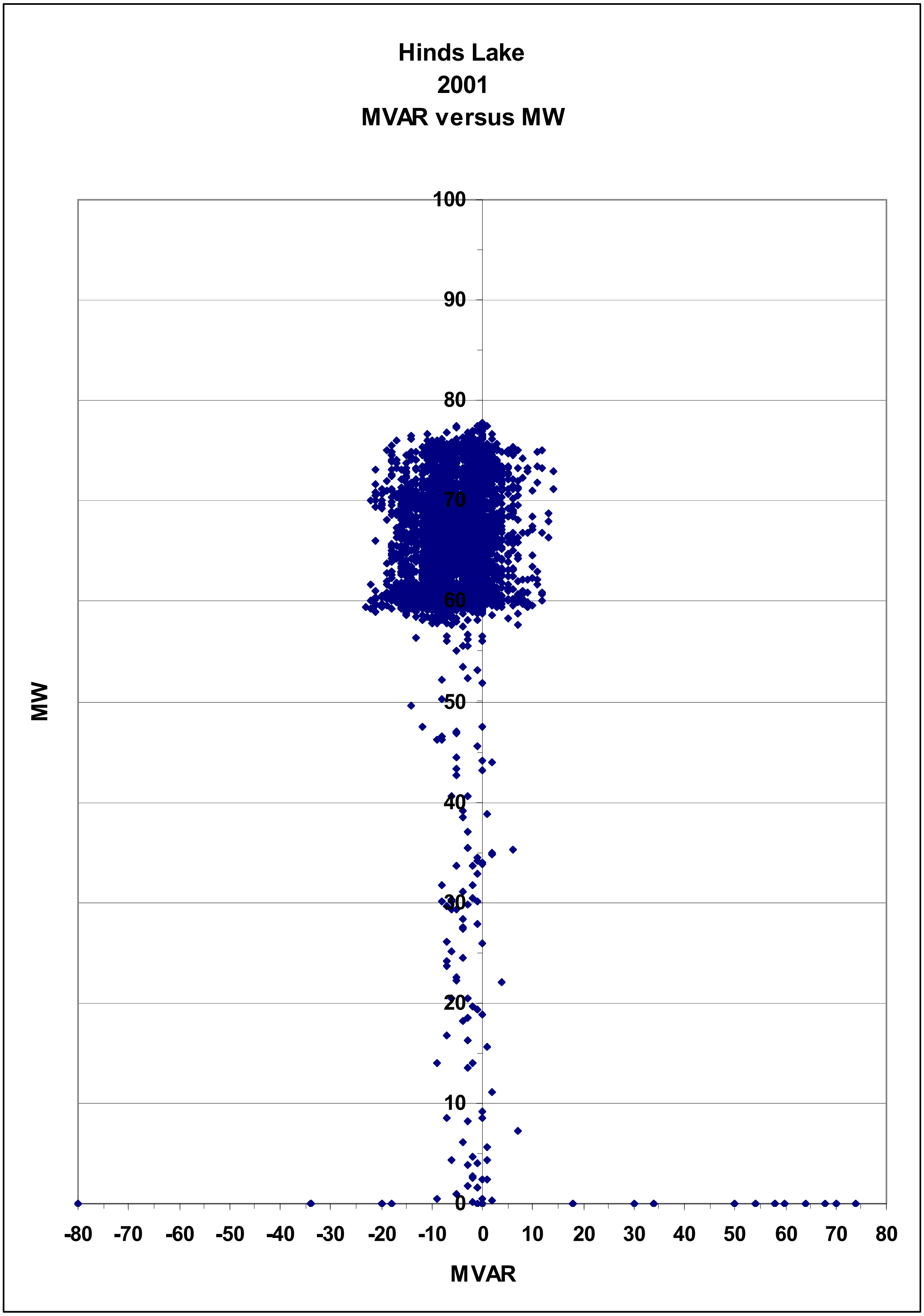
Operation of the upgraded unit at 67 MW during the summer months will increase the spinning reserve at Hinds Lake from 10.3 MW to 24.6 MW. However, the additional spinning reserve does little to improve the initial frequency drop based upon the existing under frequency load shedding schedule due to fixed water start times and governor time constants. The additional spinning reserve of the 95.6 MVA Hinds Lake unit provides marginal improvements in frequency recovery following load shedding.

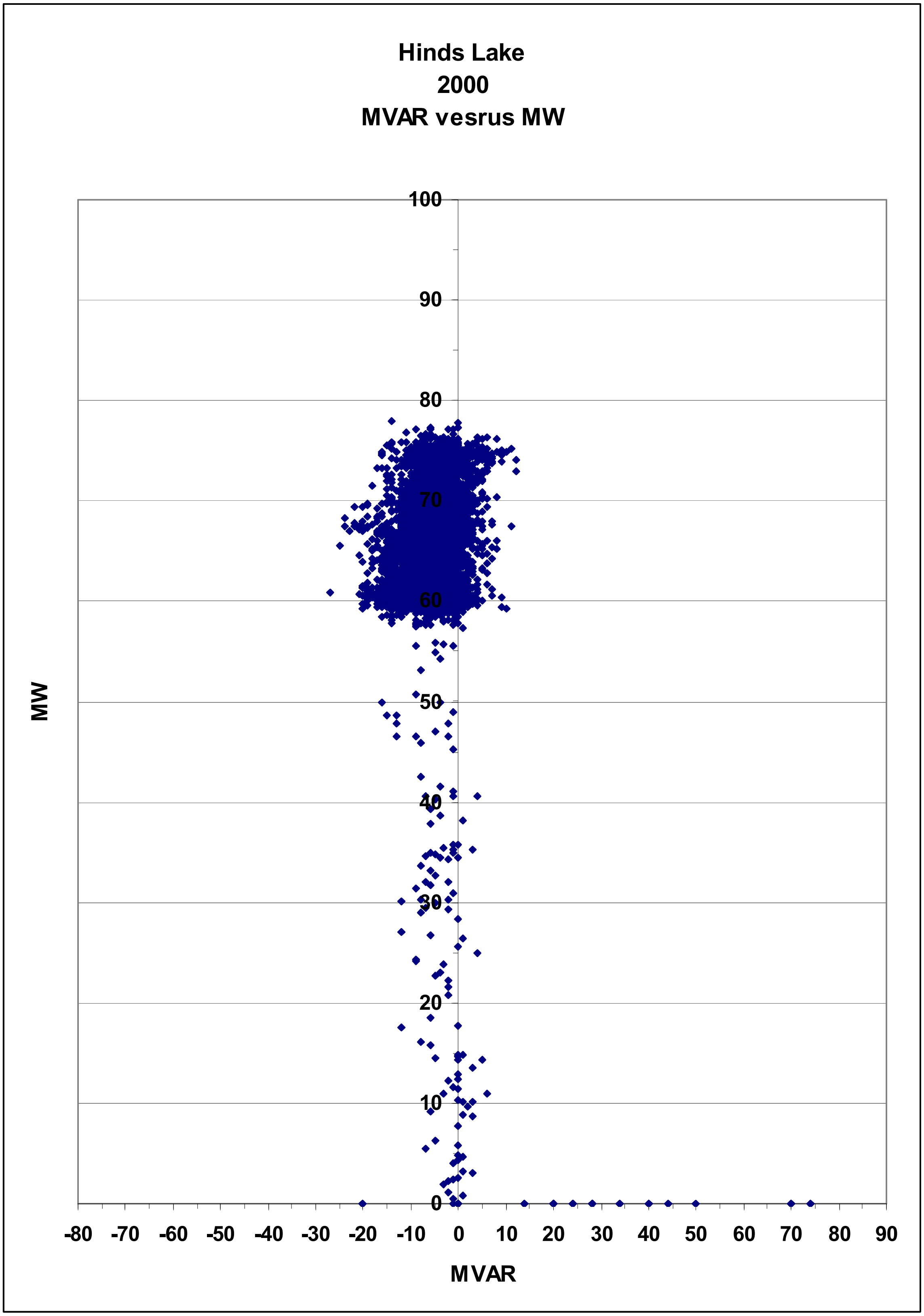
A maximum rating of 91.6 MW is recommended for the potential Hinds Lake turbine runner upgrade.

Appendix A

Hinds Lake MVAR Versus MW Curves


Hinds Lake
2002
MVAR versus MW
MW
MVAR
100
90
80
70
60
50
40
30
20
10
0
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80


Hinds Lake
2001
MVAR versus MW
MW
MVAR
100
90
80
70
60
50
40
30
20
10
0
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80


Hinds Lake
2000
MVAR vesrus MW
MW
MVAR
100
90
80
70
60
50
40
30
20
10
0
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
90

IC 126 NLH
Attachment 3
2006 NLH GRA

FILE: 123.80.20/45.00

HYDRO
THE POWER OF COMMITMENT

BAY D'ESPOIR GENERATING STATION

UNIT 7 RUNNER REPLACEMENT

REGISTERED PROFESSIONAL ENGINEER
J. E. HALLAM
SIGNATURE
PROVINCE OF NEWFOUNDLAND

Approved: M.S.Rana

Generation Engineering
2004-04-06

SUMMARY

This report presents the capital costs for a replacement runner for Bay D'Espoir Unit 7 and the benefits which would result. A runner of modern design can offer increased capacity, efficiency and improved cavitation resistance. As part of the runner replacement project, the existing floating rim generator rotor would be strengthened, to eliminate the potential risk of rotor unbalance and unit outage as a result of an overspeed, a situation which has occurred several times in that past. The report does not contain recommendations pertaining to the viability of the project, as this will be determined by System Planning.

TABLE OF CONTENTS

1. INTRODUCTION

Discussions with GE Hydro concerning Unit 7 were initiated in the fall of 2000, to discuss our concern with the floating rim rotor. Unit 7 was constructed with a floating rim type spider, which is much less rigid than more conventional designs. This type of spider construction was used at many installations at about that time. This has caused serious problems on a number of occasions following over speed events. When subjected to an over speed, the floating rim sometimes does not return to its original position, resulting in a dynamic unbalance, which causes unacceptably high vibration. The vibration must be corrected by rebalancing the rotor, a time consuming process which removes the unit from production until it can be completed.

During these discussions, GE Hydro indicated that it might be possible to increase the unit's capacity by as much as 10% by replacing the runner. Discussions proceeded over the following year and a half and have culminated in the receipt of two proposals from GE Hydro , dated 2002-04-16 and 2002-05-29. In both cases, the proposed runner would fit within the existing turbine without significant modifications. This report contains the proposals from GE Hydro, with an estimated cost to modify the unit as proposed by GE Hydro and an analysis of the benefits these modifications will provide.

All costs presented in this report are in January 2004 Canadian dollars.

2. FIRST PROPOSAL BY GE HYDRO

This proposal was dated 2002-04-16. The performance curve for this runner is presented in Figure 1. It indicates a slight increase in capacity (about 2.2 MW) and a slight increase in

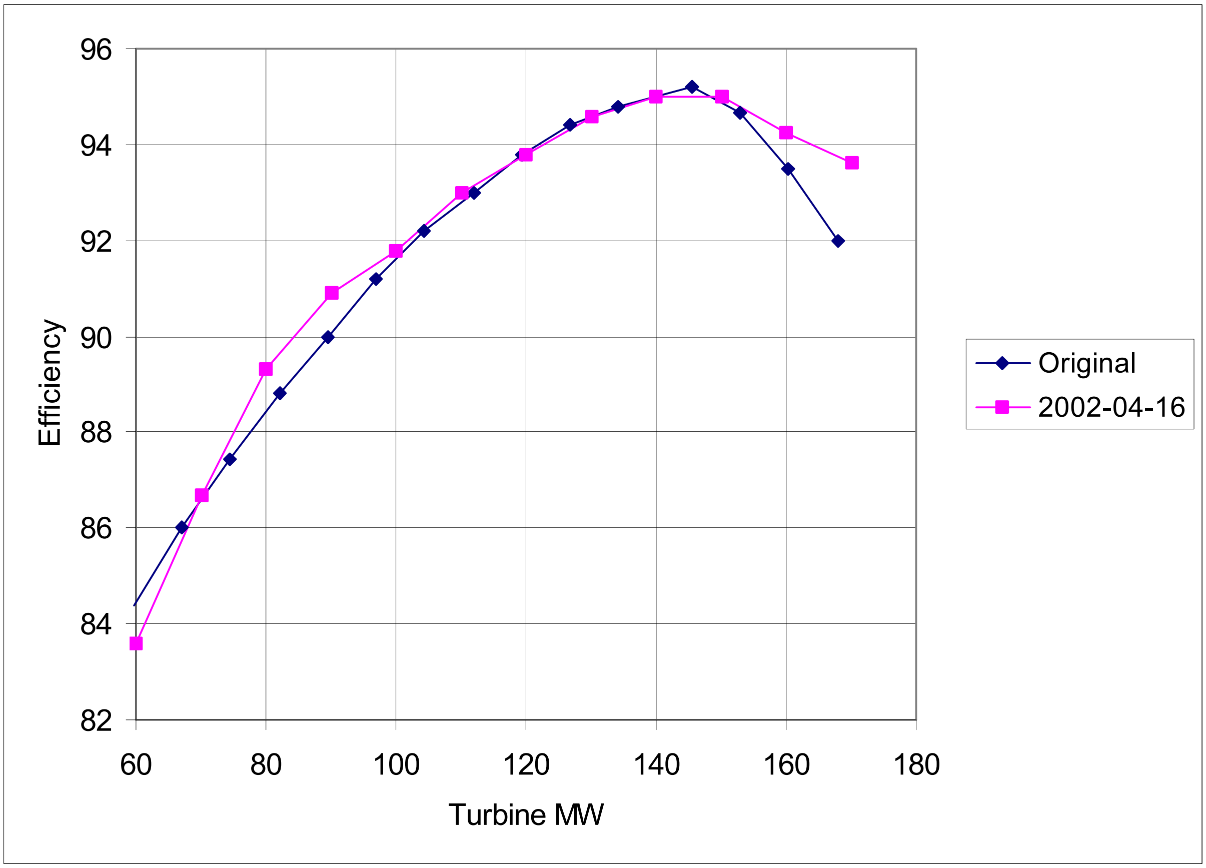


Figure 1

efficiency between 70 and 100 MW and above 150 MW. (The “Original” performance curve was obtained from the Dominion Engineering Works proposal for unit 7. It has not been verified by field testing.) GE Hydro prepared the new performance estimates based on a tail water elevation of 0.61 m, which is lower than generally encountered at Bay D’Espoir. GE Hydro was informed that, based on a review of several years of operating data, the minimum, average and maximum tail water elevations are 0.8, 2.2 and 3.2 m, respectively. GE Hydro reconsidered the performance predictions made and responded with a second proposal.

3. SECOND PROPOSAL BY GE HYDRO

This proposal was dated 2002-05-29. The performance curve for this runner is shown in Figure 2. It indicates that GE Hydro had revised their original proposal to accentuate increased capacity and efficiency at high output.

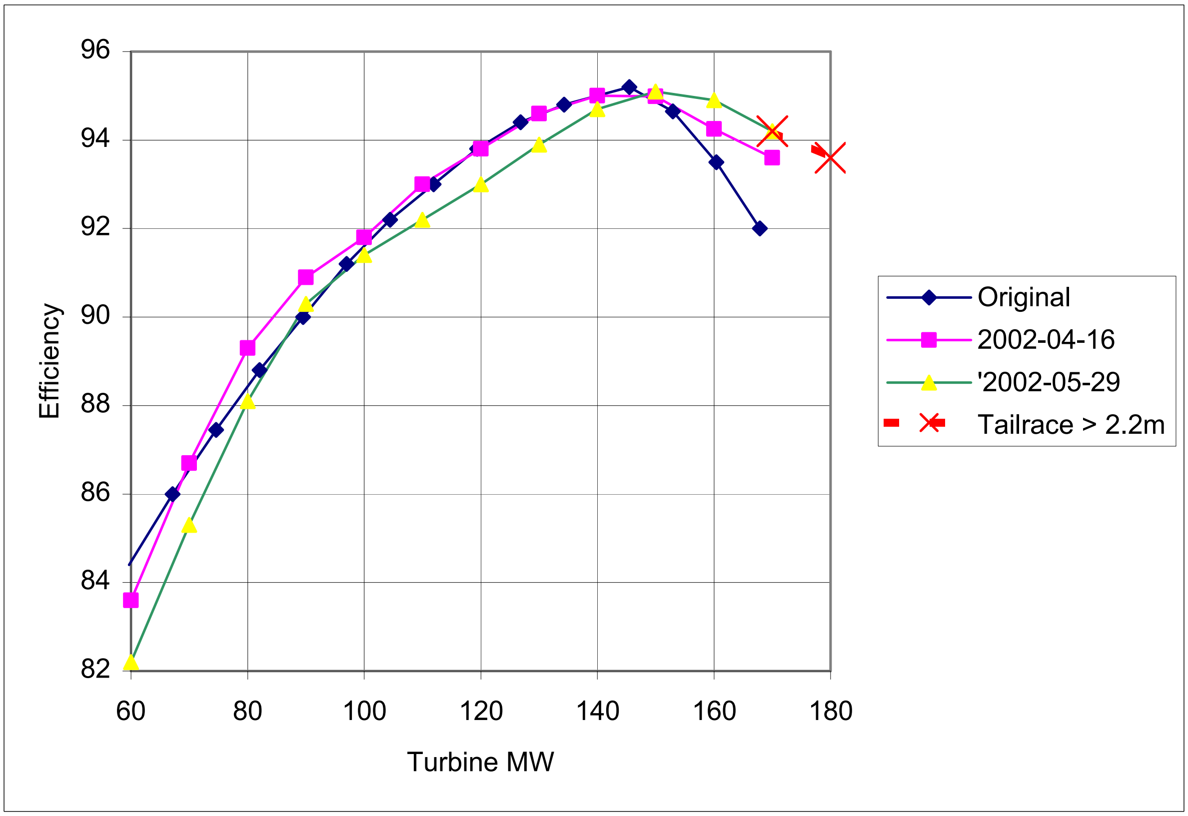


Figure 2

The curve has the same shape as the 2002-04-16 proposal but has been shifted to the right. Note also the section of this curve to the extreme right which has been identified as “Tailrace >2.2m”. GE Hydro has offered a runner which can produce significantly more MWs, depending on tail water elevation, as shown in Table 1.

Tail water Elevation (m)	Turbine MW
0.8	170
2.2	180
3.2	188

Table 1

The output is limited by the requirement to provide cavitation protection for the runner. As water flows through the runner the pressure decreases as energy is extracted from the water by the runner. Pressure decreases and under certain operating conditions can drop below the pressure at which water will boil. Bubbles form and collapse violently at a point where the pressure increases beyond the boiling point. This violent collapse of the bubbles is called cavitation and it can result in severe damage to the runner. One of the ways that cavitation can be prevented is by providing tail water protection. That is, the runner is positioned sufficiently lower than the minimum expected tail water elevation to ensure that the pressure at any point in the runner will not decrease below the point at which bubbles can form. The original runner was designed to operate cavitation free at expected tail water elevations. The design of the proposed new runner has been stretched to the limit and, in effect, beyond the limit at some tail water elevations.

Generator Rotor Spider

Unit 7 generator was designed and constructed with a floating rim. The term “floating rim” is just another way of saying that the spider is much less stiff than more conventional designs. This has caused problems several times in the past, requiring rebalancing following a unit trip and overspeed. We should consider that we have been fortunate in that we have been able to balance the unit to within acceptable (but on some occasions, less than desirable) limits quickly. We can expect this to occur again and we should also expect the situation to recur with sufficient severity that a significant delay would be experienced in returning the unit to service. This could have a detrimental affect on our ability to meet energy demands if such an event occurs during a peak production period.

Capital Cost

The capital cost estimate is summarized in Table 2, in January 2004 Canadian dollars.

Item	Capital Cost
Supply runner, spider, misc materials	$2,000,000
Install runner, spider, misc materials	$275,000
Engineering and Project Management	$155,000
Hydro forces	$175,000
Environment	0
Contingency	$261,000
Allowance for Funds During Construction	Not Included
Corporate Overheads	$172,000
Escalation	Not Included
Total	$3,038,000

Table 2

This is a prefeasibility class estimate and has an accuracy of + or - 15%. See Appendix I for the project cash flow.

4. DISCUSSION

Capacity Increase

The extent to which the capabilities of the proposed new runner could be exploited is limited by the need to provide cavitation protection. Expressed another way, the maximum output is limited by the tail water elevation. Tail water elevation at Bay D'Espoir Unit 7 is affected by three principal variables: total flow through Units 1-6 in Powerhouse 1, flow through Unit 7 in Powerhouse 2 and tide. Hourly operating data for a recent three year interval (1999-01-01 to 2002-04-26) was reviewed and Table 3 indicates the number of hours Unit 7 operated at various tall water elevations for that period.

Tail water Elevation Greater Than (m)	Number of Hours	Percent of Time
0.8	37481	98%
1	36346	95%
1.2	34503	91%
1.4	32035	84%
1.6	28533	75%
1.8	24702	65%
2	20756	55%
2.2	14916	39.2
2.4	10671	28.0
2.6	7203	18.9
2.8	4271	11.2
3.0	2197	5.8
3.2	726	1.9

Table 3

From this data a tail water elevation duration curve was plotted, to indicate how the additional capacity offered by the proposed new runner is limited by tail water elevation. This is presented in Figure 3

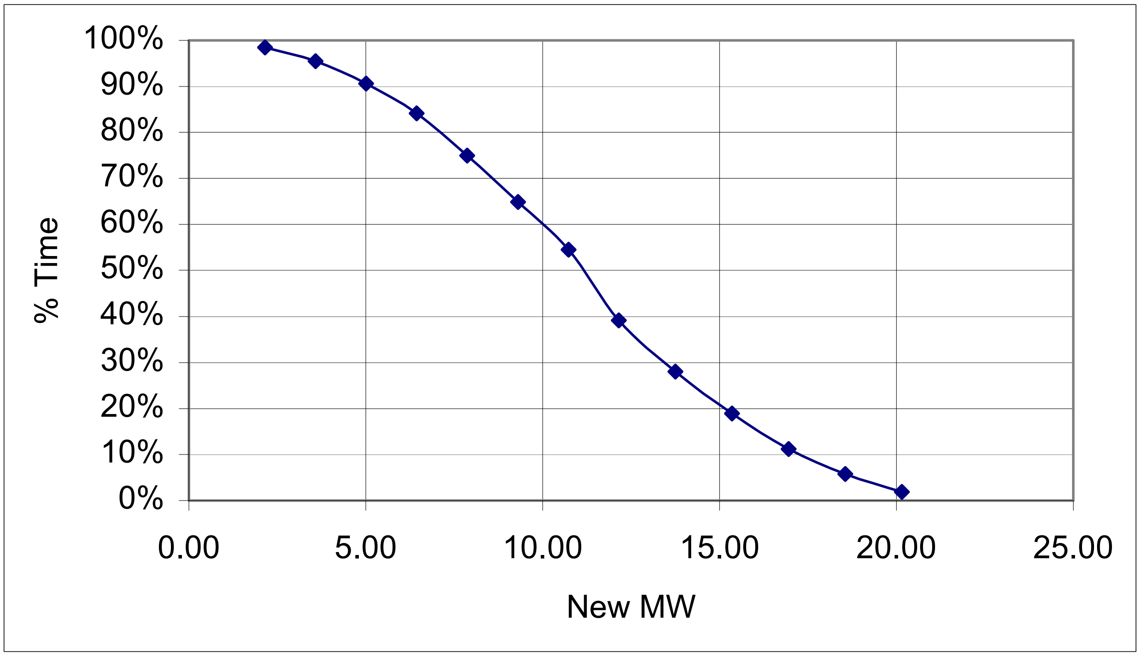


Figure 3

As an illustration of the significance of this curve, what it indicates is that we could make use of 15 additional MW of capacity only 20% of the time and 5 additional MW of capacity 90% of the time. The limitations inherent in the design of the proposed runner are apparent from this curve, especially when one considers that high tide will not necessarily coincide with system peak, which is when the additional capacity offered by the proposed runner would be of most use. Similarly, a coincidence of the required maximum output from Unit 7 with high flow rates through Units 1-6 may not occur, limiting the usefulness of the increased capacity. There is additional energy associated with this new runner in that its slight improvement in efficiency at the lower part of mid range and at the high range would improve energy production. However, as operating data indicates that Unit 7 operates only 5% of the time in this range, the energy gained through efficiency improvement would be negligible. As Unit 7 runner has not exhibited any significant corrosion, erosion or cavitation problems, there is no financial benefit to be gained by replacing the runner to address such issues.

To summarize, although GE Hydro has offered a runner with greater capacity, tail water elevation severely limits the usefulness of this additional capacity. Efficiency improvements offered are also marginal and there are no existing physical problems which would benefit from the installation of a new runner. The amount of additional capacity offered is considered to be 5 MW.

Energy Production Increase

The runner proposed by GE Hydro offers increased efficiency over a segment of the operating range, optimized based on the weighting factors provided in the original unit specification (circa 1974). GE Hydro structured its proposal in this way to facilitate comparison of the proposed runner with the originals, in the absence of absolute field test data. A review of production records for a recent three year interval (1999-01-01 to 2002-04-26) indicates that the actual operating mode is quite different from that originally expected, as indicated by the weighting factors. See Table 4

Turbine Output (MW)	Original Weighting Factor	Actual Operating factor
77	0.10	0.095
116	0.20	0.027
135	0.40	0.716
154	0.30	0.161

Table 4

The guaranteed efficiency of the original runner and of the proposed runner were compared using the Actual Operating Factor to determine the net efficiency gain of the proposed new runner. That efficiency gain, which translates directly into increased energy production, is an increase of 0.6825 % increase. (See Appendix IV for an explanation of how this increase was calculated.)There is potential to increase this by optimizing the runner design to suit our mode of operation.

Verification Of Improvements

The increase in capacity offered can be easily verified by field testing. The efficiency improvement offered is quite another matter. The correct procedure would be to test the unit before and after modification to verify that the promised improvement has been realized. The best test method which could be employed has an uncertainty, or inaccuracy, of about ± 1%. Therefore, the uncertainty band above the efficiency curve of the existing runner encompasses the efficiency curve of the new runner and vice versa. There is no way to test the unit and prove that the efficiency gain has been realized. There is no doubt that modern numerical design techniques have improved runner design and field testing of modern units has shown that turbine efficiencies have increased measurably over that past quarter century. However, if we proceed with this project, we will have to accept the efficiency improvement on faith.

Greenhouse Gas Emissions

Installation of a new runner will result in higher efficiency, which can be converted into an equivalent reduction of fuel consumption at the Holyrood Thermal Generating Station. Hydro may be able to take advantage of these reductions as carbon credits when the greenhouse gas emission reductions under the Kyoto agreement are implemented.

Other Potential Modifications

The GE Hydro has indicated that efficiency could be improved by a further 0.2% if the wicket gates were replaced by ones of revised design. The cost and benefits of this option have not been estimated, but should be investigated should this project be considered viable.

5. CONCLUSIONS

1. The project is technically feasible although a careful review will be required to ensure that GE Hydro has not pushed the envelope too close on cavitation limits.

2. The increased capacity offered has limited usefulness because of tail water elevation restrictions at higher outputs. The useful increase in capacity is 5 MW.

3. If it is decided to replace the runner, the rotor spider should be replaced to ensure that the frequency of vibration excursions caused by the floating rim does not increase, causing operational problems

4. The runner design proposed by GE Hydro was based on the efficiency weighting factors contained in the original request for proposals for the plant (circa 1974). Analysis of production records for recent years indicates that the actual mode of operation is very different. The increase in weighted efficiency of the proposed runner is 0.6825%.

6. RECOMMENDATIONS

1. The financial benefits which would accrue from replacing the existing runner should be analyzed by System Planning to determine if the project is financially viable.

2. If a decision is made to replace the runner with one having greater capacity, the generator rotor should be strengthened, consistent with conventional design standards.

3. The cavitation characteristics of the proposed runner should be carefully reviewed before proceeding with the project.

4. The production records for the most recent 10 year interval should be analyzed to establish new efficiency weighting factors. This should be reviewed with ECC to determine their preferred range of Unit operation (MW). This should then be discussed with GE Hydro with a view to modifying the proposed design to optimize the efficiency to achieve greater energy production. It should be possible to increase the efficiency gain proposed by GE Hydro (0.6825 %) to between 0.8% and 1.2%.

5. Should this project proceed, proposals should be invited from several manufacturers and

the specification should be structured to permit separate awards of the rotor spider strengthening and runner replacement. This will ensure that Hydro obtains the best alternatives for both components, which will not necessarily be proposed by one manufacturer.

6. The possibility of replacing the existing wicket gates with more hydraulically efficient units should be investigated.

APPENDIX I
Project Cash Flow

Prepared by: J. Mallam

CAPITAL BUDGET PROPOSAL

Capital Cost Estimate & Cash Flow Requirements

2004 Fiscal Year : Prepared: 25-Mar-04

BDE #7 Runner Replacement

In-Service: 31-Aug-05

AFUDC= 0.00% Annual 0.00% Mthly 0.00% Qtrly

Escalation % 2002 = 2003 = 2004 = 2005 = 2006 = 2007 = 2008 = (Est. Base: Jan-02

Period	Constr. Servcs	Equip. Purch.	Matrls Purch.	Constr. Internal	Land & Survey	External Eng.	Environ-ment	Eng.& Mgmt.	Proj./Constr Mgmt.	Inspection & Comm.	O/H @ 6.00%	Cont @ 10%	Sub Total	Escln	AFUDC	Total Project	Cash Flow (Excl AFUDC)
2004 Jan											0	0	0	0	0	0	0
Feb											0	0	0	0	0	0	0
Mar								5			0.3	0	5.3	0	0	5.3	5.3
Apr								5			0.3	0	5.3	0	0	5.3	5.3
May								5			0.3	0	5.3	0	0	5.3	5.3
Jun								5			0.3	0	5.3	0	0	5.3	5.3
Jul								5			0.3	0	5.3	0	0	5.3	5.3
Aug											0	0	0	0	0	0	0
Sep								5			0.3	0	5.3	0	0	5.3	5.3
Oct											0	0	0	0	0	0	0
Nov		200						5			12.3	0	217.3	0	0	217.3	5.3
Dec								5			0.3	0	5.3	0	0	5.3	217.3
Total 2004	0	200	0	0	0	0	0	40	0	0	14.4	0	254.4	0	0	254.4	254.4
2005 Jan								5			0.3	0	5.3	0	0	5.3	5.3
Feb		200						5			12.3	0	217.3	0	0	217.3	5.3
Mar								10			0.6	0	10.6	0	0	10.6	222.6
Apr		200						10			12.6	0	222.6	0	0	222.6	10.6
May								10			0.6	0	10.6	0	0	10.6	222.6
Jun	100	600		30				10			44.4	0	784.4	0	0	784.4	42.4
Jul	150	550						10		5	42.9	0	757.9	0	0	757.9	757.9
Aug	25			60				10		5	21.7	261	382.7	0	0	382.7	821.5
Sep				55				10		5	4.2	0	74.2	0	0	74.2	377.4
Oct				30				5		5	2.4	0	42.4	0	0	42.4	42.4
Nov		250						5		5	15.6	0	275.6	0	0	275.6	10.6
Dec											0	0	0	0	0	0	265
Total 2005	275	1800	0	175	0	0	0	90	0	25	157.6	261	2783.6	0	0	2783.6	2783.6
Beyond											0	0	0	0	0	0	0
Total Proj.	275	2000	0	175	0	0	0	130	0	25	172	261	3038	0	0	3038	3038

APPENDIX II
GE Hydro Proposals

Second Proposal

gilles.girard@ps.ge.com 05/29/2002 05:17 PM

John,

I have finally received information from our Dominique Bourque in hydraulic engineering (in all fairness to her she has been working very hard on numerous other projects at the same time).

Please see attached documents.

The maximum output of the generator is 185 MW. The maximum output of the turbine has been limited to 188 MW in order not to exceed the power that can be taken by the modified generator

The runaway speed as well as the hydraulic thrust of the new runner have been checked with the generator designers who confirmed that both were acceptable for the modified generator which we proposed with our 14 February 2002 proposal.

On the other hand, we must increase the wicket gate opening which will result in extra costs (see below)

Dominique has also performed some transient analysis calculations in order to check the over pressure and overspeed during load rejection. She concluded that we would have to modify the servomotor closing time curve so that the overspeed and over pressure are acceptable. As a result of this, we also have some additional cost detailed below to cover the necessary changes.

The price modifications are as follows:

- Increasing wicket gate opening

This consists of adding stroke to the servomotors as well as changing the pistons rods. The price includes engineering as well as refurbishment of the existing servomotors

  - Engineering $ 24,960.00
  - Servo refurbishment & New Piston Rods $ 86,910.00

- Modifications to prevent exceeding actual runaway speed and casing pressure rise

  - New check valves, flow control valve

and Dashpot modification
$ 14,360.00
- Site work to perform modifications
$ 4,050.00

Freight for all above: $ 3,780.00

Grand Total: $ 134,060.00

I hope the above will meet your new requirements as well as your expectations

Regards

Gilles
-----Original Message-----
From: JMallam@nlh.nf.ca [mailto:JMallam@nlh.nf.ca]
Sent: Tuesday, May 07, 2002 1:50 PM
To: Girard, Gilles (PS, Hydro)
Cc: RBesaw@nlh.nf.ca
Subject: Bay D'Espoir Unit 7

I have reviewed your submission dated 2002-04-16. The 0.61 m tailwater level is too low to use as a reference. Typically, the minimum level is 0.8 m, the maximum 3.2 m and the average 2.2 m. This plant is located a short distance from the ocean so the tailrace is tidal and, being long, is also affected by total plant output. The tailraces from powerhouse 1(units 1-6) and powerhouse 2 (unit 7), merge several hundred yards downstream of the plants and share a common tailrace from there to the ocean.

At powerhouse 1, the minimum tailwater level is 0.2 m, the maximum 3.0 m and the average 2.0 m.

Please review these tailwater levels and reassess what output could be achieved within the physical constraints of the existing discharge ring and draft tube, without inducing cavitation and giving due consideration to the range of tailwater levels created by tidal action and the operation of both powerhouses.

John Mallam
Newfoundland and Labrador Hydro
(709) 737-1712

Hydraulic-Writeup rev1.d CS-7004to7005.pc

Introduction

GE Hydro is proposing to replace the existing Francis runner of Unit 7 at the Bay d'Espoir Powerplant. The new runner will develop the following turbine output values for the various net heads and tailwater levels:

	Net Head: 172.517 m	Net Head: 174.45 m
Tailwater levels: 0.8 m (min)	Turbine Output: 170 MW	Turbine Output 173 MW
2.2 m (average)	180 MW	182 MW
3.2 m (max)	188 MW	188 MW

The main advantages of this new runner is to provide a turbine output increase when compared to the original rating, a gain in weighted turbine efficiency and an excellent cavitation behaviour.

Hydraulic Runner Design

GE Hydro will design one new runner specifically for the operating requirements. The new replacement runner will have 15 blades and a throat diameter of 3454.4 mm (136 inches). No modifications to the existing waterpassage components are required with our new proposed runner. The runner will rotate at the existing speed of 225 rpm.

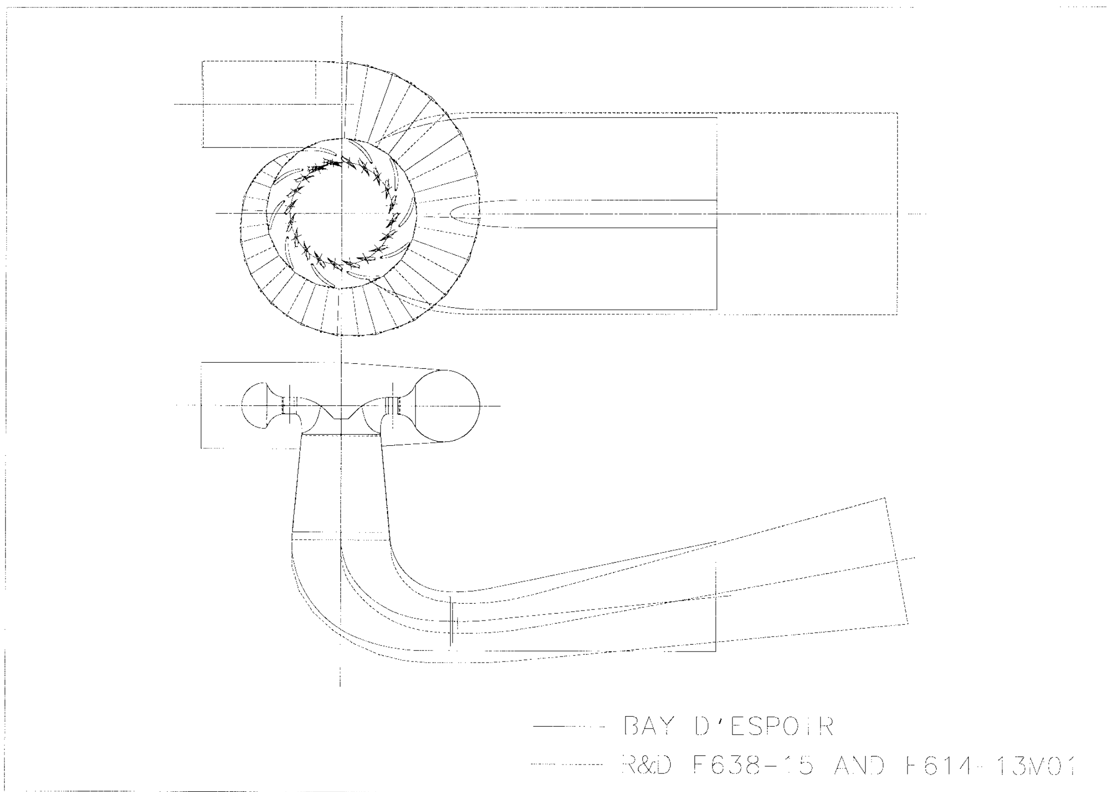
Reference models

The runner designations of GE Hydro's reference for this project are F-638-15 and F-614-13m01. These two runners were designed and model tested in 2001 within our R&D program. The model assembly used for the testing is essentially homologous to the Bay d'Espoir U7 waterpassage with the exception of the draft tube and wicket gate profile. Based on the model test results, GE Hydro has established the turbine performance that a modern runner designed for the Bay d'Espoir operating conditions would develop.

		MODEL R&D F-638-15	MODEL R&D F-614-13M01	BAY D'ESPOIR U7
THROAT DIAMETER (D_{TH})	[mm]	350.0 (model)	350.0 (model)	3454.4
SPEED COEFFICIENT AT MAX. EFF.	n11	59.74	60.26	59.175
POWER COEFFICIENT AT MAX. EFF.	P11	7.00	5.982	5.366
DISCHARGE COEFFICIENT AT MAX. EFF.	Q11	0.757	0.651	0.586
MAXIMUM MODEL EFFICIENCY	%	94.43	93.75	93.75
CASING TYPE		Full spiral case	Full spiral case	Full spiral case
CASING INLET DIAMETER:	% D_{th}	108.824	108.824	108.824
CASING AXIS DISTANCE:	% D_{th}	137.729	137.729	137.729
NUMBER OF STAY VANES	% D_{th}	10	10	10
NUMBER OF WICKET GATES	% D_{th}	20	20	20
WICKET GATE HEIGHT	% D_{th}	21.232	21.232	21.232

		Model R&D F-638-15	Model R&D F-614-13M01	Bay D'Espoir U7
Wicket gate circle diameter	% D_{th}	130.33	130.33	130.33
Runner inlet diam. (at crown)	% D_{th}	111.72	102.136	109.332
Runner exit diam. (at band)	% D_{th}	115.756	110.142	114.073
Runner band height	% D_{th}	26.547	25.793	24.013
Draft tube type		Elbow	Elbow	Elbow
Draft tube cone angle	% D_{th}	5.094°	5.094°	5.372°
Draft tube depth	% D_{th}	324.242	324.242	308.824
Draft tube length	% D_{th}	720.0	720.0	476.471
Draft tube exit height	% D_{th}	167.273	167.273	138.971
Draft tube exit width	% D_{th}	254.546	254.546	242.647
Number of pier		0	0	1
Pier distance from unit C.L.	% D_{th}	-	-	137.50
Pier width	% D_{th}	-	-	35.294

Water passage comparison between:
Bay d'Espoir U7 and R&D (runners: F-638-15, F614-13m01)



Model test
No model test is included in our proposal. The turbine performance has been established using close reference models. However, if Newfoundland Hydro requested a model test, GE Hydro will provide the associated schedule and costs.
~~Loss Analysis of the existing waterpassage.~~
In order to determine the efficiency loss of the existing assembly of unit 7, a detailed loss analysis was done.
Spiral Case
The model casing of our reference model are homologous to the Bay d'Espoir U7 casing. No efficiency correction is made.
Distributor
The stay ring, stay vanes, distributor height and wicket gate circle dimensions of our reference model are homologous to the Bay d'Espoir prototype. The wicket gate profile is however not homologous. A correction to the efficiency has been applied to account for the difference between the profiles.

Runner
No efficiency correction is made for the runner since GE Hydro is providing a new runner
Draft Tube
The existing draft tube is an elbow type. The draft tube depth and diffusion rate were reviewed and found to be acceptable.

Net Head Definition
The proposed turbine performance is based on the net head definition stated in IEC 60041 (1991)

Model to Prototype Step-Up
GE Hydro has applied a step-up value of 1.35% from model to prototype conditions. It has been applied as a constant addition to all operating points. No power step-up has been used when calculating the prototype turbine output.

It is important to note that in order to obtain the calculated step-up on the prototypes, the surface finish of the distributor, wicket gates, stay vanes and stay ring need to be in a fair condition.

Performance Curve
The expected turbine performance curves for the net heads of 172.517 m (566 feet) and 174.45 m (572.34 feet) are shown on diagram CS-7004 and CS-7005.

Maximum wicket gate opening
According to our records, the maximum wicket gate opening of the turbine is presently 23°. Based on our preliminary calculations, this opening will not be sufficient to achieve the turbine output of 188 MW under the rated net head of 172.517 m. Based on our analysis, the required maximum wicket gate opening to achieve this output value will be 28°.

New Wicket Gate Option
If new wicket gates were provided for unit 7, an efficiency gain of approximately 0.2% could be expected. This efficiency gain has not been included in the expected turbine performance efficiency. Cost for this furniture could be provided to Newfoundland Hydro upon request.

Cavitation
The new runner is guaranteed against excessive pitting due to the action of cavitation. The amount of cavitation pitting damage on the new runner will not exceed the following metal loss value:

Mass of material removed for a period of 8000 hours: $0.157 \times D_{TH}^2 = 1.87$ kg

In accordance with International Practice, the following conditions apply to our cavitation guarantee:

- The cavitation guarantee duration of operation is 8000 hours and the cavitation guarantee period is 2 years. Temporary abnormal operation shall be limited according to the recommendations described in IEC 609, article 8.2.

- The measurement and calculation of the amount of cavitation pitting shall be in accordance with IEC 609: “cavitation pitting and evaluation in hydraulic turbines, storage pumps and pump turbines.

- Our loss figures relate to weight loss caused by cavitation action only. Wear due to erosion by suspended material in the water or by chemical composition of the water is not included under the cavitation-pitting guarantee.

- GE Hydro shall be afforded the opportunity to check the machine after a reasonable operating period to be agreed with the client, and to carry out within an agreed period any work he considers necessary. If such repairs or changes are of minor nature, the cavitation period may by mutual agreement be considered as uninterrupted.

- If the runner fails to meet the guarantee for material loss as stated above, GE Hydro will repair all the damaged areas by welding and grinding.

The guarantee shall be renewed each time the turbine fails to meet the cavitation pitting guarantee.

Runaway Speed
Under the maximum net head of 175.68 m (576.4 ft), the new replacement runner for unit 7 will have a maximum runaway speed value of 405 rpm.

Hydraulic Thrust
The existing maximum hydraulic thrust value of 675 000 lb (3.0 MN) will not be exceeded.

Transient Calculations
Preliminary calculations, using an assumed closing law, were performed during the bid stage and the results were found acceptable for the speed and pressure rise. Detailed transient analysis will be performed at contract stage to confirm the values.

Guaranteed Turbine Performance and Prototype Field Test

Guaranteed Turbine Performance

It is proposed by GE Hydro to perform a pre and post Index Test to verify the turbine performance efficiency. This method is proposed to control project costs. GE Hydro would however be open to other alternative methods such as model test or prototype field efficiency test.

Turbine performance guarantees would consist in an average guaranteed weighted efficiency incremental value between the existing and new runner.

The new replacement runner will develop under the rated net head of 172.517 m, a guaranteed output value of 180 MW under an average tailwater level of 2.2 m.

An average guaranteed weighted efficiency incremental value between the new runner and the existing one has been established using the following method:

Method Pre and Post Index Test: Average Guaranteed Weighted Efficiency Incremental Value							
		Existing runner				New proposed runner	
Weighting Factor		Turbine Output				Turbine Output	
(value given in original contract)	% Rated Output *(value given in original contract)*	MW	Prototype Turbine Efficiency *(value given in the original contract)*	Step-up used between model and prototype	**Model Turbine Efficiency** *(value measured on original model test)*	**MW**	**Model Turbine Efficiency** **%**
			%	%	**%**		
w = 0.3	100	154.36	94.31	2.0	**92.31**	**180**	**92.18**
w = 0.0 (Peak)	-	141.59	95.10	2.0	**93.10**	**155.353**	**93.75**
w = 0.4	87.5	134.97	94.98	2.0	**92.98**	**157.5**	**93.72**
w = 0.2	75	115.58	92.83	2.0	**90.83**	**135**	**93.1**
w = 0.1	50	77.18	87.96	2.0	**85.96**	**90**	**88.9**
Expected Model Mean Weighted Efficiency:					**91.65**		**92.65**
Guaranteed weighted efficiency incremental value between the existing runner and the new one:					**1.00%**		

The acceptance of the new runner is based on the gain in efficiency. The absolute efficiency level (given in the above table and on curve CS-7004) is only given for information purposes. The justification of offering an incremental improvement value between the existing runner and the new one is due to the fact that it is very difficult to predict the efficiency step-up value for runner replacement projects due to the influence of the surface finish of old water passages. Moreover in the past, the specified step-up formulas (like full Moody) were also giving unrealistic values. Therefore, direct comparisons with existing prototype performance values give incorrect comparisons. The elimination of the issue of the magnitude of the possible efficiency step-up value has the advantage to compare correctly the efficiency gain between an existing and new runner.

Turbine Performance Efficiency verified by pre and post index test
As mentioned in the above section, a pre and post Index Test will be performed to verify the turbine performance efficiency.

We have included below information regarding the execution of the tests

- The index test would be performed with great care, using calibrating instruments of acceptable accuracy. Repetition of data collection at operating points would be done as required to help assure that test results are repeatable.

- Post upgrade Index Testing would be completed as soon as practical but within one year after start of commercial operation of the installed upgrade. The testing would be performed by GE Hydro using the IEC 60041 publication. A detailed test procedure would be supplied to Newfoundland Hydro prior to testing.

- Pre-Upgrade Index testing would be performed as close as practical prior to turbine upgrade outage period.

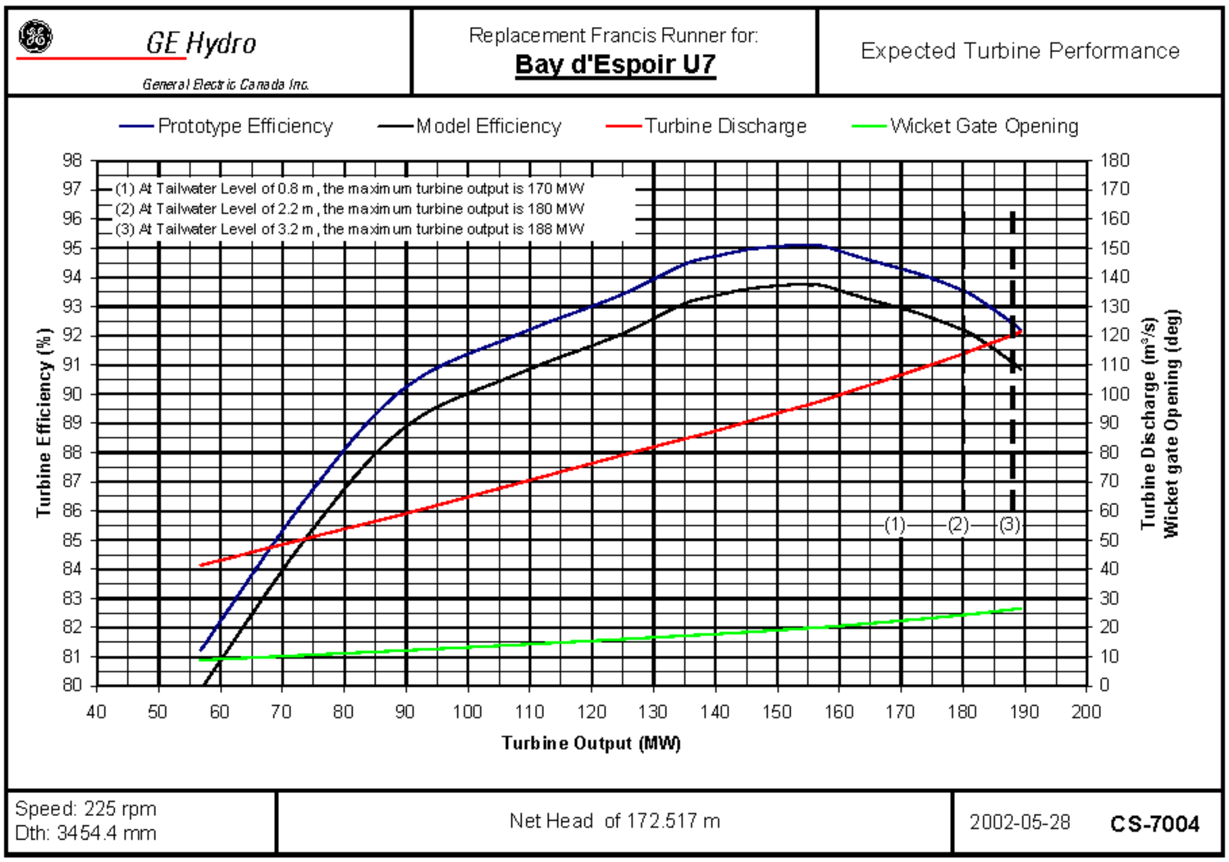
- The total efficiency uncertainty will be according to IEC 60041 publication

- Complete inspection of the machine would be done just prior to the pre-upgrade Index Test. If unusual conditions exist, discussions between GE Hydro and Newfoundland Hydro would take place in order to decide on the possible impact that the machine condition would have on performance.

- It is assumed that the condition of the turbine hydraulic waterpassage is fair, without excessive roughness. In any case, before conducting the Index Test prior to the runner removal, an inspection of all the hydraulic waterpassages including the Winter Kennedy piezometer taps and the piezometers taps at the turbine intake casing. The same type of inspection would also take place just prior to performing the Index Test of the new runner.

- GE Hydro and Newfoundland Hydro would have to agree on the generator performance curve prior to Index testing.

- A representative of Newfoundland Hydro would be at the plant site to witness both the up-grade and post-grade testing, as well as the waterpassage inspections. Prior to this testing, GE Hydro would furnish details of all test equipment, hardware and software. GE Hydro will furbish Newfoundland Hydro a complete report of each Index test performed.


GE Hydro
General Electric Canada Inc.
Replacement Francis Runner for:
Bay d'Espoir U7
Expected Turbine Performance
Prototype Efficiency
Model Efficiency
Turbine Discharge
Wicket Gate Opening
(1) At Tailwater Level of 0.8 m , the maximum turbine output is 170 MW
(2) At Tailwater Level of 2.2 m , the maximum turbine output is 180 MW
(3) At Tailwater Level of 3.2 m , the maximum turbine output is 188 MW
Turbine Efficiency (%)
Turbine Discharge (m³/s)
Wicket gate Opening (deg)
Turbine Output (MW)
(1)
(2)
(3)
Speed: 225 rpm
Dth: 3454.4 mm
Net Head of 172.517 m
2002-05-28
CS-7004

First Proposal

GE Hydro

Gilles Girard
Director Sales and Marketing, Canada

Geneal Electric Canada Inc.
795 George V, Lachine
Québec, Canada H8S 4K8

Thursday February 14, 2001

Newfoundland and Labrador Hydro
P.O. Box 12400
St John's, Newfoundland, Canada
A2B 4 K7

Att'n: Mr. Robert Beasaw
Project engineer

Subject: Bay d'Espoir Unit 7
Runner Replacement

Dear Bob,

Per our discussion of last year, we have prepared a proposal for the replacement of the runner for Unit 7 at Bay d'Espoir and we are pleased to submit herewith two (2) copies of our proposal.

As you will see in our proposal, the maximum turbine output can be increased to 168 MW, which represents a substantial increase over the actual rating of the unit. Also, the peak efficiency of the new runner can be achieved at a rating of 147.74 MW which also represents an added benefit to Newfoundland and Labrador Hydro. The overall efficiency of the turbine has also been improved over the operating range of the unit as you can see on the expected turbine performance curved attached to our proposal

During the Granite Canal negotiations, you had also mentioned that some generator work is required on that generator. During the course of last year, we had done a study for Newfoundland and Labrador Hydro to come up with a solution to your problems. Since, we are proposing to up-rate the turbine, we also looked at the impact of this increase on the generator with a view of fixing the problem of rim shifting on the existing unit. Our proposal also includes a solution to this problem.

Bob, I would be happy to meet with you and your colleagues to discuss this proposal. We believe that Newfoundland and Labrador Hydro could benefit from a runner replacement on unit 7 at Bay d'Espoir which, when combined with the generator work, will result in substantial increased benefits for that unit.

I am looking forward to hear from you.

Yours truly

Gilles Girard
Director Sales and Marketing, Canada

gilles.girard@ps.ge.com
Ph 514-485-4049

1. Introduction

GE Hydro is proposing to replace the existing Francis runner of Unit 7 at the Bay d'Espoir Powerplant. The new runner will develop a rated turbine output of 168 MW under a net head of 172.517 m and a tailwater level of 0.61 m or higher. The main advantages of this new runner is to provide a turbine output increase of 8.8% when compared to the original rating, a gain in weighted turbine efficiency and an excellent cavitation behaviour.

1.1 Hydraulic Runner Design

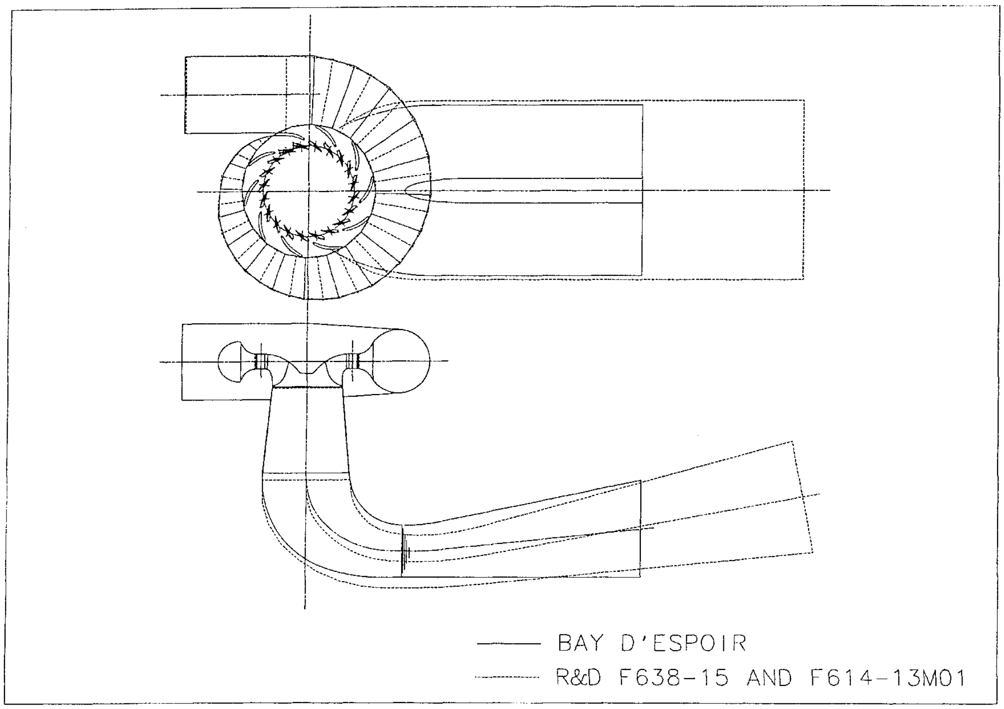
GE Hydro will design one new runner specifically for the operating requirements. The new replacement runner will have 15 blades and a throat diameter of 3454.4 mm (136 inches). No modifications to the existing waterpassage components are required with our new proposed runner. The runner will rotate at the existing speed of 225 rpm.

1.1.1 Reference models

The runner designations of GE Hydro's reference for this project are F-638-15 and F-614-13m01. These two runners were designed and model tested in 2001 within our R&D program. The model assembly used for the testing is essentially homologous to the Bay d'Espoir U7 waterpassage with the exception of the draft tube and wicket gate profile. Based on the model test results, GE Hydro has established the turbine performance that a modern runner designed for the Bay d'Espoir operating conditions would develop.

		Model R&D F-638-15	Model R&D F-614-13M01	Bay d'Espoir U7
Throat diameter (D_{TH})	[mm]	350.0 (model)	350.0 (model)	3454.4
Speed coefficient at max. eff.	n11	59.74	60.26	59.175
Power coefficient at Max. Eff.	P11	7.00	5.982	5.366
Discharge Coefficient at Max. Eff.	Q11	0.757	0.651	0.586
Maximum Model Efficiency	%	94.43	93.75	93.75
Casing type		Full spiral case	Full spiral case	Full spiral case
Casing inlet diameter:	% D_{th}	108.824	108.824	108.824
Casing axis distance:	% D_{th}	137.729	137.729	137.729
Number of stay vanes	% D_{th}	10	10	10
Number of wicket gates	% D_{th}	20	20	20
Wicket gate height	% D_{th}	21.232	21.232	21.232
Wicket gate circle diameter	% D_{th}	130.33	130.33	130.33
Runner inlet diam. (at crown)	% D_{th}	111.72	102.136	109.332
Runner exit diam. (at band)	% D_{th}	115.756	110.142	114.073
Runner band height	% D_{th}	26.547	25.793	24.013
Draft tube type		Elbow	Elbow	Elbow
Draft tube cone angle	% D_{th}	5.094°	5.094°	5.372°
Draft tube depth	% D_{th}	324.242	324.242	308.824
Draft tube length	% D_{th}	720.0	720.0	476.471
Draft tube exit height	% D_{th}	167.273	167.273	138.971
Draft tube exit width	% D_{th}	254.546	254.546	242.647
Number of pier		0	0	1
Pier distance from unit C.L.	% D_{th}	-	-	137.50
Pier width	% D_{th}	-	-	35.294

Water passage comparison between:
Bay d'Espoir U7 and R&D (runners: F-638-15, F614-13m01)



1.1.2 Model test

No model test is included in our proposal. The turbine performance has been established using close reference models. However, if Newfoundland Hydro requested a model test, GE Hydro will provide the associated schedule and costs.

1.2 Loss Analysis of the existing waterpassage.

In order to determine the efficiency loss of the existing assembly of unit 7, a detailed loss analysis was done.

1.2.1 Spiral Case

The model casing of our reference model are homologous to the Bay d'Espoir U7 casing. No efficiency correction is made.

1.2.2 Distributor

The stay ring, stay vanes, distributor height and wicket gate circle dimensions of our reference model are homologous to the Bay d'Espoir prototype. The wicket gate profile is however not homologous. A correction to the efficiency has been applied to account for the difference between the profiles.

1.2.3 Runner

No efficiency correction is made for the runner since GE Hydro is providing a new runner

1.2.4 Draft Tube

The existing draft tube is an elbow type. The draft tube depth and diffusion rate were reviewed and found to be acceptable.

1.3 Net Head Definition

The proposed turbine performance is based on the net head definition stated in IEC 60041 (1991)

1.4 Model to Prototype Step-Up

GE Hydro has applied a step-up value of 1.35% from model to prototype conditions. It has been applied as a constant addition to all operating points. This step-up is lower than obtained by the method defined in IEC 995: "Determination of the prototype performance from model acceptance tests of hydraulic machines with consideration of scale effects". No power step-up has been used when calculating the prototype turbine output.

It is important to note that in order to obtain the calculated step-up on the prototypes, the surface finish of the distributor, wicket gates, stay vanes and stay ring need to be in a fair condition.

1.5 Performance Curve

The expected turbine performance curve for the rated net head of 172.517 m (566 feet) is shown on diagram CS-6961.

1.6 Maximum wicket gate opening

The existing maximum wicket gate opening of 23° will be sufficient to achieve the guaranteed output.

1.7 New Wicket Gate Option

If new wicket gates were provided for unit 7, an efficiency gain of approximately 0.2% could be expected. This efficiency gain has not been included in the expected turbine performance efficiency. Cost for this furniture could be provided to Newfoundland Hydro upon request.

1.8 Cavitation

The new runner is guaranteed against excessive pitting due to the action of cavitation. The amount of cavitation pitting damage on the new runner will not exceed the following metal loss value:

Mass of material removed for a period of 8000 hours: $0.157 \times D_{TH}^2 = 1.87$ kg

In accordance with International Practice, the following conditions apply to our cavitation guarantee:

- The cavitation guarantee duration of operation is 8000 hours and the cavitation guarantee period is 2 years. Temporary abnormal operation shall be limited according to the recommendations described in IEC 609, article 8.2.
- The measurement and calculation of the amount of cavitation pitting shall be in accordance with IEC 609: "cavitation pitting and evaluation in hydraulic turbines, storage pumps and pump turbines.
- Our loss figures relate to weight loss caused by cavitation action only. Wear due to erosion by suspended material in the water or by chemical composition of the water is not included under the cavitation-pitting guarantee.
- GE Hydro shall be afforded the opportunity to check the machine after a reasonable operating period to be agreed with the client, and to carry out within an agreed period any work he considers necessary. If such repairs or changes are of minor nature, the cavitation period may by mutual agreement be considered as uninterrupted.
- If the runner fails to meet the guarantee for material loss as stated above, GE Hydro will repair all the damaged areas by welding and grinding.

Our guarantee is related to weight loss caused by cavitation only. Wear due to erosion by suspended material in the water or by the chemical composition of the water is not included in our cavitation pitting guarantee.
The guarantee shall be renewed each time the turbine fails to meet the cavitation pitting guarantee.

1.9 Runaway Speed

Under the maximum net head of 173.736 m (570 ft), the new replacement runner for unit 7 will have a maximum runaway speed value of 405 rpm.

1.10 Hydraulic Thrust

The existing maximum hydraulic thrust value of 675 000 lb (3.0 MN) will not be exceeded.

1.11 Transient Calculations

Preliminary calculations, using an assumed closing law, were performed during the bid stage and the results were found acceptable for the speed and pressure rise. Detailed transient analysis will be performed at contract stage to confirm the values.

2. Guaranteed Turbine Performance and Prototype Field Test

2.1 Guaranteed Turbine Performance

It is proposed by GE Hydro to perform a pre and post Index Test to verify the turbine performance efficiency. This method is proposed to control project costs. GE Hydro would however be open to other alternative methods such as model test or prototype field efficiency test.

Turbine performance guarantees would consist in an average guaranteed weighted efficiency incremental value between the existing and new runner.

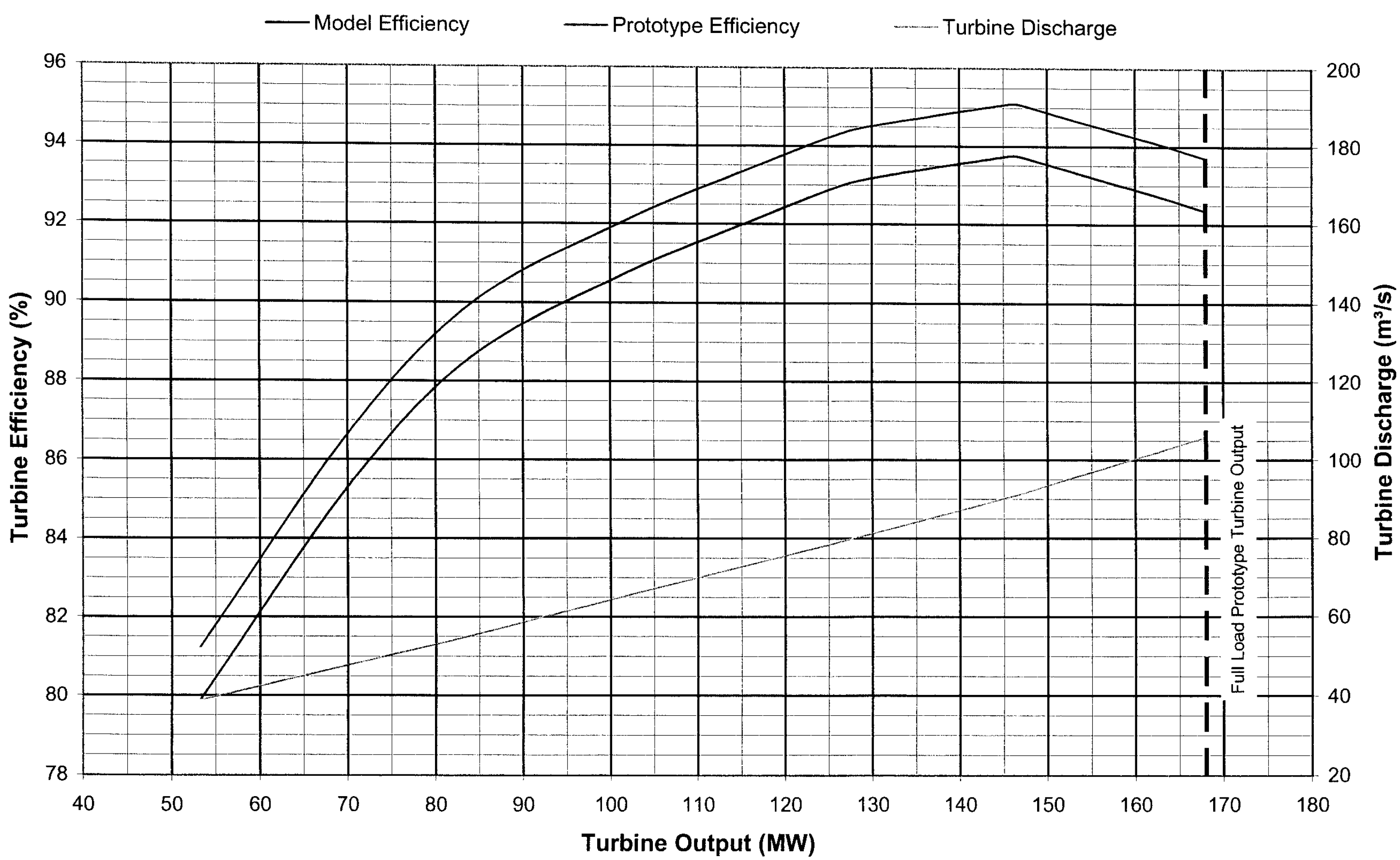
The new replacement runner will develop under the rated net head of 172.517 m, a guaranteed output value of 168 MW.

An average guaranteed weighted efficiency incremental value between the new runner and the existing one has been established using the following method:

Method Pre and Post Index Test: Average Guaranteed Weighted Efficiency Incremental Value							
		Existing runner				New proposed runner	
Weighting Factor		Turbine Output				Turbine Output	
(value given in original contract)	% Rated Output *(value given in original contract)*	MW	Prototype Turbine Efficiency *(value given in the original contract)*	Step-up used between model and prototype	**Model Turbine Efficiency** *(value measured on original model test)*	**MW**	**Model Turbine Efficiency** %
			%	%	%		
w = 0.3	100	154.36	94.31	2.0	**92.31**	**168.0**	**92.35**
w = 0.0 (Peak)	-	141.59	95.10	2.0	**93.10**	**147.74**	**93.75**
w = 0.4	87.5	134.97	94.98	2.0	**92.98**	**147.0**	**93.73**
w = 0.2	75	115.58	92.83	2.0	**90.83**	**126.0**	**92.95**
w = 0.1	50	77.18	87.96	2.0	**85.96**	**84.0**	**88.60**
Expected Model Mean Weighted Efficiency:					**91.65**		**92.65**
Guaranteed weighted efficiency incremental value between the existing runner and the new one:					**1.00%**		

The acceptance of the new runner is based on the gain in efficiency. The absolute efficiency level (given in the above table and on curve CS-6961) is only given for information purposes. The justification of offering an incremental improvement value between the existing runner and the new one is due to the fact that it is very difficult to predict the efficiency step-up value for runner replacement projects due to the influence of the surface finish of old water passages. Moreover in the past, the specified step-up formulas (like full Moody) were also giving unrealistic values. Therefore, direct comparisons with existing prototype performance values give incorrect comparisons. The elimination of the issue of the magnitude of the possible efficiency step-up value has the advantage to compare correctly the efficiency gain between an existing and new runner.

GE Hydro General Electric Canada Inc.	Replacement Francis Runner for: **Bay d'Espoir U7**	Expected Turbine Performance



Speed: 225 rpm Dth: 3454.4 mm	Net Head of 172.517 m	01/12/20 **CS-6961**

PRICING SHEET - Turbine

A)	**SUPPLY**	
	- Replacement Runner (Only):	**Cdn. $ 1,254,268.00**
	- Turbine Efficiency Pre and Post Index Test: (Please see Hydraulic write-up Page 6)	**Cdn. $ 53,333.00**
B)	**RUNNER REPLACEMENT INSTALLATION:**	**Cdn. $ 164,145.00**
C)	**TRANSPORT:**	**Cdn.$ 33,333.00**

Above Price for Installation is based on:

- 6 days per week 10 hour shifts
- Newfoundland & Lab. Hydro will have the unit dismantled
- Newfoundland & Lab. Hydro to reassemble and startup unit
- Remove shaft, clean/inspect shaft, assemble shaft to new runner, place runner shaft assembly
- Based on 2002 current rates for Granite Canal Project in Newfoundland

1. Introduction

GE Hydro has proposed to replace the existing turbine runner on unit #7 at Bay D'Espoir. The replacement runner will produce more power, be more efficient and have excellent cavitation behaviour.

2.1 Runner Characteristics

The new runner will have the following characteristics that may affect the generator design:

Rated speed –	225 rpm (unchanged)
Maximum overspeed –	405 rpm (increased from 380 rpm)
Hydraulic thrust -	< 675,000 lbs (below existing value)
Maximum turbine power -	168 MW (increased from 154.36 MW)

2.2 Effect on Generator design

The rated speed has not changed, therefore the basic generator electromagnetic is unaffected.

The increase in runaway speed (from 380 rpm to 405 rpm) would increase the maximum possible stress in the rotor rim and rotor poles by 13.5%. GE has reviewed the actual design and can confirm that the rotor rim and rotor poles can accept this increase in runaway speed without any modifications. It should be noted that the actual stress level in the rotor pole endplates will be higher than present design standards (GE Hydro estimates that ½ of the safety margin will be lost) but that this would be acceptable. GE Hydro can confirm the actual stress level at a later date.

The hydraulic thrust of the unit will not be greater than the existing runner, therefore the loads on the lower bracket and thrust bearing will not increase.

The increase in rated turbine power from 154.36 MW to 168 MW would require the generator rating to increase from 172 MVA to 184 MVA at a power factor of 0.9 (an increase of 7%). Records that GE have from the original testing of the unit #7 generator indicate there is presently margin in the operating temperature of both the rotor and stator. GE feels that the new rating of 184 MVA can be achieved with a temperature rise in both the stator and rotor below 65 C above cool air temperature.

The present equipment can accommodate the increase in mechanical power of 8.8 %.

Overall, the increase in turbine rating can be accommodated with no changes in the generator components.

2.3 Rotor Spider Design

Various correspondences have occurred between GE Hydro and Newfoundland Hydro over the subject of the rotor balance of Unit #7. GE Hydro would like to confirm that we feel the best solution to these issues is the shrinking of the rotor rim onto the rotor spider.

The study entitled "Rotor Rim Shrink Study" performed by GE Hydro in September, 2000 by Mr. Mike White and Mr. Wayne Martin examined the possibility of shrinking the rotor rim onto the present rotor spider. The conclusion stated that, with reinforcement, the present rotor spider could accept a rotor rim shrink that would be effective until 115 % of rated speed. The present industry standard for shrunk rotor rims is 125-130% of rated speed.

GE Hydro would like to propose that the rotor spider be completely replaced. The new rotor spider would be designed to transmit the increased power from the turbine and also be designed to allow the retained rotor rim to be shrunk to 130 % of rated speed.
The spider would also have a modern keying system between the rotor rim and rotor spider to maintain rotor balance at speeds above 130 % rated.

PRICING SHEET - Generator

A) **SUPPLY:**

- **Spider Replacement:** **Cdn. $ 439,091.00**

B) **Installation:**

- **Spider Replacement:** **Cdn. $ 239,641.00**

C) **TRANSPORT:** **Cdn.$ 33,333.00**

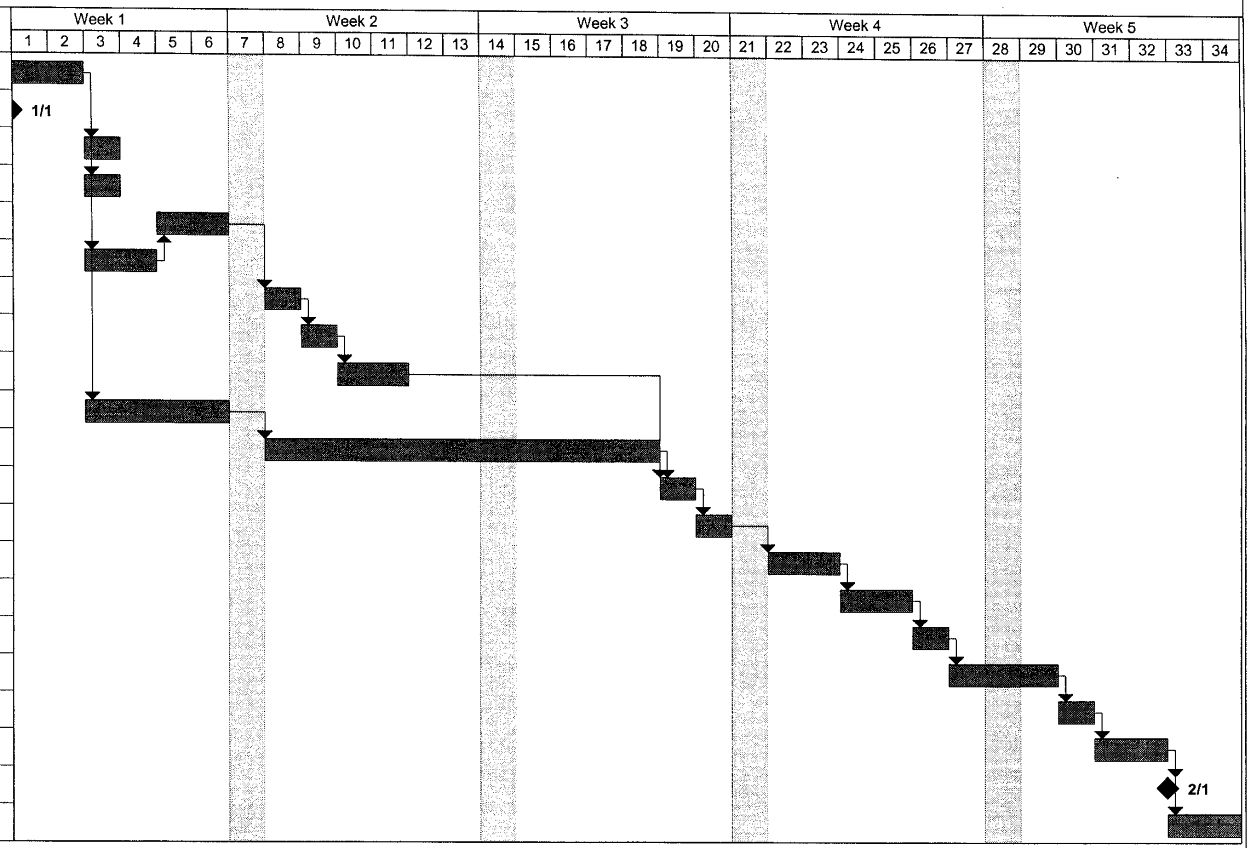
Above Price for Installation is based on:

- 6 days per week 10 hour shifts
- Newfoundland & Lab. Hydro will have the unit dismantled and placed in the erection bay
- Newfoundland & Lab. Hydro to reassemble and startup unit
- GE to send rep for startup and balancing
- Heaters and blankets included in price to be left at site
- Spider can be removed using the crane
- Customer will ream coupling holes during reassembly
- One initial heating cycle required for elevation and centering
- Rim can be adjusted by two additional heating cycles risk 10%
- Current union rates recorded for Granite Canal 2002
- Main leads and supports will be reused

Newfoundland And Labrador Hydro
Bay d'Espoir Unit 7
Spider Replacement

GE Hydro

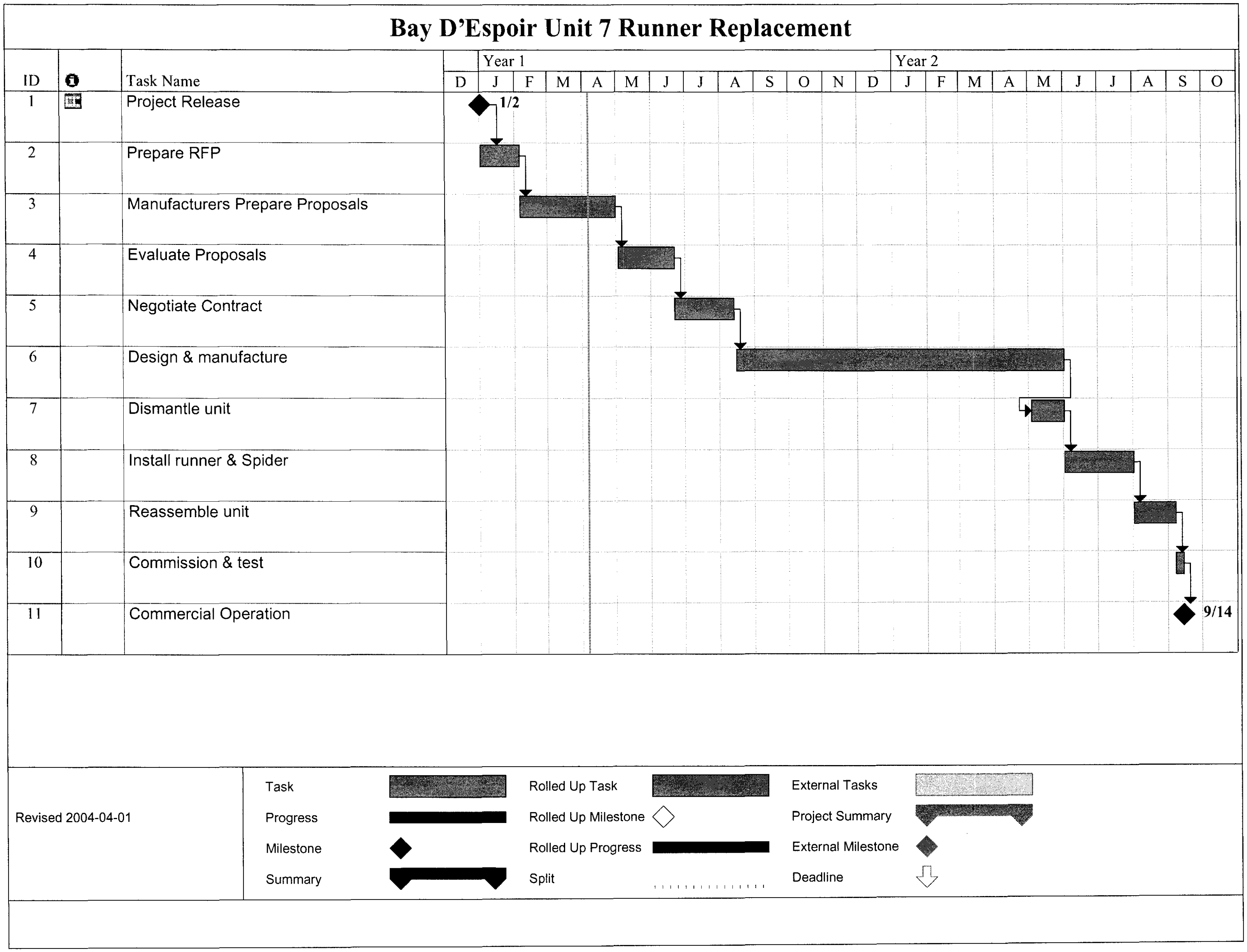
ID		Task Name
1		Mobilize
2		Dismantle by others
3		Setup distribution panel
4		Install termocouples
5		Install heaters & blankets
6		Initial roundess & verticality
7		Heat & remove keys
8		Remove main leads
9		Remove existing spider
10		Setup spider sections
11		Weld spider joints
12		Install new spider
13		Install main Leads
14		Heat & adjust rim
15		Rim Measurements
16		Heat & adjust rim
17		Rim Measurements
18		Heat & final adjustment
19		Final Measurements
20		Reassemble by others
21		Demobilize



Project: Bay d'Espoir Spider
Date: Wed 2/13/02

Task | Progress | Milestone | Summary

APPENDIX III
Project Schedule



Bay D'Espoir Unit 7 Runner Replacement
Year 1
Year 2
ID
Task Name
D J F M A M J J A S O N D J F M A M J J A S O
1 Project Release
1/2
2 Prepare RFP
3 Manufacturers Prepare Proposals
4 Evaluate Proposals
5 Negotiate Contract
6 Design & manufacture
7 Dismantle unit
8 Install runner & Spider
9 Reassemble unit
10 Commission & test
11 Commercial Operation
9/14
Revised 2004-04-01
Task
Progress
Milestone
Summary
Rolled Up Task
Rolled Up Milestone
Rolled Up Progress
Split
External Tasks
Project Summary
External Milestone
Deadline

APPENDIX IV
Efficiency Increase Calculation

When Unit 7 was designed by Dominion Engineering Works (Now GE Hydro) in the mid 1970s, it was optimized to maximize the weighted average efficiency, based on weighting factors specified in Hydro's request for proposals. A review of operating records for a recent three year period indicated that the unit is operated in a different manner than was predicted by the weighting factors (see Table 4, page 7). The proposal submitted by GE Hydro was based on the original operating factor and the efficiency increase they predict for the new runner is the difference between the efficiency of the original runner and the proposed new runner at several operating points, multiplied by the original weighting factors. For the purpose of this analysis, this methodology was followed, but new weighting factors were derived based on the recent three year period of operating experience. The results are summarized in the table below:

From GE proposal 2002-05-29					
Turbine Output (MW)	Original Model Efficiency (%)	New Model Efficiency (%)	New Weighting Factor	Original Model Efficiency (%)	New Model Efficiency (%)
115.58	90.83	93.10	0.10	8.70	8.92
134.97	92.98	93.72	0.03	2.54	2.56
141.59	93.10	93.75	0.72	66.70	67.16
154.36	92.31	92.18	0.16	14.82	14.80
			Weighted efficiency:	92.7525	93.4349
			Difference:		**0.6825**

This analysis indicates that the energy production increase we would realize would be 0.6825%, not 1.00% as stated by GE Hydro. There is no doubt that GE Hydro could redesign the runner to increase its weighted efficiency, based on our new weighting factors and this should be investigated should this project proceed.

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Acres International Limited
St. John's, Newfoundland

April 1994

Upstream Regulation Structure
Paradise River
Final Report

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Introduction

1 Introduction

Newfoundland and Labrador Hydroelectric Corporation (Hydro) owns and operates seven hydroelectric generating plants on the island of Newfoundland, with a total installed capacity of about 900 MW. One of Hydro's plants is the Paradise River Development located near Monkstown on the Burin Peninsula. The plant, commissioned in 1989, has a capacity of 8 MW with an average annual energy production of approximately 36 GWh.

The plant is operated as a run-of-the-river system. As the forebay has little storage, any water in excess of turbine flow capacity is spilled. In 1991, Acres carried out a prefeasibility study for the Canadian Electrical Association, sponsored by Energy, Mines and Resources Canada and Hydro, on the benefits of providing regulation at an upstream pond to reduce the amount of spill.

The results of the prefeasibility study showed that a small rollcrete or gabion dam would be the most suitable structure, with culverts to provide unattended hydraulic control. A suitable location for the structure was selected, and a cost/benefit analysis indicated that the project was attractive. The site identified for a flow regulation structure was at the outlet of Dunn's Pond, approximately 1 km northwest of the Burin Peninsula Highway, Route 210 (Plate 1). The drainage area above Dunn's Pond (281 km^2) accounts for about 60 percent of the 477 km^2 project basin.

After the prefeasibility study was complete, Hydro carried out a survey of the area and identified an alternative dam alignment for the regulation structure. This alignment is identified as Axis B in Plate 2. Additional soundings showed that the water at the original alignment (Axis A) in the prefeasibility was much deeper than had been assumed. Consequently both dam volume and costs for diversion during construction would increase. A brief review indicated that a dam located at the second alignment would be more economic.

The purpose of the present work was to finalize the design parameters for the project, in particular to optimize the full supply level and dam type. This report presents a brief description of the location and geology, documents the hydrological analysis and optimization studies, and presents the recommended design. Information on changes in water levels in Dunn's Pond is also presented for Hydro's use in preparing an environmental registration.

Geotechnical Considerations

2 Geotechnical Considerations

2.1 General

2.1.1 Available Data

The geotechnical site conditions were identified by at-site geological mapping of exposed bedrock at a reconnaissance level, supplemented by a review of published geological documents and air photo interpretation. The following geotechnical description was prepared using this information.

2.1.2 Site Description

The proposed site is located approximately 1 km upstream from the confluence of Dunn's Brook and Paradise River. The terrain consists of gently rolling, locally steep hills and northeast-trending ridges. Local relief is in the order of 50 m to 60 m. Dunn's Brook flows through a U-shaped valley eroded in the bedrock terrain. In addition to the main dam across the present river bed, a saddle dam is required to cut off an adjacent old river bed. The two dams are about equal in size, and are referred to here as the main dam and the saddle dam. The main dam is located across the river immediately upstream of a small waterfall, while the saddle dam is located upstream of a small escarpment, with little or no runoff. The alignment of each dam is indicated on Plate 2 as Axis B.

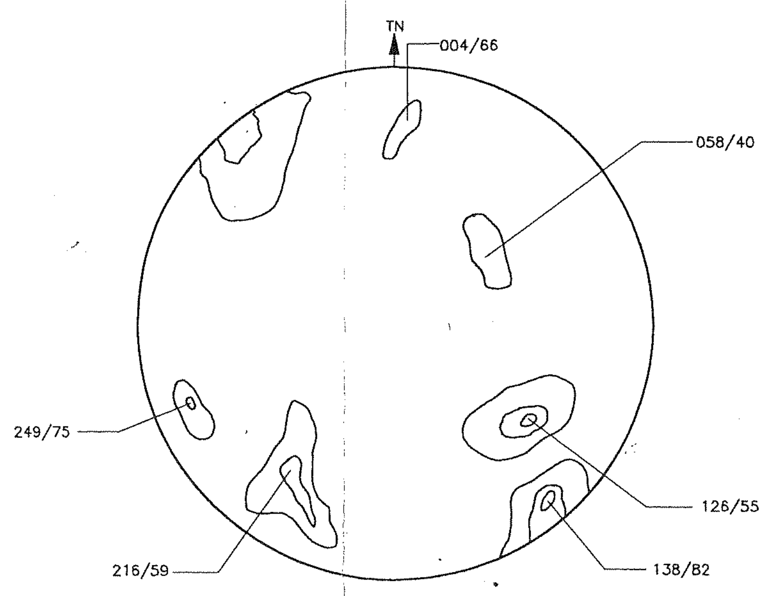
2.2 Site Geology

2.2.1 Surficial Geology

The region is comprised of bedrock terrain with sparse, non-extensive deposits of surficial materials, generally less than two metres deep. Significant soil features are presented in Figures 2.1 and 2.2.

LEGEND:

- BOG, LESS THAN 2M THICK
- SAND AND GRAVEL TERRACE
- OUTCROP BEDROCK
- TALUS SLOPES, LESS THAN 2m THICK
- ALLUVIAL, LESS THAN 2m THICK
- FOLIATION
- DIP DIRECTION (DIP)
- TN/GN TRUE NORTH/GRID NORTH

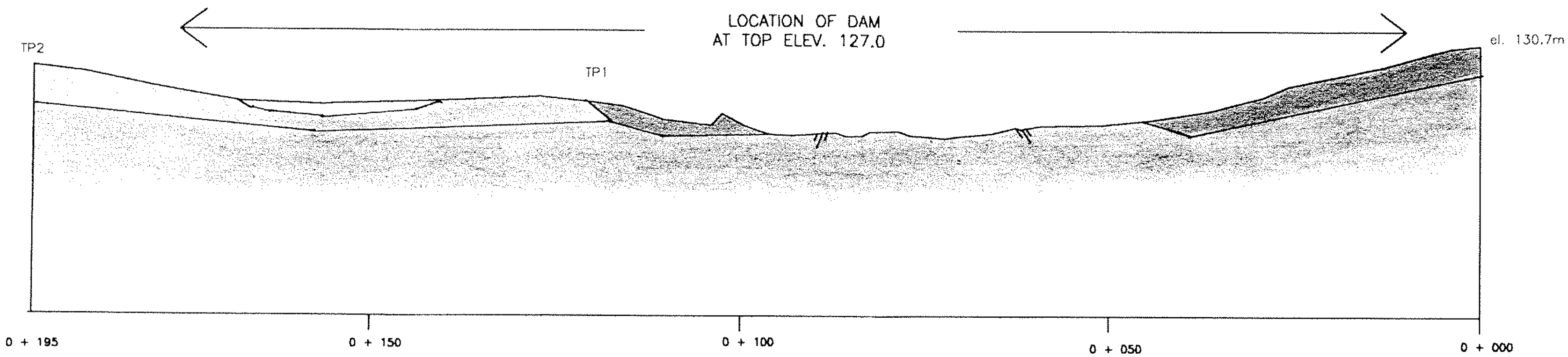


n=32

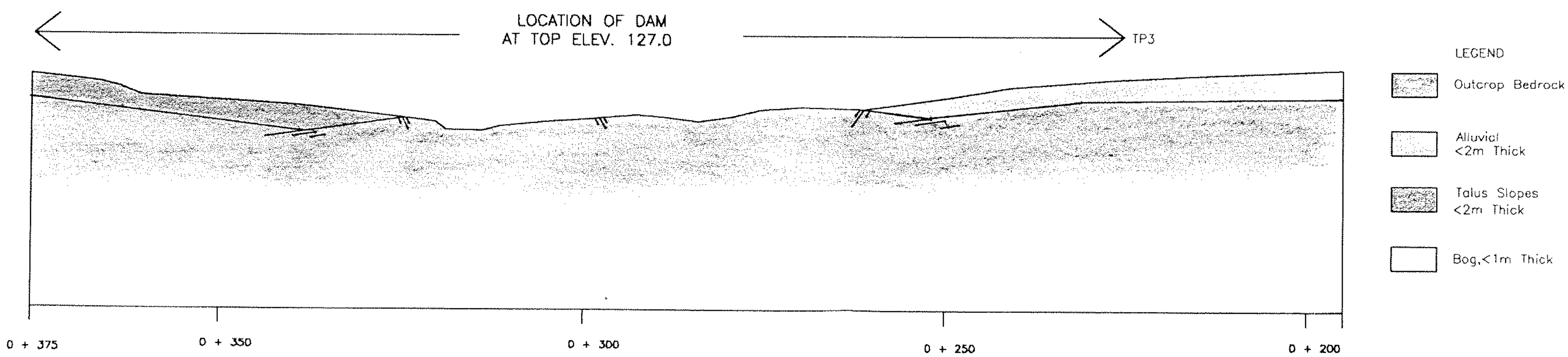
DUNN'S RIVER CONTROL DAM
UPPER HEMISPHERE POLAR PLOT

NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
GEOLOGICAL PLAN

FIG. 2.1



GEOLOGICAL PROFILE DAM 1 FOUNDATION



GEOLOGICAL PROFILE DAM 2 FOUNDATION

NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
FOUNDATION CONDITIONS

FIG. 2.2

ACRES

The right abutment of the main dam and the left abutment of the saddle dam (looking downstream) consist of bedrock terrain with a thin veneer of soil and talus, generally less than two metres deep. Scattered bedrock outcrop and frequent erratic boulders, varying from 1 m to 5 m in size, were noted in both abutments.

A small knoll forms both the left abutment for the main dam and the right abutment for the saddle dam. It consists of a moraine with an estimated depth not greater than two meters, underlain by bedrock. The actual depth to bedrock could not be determined in the field and should be confirmed in final design.

The deepest sections of both dams are founded on outcrop bedrock. Some minor grubbing and bedrock cleaning is expected to be required for foundation preparation.

No potential impervious fill deposits were noted during the investigation.

2.2.2 Bedrock Geology

Bedrock in the region consists of Precambrian Age, Anderson's Cove Formation. This unit consists of gray, subarkosic sandstone, fine to medium grained, faintly weathered at the surface but generally fresh. The rock is strong. Occasional thinly laminated, slate-like rock outcrops of limited extent were noted. The strike of the lamination was parallel to bedding.

Joint and bedding planes were closely to widely spaced, and inclined at medium to high angles, as indicated in Figure 2.1. Joints are frequently tight and occasionally healed with quartz. No major discontinuities were noted in the area.

2.3 Site Assessment

Based on available information, the site appears to be structurally suitable for a small dam. Foundation preparation along the abutments should be minimal prior to placement. Most of the grubbing will be at the right abutment of the main dam and the left abutment of the saddle dam where overburden depth is estimated to be 0 m to 2 m with exposed bedrock in many areas of the dam foundation.

The depth to bedrock at the center abutment will have to be determined for design purposes. This could be done most economically by either drilling or test pitting.

Hydrology / Energy Benefits

3 Hydrology / Energy Benefits

Hydrologic analysis was required for two purposes

- estimating energy benefits;
- estimating flood flows for spillway design, diversion during construction, and intermediate floods to assess environmental effects.

The first sections of this chapter describe the analysis of the energy benefits. The following sections present the flood flow analysis.

3.1 Estimation of Energy Benefits

The energy benefits were estimated by comparing spill volumes at Paradise River before and after construction of a dam at Dunn's Pond. Several alternative cases of different sill elevations were simulated for input to the economic optimization (described in Section 5). The sill elevations correspond to the maximum elevation of Dunn's Pond before it starts to spill. They are roughly equivalent to the maximum operating level in a reservoir but they are not the normal or average level. The before and after cases were simulated using Acres Reservoir Simulation Program (ARSP).

The hydrological analysis for energy benefits required

- assessment of data;
- set up of reservoir model;
- determination of relationship between hourly and daily Paradise River data sets as well as daily Paradise River and Piper's Hole data sets;
- simulation of long term operation to estimate benefits at various sill elevations;
- comparison of improvements with a gate-controlled (rather than hydraulically controlled) outlet.

3.1.1 Assessment of Available Data

Three sets of flow data were available

- hourly data from the Paradise River Generating Station for spill events in 1991 and 1992;
- daily data from the station for the period of 1989 - 1992;
- daily flows from the adjacent Piper's Hole River basin for the period 1953 - 1993, measured by Water Survey of Canada.

The daily and hourly data from the station consisted of headpond elevation and generation output (KWh). The total flow at the station was obtained (by Hydro) by converting the energy output to flow, and calculating spill based on headpond elevation. The inflow into the station was then calculated by backrouting.

The Piper's Hole River record provides a good basis for estimating the long term energy benefits, but for this project, it was important to determine the relationship between spill estimated using the Piper's Hole River record and actual recorded spill in relatively short flood events. There are two factors which could cause inaccuracies in simulating minor floods if the Dunn's Pond inflows were obtained directly from Piper's Hole River.

1) Size: Piper's Hole River drainage basin is considerably larger than the Dunn's Pond basin (764 km^2 compared to 281 km^2), and peaks would therefore be expected to be relatively lower.

2) Time Step: If a daily time step is used, some of the peaks may be missed. This effect is not so important in the spring, because peaks tend to be spread out over several days, but at other times of year the daily peak inflow might be considerably less than the hourly peak.

Both of these conditions would lead to an underestimate of spill savings.

For an overall assessment of energy at Paradise River, proration from Piper's Hole River would be acceptable. In the particular case of a detailed examination of many relatively small events at Dunn's Pond, however, it was important to check the magnitude of these effects.

An examination of plots of the daily data from the station showed that in fact the magnitude of the peaks from the two basins was often similar, even though the Paradise River basin is only about 60% of the size of the Piper's Hole River basin (477/764 km^2). Generally, the total volume was less because the floods receded more quickly. Figure 3.1 shows this effect during spill events for the typical flood events of October and November of 1990.

3.1.2 Model Setup and Initial Checks

Acres Reservoir Simulation Program (ARSP) was used to model the selected basin inflows. Figure 3.2 illustrates the network used for the Paradise River reservoir system. Required physical input includes

- stage/discharge curves;
- volume/area/elevation curve for Dunn's Pond.

The stage/discharge curve for the control section at the outlet of Dunn's Pond was developed from survey data, 1:500 scale topographic maps with 1 m contours, and several sets of photographs. The flow at the times of the photographs could be reasonably estimated from station data, and served as checks. No soundings were available, so the underwater portion had to be assumed and adjusted by trial and error. The estimated stage/discharge curve for the natural case and the volume/area/elevation curve for Dunn's Pond is presented in Appendix A.

The volume/area/elevation curve was developed from the 1:5,000 scale reservoir mapping. A discrepancy was originally detected when comparing elevations from

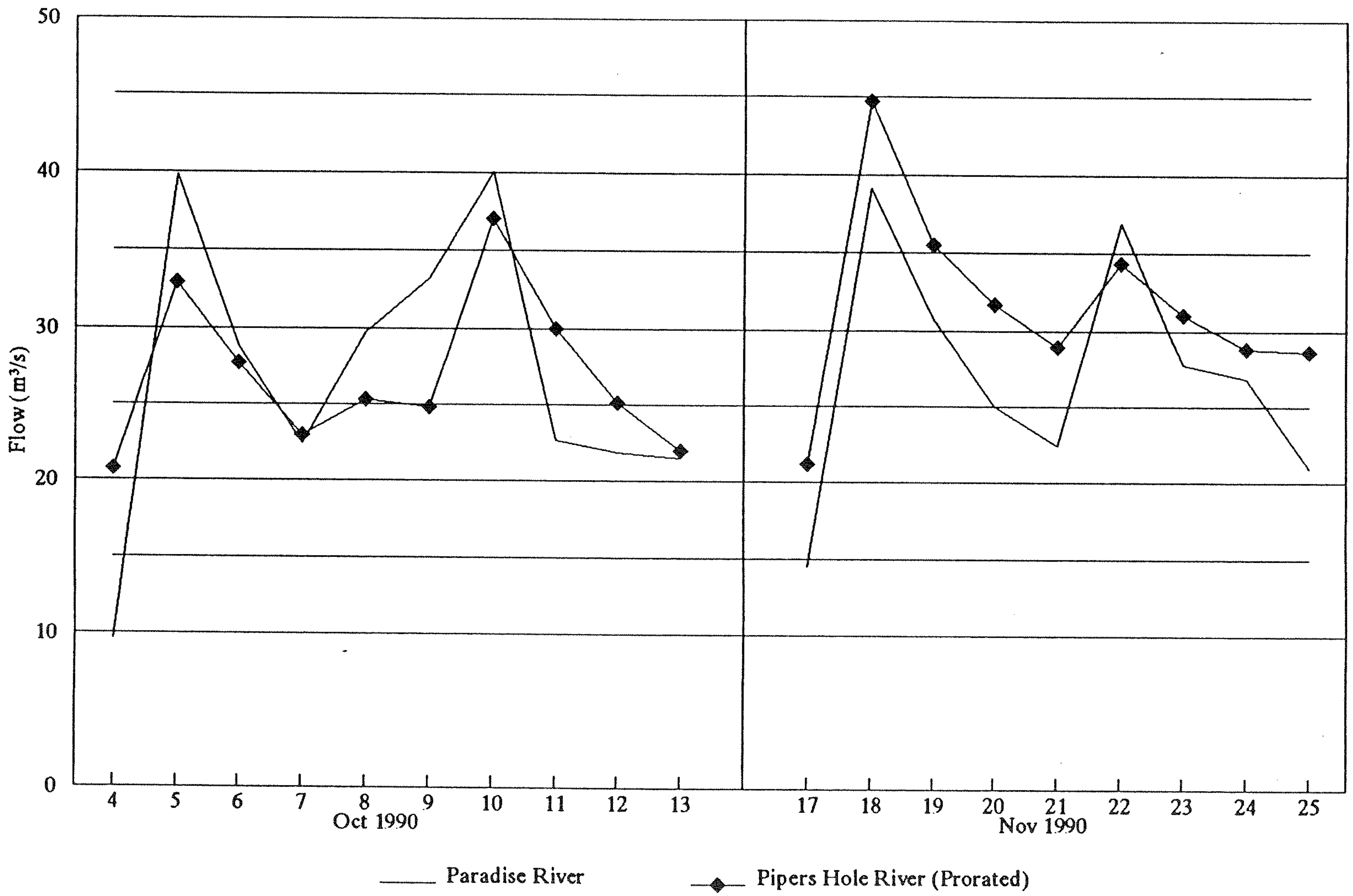


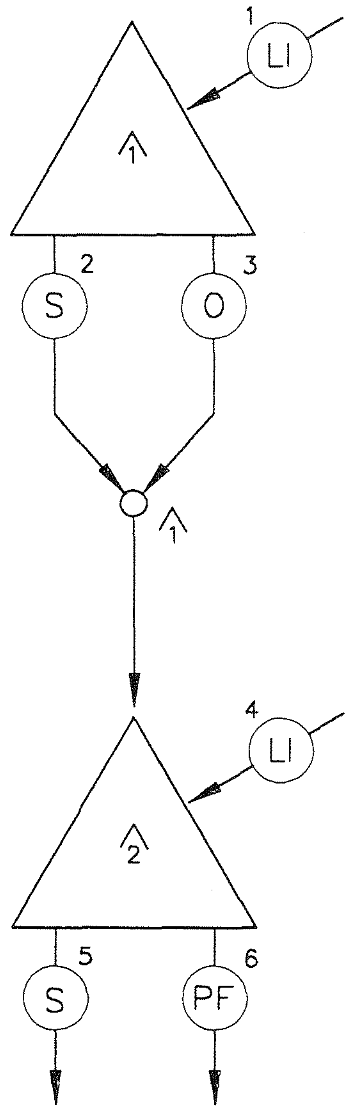
FIG. 3.1

NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

RECORDED vs. PRORATED INFLOWS

ACRES



LEGEND

- RESERVOIR NODE
- PF — POWER FLOW CHANNEL
- S — SPILL CHANNEL
- LI — LOCAL INFLOW
- O — ORIFICE FLOW
- JUNCTION NODE
- 1 — NODE NUMBER

RESERVOIRS:

1 – DUNN'S POND
2 – PARADISE RIVER HEADPOND

NODES:

1 – PARADISE RIVER BELOW DUNN'S POND

CHANNELS:

1 – DUNN'S POND INFLOW
2 – DUNN'S POND SPILL
3 – DUNN'S POND ORIFICE FLOW
4 – PARADISE RIVER INTERMEDIATE INFLOW
5 – PARADISE RIVER SPILL
6 – PARADISE RIVER POWER FLOW

FIG 3.2

NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
MODEL SCHEMATIC

ACRES

map contours to the point elevations as determined from the field survey. In particular, the survey data indicated elevations approximately 0.5 m to 1.0 m greater than that of the contours. It was therefore decided to adjust the elevation of each contour on the map in the region of the dam axis to make it conform to the survey data. This adjustment of the volume/area/elevation curve results in slightly conservative estimates of energy benefits. A sensitivity analysis of volume/area/elevation curve adjustments and the resulting spill volumes at the Paradise River station indicated that the dam height optimization is not sensitive to such adjustments.

Stage/discharge curves were also prepared for each alternative structure height considered with sill elevations from 120 m to 126 m, as required for the preliminary hourly simulations as shown in Table 3.1. These curves were developed assuming orifice releases through a box culvert with dimensions 4.0 m x 0.6 m. (Note that this arrangement was later changed to two pipe arch culverts for ease of construction and more effective distribution of stresses. The discharge curve and costs are similar for both arrangements).

For the purposes of economic optimization, the base of the culvert corresponds to the assumed bottom of the natural river bed (117.5 m). However, the comparison of intermediate flood levels before and after construction (Section 3.2.3) assumes the invert of the box culvert at an elevation of 118.0 m. This corresponds to a map reference approximately 0.5 m greater than the survey datum.

A rectangular overflow spillway was also assumed in developing the stage/discharge curve with a discharge coefficient of 1.8 and dimensions of 100.0 m x 1.0 m. Although this arrangement is arbitrary, it represents a typical solution to the task at hand. Modifications to this arrangement can be expected in the final design to satisfy the required stage/discharge relationship at the optimum sill elevation, with negligible effect on cost.

Table 3.1

Preliminary Hourly Simulation Results

EVENT	SPILL AT PARADISE RIVER GENERATING STATION (m^3*10^3)													
	NATURAL		STRUCTURE SILL ELEVATION (m)											
Date	Recorded	Theoretical	120.0	(savings)	122.0	(savings)	123.0	(savings)	124.0	(savings)	125.0	(savings)	126.0	(savings)
Feb 16/91	34700	34300	30100	4200	19900	14400	13300	21000	12600	21700	12600	21700	12600	21700
Spring	19000	–	14800	4200	4600	14400	0	19000	0	19000	0	19000	0	19000
Apr 23/91	3000	3100	300	2800	0	3100	0	3100	0	3100	0	3100	0	3100
Oct 03/91	23900	24400	21800	2600	11900	12500	7600	16800	7600	16800	7600	16800	7600	16800
Nov 01/91	600	600	0	600	0	600	0	600	0	600	0	600	0	600
Nov 13/91	300	200	0	200	0	200	0	200	0	200	0	200	0	200
Jan 25/92	3400	3400	300	3100	0	3400	0	3400	0	3400	0	3400	0	3400
Spring	92000	–	87800	4200	77600	14400	70800	21200	64000	28000	57200	34800	50400	41600
Jun 22/92	200	0	0	0	0	0	0	0	0	0	0	0	0	0
Oct 14/92	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Oct 27/92	1800	0	0	0	0	0	0	0	0	0	0	0	0	0
Dec 10/92	500	0	0	0	0	0	0	0	0	0	0	0	0	0
Dec 21/92	4000	4200	1700	2500	300	3900	300	3900	300	3900	300	3900	300	3900

$ Value = 0.341 $/kWh

Energy Factor = 0.09 kWh/m3

SAVINGS (Present Worth)

	STRUCTURE SILL ELEVATION (m)					
	120.0	122.0	123.0	124.0	125.0	126.0
1991						
(m^3*10^3)	14600	45200	60700	61400	61400	61400
($kWh*10^3$)	1308	4050	5439	5501	5501	5501
($1000)	$446	$1,381	$1,855	$1,876	$1,876	$1,876
1992						
(m^3*10^3)	9800	21700	28500	35300	42100	48900
($kWh*10^3$)	878	1944	2554	3163	3772	4381
($1000)	$299	$663	$871	$1,079	$1,286	$1,494
Average	$400	$1,000	$1,400	$1,500	$1,600	$1,700

Initial ARSP runs were performed to simulate the natural case for selected spill events. Two sets of inflow sequences were prepared to determine the extent of the natural attenuation of Dunn's Pond. The first set of sequences was simply a proration from Paradise River, i.e., Dunn's Pond inflows = 281/477 times Paradise River inflows. The second set was obtained by backrouting flows through Dunn's Pond in selected spill events with hourly records. Dunn's Pond outflows were estimated from the Paradise River record as the difference between plant inflow and the intermediate flow below Dunn's Pond (estimated by proration as 196/477 times plant inflow).

A comparison of simulated results using both sets of these runs indicated that the combined spill volume and turbine use at the station as calculated by the model was approximately equal to that actually recorded by Hydro. These results acted as a calibration and verified that the model was set-up and running as intended.

In addition to a natural case calibration, a further analysis was performed at a sill elevation of 123.0 m to study the relationship between backrouted inflows and resulting outflows. The results at the Paradise River station were similar, indicating that inflow sequences derived by proration would give good results, and additional backrouting was not required. Plant inflows calculated by the model were similar to those calculated using backrouted Dunn's Pond inflows. It was therefore decided to assume Dunn's Pond inflows equal to backrouted Dunn's Pond outflows for remaining hourly and daily simulations.

3.1.3 Hourly Simulations

The Paradise River drainage basin was simulated on an hourly time step basis for 13 spill events from January 1991 to December 1992. Each event was modeled using the appropriate stage/discharge curve corresponding to sill elevations of 120 m, 122 m, 123 m, 124 m, 125 m, and 126 m. The results of these runs are shown in Table 3.1. Also presented in this table is the total spill volume at the station for each event. The table indicates both the recorded spill volumes by Hydro and the theoretical volumes calculated according to the operating rules specified in the model (i.e. project does not spill until station inflow exceeds a maximum turbinable flow of 25.5 m^3/s).

Net benefits for each scenario (i.e. alternative sill elevations) are based on the decrease in spill observed at the station as a result of having a control structure at Dunn's Pond. An energy factor of 0.09 kWh/m^3 was used to convert spill savings to energy benefits, and a present worth value of $0.341/kWh as a means of calculating the overall present worth benefits of the project (provided by Hydro). These benefits were subsequently used in conjunction with the Capital Cost Estimate (CCE) in the optimization analysis.

For the purpose of establishing a preliminary benefit/cost analysis, the average annual spill savings for 1991 and 1992 were used. This was based on the hourly simulation results as indicated in Table 3.1. Of the 13 events modeled, 11 are non-spring events with a short runoff duration (i.e. less than 1-2 weeks).

Spill saved during the yearly spring runoff event was estimated using a slightly different approach. Since spring floods have a much greater duration and frequency than those experienced in the remainder of the year, it is often not possible for Dunn's Pond to draw down to low supply level between events. It was therefore assumed that the total live storage volume of both Dunn's Pond (with a control structure) and the project headpond can only be saved once during the entire spring runoff period.

3.1.4 Daily Simulations

The results of the hourly simulations showed that the model results were very close to those recorded, and that considerable volumes of spill could be saved. In order to estimate the long term benefits, it was necessary to relate these hourly results for 2 years to the daily flows of the corresponding period. In addition, the daily flows for the period of operation at Paradise River was related to the long term daily flow record at Piper's Hole River.

In order to relate the two sets of daily flows, the model was run using daily flow records of Paradise River and Piper's Hole River. Both flow records were prorated based on drainage area to develop an appropriate inflow sequence for Dunn's Pond. Each inflow sequence was run at sill elevations ranging from 122 m to 125 m, as well as the natural case. The calculated spill at the station for each scenario is presented in Table 3.2 for the four years of overlapping records.

An adjustment factor was calculated to convert the Piper's Hole River daily record to the Paradise River daily flows for the four years of overlapping flow records. The adjustment factors, which are shown in the table range from about 1.4 to 1.8. These results confirm that the response of the Paradise River (or Dunn's Pond) basins is flashier than Piper's Hole River, even when some storage is provided, and that this effect must be taken into account in a detailed examination of spill.

To relate the daily and hourly flows for the two years of record for which the station has both hourly and daily flow data, the calculated spill at the station was compared using both techniques. At each sill elevation an adjustment factor was calculated to correct for spill volumes over or under estimated when using the longer time step (i.e. daily instead of hourly). Table 3.2 shows that an average adjustment factor of 0.96 with a control structure in place and 1.06 for the natural case can be used to adjust Paradise River daily results to correspond to Paradise River hourly results. In other words, once a structure is in place, the peaks are sufficiently attenuated that a daily time step will yield reliable estimates.

Using the developed adjustment factors, the long term spill for each scenario can be estimated based on the 40 year Piper's Hole River record. The long term average annual spill at the station based on a Piper's Hole River daily simulation is shown in Table 3.3. These results are adjusted to reflect the long term expected spill volumes based on daily flows. The spill volumes are further adjusted to convert from daily to hourly calculated flows. The average annual spill estimates reflect the expected results of a theoretical long term Paradise River hourly simulation.

Table 3.2

Daily Simulation Results

Paradise River Spill ($m^3 \times 10^6$)

Natural Case

Method of Calculation	1989	1990	1991	1992
A) Recorded (Daily Totals)	82.1	176.5	74.2	103.7
B) Simulated hourly (Paradise River)	–	–	81.6	99.6
C) Simulated daily (Paradise River)	62.4	151.3	75.6	95.9
D) Simulated daily (Piper's Hole)	33.1	87.2	99.9	75.6

Avg ratio from (C) to (B) = 1.06
Avg ratio from (D) to (C) = 1.41

Sill@122

Method of Calculation	1989	1990	1991	1992
A) Recorded (Daily Totals)	–	–	–	–
B) Simulated hourly (Paradise River)	–	–	36.4	77.9
C) Simulated daily (Paradise River)	46.4	97.2	40.8	72.8
D) Simulated daily (Piper's Hole)	19.8	52.7	60.7	56.6

Avg ratio from (C) to (B) = 0.98
Avg ratio from (D) to (C) = 1.54

Sill@122.5

Method of Calculation	1989	1990	1991	1992
A) Recorded (Daily Totals)	–	–	–	–
B) Simulated hourly (Paradise River)	–	–	–	–
C) Simulated daily (Paradise River)	43.1	90.4	33.1	68.0
D) Simulated daily (Piper's Hole)	16.5	48.3	51.9	52.0

Avg ratio from (C) to (B) = –
Avg ratio from (D) to (C) = 1.61

Sill@123

Method of Calculation	1989	1990	1991	1992
A) Recorded (Daily Totals)	–	–	–	–
B) Simulated hourly (Paradise River)	–	–	20.9	71.1
C) Simulated daily (Paradise River)	39.7	85.6	28.3	63.4
D) Simulated daily (Piper's Hole)	13.2	45.2	44.5	47.5

Avg ratio from (C) to (B) = 0.93
Avg ratio from (D) to (C) = 1.72

Sill@123.5

Method of Calculation	1989	1990	1991	1992
A) Recorded (Daily Totals)	–	–	–	–
B) Simulated hourly (Paradise River)	–	–	20.2	67.7
C) Simulated daily (Paradise River)	36.4	82.6	26.3	58.9
D) Simulated daily (Piper's Hole)	11.5	42.2	39.7	43.6

Avg ratio from (C) to (B) = 0.96
Avg ratio from (D) to (C) = 1.78

Sill@124

Method of Calculation	1989	1990	1991	1992
A) Recorded (Daily Totals)	–	–	–	–
B) Simulated hourly (Paradise River)	–	–	20.2	64.3
C) Simulated daily (Paradise River)	33.1	79.5	26.3	54.8
D) Simulated daily (Piper's Hole)	11.5	39.2	37.0	41.0

Avg ratio from (C) to (B) = 0.97
Avg ratio from (D) to (C) = 1.74

Sill@125

Method of Calculation	1989	1990	1991	1992
A) Recorded (Daily Totals)	–	–	–	–
B) Simulated hourly (Paradise River)	–	–	20.2	57.5
C) Simulated daily (Paradise River)	26.8	73.6	26.3	49.7
D) Simulated daily (Piper's Hole)	11.4	34.3	37.0	36.9

Avg ratio from (C) to (B) = 0.96
Avg ratio from (D) to (C) = 1.64

Table 3.3

Long Term Benefits

	Average Annual Spill Volume at Paradise River Station (m³*10^6)						
	Natural	Sill@122	Sill@122.5	Sill@123	Sill@123.5	Sill@124	Sill@125
Simulated daily (Piper's Hole)	88.7	58.1	52.6	48.4	45.2	42.8	39.6
Paradise River Daily (adjusted from P.H. Daily)	125.2	89.1	84.6	83.1	80.6	74.6	64.9
Paradise River Hourly (adjusted from P.R. Daily)	132.6	85.6	81.2	79.8	77.4	71.6	62.3
Spill saved (descrease from natural)		47.1	51.4	52.8	55.2	61.0	70.3
Energy Benefit (GWh)		4.24	4.63	4.76	4.97	5.49	6.33
$ Benefit (Present Worth)		$1,444,200	$1,577,800	$1,621,700	$1,695,200	$1,873,000	$2,157,000

Table 3.3 also shows the estimated long term energy benefits. The estimated annual spill for each sill elevation is subtracted from that of the natural case to determine the theoretical spill saved for each case. The average annual spill saved is multiplied by the energy factor and present worth value as described in Section 3.1.3 to estimate the present worth benefits of each case.

3.1.5 Gate Controlled Outflows

Additional flow regulation at the outlet of Dunn's Pond can be provided through the use of control gates. These gates would be closed whenever flows downstream of Dunn's Pond exceed the turbine capacity at the Paradise River station, and reopened when the flows recede. Without the gate, there is always some flow through the culverts, even when the flows downstream exceed turbine capacity. The control gates have most benefit during short duration non-spring flood events, when the levels in Dunn's Pond do not rise above the spillway sill. During spring floods much of the release from Dunn's Pond is over the spillway, so gate control of the culverts has little effect.

The average annual spill volume at the Paradise River generating station was estimated assuming control gates at the culvert, for the optimum sill elevation of 122.5 m (determined as described in Section 5). Using both the Paradise River daily record and the long term Piper's Hole River record, control of the outlet culverts at Dunn's Pond was estimated to result in an additional annual spill savings of at least 3.3×10^6 m^3, equivalent to about $100,000 (present worth), over an uncontrolled arrangement. A cost / benefit analysis of a controlled outlet would require a detailed analysis of the operation of the gates and all related costs. This analysis would include consideration for items such as the time lag between Dunn's Pond outflows and plant headpond inflows, as well as the logistics of ensuring continuous access to the gates by an operator, or provision of automatic control.

3.2 Floods

3.2.1 Spillway Design Flood

Environment Canada's Consolidated Frequency Analysis (CFA) package as well as inhouse software were used to estimate the project design floods. Both daily and instantaneous peak flows were examined for the flow records at both Piper's Hole River (02ZH001) and Rattle Brook (02ZG004). The selected design flood is 500 m^3/s (instantaneous) with an estimated return period of 10,000 years.

This design criteria satisfies the recommended guidelines of both the Institute of Civil Engineering, London and the U.S. Army Corps of Engineers.

3.2.2 Construction Diversion Flood

The construction diversion flood analysis was based on a four month (June to September) construction season. Summer flows from the Piper's Hole River daily extreme record were prorated based on drainage area to Dunn's Pond. The daily flows were then indexed by 20 percent to 50 percent to estimate instantaneous maximum flows. Based on this analysis, a design construction diversion flood of 50 m^3/s was calculated, with a return period of about 5 years.

3.2.3 Intermediate Floods

In addition to establishing the project design floods as explained above, intermediate flood flows were also estimated as shown in Table 3.4. Using the stage/discharge curves developed for both the natural case and for a sill elevation of 122.5 m (see Section 3.1.2 for discussion), the water levels corresponding to flood flows of various return periods were estimated. For typical flood events occurring once every two to five years on the average, high water levels are estimated at elevation 123.5 m with a structure in place and 120 m in the natural case.

Figures 3.3 and 3.4 illustrate the end-of-week elevations in Dunn's Pond for the pre and post construction cases respectively, shown to map reference elevation.[1] These plots are the results of the 4 - year daily simulations based on flows recorded at the station. An annual peak elevation in the spring of each year is evident in both plots with smaller floods commonly experienced in the late fall and early winter. Figures 3.5 to 3.8 show graphical representations of Dunn's Pond pre and post construction water levels for each of the four years simulated.

Table 3.4

Water Level Frequency Analysis

Q (m^3/s)	Tr (yrs)	Elevation (m)		Flooded Area (km^2)		
		Natural	**Post Construction**	**Natural**	**Post Construction**	**Increase**
80	2	119.7	123.5	4.90	7.05	2.15
120	5	120.0	123.7	5.40	7.10	1.70
140	10	120.1	123.8	5.45	7.15	1.70
190	50	120.5	124.0	5.65	7.25	1.60

1 Map reference elevation = survey datum + 0.5 m

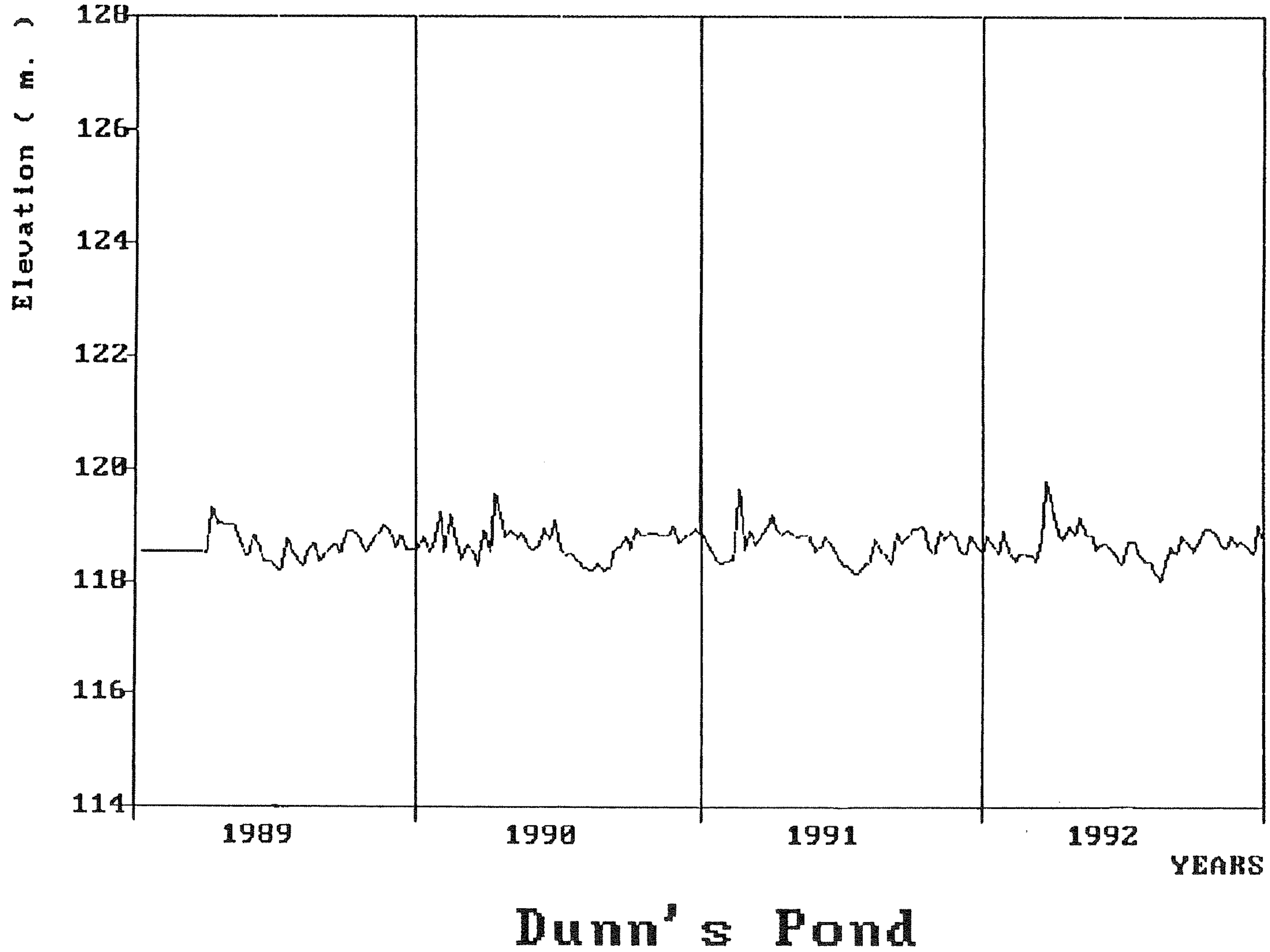


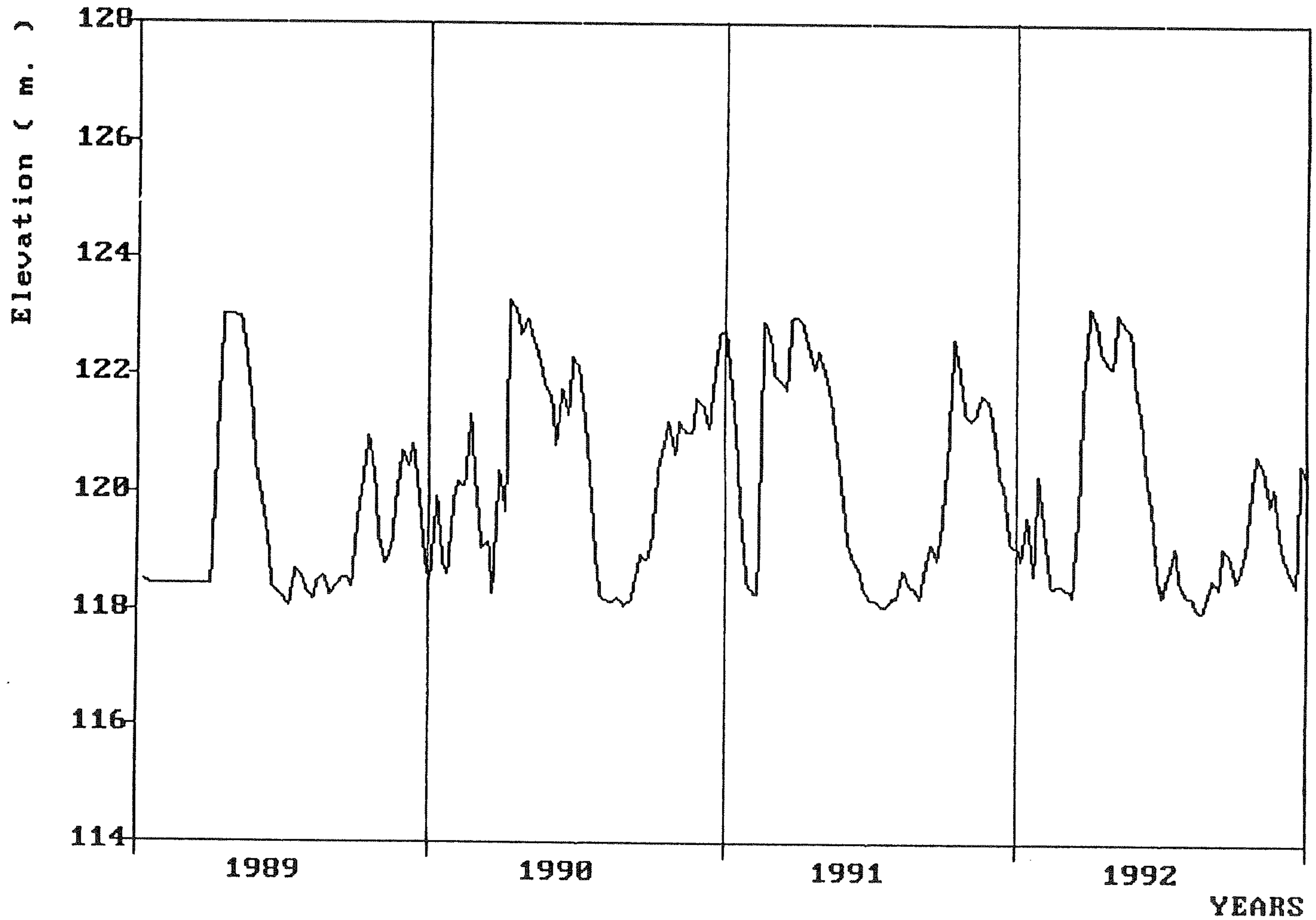
FIG. 3.3

NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

SIMULATED PRE–CONSTRUCTION WATER LEVELS

ACRES

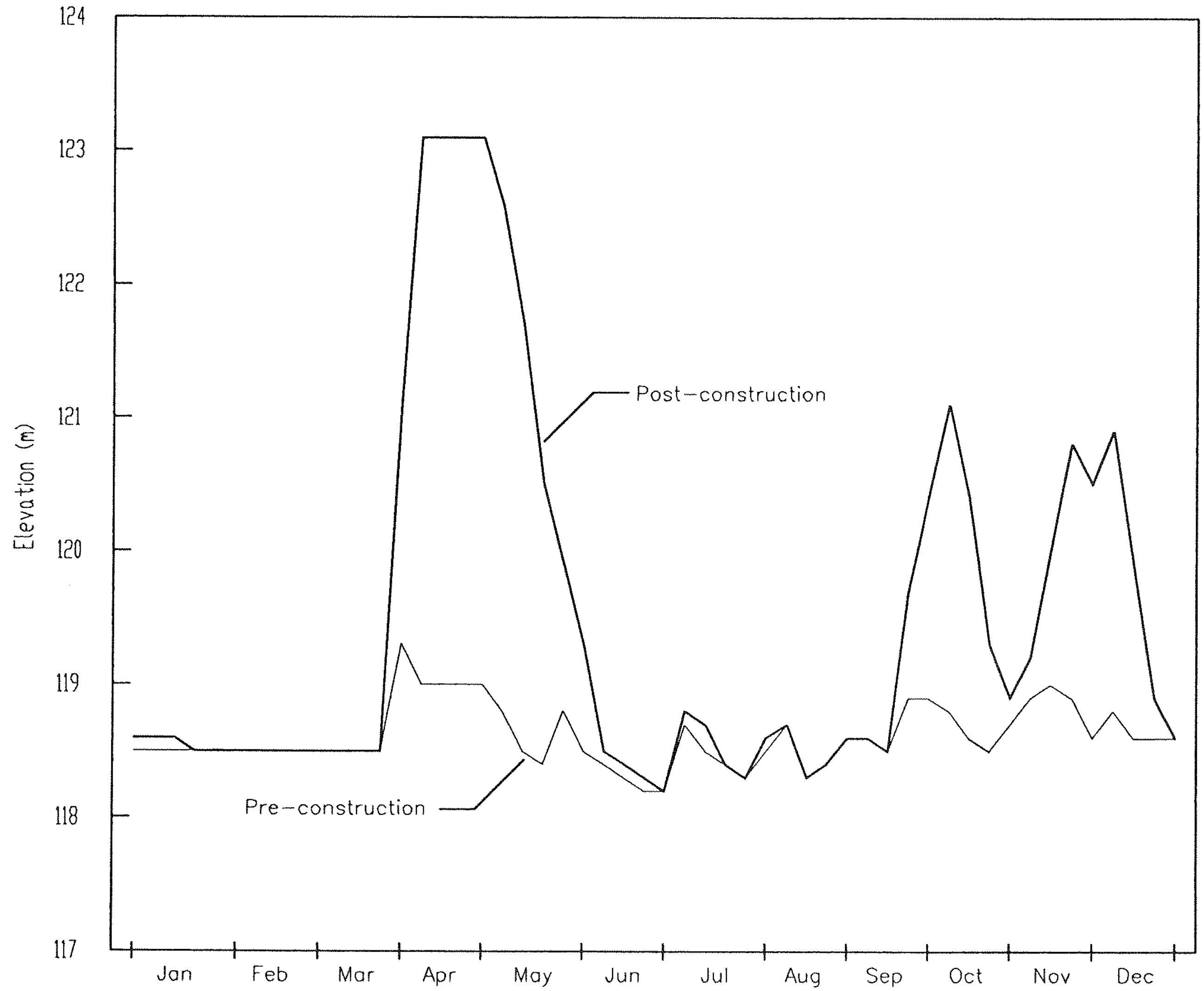


Dunn's Pond

NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
SIMULATED POST–CONSTRUCTION WATER LEVELS

FIG. 3.4

ACRES



NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

SIMULATED DUNN'S POND WATER LEVELS – 1989

FIG. 3.5

ACRES

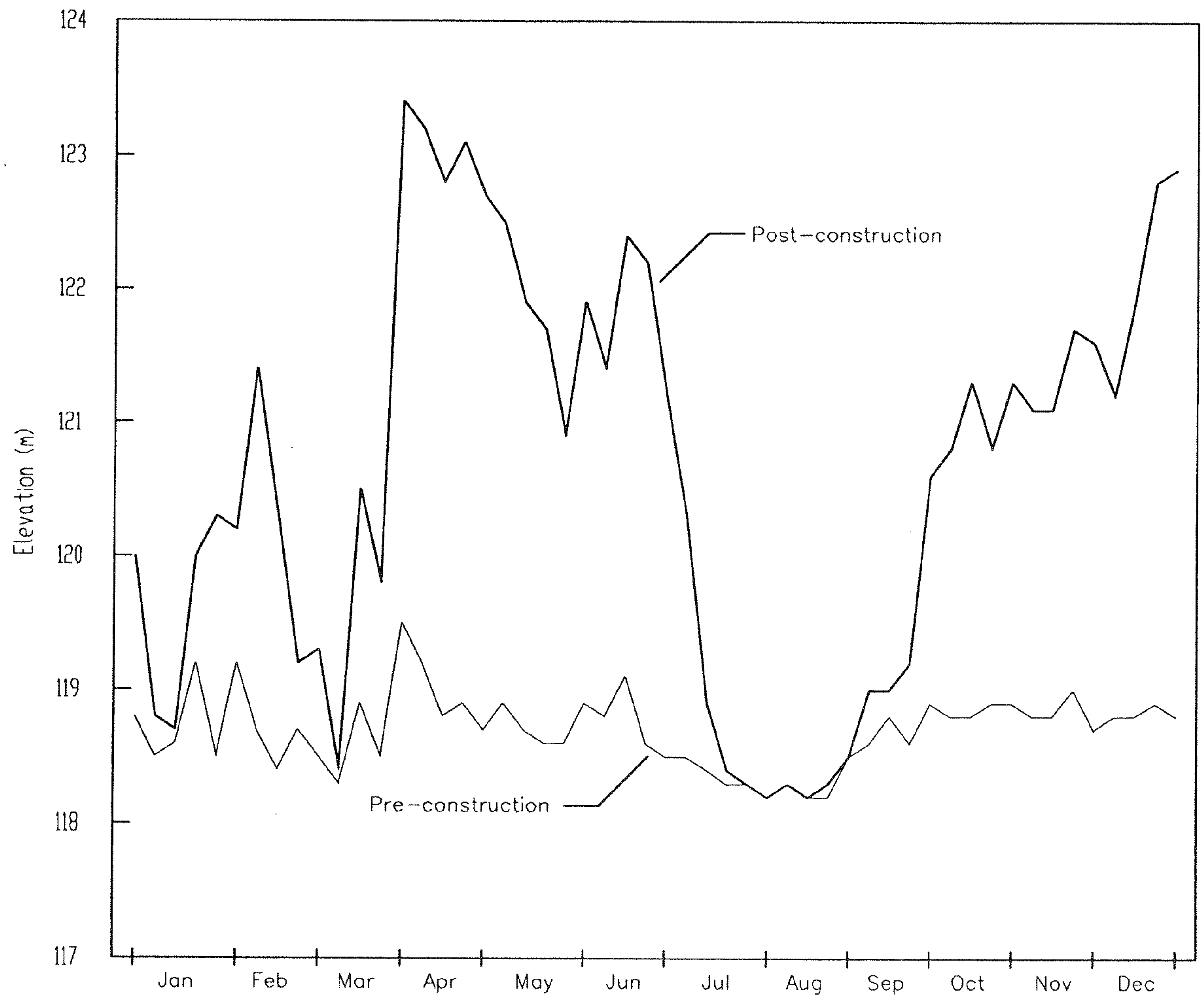


FIG. 3.6

NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

SIMULATED DUNN'S POND WATER LEVELS – 1990

ACRES

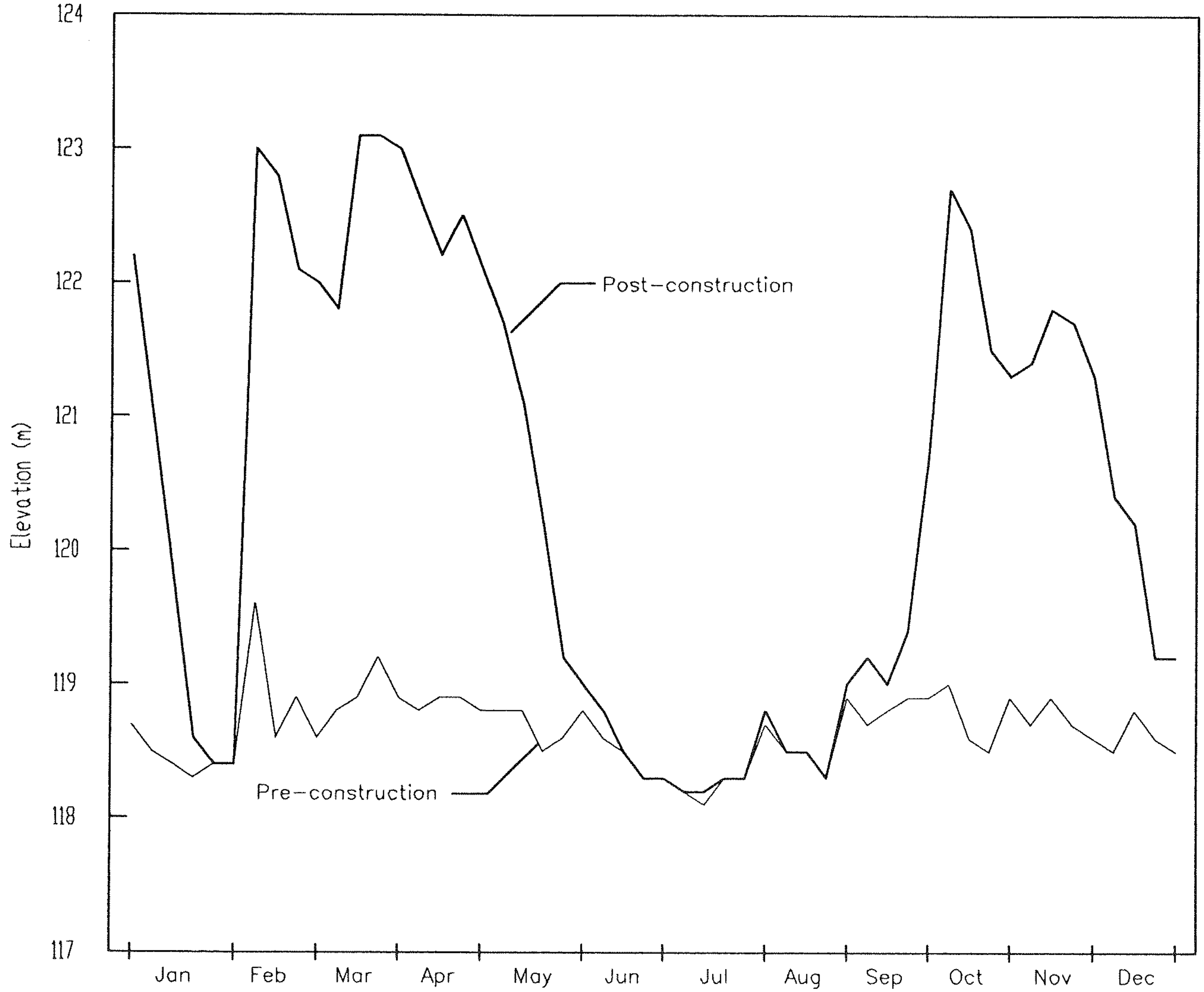


FIG. 3.7

NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

SIMULATED DUNN'S POND WATER LEVELS – 1991

ACRES

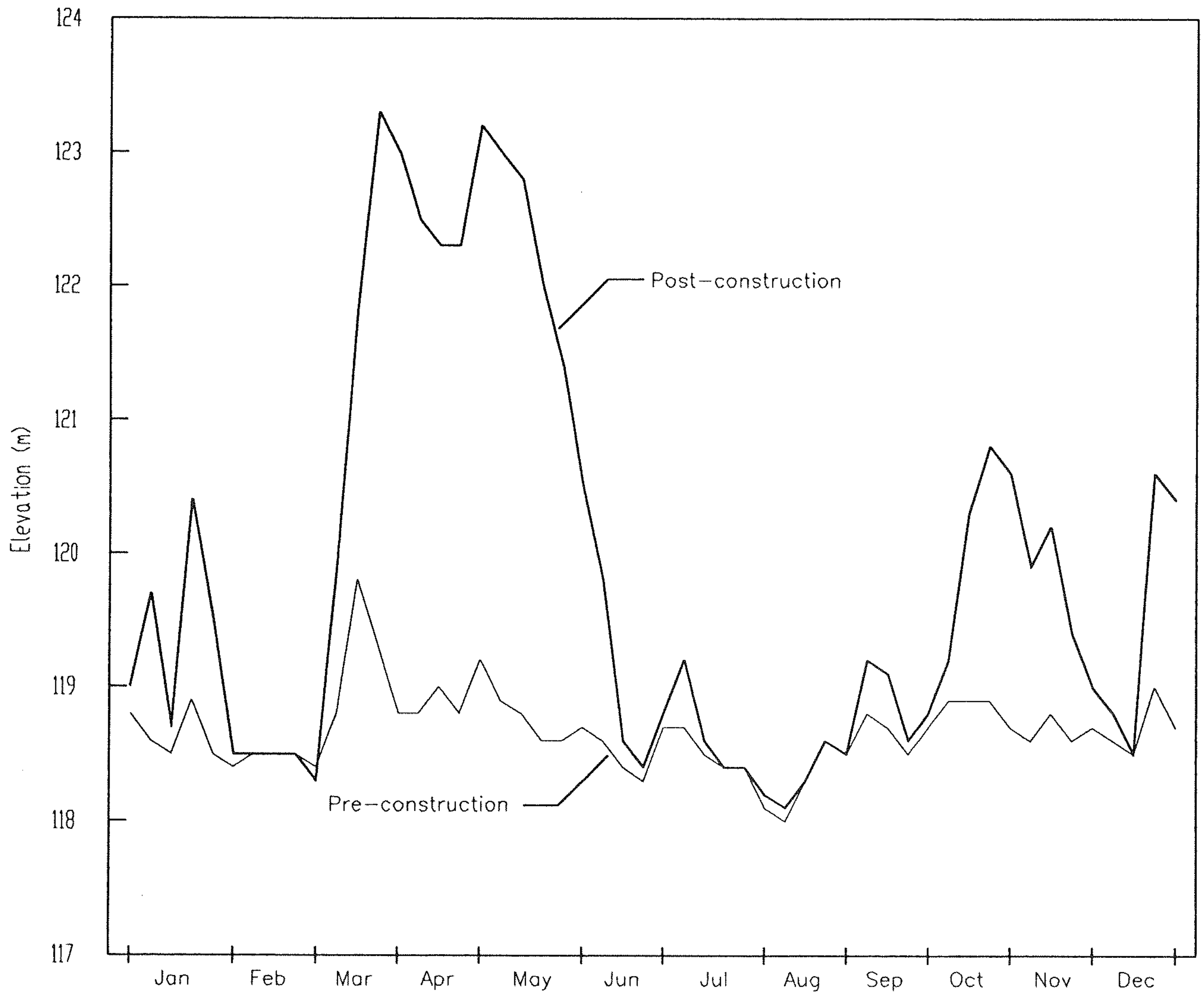


FIG. 3.8

NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

SIMULATED DUNN'S POND WATER LEVELS – 1992

ACRES

Design Considerations

4 Design Considerations

4.1 General

The primary objective of engineering at the feasibility level is to determine the parameters for final design and the costs associated with the construction of various structure types and sizes. Dam types considered in this study include conventional concrete, roller compacted concrete, timber crib, gabions, and rockfill. Various combinations of these dam types were considered to determine the most economic regulating structure at each sill elevation. The structure costs, along with the corresponding sill elevation benefits, are the basis for the economic optimization analysis as outlined in Section 5.

4.2 Dam Alignments

Two alternatives were identified at the outset of the feasibility study as a possible location for the structure. Both structure locations are shown in Plate 2. Axis B is located at the natural control of Dunn's Pond outlet. This arrangement requires a saddle dam with a possible length ranging from 120 m to 180 m, depending on the crest elevation selected. Axis A is located upstream of the control with a total length of about 160 m. At this location of the river, surveys have indicated water depths of up to 4.5 m with unknown depths of silt in the river bed. Subsequent cost estimates showed that additional provisional sums to account for the uncertainty of river bed conditions would discount this alignment as an economically viable solution.

4.3 Preliminary Designs

The structure types listed below were reviewed for suitability as self-regulating structures at Dunn's Pond outlet.

- Conventional Concrete Dam
- Roller Compacted Concrete Dam (RCC)
- Timber Crib Dam
- Rockfill Freeboard Dyke
- Gabion Dam

Various combinations of the above structures were considered at Axis B for both the main dam in the river channel and the saddle dam. Because the rockfill structure is non-overtoppable, it was only considered as a freeboard dyke saddle dam.

Cross-sections of each structure type were developed for the purpose of estimating preliminary quantities only. The dimensions to be used for construction will be determined during final design.

4.4 Dam Profile

For each combination of dam type and sill elevation, a unique dam profile was developed. Each arrangement is designed with a spillway capacity of 500 m^3/s, corresponding to a flood with an approximate return period of about 10,000 years. In cases where both the main dam and saddle dam were overtoppable, a two-step spillway profile was developed. This approach would restrict all spill up to 150 m^3/s (1:10 year flood) to the spillway in the main dam, thus confining flow to the original streambed.

Plates 3 and 4 show the dam profiles for the eventual desired arrangement (as determined in Section 5). In this case the saddle dam is a nonovertoppable rockfill structure in which all floods are passed by the two-step spillway of the main dam.

4.5 Diversion During Construction

To allow access to the riverbed during construction, a two phase dewatering operation will be used. A cofferdam constructed to elevation 120.0 m will be used to divert all flow away from the south shore of the river. The riverbed will be cleaned and the dam will be built around the pipe arch culverts placed at invert elevation 117.5 m. This portion of the dam will be constructed up to elevation 122.5 m leaving a 14.0 m wide notch directly above the pipes with sill elevation at 119.5 m.

During the second phase of the dewatering, a cofferdam will be constructed to elevation 122.0 m to divert flow away from the north shore towards the pipes and notch as explained above. The dam will then be constructed to elevation 122.5 m in the dewatered section of the riverbed and up the north bank. The diversion arrangement has the capability to pass 50 m^3/s at a water surface elevation of 121.0 m. This flow corresponds to a summer flood (i.e. June to September) with a return of about 5 years.

During the construction sequence, the rockfill portion of the structure will proceed ahead of the concrete portion to ensure that the area will not be flooded if a major storm occurs during construction. A temporary bridge will be provided across the river downstream of the damsite to allow for equipment access to the north side.

4.6 Concrete Dam

The concrete portion of the structure will be a gravity section. An energy dissipating flip bucket will be provided in the section of the dam in the old riverbed. In general, a dam height of less than two metres will not require a flip bucket.

General design considerations will be

- Top width	1.5 m
- Upstream face	vertical
- Downstream slope	0.7 H to 1.0 V
- Concrete Strength	20 MPA
- Maximum Aggregated Size	150 mm
- Reinforcing Steel	Temperature only in dam
	To be designed for walls and flip bucket

- Foundation Treatment	Rock quality to determine foundation grouting and drainage requirements

4.7 Rockfill Dam

The freeboard rockfill dam will be constructed from station 0+227 to 0+371 as shown on Plate 4. This structure will be comprised of a dumped and equipment compacted rockfill portion with a filter and dumped impervious blanket on the upstream face. Depending on the gradation of the impervious material, the surface may be self protecting or require a granular or rockfill blanket to prevent erosion.

General design considerations are

Rockfill:	3 m top width 1.5 H to 1.0 V slopes 500 mm to fine rock, well graded
Rockfill for Slope Protection:	300 mm to fine rock, well graded
Filter:	1.5 m thick Gradation to be from final design Foundation cleanup to bedrock
Impervious:	2 m top width 2.0 H to 1.0 V upstream slope 12% maximum passing No. 200 sieve Gradation to be set in final design Compaction by working equipment Foundation cleanup to bedrock

4.8 Gabion Dam

The gabion option was reviewed after the range of crest elevations were determined. Two key points were noted at this stage regarding the use of a gabion structure as an appropriate alternative.

- Since the range of the dam heights is 4 m to 8 m, and the structure would have to be capable of passing a flood of 500 m^3/s, it was felt that a contained rockfill structure would not meet the desired safety requirements.

- There is a concern with the method of applying a water tight membrane that would withstand settlement in the gabions and ice problems during the winter.

In consideration of the potential problems associated with an overflow gabion structure of this size, it was decided to eliminate gabions at this stage.

4.9 Culvert

Two 1880 x 1260 arch culverts will pass normal flows through the structure and attenuate flows during flood events. This arrangement adequately satisfies the hydraulic requirements as well as structural considerations.

The present design assumes the following

- size: 1880 mm x 1260 mm;
- material: asphalt coated;
- 117.5 m invert;
- culvert set in concrete;
- no trash racks at entrance.

This arrangement has been selected because it provides the required hydraulic function, and is relatively inexpensive and simple to construct. When the foundation conditions and invert elevations are known in greater detail, this arrangement can be modified, as long as the discharge characteristics are approximately maintained.

Economic Optimization

5 Economic Optimization

The economic optimization required the selection of

1) optimum dam type (or combination of types) from among

- rockfill (saddle dam only);
- concrete (conventional);
- timber crib;
- roller-compacted concrete;

2) optimum spillway sill elevation, expected to be in range of 120 m to 126 m (limited by topography at the higher elevations).

5.1 Preliminary Analysis

A preliminary analysis was first carried out based on the cost of dam volumes only to reduce the number of options of feasible dam types and sill elevations. On a project of this nature and magnitude, the cost due to dam volumes is expected to account for approximately 50% of the Capital Cost Estimate (CCE). All other items in the CCE are not expected to vary appreciably with dam height.

The possible structure arrangements include combinations of conventional concrete, roller-compacted concrete (RCC), and timber crib as either the main dam or saddle dam, along with a possible rockfill freeboard dyke as a saddle dam only. The objective of the preliminary economic analysis was to identify the most cost efficient combination of the main dam and saddle dam at each sill elevation.

The first step in the analysis was to directly compare RCC and conventional concrete costs. Based on Acres recent experience (NUGs), review of literature, and prices from a local contractor who bid on the Lake Robertson RCC dam (see Appendix B), a unit price of $300/m^3 was assumed for RCC. Because these cost estimates were developed for larger dams than those proposed for this project, and due to the learning curve expected to be associated with construction of an RCC dam, this price is considered to be at the lower end of the expected range.

The results of the conventional concrete/RCC comparison are shown in Table 5.1. A unit price of $\$400/m^3$ was assumed for conventional concrete. This table indicates costs for an RCC dam 20% to 25% greater than conventional concrete. Because of this greater cost, and the higher risk associated with the RCC unit price, conventional concrete was selected in preference to RCC.

The dam volumes were then calculated for the six remaining alternatives and are summarized in Table 5.2. The results indicate that both concrete / rockfill and timber crib / rockfill are the most economically viable arrangements by a considerable amount. It was therefore decided to do a detailed cost / benefit analysis comparing these two arrangements.

5.2 Dam Type Selection

A final dam type selection is based on total project costs for each arrangement at various sill elevations. This is provided through the capital cost estimates as described in Section 7. These cost estimates include total direct costs, 10% contingency, and 15% management and engineering. Interest during construction and owner's costs are not included.

Table 5.3 provides a cost / benefit analysis of the two arrangements. In both cases, the project is economically viable at sill elevations in the range of 122.0 m to 124.0 m. The benefit-to-cost ratios (B/C) suggest an optimum project at the lower elevations regardless of the structure type selected. The desired structure type can thus be chosen independent of the preferred dam height as evaluated in Section 5.3 below.

Both types of structures were then compared quantitatively and qualitatively at the 122.0 m to 124.0 m range. Quantitatively, the timber crib/rockfill structure is about ten to fifteen percent cheaper. On a qualitative basis, the concrete structure is preferred because of its greater flood handling abilities, durability and spillway constructability. In addition, there is greater uncertainty associated with the unit price used for timber crib than that of concrete. After discussions with Hydro, a concrete structure was selected for the main dam, with a rockfill freeboard saddle dam.

5.3 Dam Height Selection

Having previously established optimum sill elevations in the range of 122.0 m to 124.0 m, further refinements of the dam quantities and subsequent cost estimates were required for a final selection of dam height. Capital costs were calculated using detailed quantity take-offs at sill elevation intervals of 0.5 m between 122.0 m and 124.0 m. These costs are shown with the corresponding energy benefits in Table 5.4.

Also presented in Table 5.4 is the overall project cost of each scenario. This cost is defined as the sum of capital cost plus the opportunity cost of not maximizing energy benefits. Because maximum benefits are realized at an elevation of 124.0 m (within the range considered), the difference in these benefits and those calculated at lower elevations is calculated as an opportunity cost. The optimum project is then defined as the arrangement which minimizes overall project cost.

Figure 5.1 shows a graphical representation of benefits, capital costs, and overall project costs. Overall costs are minimized in the range of 122.0 m to 122.5 m. After consultation with Hydro, 122.5 m was selected as the preferred sill elevation. This arrangement maximizes benefits within the range of minimum overall project costs (i.e. 122.0 m to 122.5 m).

Table 5.1

Preliminary Comparison of RCC and Conventional Concrete Dam Costs

Sill Elevation	Conventional Concrete	RCC
122.0	$ 914,000	$1,154,000
124.0	$1,744,000	$2,085,000
126.0	$2,818,000	$3,488,000

Table 5.2

Preliminary Dam Volumes and Costs

Dam Type	Unit Price ($/m³)
Concrete	400
Timber Crib	220
Rockfill	20
Impervious	15

Alternative #1

Sill (m)	Main Dam Concrete	Saddle Dam Concrete	Dam Cost
122.0	1186	1098	$913,600
123.0	1506	1518	$1,209,600
124.0	2205	2154	$1,743,600
125.0	2690	2926	$2,246,400

Alternative #2

Sill (m)	Main Dam Concrete	Saddle Dam Timber Crib	Dam Cost
122.0	1186	1585	$823,100
123.0	1506	2326	$1,114,100
124.0	2205	3443	$1,639,500
125.0	2690	4858	$2,144,800

Alternative #3

Sill (m)	Main Dam	Saddle Dam		Dam
	Concrete	Rockfill	Impervious	Cost
122.0	1146	4719	1650	$577,500
123.0	1521	8302	3079	$820,600
124.0	2013	10472	3959	$1,074,000
125.0	2780	13551	5233	$1,461,500

Alternative #4

Sill (m)	Main Dam Timber Crib	Saddle Dam Concrete	Dam Cost
122.0	1623	1098	$796,300
123.0	2229	1518	$1,097,600
124.0	3326	2154	$1,593,300
125.0	4305	2926	$2,117,500

Alternative #5

Sill (m)	Main Dam Timber Crib	Saddle Dam Timber Crib	Dam Cost
122.0	1623	1585	$705,800
123.0	2229	2326	$1,002,100
124.0	3326	3443	$1,489,200
125.0	4305	4858	$2,015,900

Alternative #6

Sill (m)	Main Dam	Saddle Dam		Dam
	Timber Crib	Rockfill	Impervious	Cost
122.0	1603	4719	1650	$471,800
123.0	2254	8302	3079	$708,100
124.0	3101	10472	3959	$951,000
125.0	4356	13551	5233	$1,307,800

Table 5.3

Economic Comparison of Dam Types

Water Value =	0.341 $ / kWh
Energy Factor =	0.09 kWh / m³

CONCRETE + ROCKFILL

FSL	Savings Spill (m3*10³)	Savings Benefit ($)	Incremental Benefit ($)	Total Cost ($)	Incremental Cost ($)	Benefit/Cost Ratio
Present	–	–		–		
			$1,444,000		$1,022,000	
122	47100	$1,444,000		$1,022,000		1.41
			$178,000		$316,000	
123	52800	$1,622,000		$1,338,000		1.21
			$251,000		$465,000	
124	61000	$1,873,000		$1,803,000		1.04
			$284,000		$483,000	
125	70300	$2,157,000		$2,286,000		0.94

TIMBER CRIB + ROCKFILL

FSL	Savings Spill (m3*10³)	Savings Benefit ($)	Incremental Benefit ($)	Total Cost ($)	Incremental Cost ($)	Benefit/Cost Ratio
Present	–	–		–		
			$1,444,000		$887,000	
122	47100	$1,444,000		$887,000		1.63
			$178,000		$308,000	
123	52800	$1,622,000		$1,195,000		1.36
			$251,000		$455,000	
124	61000	$1,873,000		$1,650,000		1.14
			$284,000		$501,000	
125	70300	$2,157,000		$2,151,000		0.99

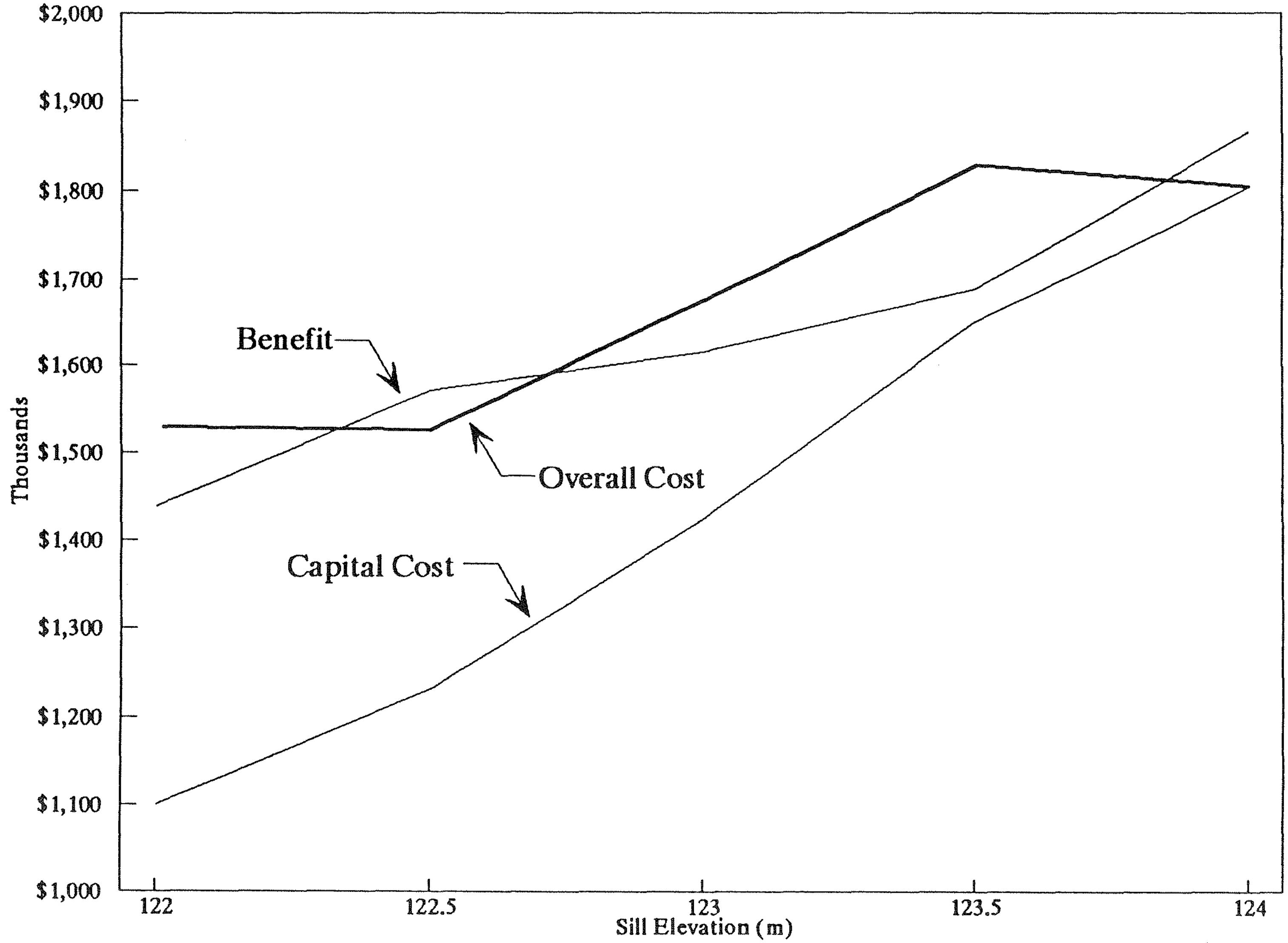
Table 5.4

Final Dam Height Optimization

Sill Elevation (m)	Energy Benefit ($)	Opportunity Cost * ($)	Capital Cost ($)	Overall Project Cost ** ($)
122	$1,444,000	$429,000	$1,100,000	$1,529,000
122.5	$1,578,000	$295,000	$1,232,000	$1,527,000
123	$1,622,000	$251,000	$1,424,000	$1,675,000
123.5	$1,695,000	$178,000	$1,650,000	$1,828,000
124	$1,873,000	$0	$1,803,000	$1,803,000

* Opportunity Cost = cost of not maximizing potential benefits

** Overall Project Cost = Capital Cost + Opportunity Cost



NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

DAM HEIGHT OPTIMIZATION

FIG. 5.1

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Final Project Arrangement

6 Final Project Arrangement

The recommended project is shown in Plates 2 to 5. It consists of a concrete dam across the main channel and a rockfill saddle dam. An orifice in the main dam will pass normal flows through the structure and attenuate flood flows. The characteristics of the project components are as follows.

Main Dam

• Type	Conventional Concrete
• Total length	155 m
• Spillway	
- Main Section	
Length	78 m
Sill Elevation	122.5 m
- Stepped Section	
Length	56 m
Sill Elevation	123.5 m
- Total Spill Capacity	500 m^3/s
• Crest Elevation at Abutment	124.6 m
• Maximum Height	5.0 m

Saddle Dam

• Type	Rockfill
• Total Length	144 m
• Crest Elevation	125.5 m
• Maximum Height	7.4 m

Orifice

• Type	2 Arch Culverts
• Size	1880 mm x 1260 mm
• Material	Asphalt Coated
• Invert	117.5 m

Cost and Schedule

7 Cost and Schedule

The dam cross-sections developed for the purpose of estimating preliminary quantities served as a basis for the economic optimization analysis. In preparation of a final CCE, further refinement of the concrete and rockfill dam cross sections were required.

After a review of the geotechnical information and the site walkover, an overburden depth of 0.5 m was assumed for the purpose of quantity calculation.

Foundation cleanup was considered under the gravity concrete section and the impervious and filter section of the rockfill dam. The rockfill would be placed on bedrock or acceptable foundation as determined at the time of construction. Quantities for concrete, rockfill, filter material and impervious fill were calculated from 10 m sections using the field surveyed centerline. These quantities were checked against the elevations on the topographic map from elevation variances upstream and downstream of the centerline.

7.1 Capital Cost Estimate

The capital cost estimate was developed using calculated quantities and unit prices from recent projects, budget quotes from contractors, and discussions with contractors regarding anticipated prices for 1994. This estimate reflects relatively low current prices due to the current inactivity in the construction industry and should be valid for the next year or two.

Table 7.1 shows a breakdown of the capital cost estimate for the concrete / rockfill structure at sill elevation of 122.5 m. The total direct cost is expected to be accurate to within ±10 percent.

Ordinary environmental costs associated with construction (e.g. site cleanup and minor restoration) are contractor's responsibility are included in the cost estimate.

Table 7.1

Capital Cost Estimate (January,1994 $)

Description	Quantity	Unit	Cost	Amount	Total
General					
Mobilization	1	LS	$10,000	$10,000	
Camp cost	1	LS	$20,000	$20,000	
Roads	1	km	$50,000	$50,000	
Bridge	1	LS	$10,000	$10,000	
Unwatering	1	LS	$10,000	$10,000	
		Sub Total (General)		$100,000	$100,000
Dam					
Excavation – common	2700	cm	$6	$16,200	
Excavation – rock	50	cm	$40	$2,000	
Foundation cleanup	2000	sm	$8	$16,000	
Grouting/Drainage	1	LS	$10,000	$10,000	
Concrete – mass	1600	cm	$400	$640,000	
Concrete – walls	20	cm	$600	$12,000	
Culvert	20	m	$500	$10,000	
Rockfill	6800	cm	$18	$122,400	
Impervious	2100	cm	$12	$25,200	
Filter	1000	cm	$20	$20,000	
		Sub Total (Dam)		$873,800	$873,800
		Total Direct			$973,800
		Contingency (10%)			$97,400
		Total Construction Cost			$1,071,200
		Management & Engineering (15%)			$160,700
		Total Project Cost			$1,231,900

* Note: IDC and owner's cost not included

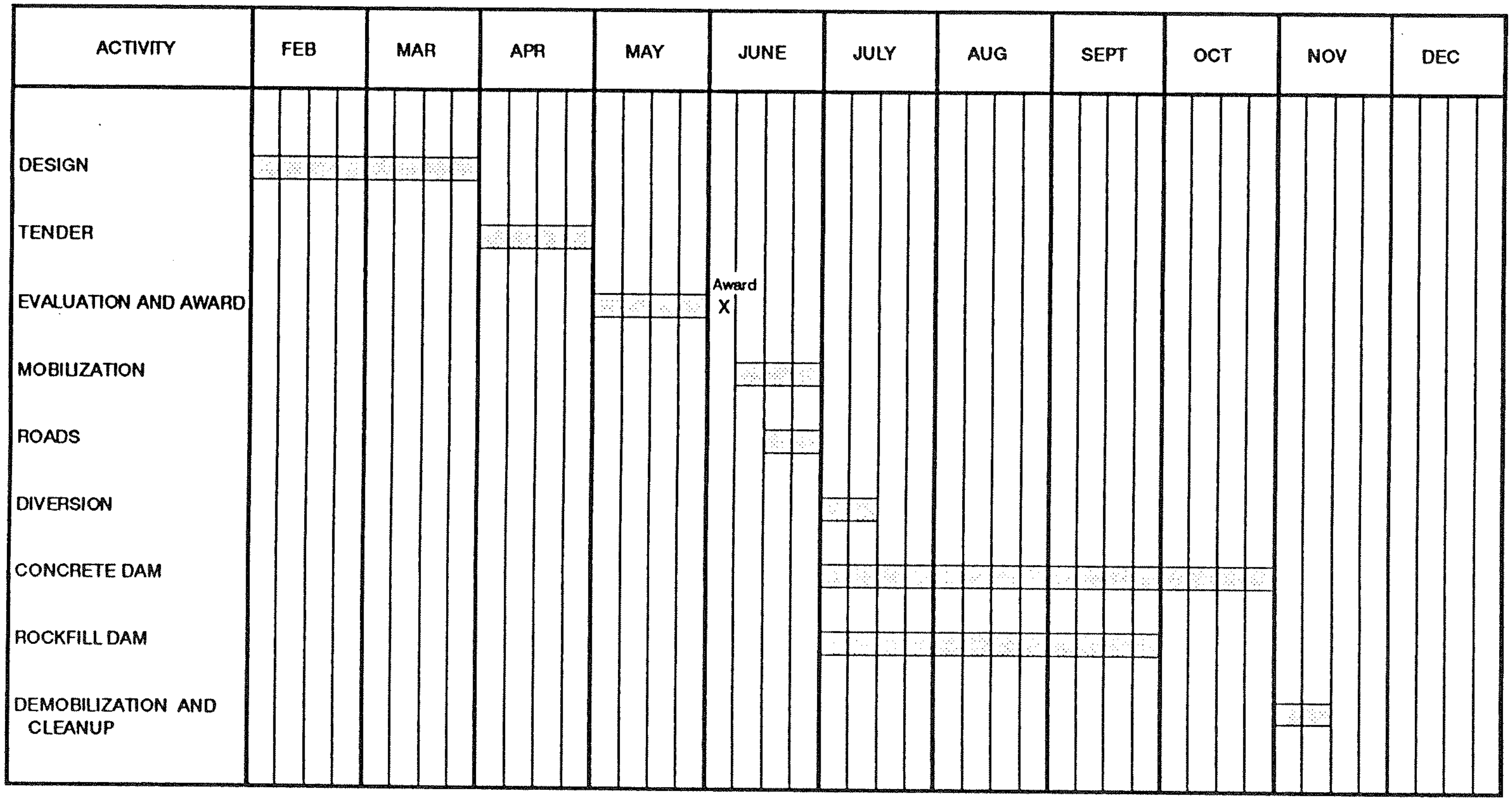
7.2 Construction Schedule

Due to the relatively small quantities, the project will be carried out in one construction season. Assuming a project approval date of February 1, the design and tendering will be completed by the end of April. The construction contract would then be awarded by June 1. This would enable the contractor to mobilize and complete the construction of the access bridge before the end of June.

Work would start immediately on the rockfill dam portion. The completion of this section of work before the concrete dam can be raised to full height over the entire length is necessary to meet flood requirements. The diversion, concrete dam and installation of the culvert will be carried on in parallel with the rockfill dam activity.

Completion of construction and demobilization should be completed by the end of October.

The construction schedule is illustrated in Figure 7.1.



* Note: This schedule assumes an approval to proceed date of February 1

NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
CONSTRUCTION SCHEDULE

FIG. 7.1

ACRES

Conclusions

8 Conclusions

The construction of an upstream regulating structure at Paradise River near the outlet of Dunn's Pond is feasible. The optimum arrangement for the structure consists of a concrete overflow section with a spillway sill elevation of 122.5 m and a rockfill section with a crest elevation of 125.5 m. This arrangement has an estimated total cost of $1,232,000 with an estimated average annual energy benefit of 4.63 GWh.

The structure will be self-regulating with normal flows passing through two 1880 mm by 1260 mm pipe arch culverts through the concrete portion of the dam in the natural river bed.

Appendix A

Volume/Area/Elevation Curve
Stage/Discharge Curves

Dunn's Pond Volume/Area/Elevation Curve

Elevation (m)	Area (km^2)	Volume (m^3)
115	0.00	0.00
116	1.00	0.50
118.6	2.81	5.45
119	3.70	6.76
120	5.40	11.31
121	5.90	16.96
122	6.35	23.08
123	6.80	29.66
124	7.25	36.68
125	7.70	44.16
126	8.00	52.01

Areas above 118.6 m taken from 1:5000 scale mapping, and correspond to flooded areas identified in Plate 5. Areas below 118.6 m estimated. See Section 3.1.2 for additional comments.

Natural Outlet Stage/Discharge Curve

Elevation (m)	Discharge (m^3/s)
117.9	0.0
118.0	0.2
118.1	0.5
118.2	0.9
118.3	1.4
118.4	2.0
118.5	2.6
118.6	3.9
118.7	5.8
118.8	9.5
118.9	14.7
119.0	21.0
119.2	36.1
119.4	54.1
119.6	74.5
119.8	97.0
120.0	121.5
120.2	147.9
120.4	175.9
120.5	190.5

Stage/Discharge Curves used in Optimization

Sill @ 122.0 m

Elevation (m)	Discharge (m³/s)		
	Orifice	Spillway	Total
117.5	0.0		0.0
117.6	0.2		0.2
118.1	3.4		3.4
118.2	4.8		4.8
118.5	6.6		6.6
119.0	8.1		8.1
119.5	9.3		9.3
120.0	10.4		10.4
120.5	11.4		11.4
121.0	12.3		12.3
121.5	13.2		13.2
122.0	14.0	0.0	14.0
122.1	14.1	5.7	19.8
122.5	14.7	63.6	78.3
123.0	15.5	180.0	195.5

Sill @ 123.0 m

Elevation (m)	Discharge (m³/s)		
	Orifice	Spillway	Total
117.5	0.0		0.0
117.6	0.2		0.2
118.1	3.4		3.4
118.2	4.8		4.8
118.5	6.6		6.6
119.0	8.1		8.1
119.5	9.3		9.3
120.0	10.4		10.4
120.5	11.4		11.4
121.0	12.3		12.3
121.5	13.2		13.2
122.0	14.0		14.0
122.5	14.7		14.7
123.0	15.5	0	15.5
123.1	15.6	5.7	21.3
123.5	16.1	63.6	79.7
124.0	16.8	180.0	196.8

Sill @ 124.0 m

Elevation (m)	Discharge (m³/s)		
	Orifice	Spillway	Total
117.5	0.0		0.0
117.6	0.2		0.2
118.1	3.4		3.4
118.2	4.8		4.8
118.5	6.6		6.6
119.0	8.1		8.1
119.5	9.3		9.3
120.0	10.4		10.4
120.5	11.4		11.4
121.0	12.3		12.3
121.5	13.2		13.2
122.0	14.0		14.0
122.5	14.7		14.7
123.0	15.5		15.5
123.5	16.1		16.1
124.0	16.8	0.0	16.8
124.1	16.9	5.7	22.6
124.5	17.4	63.6	81.0
125.0	18.1	180.0	198.1

Sill @ 125.0 m

Elevation (m)	Discharge (m³/s)		
	Orifice	Spillway	Total
117.5	0.0		0.0
117.6	0.2		0.2
118.1	3.4		3.4
118.2	4.8		4.8
118.5	6.6		6.6
119.0	8.1		8.1
119.5	9.3		9.3
120.0	10.4		10.4
120.5	11.4		11.4
121.0	12.3		12.3
121.5	13.2		13.2
122.0	14.0		14.0
122.5	14.7		14.7
123.0	15.5		15.5
123.5	16.1		16.1
124.0	16.8		16.8
124.5	17.4		17.4
125.0	18.1	0.0	18.1
125.1	18.2	5.7	23.9
125.5	18.6	63.6	82.2
126.0	19.2	180.0	199.2

Appendix B

Cost Information From Contractors

ACRES **Calculations** By M. Mills Date Mar. 11/94 Project No. P10726.00

SUBJECT Checked ____ Date ____ Calculation No. ____ Page ____ of ____

B-1

RCC COSTS FOR LAKE ROBERTSON

From McNamara Construction:

Volume:	24,000 m^3 5,000 m^3 Concrete Facing
RCC including Fly Ash and Cement	\$190/m^3
Concrete	\$550/m^3
24,000 m^3 @ \$190/m^3	\$4,560,000
5,000 m^3 @ \$550/m^3	\$2,750,000
29,000 m^3	\$7,310,000

Average \$252

20% increase for small dam

RCC = \$300/m^3

ACRES

Calculations

By T. Chislett Date March 1994 Project No. P10726.00

Checked ________ Date ________ Calculation No. ________

Page ____ of ____

SUBJECT

BUDGET PRICES FOR DAM

SOURCE	Mass Concrete ($\$/m^3$)	Rockfilled Timber Crib: Crib ($\$/m^3$)	Rockfilled Timber Crib: Sheeting ($\$/m^2$)
Trident Construction (January 6, 1994)	400-450	175	40-45
Trent Construction (December 28, 1993)	540	170	48
Little Harbour River Hydroelectric Development (August, 1993)	*200		
Southwest River Hydroelectric Development (August, 1993)	*300		

*NOTE: Unit prices derived from recent projects, as well as budget prices from contractors and suppliers.

B-3

ACRES **Record of Telephone Call**

File P10726.00

Date January 26, 1994

Date of Call January 26, 1994

Project Paradise River Sheet 1 of 1

Subject Budget Price for Arch Culvert

Call (to) (from) Roger Goobie Tel. No.
(Company)

Discussion between ARMCO and Tony Chislett
(of Company) (of Acres)

Details of Call

TC requested a budget price for the supply of two 1880 x 1260 arch culverts.

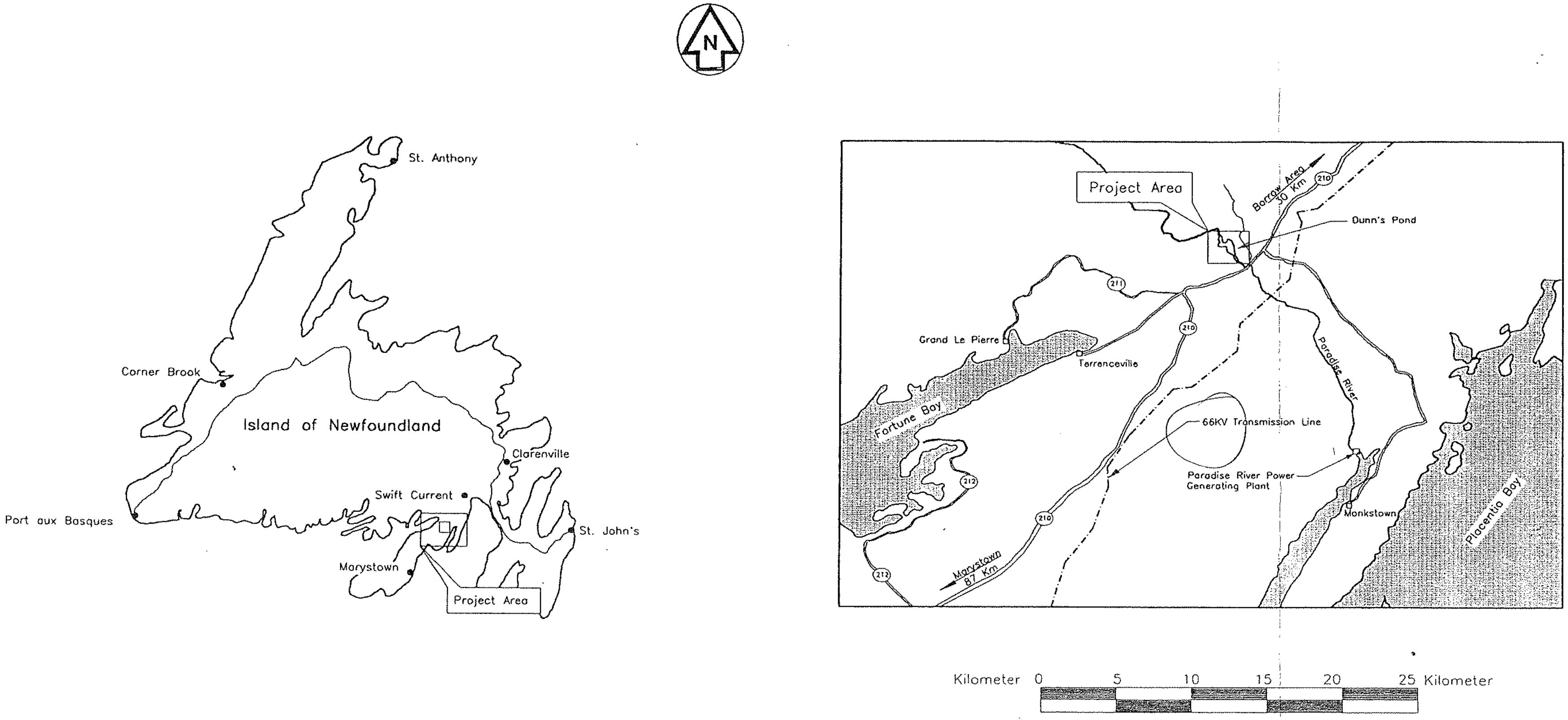
RG quoted $208.00 per linear metre (plus tax).

Action Required

Copies to (1)
(2)
(3)
(4)

Circulation to (1)
(2)
(3)
(4)
(5)
(6)
(7)
(8) File

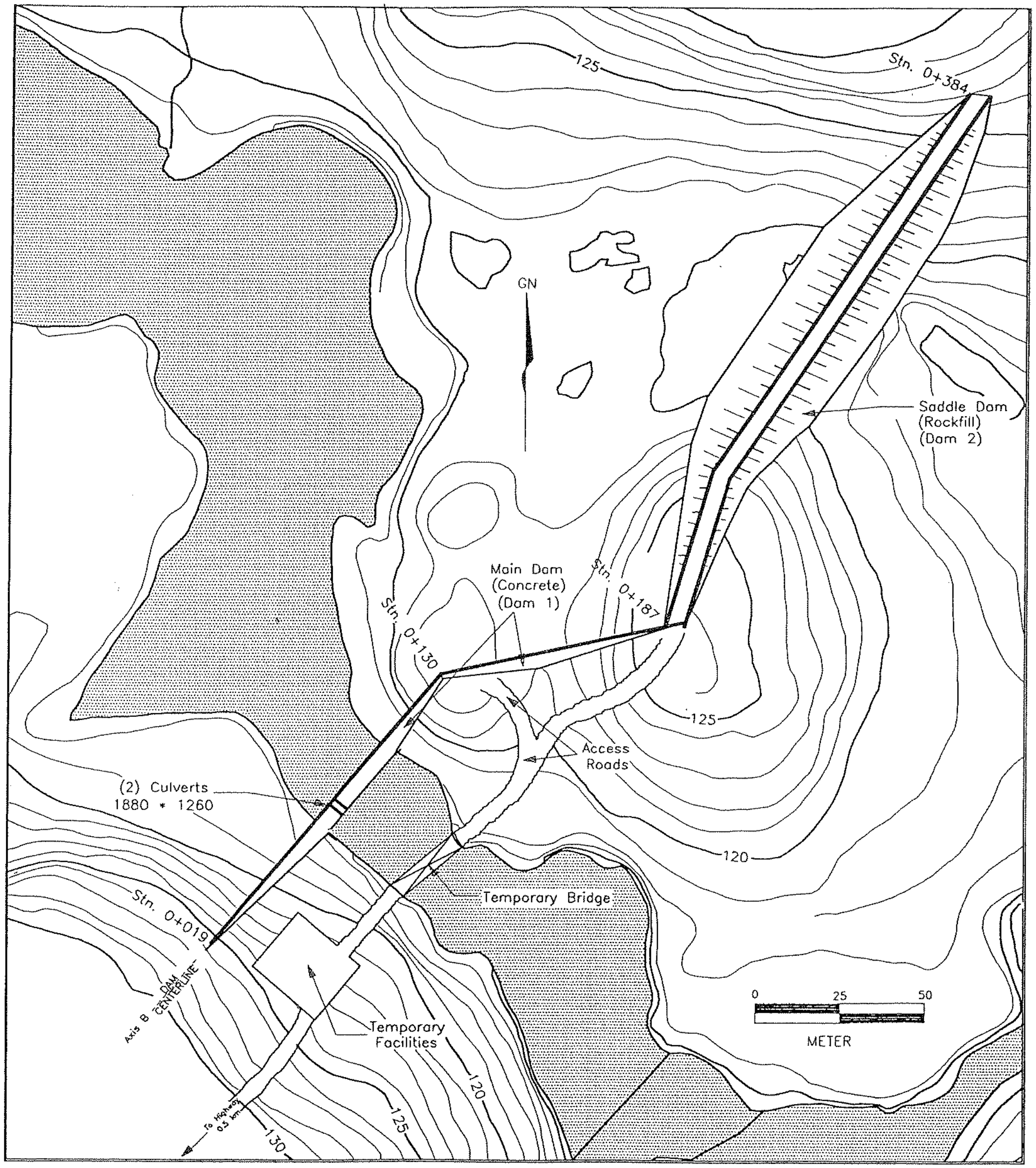
Plates

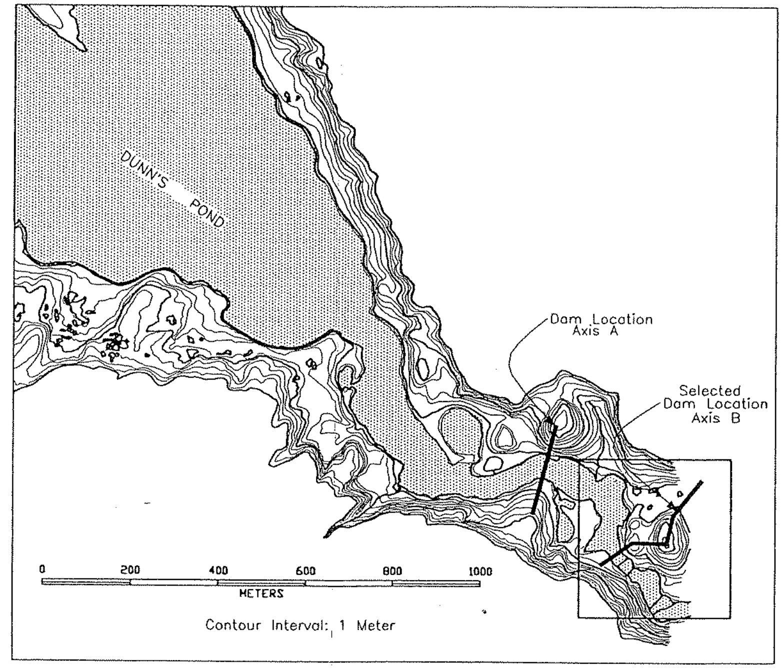


NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
LOCATION PLAN

PLATE 1

ACRES





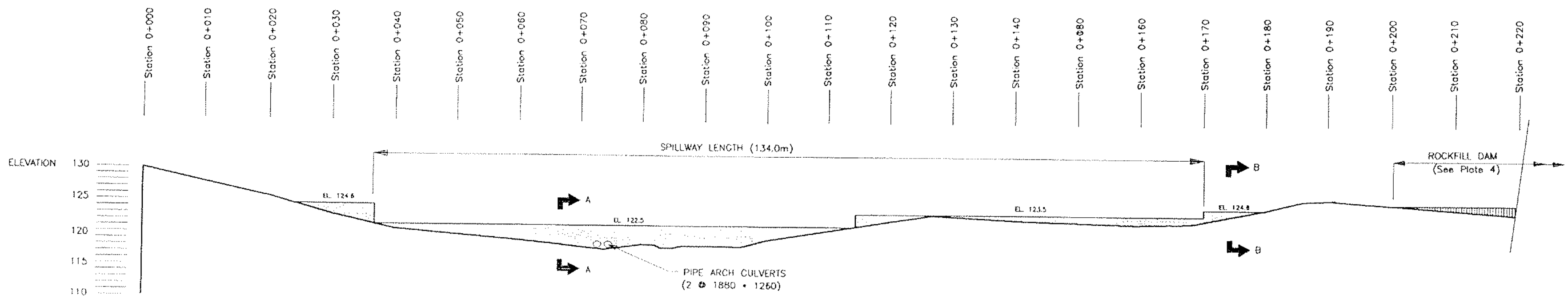
NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

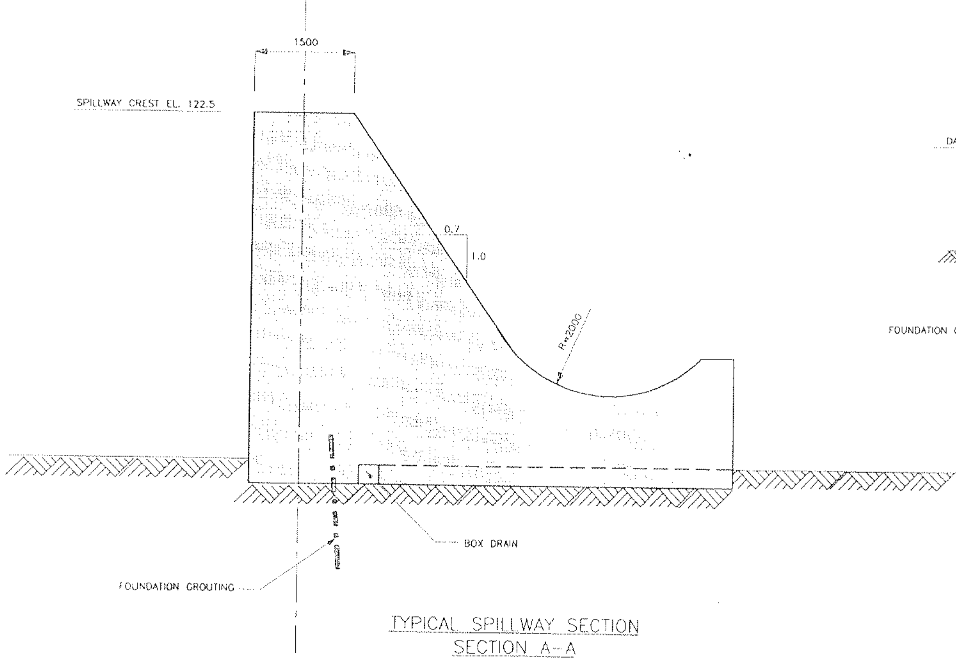
PROJECT ARRANGEMENT

PLATE 2

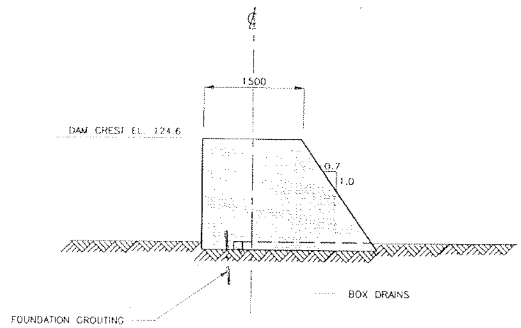
ACRES



DAM CENTERLINE PROFILE



TYPICAL SPILLWAY SECTION
SECTION A–A



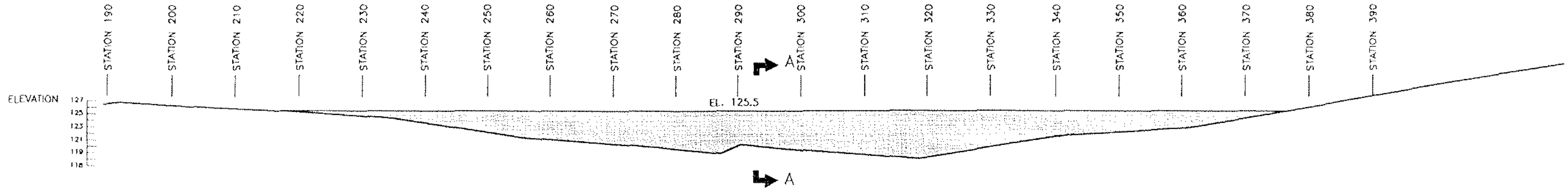
TYPICAL DAM
SECTION B–B

STAGE/DISCHARGE CURVE

Elevation	Discharge (m^3/s)	
(m)	Culverts	Spillway
117.5	0	--
118.5	3.6	--
119.0	6.0	--
119.5	7.8	--
120.0	9.3	--
120.5	10.6	--
121.0	11.8	--
121.5	12.8	--
122.0	14.0	--
122.5	14.8	0
123.0	15.2	46.9
123.5	15.9	132.6
124.0	16.6	273.3
124.6	17.2	500.4

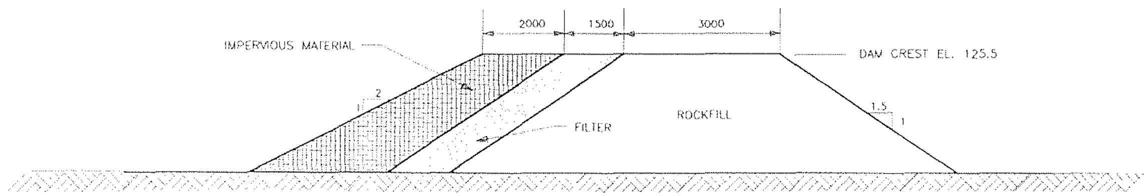
NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
CONCRETE DAM – PROFILE AND SECTIONS

PLATE 3
ACRES



ROCKFILL DAM PROFILE
SECTION A–A

METER 0 10 20 METER

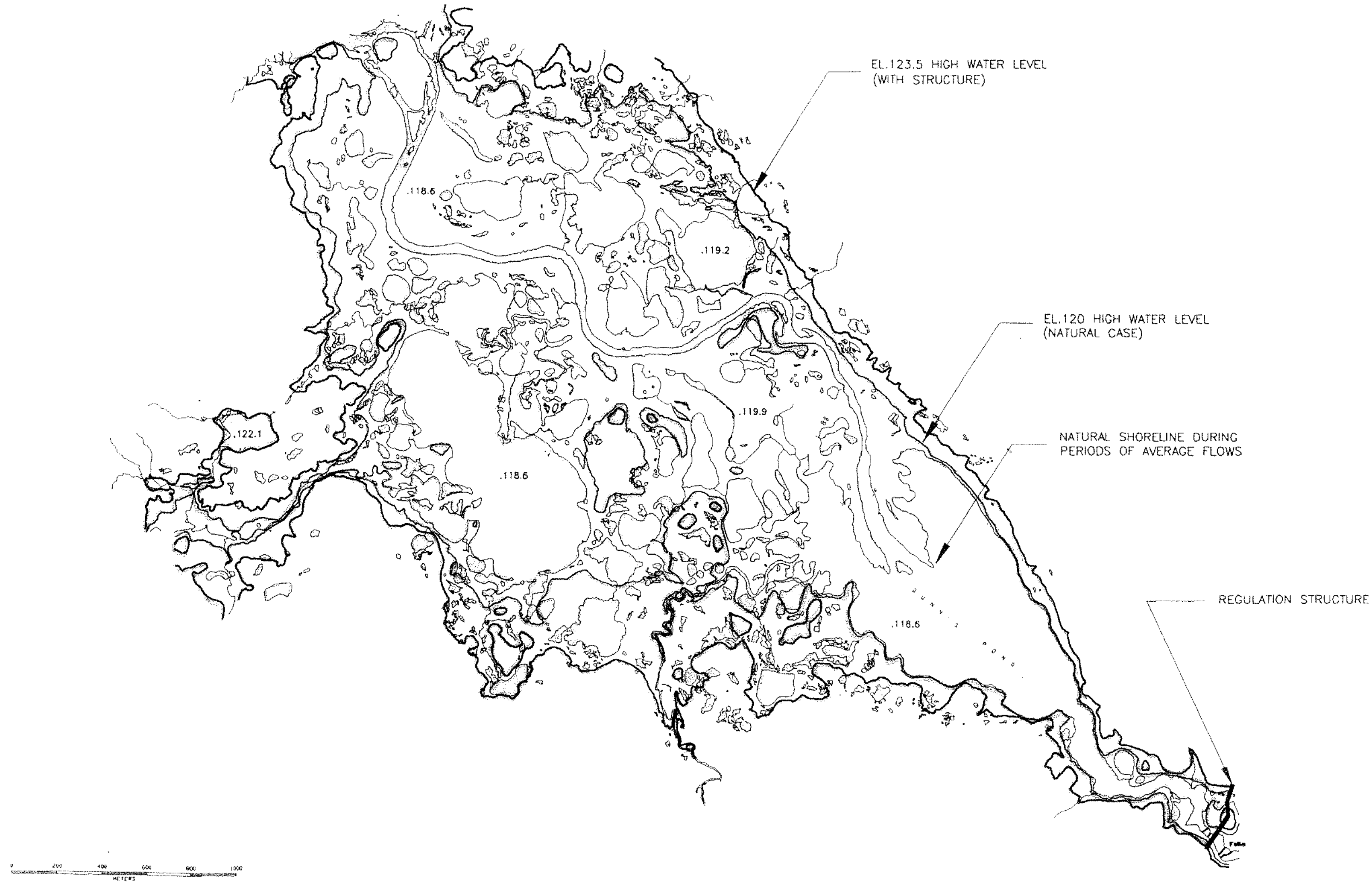


TYPICAL ROCKFILL DAM SECTION
SECTION A–A

METER 0 2 4 METER

NEWFOUNDLAND AND LABRADOR HYDRO
PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER
ROCKFILL DAM – PROFILE AND SECTION

PLATE 4
ACRES



NEWFOUNDLAND AND LABRADOR HYDRO

PROPOSED UPSTREAM REGULATION STRUCTURE – PARADISE RIVER

DUNN'S POND WATER LEVELS

PLATE 5

ACRES

PARADISE RIVER

WATER REGULATION STRUCTURE

PROJECT OVERVIEW

JUNE 1994

PARADISE RIVER WATER REGULATION STRUCTURE

PROJECT OVERVIEW

In February 1991 ACRES, through the auspices of CEA, produced a report entitled "*Selection of Economic Structure Types to Provide Upstream Regulation and Reduce Spills during Small Flood Events in Remote Locations*". Although this report was not done primarily for Newfoundland and Labrador Hydro (Hydro), it was jointly sponsored by Hydro and Energy Mines and Resources Canada. This report selected Paradise River as a suitable site for construction of a prototype structure to regulate spill. Several types of spill control structures were studied but the principal findings of the study showed that either a rollcrete or gabion type design was an attractive option for the Paradise River site. The report recommended that further investigation into these options be conducted.

Structure Design

Further investigations by Hydro's Engineering Department showed that the gabion type structure was more economical than the rollcrete structure since there were concerns that additional high costs would be incurred if a concrete shell for ice protection was required for the rollcrete dam. Provision was made in the cost estimate for timber sheeting of the upstream face of the Gabion dam for ice protection.

Capital Budget Proposal

In the Spring of 1993 and for the 1994 Capital Budget process, a proposal was made for the construction of the water regulation structure. Based on available information at that time, Engineering, together with System Planning showed that the project was economical. The Capital

Cost estimate was $1.2 million for in-service in October, 1995. The expected energy gain from the project was between 5.12 GW.h and 5.26 GW.h depending on which energy conversion factor was used. The fuel series used in the analysis at the time was that issued by Economic Analysis dated October 1992. An analysis of the project's feasibility by System Planning showed that the payback period was 8 years. This looked very attractive but because of the uncertainty associated with expected energy levels and costs, Management decided that a Consultant be commissioned to undertake a study to bring the project to a feasibility level and thus, firm up the expected energy to be gained as well as the costs.

Choice of Consultant

In the Fall of 1993, Engineering issued a *Request for Proposals for Engineering Design Work for Proposed Upstream Regulation Structure(s) at Paradise River Generating Plant*. Three responses were received by Hydro; from ACRES, Shawmont and BAE Group. Engineering reviewed the three proposals and recommended that ACRES be awarded the work. Although either consultant could adequately do the work, Engineering felt that ACRES' proposal was much better in hydrotechnical areas and provided the best value for the money spent.

The Work

ACRES, in its analysis, optimized the height of the water regulation structure to the elevation where the last increment of elevation would produce an energy savings equal to the incremental cost of energy of the alternative. The alternative was the marginal cost of fuel at Holyrood. Using the fuel series provided by Economic Analysis dated October 1993 it was shown that the value of 1 kW.h over a 30-year project economic life was 340.8 mills/kW.h. Using this value, ACRES established that the optimum elevation for the regulation dam would be 122.5 m. The value of expected energy gain was established as 4.63 GW.h. In April 1994, ACRES produced for Hydro a report,"*Upstream Regulation Structure Paradise River - Final Report*". The costs stated in the report did not include IDC, Owner's cost nor Environmental costs. When these costs were added the total capital cost was $2.0 million for in-service October 1996.

Environmental Issues

The initial environmental review of the proposed Paradise River Water Control Structure in 1992 suggested that registration would be required under the Environmental Assessment Act. No significant environmental concerns were immediately identified, but given Hydro's previous experience with Paradise River, it was felt that an environmental preview report (EPR) might be required. Consquently, a budget of $50,000.00 was identified to cover costs associated with registration and completion of an EPR. This budget did not include environmental compliance monitoring or the cost of mitigation.

In the Spring of 1993, the project was reviewed, and possible concerns related to waterfowl habitat were identified. A meeting was held with representatives of the Canadian Wildlife Service (CWS) who informed Hydro that the proposed project area is located within the most productive waterfowl habitat on the Island, and also that the area had been designated for habitat enhancement under a federal/provincial agreement. Apparently the proposed project area contains important nesting, rearing and staging habitat for waterfowl (particularly Canada geese), and is also a popular hunting area. CWS indicated that they would have significant concerns about the impacts of the proposed project on waterfowl, and that they would recommend an environmental impact statement if the project was registered.

Prior to initiating further discussions with regulatory agencies or registering the project, it was decided to undertake an engineering study to better define the project. The environment budget was reforecasted to include $5,000.00 in 1993 for review of the engineering study, and preparation of a registration, and $100,000.00 for preparation of an EIS in 1994-95. This represented an increase of $55,000.00 above the previous estimate. Again, this budget did not include environmental compliance monitoring, environmental effects monitoring or the costs of mitigation.

Upon completion of the engineering study in early 1994 by ACRES, the environmental component of the project was again reviewed. Using a number of assumptions about environmental concerns which could not be verified without registering the project, it was assumed that environmental approval for the EIS could not be expected prior to June, 1996. This schedule assumed that waterfowl surveys would be initiated in May, 1994. The cost of the component studies and the EIS was estimated at $223,000.00 (including 10% contingency). The costs of environmental compliance monitoring and environmental effects monitoring were estimated to be $72,000.00 and $120,000.00 respectively, bringing the total estimated environmental cost to $415,000.00. The costs of mitigation could not be estimated.

Further Analysis

Using a new fuel series issued by Economic Analysis in April 1994, System Planning again evaluated the cost of a kW.h over a 30-year project economic life. The value is 283.1 mills/kW.h which is a significant decrease from the 340.8 mills/kW.h used by ACRES. Engineering felt that it would not be worthwhile for ACRES to optimize the regulation dam elevation to a lower value than 122.5 m because of diminishing expected energy gains.

With the new fuel series and the expected energy gain of 4.63 GW.h, the payback period for the $2.0 million project is beyond the 30 year (and even the 60 year) life of the project. When two sensitivity analyses,ie., -10% change in Capital cost and +10% change in fuel prices, are done the payback periods in both cases are beyond 30 years. It is interesting to note that if the overall in-service cost had remained at $1.2 million and expected energy of 5.12 GW.h the project payback period would be 13 years using the April 1994 fuel prices. With an expected energy of 4.63 GW.h the payback period would be 16 years.

Conclusion

In retrospect, the decision to have a Consultant undertake a study to firm up project costs and expected energy gains was a prudent one. Since that time there have been three factors that have negatively affected the project's viability. These are:

- Lower expected energy gains.
- Lower fuel prices.
- Higher overall costs.

Therefore, it is recommended that this project not be included in the 1995 Capital Budget process and postponed for later evaluation.

Newfoundland and Labrador Hydro
P.O. Box 12400
Hydro Place, Columbus Drive
St. John's, Newfoundland
A1B 4K7

Reservoir Improvements at Burnt Pond: Spruce Pond Alternative

June 1990

Acres International Limited
St. John's, Newfoundland

ACRES

June 22, 1990
P09431.01

Newfoundland and Labrador Hydro
P.O. Box 12400
Hydro Place, Columbus Drive
St. John's, Newfoundland
A1B 4K7

Attention: Mr. Harvey Young
Director of Engineering Design

Dear Mr. Young:

Reservoir Improvements at Burnt Pond: Spruce Pond Alternative

Enclosed are 12 copies of our report on the Spruce Pond Alternative.

Our conclusion is that it is probably not economic to build a dyke at Spruce Pond for flood handling during the PMF. For energy savings, however, a single timber crib dyke looks quite attractive.

If you have any questions on this work, please feel free to call.

Yours very truly,

for Maurice S. Mills
Regional Manager

SHR:gm
Encl.

ACRES INTERNATIONAL LIMITED
4th Floor, Beothuck Building, Crosbie Place
P.O. Box 13337, Station A, St. John's, Newfoundland A1B 4B7
Telephone 709-754-1710 Facsimile 709-754-2717

Vancouver, Calgary, Winnipeg, Toronto, Burlington, Niagara Falls, Halifax, Sydney

Table of Contents

Summary

The purpose of the study was to determine whether it is economically and technically feasible to build a dyke at the outlet of Spruce Pond.

The benefits could be

1. energy savings through reduced spill at White Bear Spillway
2. capital cost savings by not having to provide flood protection (fuse plug) at Burnt Pond.

Several alternatives were examined (2 dykes, one at the outlet to each of two ponds, and overtoppable/nonovertoppable designs).

The most economic alternative is one timber crib overflow dyke at Spruce Pond, for energy benefits only. The approximate cost is estimated to be $4.7 million, with annual energy benefits of about $625,000 to $700,000.

The cost to provide flood protection is an additional $1.2 million (estimated at a total of $5.9 million) compared with the cost of a fuse plug (less than $1 million).

1 Introduction

1.1 Purpose

The purpose of this report is to evaluate the technical and economic feasibility of building one or two small dykes at Spruce Pond. These dykes would provide

- increased flood handling protection during the Probable Maximum Flood (PMF) at Burnt Pond, and
- energy savings by reducing the spill through White Bear Spillway during minor floods.

1.2 Background

The 1986 Flood Handling Alternatives Report identified a need for increased flood protection at Burnt Pond. Several possible alternatives were proposed, among them a fusible plug installed upstream of Burnt bridge, and increased upstream storage at Spruce Pond. The cost of providing upstream storage is considerably higher than providing a fuse plug, and thus the former option is viable only if it can provide additional energy savings to Newfoundland and Labrador Hydro (NLH). An initial review of NLH records indicates that savings in spill through White Bear Spillway could average approximately a million dollars a year.

In order to assess the viability of this proposal, improved mapping was required to show that closure of the contours occurred at the location of the proposed dykes. This was undertaken by Kenting Earth Science International Corporation using aerial photography. Analysis of the mapping provided by Kenting confirmed that the contours do in fact close.

2 Hydrology

Preliminary choices were made for the dyke locations at Spruce Pond North and South as shown in Figure 1. Two locations were selected at the narrowest sections across the outlet from Spruce Pond North and Spruce Pond South respectively. A third location was considered further downstream where one larger dyke would control the outflow from both Spruce Pond North and Spruce Pond South. This option was eliminated because of the excessive quantity of fill required.

2.1 Inflow Hydrographs

Inflow hydrographs were developed for both the January 1983 flood event and the PMF event. In the 1983 event, total inflows into Burnt Pond were obtained from NLH data sheets, and these inflows were then backrouted and distributed among the basins according to drainage area. Table I indicates the drainage areas for the ponds under consideration:

Table I

Basin	**Drainage Area (m2)**	**% Of Total Area**
Burnt Pond Local	210	30.4
Spruce North	335	48.4
Spruce South	147	21.2

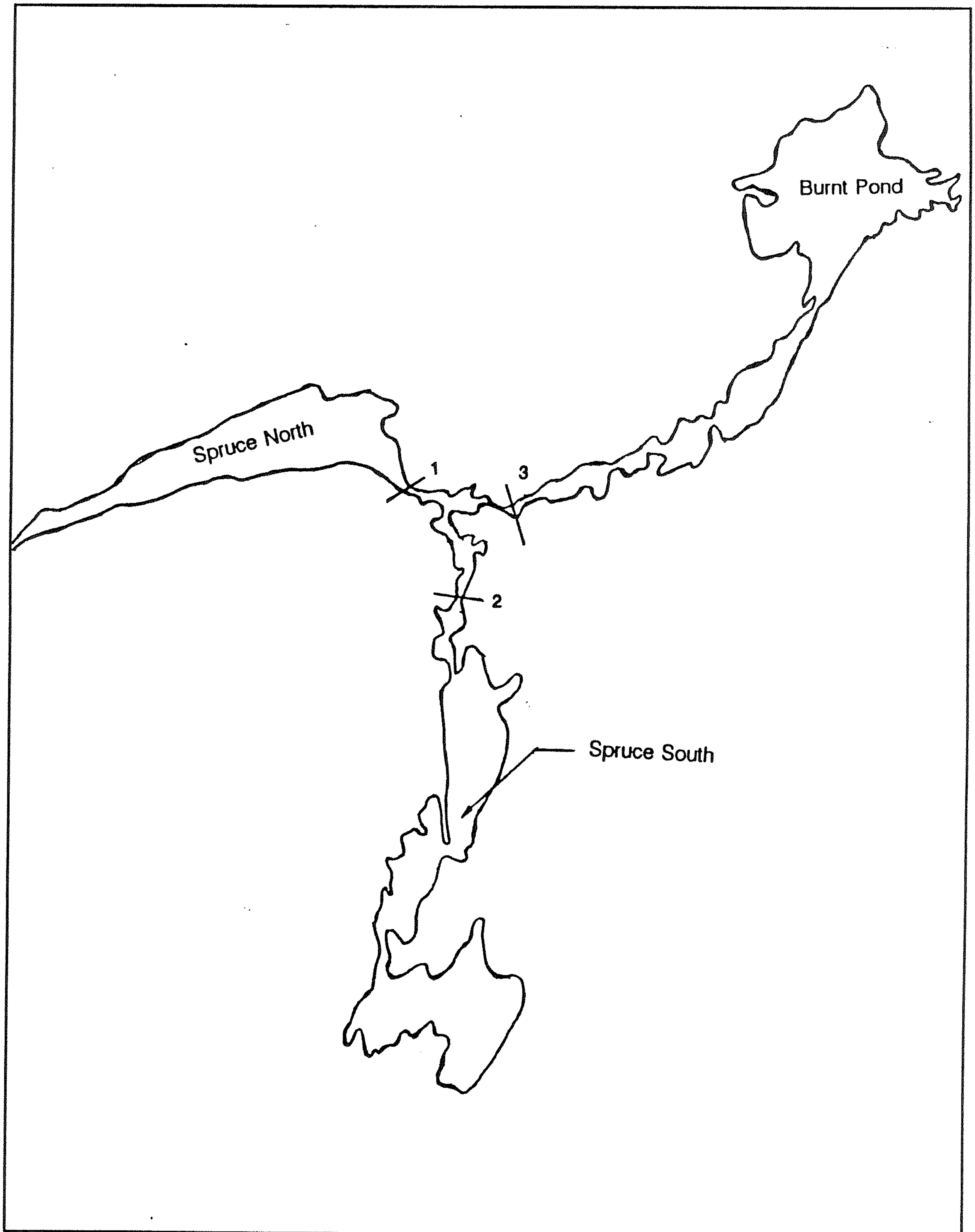


Figure 1 - Preliminary Dyke Locations

As can be seen from the figures above, the drainage area of Spruce North is considerably larger than that of Spruce South. The north pond accounts for approximately 70% of the inflow to Burnt Pond excluding local inflow.

Inflow hydrographs for the PMF were developed from the 1986 Flood Handling Alternatives Study. Appendix A contains the inflow hydrographs for both the January 1983 flood event and the PMF event.

2.2 Storage and Outflow Curves

Storage in each of the ponds at increasing elevations was determined from areas taken from the 1:50 000 scale mapping. With better mapping, these storage curves could be improved. Reservoir data are presented in Appendix B.

Outflow curves are dependent on the type of structure selected. Several structures were considered in the initial stages. A slotted overflow weir was eliminated from the analysis because of the difficulty in constructing a high narrow slot in a dyke. Two other types of structures were considered, namely low overflow dykes, and high rockfill dykes that would not be overtopped. The following combinations were assessed:

- one overflow dyke to elev. 327.5 m at the outlet of Spruce North

- two overflow dykes (at Spruce North to elev. 327.5 m and Spruce South to elev. 328.5 m),

- one rockfill dyke at Spruce North to elev. 332.5 m, and

- two rockfill dykes (at Spruce North to elev. 332.5 m and Spruce South to 330.0 m).

All of the above structures would be of relatively simple construction. The overflow dykes could be constructed of timber cribwork similar to the overflow spillways at Granite Lake. The rockfill structures would be made as impervious as possible, but some leakage could be accepted if it did not endanger the structure. Both types of dykes would have culverts to pass low and average flows to maintain ponds at normal water levels. The overflow structures would be designed to overflow at their crest elevations; however, the rockfill structures would handle floods entirely thorough storage. Outflow hydrographs for each alternative are contained in Appendix B.

2.3 Flood Routing

The Acres Reservoir Simulation Program (ARSP) was used to model both the January 1983 and PMF floods. The appropriate storage and outflow curves were input for each option and for each event. The elevations in each of the ponds and the spill at White Bear Spillway were then noted. Table 2 summarizes the results of the flood modeling.

22-Jun-90

TABLE II

SUMMARY OF POND ELEVATIONS AND SPILL AT WHITE BEAR SPILLWAY

	Max Elev at Sp North	Change in Elev at Sp North	Max Elev at Sp South	Change in Elev at Sp South	Max Elev at Burnt	Spill at WBS (Mm3)
January 1983 Event						
1) One Dyke Option (at Spruce North)	327.5	6	-	-	313.94	20
3) Two Dyke Option	327.5	6	328.5	2.5	313.87	0
PMP Event						
1) One Dyke Option (at Spruce North)	332.5	11	-	-	315.0	NC
2) Two Dyke Option	331.2	9.7	330.3	4.3	315.0	NC

NOTE:	Crest	Top of Core
BSHC	315.5 to 314.9	315.5 to 314.9
Burnt	316.40	315.47

Note: All elevations and dimensions in metres except where indicated

NC is not calculated

3 Energy Benefits and Costs

Cost estimates for each of these alternatives were prepared along with the associated energy benefits. A detailed breakdown of cost estimates for each option can be found in Appendix C.

For the rockfill alternatives, costs were taken from recent contracts and adjusted to account for different conditions. For the timber crib options, unit costs were obtained from the Department of Public Works, a consultant, and a contractor. These costs ranged from \$200 to \$300/m^3 and are somewhat conservative since they are principally for wharf construction. In dyke construction the timber cribs could be constructed in the dry and backfilled with a backhoe. A separate item for foundation preparation over the whole site was conservatively included, even though little preparation would be required under about two-thirds of the timber crib dyke.

An initial review of NLH's records indicate that average savings for NLH could amount to between half a million and one million dollars. Table 3 summarizes the benefits and costs of each dyke option.

As can be seen from Table 3, in both of the overtoppable options, the fuse plug is still required at Burnt Pond to handle the PMF event. The risk at Burnt Sidehill Canal is somewhat reduced but not eliminated. In both of the rockfill options, the requirement for a fuse plug is eliminated.

Energy benefits were calculated using the following assumptions:

a) average annual spill saved = 42.5 Mm^3

b) energy value of water = 0.5587 GWh/Mm^3

c) value of energy in fuel displacement = \$0.033 /kWh
(600 kWh/bbl; \$20/bbl)

d) in events less than or equal to Jan 1983: 100% spill saved with two dykes; 80-90% saved with one dyke.

It should be noted that energy benefits especially for the one dyke option must be confirmed by detailed evaluation of each of the actual recorded floods.

20-Jun-90

TABLE III
SUMMARY OF BENEFITS AND COSTS

A – ENERGY BENEFITS ONLY (Risk at BSHC reduced – fuse plug still required)			
Alternative	Structure	Benefits	Cost
1) One Dike Spruce North	overtoppable Top 327.5 m Height 6.0 m	save 80–90% of spill at WBS approx $625,000–$700,000/yr	$4.7 million
2) Two Dikes Spruce North Spruce South	overtoppable Top 327.5 m Height 6.0 m Top 328.5 m Height 4.5 m	save all spill in events ≤Jan 83 approx $780,000/yr	$11.3 million
B – ENERGY BENEFITS AND PMF PROTECTION AT BURNT (No fuse plug required)			
Alternative	Structure	Benefits	Cost
1) One Dike Spruce North	rockfill Top 332.5 m Height 11.0 m	save 80–90% of spill at WBS approx $625,000–$700,000/yr	$5.9 million
2) Two Dikes Spruce North Spruce South	rockfill Top 332.5 m Height 11.0 m Top 330.0 m Height 6.0 m	save all spill in events ≤Jan 83 approx $780,000/yr	$9.8 million

4 Environmental Concerns

The three major environmental concerns are expected to be construction of the access road, fish passage, and changed water levels. Construction of an access road from the Burgeo road would likely be the key concern. The area is known to be near the caribou migration path and a new road would provide better access for hunters and poachers. One possible solution would be to construct a winter road and bring all heavy equipment and timber onto the site ready for the next summer. Other supplies could be barged in during the summer.

Another environmental concern could be providing adequate fish passage. Locating the culverts in the natural stream bed would minimize disruption. If timber cribwork is used for the low dyke option it is possible that an opening could be left in the crib so that the natural stream bed is not altered.

The third concern is raised water levels. The ponds would be maintained at their normal levels most of the time, but in major flood events (perhaps every year or two) levels would rise as much as several metres gradually draining down over several weeks. A detailed analysis of historic floods is required to determine the pond trajectories during floods.

5 Conclusions and Recommendations

The most economic alternative is one timber crib overflow dyke at Spruce Pond North. Although it is not a complete flood handling solution and a fuse plug is still required, it should be considered on its merits of energy savings alone.

Appendix A
Inflow Hydrographs

JAN 83 INFLOW HYDROGRAPHS (m3/s)

DAY	MONTH	HOUR	BURNT POND	SPRUCE NORTH	SPRUCE SOUTH
11	JAN	0	0	0	0
11	JAN	6	0	0	0
11	JAN	12	50	0	0
11	JAN	18	103	0	0
12	JAN	0	23	19	9
12	JAN	6	29	24	11
12	JAN	12	34	29	13
12	JAN	18	40	33	15
13	JAN	0	56	89	39
13	JAN	6	72	115	51
13	JAN	12	130	206	90
13	JAN	18	133	213	93
14	JAN	0	275	436	191
14	JAN	6	223	355	156
14	JAN	12	172	274	120
14	JAN	18	137	219	96
15	JAN	0	159	253	110
15	JAN	6	132	211	93
15	JAN	12	118	187	82
15	JAN	18	103	164	72
16	JAN	0	104	166	73
16	JAN	6	94	150	66
16	JAN	12	88	140	61
16	JAN	18	0	0	0

PMP INFLOW HYDROGRAPHS (m3/s)

DAY	MONTH	HOUR	BURNT POND	SPRUCE NORTH	SPRUCE SOUTH
15	MAR	0	9	14	6
15	MAR	6	40	63	28
15	MAR	12	86	136	60
15	MAR	18	126	201	88
16	MAR	0	162	256	113
16	MAR	6	203	323	142
16	MAR	12	263	419	184
16	MAR	18	338	538	236
17	MAR	0	397	632	277
17	MAR	6	428	681	299
17	MAR	12	449	714	313
17	MAR	18	470	749	328
18	MAR	0	480	764	335
18	MAR	6	469	747	327
18	MAR	12	454	723	317
18	MAR	18	408	650	284
19	MAR	0	319	508	222
19	MAR	6	232	369	162
19	MAR	12	165	263	115
19	MAR	18	122	195	85
20	MAR	0	88	141	62
20	MAR	6	64	102	45
20	MAR	12	46	74	32
20	MAR	18	33	53	23

Appendix B
Storage and Outflow Curves

STORAGE CURVES

Burnt Pond

Elevation (m)	Area (m2)
310.00	15.60
311.00	16.12
312.00	16.63
313.00	17.13
314.00	17.63
315.00	18.64

Spruce Pond North

Elevation (m)	Area (m2)
320.70	0.30
321.50	8.60
324.75	11.20
328.00	13.80
343.00	25.00

Spruce Pond South

Elevation (m)	Area (m2)
324.50	0.30
326.00	6.00
326.50	6.10
328.00	18.20
335.00	25.00

OUTFLOW CURVES

One Dam Overflow Option

Spruce North Elevation	Flow (m3/s)
321.5	0.0
322.0	10.0
323.2	13.5
324.8	26.4
326.5	33.9
327.5	38.0
328.0	145.0
330.0	1570.0
332.5	5090.0
334.0	8260.0

Two Dam Overflow Option

Spruce North Elevation	Flow (m3/s)
321.5	0.0
322.0	10.0
323.2	13.5
324.8	26.4
326.5	33.9
327.5	38.0
328.0	145.0
330.0	1570.0
332.5	5090.0
334.0	8260.0

Spruce South Elevation	Flow (m3/s)
326.0	0.0
326.5	6.7
327.0	8.0
327.7	9.0
328.5	14.2
330.1	935.0
330.5	1420.0
331.0	2305.0
332.6	8610.0
334.3	14630.0

OUTFLOW CURVES

One Dam Rockfill Option

Spruce North

Elevation	Flow (m3/s)
321.5	0.0
322.0	10.0
322.5	12.0
323.2	13.5
324.8	26.4
326.5	33.9
329.8	43.7
330.5	45.0
332.5	50.0
334.0	970.0

Two Dam Rockfill Option

Spruce North

Elevation	Flow (m3/s)
321.5	0.0
322.0	10.0
322.5	12.0
323.2	13.5
324.8	26.4
326.5	33.9
329.8	43.7
330.5	45.0
332.5	50.0
334.0	970.0

Spruce South

Elevation	Flow (m3/s)
326.0	0.0
326.5	6.7
327.0	8.0
327.7	9.0
328.5	14.2
329.3	17.6
330.1	35.2
331.0	680.0
332.6	3480.0
334.3	7880.0

Appendix C
Cost Estimates

PRELIMINARY ESTIMATE BASED ON CONCEPTUAL DESIGN 20–Jun–90

CONTRACT P09431 – SPRUCE POND
Prepared by: Acres International Ltd.
Client: Nfld & Lab Hydro

ONE DAM OVERFLOW OPTION				
DESCRIPTION	UNIT OF MEASURE	ESTIMATED QUANTITY	UNIT PRICE	AMOUNT
MOBILIZATION	L.S.	1	$50,000	$50,000
CLEARING	ha	5	$2,000	$10,000
ACCESS	km	11	$50,000	$550,000
CAMP COSTS	L.S.	1	$160,000	$160,000
MANAGER'S FACILITIES	L.S.	1	$60,000	$60,000
FOUNDATION PREPARATION				
Unclassified Excavation	m3	5000	$10	$50,000
Rock Excavation	m3	500	$50	$25,000
Cofferdam	L.S.	1	$30,000	$30,000
Unwatering (Pumps)	L.S.	1	$10,000	$10,000
TIMBER CRIB	m3	10000	$275	$2,750,000
		TOTAL CAPITAL COST:		$3,695,000
		CONT (15%)		$554,250
				$4,249,250
		ENG FEES (10%)		$424,925
			TOTAL:	$4,674,175

$4.7 million

PRELIMINARY ESTIMATE BASED ON CONCEPTUAL DESIGN 20-Jun-90

CONTRACT P09431 - SPRUCE POND
Prepared by: Acres International Ltd.
Client: Nfld & Lab Hydro

TWO DAM OVERFLOW OPTION				
DESCRIPTION	UNIT OF MEASURE	ESTIMATED QUANTITY	UNIT PRICE	AMOUNT
MOBILIZATION	L.S.	1	$50,000	$50,000
CLEARING	ha	5	$2,000	$10,000
ACCESS	km	14	$50,000	$700,000
CAMP COSTS	L.S.	1	$160,000	$160,000
MANAGER'S FACILITIES	L.S.	1	$60,000	$60,000
FOUNDATION PREPARATION				
Unclassified Excavation	m3	10000	$10	$100,000
Rock Excavation	m3	1000	$50	$50,000
Cofferdam	L.S.	2	$30,000	$60,000
Unwatering (Pumps)	L.S.	2	$10,000	$20,000
TIMBER CRIB	m3	28000	$275	$7,700,000
		TOTAL CAPITAL COST:		$8,910,000
		CONT (15%)		$1,336,500
				$10,246,500
		ENG FEES (10%)		$1,024,650
			TOTAL:	$11,271,150

$11.3 million

PRELIMINARY ESTIMATE BASED ON CONCEPTUAL DESIGN 20-Jun-90

CONTRACT P09431 - SPRUCE POND
Prepared by: Acres International Ltd.
Client: Nfld & Lab Hydro

ONE DAM ROCKFILL OPTION					
DESCRIPTION	UNIT OF MEASURE	ESTIMATED QUANTITY	UNIT PRICE	AMOUNT	
MOBILIZATION	L.S.	1	$50,000	$50,000	
CLEARING	ha	5	$2,000	$10,000	
ACCESS	km	11	$50,000	$550,000	
CAMP COSTS	L.S.	1	$160,000	$160,000	
MANAGER'S FACILITIES	L.S.	1	$60,000	$60,000	
FOUNDATION PREPARATION					
Unclassified Excavation	m3	10000	$10	$100,000	
Rock Excavation	m3	500	$50	$25,000	
Cofferdam	L.S.	1	$50,000	$50,000	
Unwatering (Pumps)	L.S.	1	$10,000	$10,000	
FILL					
Rock Fill	m3	118000	$25	$2,950,000	
Filter	m3	8000	$60	$480,000	
CULVERTS	m	180	$1,000	$180,000	
		TOTAL CAPITAL COST:		$4,625,000	
		CONT (15%)		$693,750	
				$5,318,750	
		ENG FEES (10%)		$531,875	
			TOTAL:	$5,850,625	$5.9 million

PRELIMINARY ESTIMATE BASED ON CONCEPTUAL DESIGN 20-Jun-90

CONTRACT P09431 - SPRUCE POND
Prepared by: Acres International Ltd.
Client: Nfld & Lab Hydro

TWO DAM ROCKFILL OPTION				
DESCRIPTION	UNIT OF MEASURE	ESTIMATED QUANTITY	UNIT PRICE	AMOUNT
MOBILIZATION	L.S.	1	$50,000	$50,000
CLEARING	ha	5	$2,000	$10,000
ACCESS	km	14	$50,000	$700,000
CAMP COSTS	L.S.	1	$160,000	$160,000
MANAGER'S FACILITIES	L.S.	1	$60,000	$60,000
FOUNDATION PREPARATION				
Unclassified Excavation	m3	20000	$10	$200,000
Rock Excavation	m3	1000	$50	$50,000
Cofferdam	L.S.	2	$50,000	$100,000
Unwatering (Pumps)	L.S.	2	$10,000	$20,000
FILL				
Rock Fill	m3	207000	$25	$5,175,000
Filter	m3	16000	$60	$960,000
CULVERTS	m	260	$1,000	$260,000
		TOTAL CAPITAL COST:		$7,745,000
		CONT (15%)		$1,161,750
				$8,906,750
		ENG FEES (10%)		$890,675
			TOTAL:	$9,797,425

$9.8 million

NEWFOUNDLAND AND LABRADOR HYDRO

KITTY'S BROOK DEVELOPMENT PRE-FEASIBILITY STUDY

OCTOBER, 1986

Prepared by:

SHAWMONT NEWFOUNDLAND LIMITED

SMR-21-86 October, 1986

ShawMont Newfoundland Limited

BALLY ROU PLACE
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Newfoundland & Labrador Hydro
P. O. Box 9100
St. John's, Newfoundland
A1A 2X8

Attention: Mr. L. G. Sturge
Manager of Engineering

Gentlemen:

We take pleasure in submitting our final report entitled "Kitty's Brook Development - Pre-Feasibility Study".

In meeting the objectives of the study, to optimize the development, we have prepared a report which indicates that the Kitty's Brook Development is technically feasible. The optimization was completed using an energy value of $0.80/kWh which is the estimated present worth of a single kilowatt-hour of energy from an alternate source, over a 60 year plant life.

The benefit of additional energy generation at the Deer Lake Generating Plant due to improved regulation of the run-off from the Kitty's Brook, Chain Lakes and Sheffield Brook drainage areas, and the addition of Burnt Berry Brook and Barneys Brook drainage areas, has not been addressed in this study.

We recommend that the additional studies outlined herein, be carried out to further evaluate this development.

We wish to thank the Hydro staff who assisted us and contributed to the preparation of this Report and to express our appreciation for the opportunity of carrying out this most interesting study.

Your very truly,

ADPeach

A. D. Peach, P. Eng.
Vice President &
General Manager

DHB:jvw

SUMMARY

The diversion of Kitty's Brook is considered to be technically feasible. The results of this study show that the diversion of additional drainage areas upstream of Kitty's Brook are not only technically feasible but the more drainage areas that are added, the more economical the development becomes.

Based on limited field information and a review of all possible alternatives for each diversion, a layout for diversion of five drainage areas has been developed. The development comprises a series of diversion dams and canals to divert the headwaters of Kitty's Brook, Chain Lakes, Sheffield Brook, Burnt Berry Brook and Barneys Brook into the existing Hinds Lake Development via Goose Pond.

The structure sizes in each diversion were established by optimizing structure costs against the value of energy from an alternative source. An energy value of $0.80 per kWh, representing the present worth (mid 1986) of a single kilowatt-hour of energy over an estimated 60 year plant life, was used in the optimizations. This value of energy is based on the average of the estimated present worths of energy from a Labrador infeed and from a thermal plant.

Provision of storage facilities on Chain Lakes (the only suitable storage site in the entire development) resulted in marginally more economic energy than a layout with no storage. The greatest benefit of this storage is the reduction in size of the water conveyance structures for Chain Lakes and Kitty's Brook. Reductions in design flow capacity of 55% and 29% for Chain Lakes and Kitty's Brook, respectively, result from provision of storage in Chain Lakes.

The Kitty's Brook Development would deliver an average flow of 12.4 m^3/s to Hinds Lake, resulting in an average annual energy production of 200 gWh. The annual firm energy is estimated to be approximately 70% of the total annual energy.

The estimated capital cost of the optimized development is $135,350,000 including escalation and interest during construction. The costs are in mid 1986 dollars and for this study, the escalation was calculated on an assumed construction period between July, 1988 and December, 1990. The cost of energy is 86.8 mils per kWh, based on an annual cost of $16,442,000 which was calculated using an annual operating charge rate of 12.133%.

SUMMARY (Cont'd)

The following summarizes the costs and energy benefits for the diversion of Kitty's Brook alone and shows the resultant costs and benefits when additional drainage areas are added (including storage in Chain Lakes).

SCHEME	TOTAL CAPITAL COST ($ X 1000)	AVERAGE ANNUAL ENERGY (gWh)	ANNUAL FIRM ENERGY (gWh)	ENERGY COST (mils/ kWh)
KB	78,960	90	63	106.4
KB+CL	100,746	129	90	94.8
KB+CL+US	119,000	169	118	85.4
KB+CL+US+BB	124,850	179	125	84.6
KB+CL+US+BB+B	135,350	200	140	82.1

The proposed project schedule shows completion of a field investigation program and a feasibility report in Year 1. A total of 36 months would be required from project commitment in January of Year 2, when engineering would commence, to the scheduled date of completion in December of Year 4. Construction would be over three construction seasons starting in Year 2.

RECOMMENDATIONS FOR FURTHER STUDY

Prior to undertaking any more field work, further studies will be required to establish the project parameters in more detail. These will include:

1. A review of a bored tunnel alternative to the sidehill canal for the Kitty's Brook diversion. Last minute information received from a tunnel boring machine supplier indicated that the cost of this alternative could be attractive and could significantly reduce the project capital cost.

2. Optimization of storage in Chain Lakes. The present study allows costs for the maximum required storage to fully regulate the inflows to Chain Lakes since time did not permit optimizing the storage.

3. A review of the possible addition of a small amount of drainage area at the eastern end of Chain Lakes. This would be accomplished by relocating the cut-off dam, if the optimization of Chain Lakes storage indicates a cut-off dam is required.

RECOMMENDATIONS FOR FURTHER STUDY (Cont'd)

4. Optimization of the location of the cut-off dykes in the Burnt Berry Brook diversion. Relocation could reduce structure costs but would reduce available drainage area.

5. A review of the diversion dam location in the Barneys Brook Diversion. Relocation of the dam and canal farther downstream would increase structure costs but would increase the available drainage area.

6. Optimization of the overflow spillway crest lengths. The crest lengths of each spillway must be optimized against the applicable flood surcharge. Increasing the crest length would increase spillway costs but reduce dam height and/or canal depth.

In addition to the above studies required prior to undertaking any more field work, the following additional work will be required prior to committing the project.

1. A survey program to obtain topography of all construction sites, construction camps and access roads.

2. A geotechnical program to provide foundation information for all structures and to locate construction materials.

Following the field work, time will be required to update this report to feasibility report status to define the scope of work in sufficient detail for an accurate cost estimate. Based on the results of the field work and the studies noted above, further studies will be required as follows:

1. Optimization of the hydraulic cross-section for each canal. For this study the canal widths were set at 5m and the canal design flow was varied by increasing the depth.

2. A review of the geotechnical aspects of the dams to address the availability of construction materials available at each site and of the canals to address the variation in water levels during canal operation.

RECOMMENDATIONS FOR FURTHER STUDY (Cont'd)

3. A freeboard study for all earth/rockfill dams. For this study a freeboard of 2.5 - 3.0 m was assumed depending on dam location and fetch.

4. A review of the conceptual layouts of all structures.

5. A regulation study to confirm the design flows and spill at each diversion and to confirm that Hinds Lake Reservoir will fully regulate the new inflows.

6. A review of the construction schedule and costs to determine the economics of winter construction to provide for earlier project completion and reduced IDC costs. For this study, it was assumed that no construction would be carried out in the winter months.

KITTY'S BROOK DEVELOPMENT

PRE-FEASIBILITY STUDY

OCTOBER, 1986

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PLATES

PART ONE

INTRODUCTION

1.1 AUTHORIZATION

This study has been undertaken in accordance with the Scope of Work agreed to in a meeting held on April 10, 1986 and as subsequently outlined in ShawMont Newfoundland Limited's proposal dated April 15, 1986 and subsequent revisions on April 16, 1986 and April 23, 1986. The study was authorized with a purchase order dated May 13, 1986.

1.2 OBJECTIVES OF STUDY

The objectives of the pre-feasibility study were to:

1. Determine the technical feasibility of the diversion into the existing Hinds Lake Development of:

   -- firstly, Kitty's Brook and Chain Lakes, and
   -- secondly, three additional upstream diversions, namely: Upper Sheffield Brook, Burnt Berry Brook and Barneys Brook.

2. To provide a preliminary estimate of project cost for the Kitty's Brook/Chain Lakes diversion and incremental costs for the addition of each upstream diversion.

3. Recommend additional engineering and field studies required to bring the report to the full feasibility level.

1.3 PREVIOUS ENGINEERING WORK

During the design phase of the Hinds Lake Development, ShawMont identified five drainage areas located to the east of the Goose Pond Diversion, which had the potential for diversion into the Hinds Lake Development. These drainage areas are known as: Kitty's Brook, Chain Lakes, Upper Sheffield Brook, Burnt Berry Brook and Barneys Brook. A preliminary assessment of the economic feasibility of these diversions was undertaken at that time and a short letter report was submitted to Newfoundland and Labrador Hydro on January 16, 1978.

During the period 1984-1985, Newfoundland and Labrador Hydro conducted further investigation into the potential of these diversions through an in-house office study in conjunction with a limited amount of field survey and "walk-over" inspection work.

1.3 PREVIOUS ENGINEERING WORK (Cont'd)

The current assignment was initiated to verify Newfoundland & Labrador Hydro's findings using information and data developed during their in-house studies and a scope of work for this study was developed as described in the following section.

1.4 SCOPE OF WORK

The scope of work for this study included the following:

a) Review of all existing data and work previously completed by Newfoundland and Labrador Hydro and the identification of information gaps;

b) Investigation and review of hydrology for the diversions to assess water resources including obtaining run-off and flow duration characteristics;

c) Investigation of various conceptual layouts to maximize water diversion from Kitty's Brook and Chain Lake Watersheds into Goose Pond and the Hinds Lake System;

d) Provision of preliminary design for structures required, preparation of cost estimates and cost/benefit analysis for the development;

e) Undertake a site visit to confirm the viability of the schemes under consideration. The site visit to entail a "walk-over" visual investigation to determine suitability of structure design philosophy and also to confirm as far as practical, geotechical and other physical aspects of the sites, (No survey work or sub-surface investigation were allowed for in the study);

f) Based on inspection, selection of preferred layout for development, modification of preliminary designs and re-examination of estimates if required;

g) Based on preferred scheme for Kitty's Brook and Chain Lakes diversions, examine and determine effect and incremental costs for adding diversions from upstream watersheds, namely: Upper Sheffield Brook, Burnt Berry Brook and Barneys Brook;

1.4 SCOPE OF WORK (Cont'd)

h) Compilation of pre-feasibility report which will detail the development options with cost/benefit information and provide recommendations for further work and field investigation programs as necessary.

1.5 STUDY PLAN

The study program was divided into two parts. ShawMont were required to investigate the regional hydrology covering all five diversions under consideration as well as investigating the Kitty's Brook and Chain Lakes diversions. Newfoundland and Labrador Hydro were responsible for investigating the three upstream diversions using hydrological parameters furnished from ShawMont's study. The results of both investigations were then compiled by ShawMont and included in this report.

For the Kitty's Brook and Chain Lakes diversions, ShawMont initially compared a number of alternative conceptual layouts for each diversion which included preliminary optimizations to determine the most cost effective structure arrangement for each. The most cost effective layout was then developed in more detail for various design flow options and cost curves were prepared for each structure and each diversion. The cost of each diversion, including associated costs to increase the flow capacity of all downstream diversions, was then optimized against the benefits derived from each design flow option.

Concurrent with ShawMont's work on the two downstream diversions, Hydro carried out a similar study of the three upstream diversions. The resulting optimized layouts were then added successively beginning at Kitty's Brook and working towards Barneys Brook to determine the optimum development layout.

Cost estimates were then prepared from the cost curves for each structure of the optimized layout.

PART TWO

DESCRIPTION OF SITE

2.1 GENERAL DESCRIPTION

The proposed Kitty's Brook Development is located to the east of the existing Hinds Lake Development and to the south of Sandy Lake and Birchy Lake of the Grand Lake system. The proposed development will divert the headwaters of several rivers, which presently flow in a generally northern direction to Sandy Lake, Birchy Lake and Halls Bay, into the Hinds Lake Development via Goose Pond. The following summarizes the diversions and the present outlets of their respective rivers:

Diversion	River	Present Outlet
Kitty's Brook	Kitty's Brook	Sandy Lake
Chain Lakes	Chain Lakes Brook	Sandy Lake (via Kitty's Brook)
Upper Sheffield	Upper Sheffield Brook	Birchy Lake (via Sheffield Brook)
Burnt Berry	Burnt Berry Brook	Halls Bay
Barneys Brook	Barneys Brook	Halls Bay (via West Brook)

Three of the watersheds under consideration presently drain via Kitty's Brook, Chain Lakes and Upper Sheffield Brook into the Grand Lake system and their flows ultimately pass through the hydroelectric station at Deer Lake. However, Burnt Berry Brook and Barney's Brook flow to the sea at Halls Bay.

The proposed development will divert the flow of the headwaters of these rivers through the Hinds Lake Development (via the Goose Pond Diversion) and will utilize the 217 m of head between Hinds Lake and Grand Lake. Additional benefits (not considered in this study) would be derived at Deer Lake by the diversion of flows from Burnt Berry Brook and Barneys Brook, into Grand Lake and through the Deer Lake Generating Station.

The Hinds Lake plant was designed for peaking support and has a relatively low capacity factor of 50% (average). The Hinds Lake reservoir is large enough to absorb all of the diverted flow and additional flow from the proposed development would increase the energy production at this plant.

The Kitty's Brook Development would include construction of diversion dams on each of the rivers and diversion canals through or around the divides between the watersheds. The major diversion structure and the key to the entire development would be the Kitty's Brook sidehill canal/pipeline. This structure would have a total length of

2.1 GENERAL DESCRIPTION (Cont'd)

18,400 m around the base of the escarpment of the Buchans Plateau, between Kitty's Brook and Goose Pond. A general layout of the development is shown on Plate 1.

2.2 SERVICE AND ACCESS

The Kitty's Brook canal is accessible from the community of Howley, at the end of route 401 from the Trans Canada Highway, via an existing construction road between Howley and the Goose Pond Dam of the Hinds Lake Development. This road will require minor upgrading. Construction access along the canal would be required beyond the end of this road and the permanent access to the canal would be provided as part of the canal construction by utilizing the top of the embankment as the road surface.

Access to the Chain Lakes structures (at the west end of Chain Lakes) and the Kitty's Brook dam would require construction of approximately 10 km of new access road from the end of an existing woods road off the Trans Canada Highway at Birchy Narrows (narrows between Birchy Lake and Sandy Lake). The existing woods road would probably require minor upgrading.

Access to the Upper Sheffield structures would require construction of approximately 3 km of new access road from the above existing woods road to the downstream end of the canal and another 8 km to the dam site. From this point another 6 km of access road would be required to access the Burnt Berry structures. Another 9 km of access road would be required to access the Barneys Brook structures.

2.3 TOPOGRAPHY AND GEOLOGY

The Kitty's Brook Development is located in the west-central portion of the Island near the summit of the Buchan's Plateau where the ground elevation is generally about 450 m above sea level.

The topography generally comprises gently undulating terrain with rounded hills and broad valleys. Drainage is poor and numerous bogs are present. A notable feature of the terrain is the northeast/southwest trend of the rivers and ponds in the area which lie in a series of geologically controlled troughs. The whole area was glaciated with the pre-glacial surface being scoured and subsequently covered with a discontinuous layer of till of varying thickness. Kitty's Brook diverges from the northeast/southwest trend and flows, in a northwest direction, down through the escarpment at the northwest side of the Buchan's Plateau.

2.3 TOPOGRAPHY AND GEOLOGY (Cont'd)

The terrain in the area of the Kitty's Brook dam is characterized by a heavily treed, steep sided valley, the west side of which is a colluvial complex slope. The steep sided valley gives way to a broader valley with flatter side slopes farther downstream as the river swings northwest toward Sandy Lake. The canal route is heavily treed except for the last 2 - 3 kms near Goose Pond where the route is essentially barren. Along the canal route, overburden depth* is expected to vary from deep (5 - 10 m), where hummocky moraines predominate, to very shallow where bedrock outcrops occur. Bedrock outcrops were identified intermittently in the Kitty's Brook valley and along the canal route, except for the last 2 - 3 kms near Goose Pond where bedrock outcrops predominate.

The terrain in the Chain Lakes area is gently undulating and is characterized by sparse tree cover and deep overburden comprising hummocky moraines of variable textured materials. Large angular boulders are common in this area.

The terrain in the area of the upstream diversions (Upper Sheffield, Burnt Berry and Barneys Brook) is generally barren with little tree cover. Along the Upper Sheffield canal the overburden is coarse textured glacial till with the depth varying from deep (>5 m) near Chain Lakes to very shallow (<1 m) at the dam site. At Burnt Berry and Barneys Brook areas, the glacial till overburden is very shallow and bedrock outcrops are common.

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* Depths of overburden are based on API work carried out by Newfoundland and Labrador Hydro.

PART THREE

HYDROLOGY

3.1 INTRODUCTION

Kitty's Brook Development involves diversion of a series of watersheds via Goose Pond into the Hinds Lake Development (see Plate 1). The ultimate development includes diversions from five basins, with flow routing as below:

B ->BB ->US ->CL ->KB ->Hinds Lake (via Goose Pond)

where B	=	Barneys Brook
BB	=	Burnt Berry Brook
US	=	Upper Sheffield Brook
CL	=	Chain Lakes
KB	=	Kitty's Brook

The total diverted area would be 422.1 km^2 with a mean runoff of 12.41 m^3/s. This represents a potential energy benefit at Hinds Lake of 200 gWh/year or an increase of 63% in the present energy output of the Hinds Lake Generating Station.

In the original design of the Hinds Lake Development a relatively large live storage volume of 252 Mm^3 (storage ratio = 0.39)* was provided in order to obtain a high degree of regulation and maintain a firm flow to the plant of approximately 85% of the average basin flow. As shown in Section 3.3, this large storage volume is sufficient to fully regulate the delivered flows from Kitty's Brook Development.

The main benefits of the Kitty's Brook Diversion scheme are seen to be the increase in firm and secondary energy supply to the Island's hydro system.

Accordingly, the approach that was taken in assessing the hydrology of the Kitty's Brook Development assumes a run-of-river mode of operation for each diversion with regulation of the delivered flows provided by the Hinds Lake Reservoir.

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* $1 Mm^3 = 1 \times 10^6 m^3$

$$\text{Storage Ratio} = \frac{\text{Storage Volume}}{\text{Mean Annual Run-off Volume}}$$

3.1 INTRODUCTION (Cont'd)

The objectives of the hydrology studies for Kitty's Brook Development were:

-- estimation of average basin flows,

-- determination of water use/capacity relationships (water use curves) for each basin,

-- review of the capability of Hinds Lake Reservoir to re-regulate additional inflows and estimation of water use factors to account for spillage at Hinds Lake,

-- selection of design flood criteria,

-- determination of design flood peaks and spillway design flows,

-- determination of energy benefits.

3.2 AVERAGE BASIN FLOWS

The diverted areas lie midway between the Hinds Brook and Sheffield Brook watersheds and hence can be expected to have runoff values intermediate between the values measured at the gauges on these rivers. For purposes of estimating average basin flows a single runoff rate has been applied. This value was taken as the mean of runoff rates from the above basins, as determined from long term stream flow records [Hinds Brook = 30 years and Sheffield Brook = 19 years]. The resulting value of 927 mm [29.4 l/s] was then applied to each diverted area to determine basin flows as given in Table 3.1 (at the end of this section).

3.3 WATER USE CURVES

Unitized water use curves were developed for each basin using Acres Pre-feasibility Method (1)*. These curves are given in Figures 3.1 to 3.6 (at the end of this section). These curves give usable (turbinable) flow as a function of canal design flow capacity, average annual flow and storage. The nomenclature used is defined as follows:

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* Corresponds to listing in Reference Section of the report.

3.3 WATER USE CURVES (Cont'd)

$$q_t = \frac{Q \text{ usable}}{Q \text{ average}}$$

$$q_d = \frac{Q \text{ design}}{Q \text{ average}}$$

$$S = \frac{\text{Storage Volume}}{\text{Mean Annual Flow Volume}}$$

where: q_t = unitized usable (or turbinable) flow

q_d = unitized design flow (plant flow or canal capacity)

Q usable = usable (turbinable) flow

Q design = design flow (plant flow or canal capacity)

Q average = average flow

S = Storage Ratio

Q usable, Q average and Q design are in m^3/s
Storage Volume and Mean Annual Flow Vol. are in Mm^3

In addition to the Hinds Lake reservoir, only Chain Lakes offered potential for development of storage for flow regulation. The water use curves for Hinds Lake and Chain Lakes account for the availability of storage by considering the storage ratio. In computing the benefits of storage, a run-of-river mode of operation is assumed, i.e. surplus water is held in the reservoir on a temporary basis and withdrawn as quickly as possible after peak flows recede.

The Hinds Lake water use curve, Figure 3.6, was used to establish the maximum usable flow for the system, as each basin was added. Any excess in delivered flow, as each basin was added, over the maximum useable flow at Hinds Lake would require spillage at Hinds Lake and subsequent adjustment of the useable flow at each basin. However, as shown in Table 3.1, the delivered flow in each case was not in excess and, therefore, all of the delivered flow was useable.

A similar assessment was carried out to evaluate the benefit to the overall system of adding storage in Chain Lakes. In this assessment total storage was "lumped" in

3.3 WATER USE CURVES (Cont'd)

Hinds Lake and the Hinds Lake water use curve used to evaluate overall water use. Since upstream storage is less effective in regulating the system than storage at the system outlet (i.e. Hinds Lake) it was necessary to reduce Chain Lakes storage to an equivalent/effective volume at Hinds Lake. This adjustment was made by extrapolation of relationships developed for another study and is somewhat approximate. The results are shown in Table 3.2.

These analyses showed that the Hinds Lake Reservoir was able to regulate additional inflows and that loss of delivered water due to spillage at Hinds Lake was negligable.

3.4 STORAGE IN CHAIN LAKES

An assessment was carried out to evaluate the storage requirement in Chain Lakes to fully regulate inflows from this basin and each additional upstream basin. In addition, the effects of storage on the downstream structures was determined. The results of this assessment are given in Table 3.3.

A review of the perimeter topography of Chain Lakes and a storage volume curve for the reservoir indicated that the maximum practical storage volume would be about 57 Mm^3. A review of the Chain Lakes water use curve indicated that the maximum storage ratio would be about 0.30. With these parameters set as maximum values, the volume of storage required to regulate the inflows into, and the resultant design flow out of, Chain Lakes was determined.

for example

for CL + US + BB + B, $Qav = 6.94\ m^3/s$

$$\begin{aligned}\text{Storage Volume} &= S \times \text{Mean Annual Flow Volume}\\ &= 0.30 \times 6.94 \times 365 \times 24 \times 3600\\ &= 65.6\ Mm^3 > 57\ Mm^3\end{aligned}$$

$\therefore$ Use Storage Volume = 57 Mm^3

$$\text{from this, } S = \frac{57 \times 10^6}{6.94 \times 365 \times 24 \times 3600} = 0.26$$

from Chain Lakes water use curve for $q_t = 1.00$, $q_d = 1.65$.

$\therefore$ Q design = $1.65 \times 6.94 = 11.5\ m^3/s$.

3.5 SELECTION OF DESIGN FLOOD CRITERIA

Reservoirs and structures at each diversion point were classified following Table 3.4, from the I.C.E. Flood Guidelines (2).

Diversion dams at Barneys, Burnt Berry, Upper Sheffield, Chain Lakes and Kitty's Brooks were all classified into Category "C", for which the following criteria are recommended:

-- minimum standard, suitable for timber crib/concrete construction - use larger of Q150/0.2 PMF.

-- general standard, for embankment dams - use larger of Q1000/0.3 PMF.

For provision of storage on Chain Lakes, which would contain a relatively large volume of water, the storage dams would be classified into Category "B", for which the design standard is:

-- general standard, for embankment dams - larger of Q10000/0.5 PMF.

3.6 DESIGN FLOODS

Design flood peaks were determined using either the RFFA formulae(3) or based on SNL's regional PMF analysis. The parameters of the design floods are summarized in Table 3.5.

The results of the flood studies are summarized in Table 3.5.

3.7 ENERGY BENEFITS

Total energy benefits for each basin were determined by applying a factor of 1958.4 kW per m^3/s to the increment in usable flow obtained from each diversion. This factor was based on operating experience at Hinds Lake and may be calculated using:

$$kw = 9.81 \times H \times e \times Q$$

where:
H = average head at Hinds Lake Plant
e = overall plant efficiency
Q = flow

from which, for 1 m^3/s

factor = $9.81 \times 217 \times 0.92 \times 1 = 1958.4$ kw.

3.7 ENERGY BENEFITS (Cont'd)

An estimate of the annual firm flow available from each scheme of development, as additional drainage areas were added, was based on the Hinds Brook flow records. The annual firm flow, defined as the minimum total flow available for the 12 driest consecutive months on record, was found to occur between June, 1960 and May, 1961 and represented approximately 70% of the long term average flow for this river. For this study therefore, since energy is directly proportional to flow, the firm energy for each scheme was estimated as 70% of the total energy available.

The energy benefits for each scheme of development are summarized in Tables 3.1 and 3.2.

TABLE 3.1

SUMMARY OF WATER SUPPLY DETERMINATIONS

(NO STORAGE IN CHAIN LAKES)

SCHEME	D.A.	D.A.	Qav	Qav	Water Supply Factors at Hinds Lake				Usable Flow by Basin			Energy Benefit per Annum (4)	
								Max. Qus at Plant (1)	Qus (2)	Qus	Water Use Factor (3)	Total	Firm
	km^2	km^2	(m^3/s)	(m^3/s)	S	q_d	q_t	(m^3/s)	(m^3/s)	(m^3/s)		(gWh)	(gWh)
Hinds Lake	650.9	650.9	20.11 (5)	20.11	0.394	1.97	1.000	20.11	20.11	20.11	1.00	345	294
KB	186.2	837.1	5.47	25.58	0.310	1.55	1.000	25.58	5.23	25.34	1.00	90	63
KB+CL	87.7	924.8	2.58	28.16	0.282	1.41	1.000	28.16	2.30	27.64	1.00	129	90
KB+CL+US	80.9	1005.7	2.38	30.54	0.260	1.30	0.995	30.39	2.33	29.97	1.00	169	118
KB+CL+US+BB	20.5	1026.2	0.60	31.14	0.255	1.28	0.991	30.86	0.56	30.53	1.00	179	125
KB+CL+US+BB+B	46.8	1073.0	1.38	32.52	0.245	1.22	0.978	31.80	1.21	31.74	1.00	200	140
Sub-Total Diversions	422.1		12.41										
Total HL + Diversions	1073.0	1073.0	32.52	32.52									

Legend

D. A. = Drainage Area

Qav = Average Flow

S = Storage Ratio = $\frac{\text{Storage Volume}}{\text{Mean Annual Run-off Volume}}$

$q_d = \frac{\text{Q design}}{\text{Q average}}$

$q_t = \frac{\text{Q usable}}{\text{Q average}}$ (based on qd in figure 3.6)

Qus = Usable (turbinable) Flow

Notes

(1) Maximum usable flow at Hinds Lake Generating Station
(2) Usable flow delivered from each basin based on optimization
(3) When the total usable flow delivered to plant (Qus) $\leq$ maximum usable flow at plant, then Water Use Factor = 1.00
(4) Energy benefits shown are for each scheme of development, as each new drainage area is added.
(5) Hinds Lake average flow is based on long term average flow less fish compensation flows.

TABLE 3.2

SUMMARY OF WATER SUPPLY DETERMINATIONS

(WITH STORAGE IN CHAIN LAKES)

SCHEME	D.A.	D.A.	Qav	Qav	Water Supply Factors at Hinds Lake				Usable Flow by Basin			Energy Benefit (4) per Annum	
	km^2	km^2	(m^3/s)	(m^3/s)	S	q_d	q_t	Max. Qus at Plant (1) (m^3/s)	Qus (2) (m^3/s)	Qus (m^3/s)	Water Use Factor (3)	Total (gWh)	Firm (gWh)
Hinds Lake	650.9	650.9	20.11 (5)	20.11	0.394	1.97	1.000	20.11	20.11	20.11	1.00	345	294
KB	186.2	837.1	5.47	25.58	0.310	1.55	1.000	25.58	5.23	25.34	1.00	90	63
KB+CL	87.7	924.8	2.58	28.16	0.284	1.41	1.000	28.16	2.30	27.64	1.00	129	90
KB+CL+US	80.9	1005.7	2.38	30.54	0.270	1.30	1.000	30.39	2.33	29.97	1.00	169	118
KB+CL+US+BB	20.5	1026.2	0.60	31.14	0.267	1.28	0.996	30.86	0.56	30.53	1.00	179	125
KB+CL+US+BB+B	46.8	1073.0	1.38	32.52	0.265	1.22	0.986	32.80	1.21	31.74	1.00	200	140
Sub-Total Diversions	422.1		12.41										
Total HL + Diversions	1073.0	1073.0	32.52	32.52									

Legend

D. A. = Drainage Area

Qav = Average Flow

S = Storage Ratio = $\frac{\text{Storage Volume}}{\text{Mean Annual Run-off Volume}}$

q_d = $\frac{\text{Q design}}{\text{Q average}}$

q_t = $\frac{\text{Q usable}}{\text{Q average}}$ (based on qd in figure 3.6)

Qus = Usable (turbinable) Flow

Notes

(1) Maximum usable flow at Hinds Lake Generating Station
(2) Usable flow delivered from each basin based on optimization
(3) When the total usable flow delivered to plant (Qus) $\leq$ maximum usable flow at plant, then Water Use Factor = 1.00
(4) Energy benefits shown are for each scheme of development, as each new area is added.
(5) Hinds Lake average flow is based on long term average flow less fish compensation flows.

TABLE 3.3

STORAGE IN CHAIN LAKES

SCHEME	Q_{av}	STORAGE VOLUME (1)	$S^{(2)}$	q_t	q_d (3)	Qd	FULL SUPPLY LEVEL (4)
CL	2.58	24.4	0.30	1.00	1.4	3.61	341.7
CL + US	4.96	46.9	0.30	1.00	1.4	6.94	345.5
CL + US + BB	5.56	52.6	0.30	1.00	1.4	7.78	346.2
CL + US + BB + B	6.94	57.0	0.26	1.00	1.65	11.50	347.0

(1) Maximum storage volume = 57 Mm^3

(2) Maximum S = 0.30

(3) q_d from water use curve for q_t = 1.00

(4) from CL storage volume curve.

TABLE 3.4

RESERVOIR FLOOD AND WAVE STANDARDS BY DAM CATEGORY

Category	Initial reservoir condition	Dam design flood inflow			Concurrent wind speed and minimum wave surcharge allowance
		General standard	Minimum standard if rare overtopping is tolerable	Alternative standard if economic study is warranted	
A. Reservoirs where a breach will endanger lives in a community	Spilling long term average daily inflow	Probable Maximum Flood (PMF)	0.5 PMF or 10 000 year flood (take larger)	Not applicable	Winter: maximum hourly wind once in 10 years (Fig. 4) Summer: average annual maximum hourly wind (Fig. 3) Wave surcharge allowance not less than 0.6 m
B. Reservoirs where a breach (i) may endanger lives not in a community (ii) will result in extensive damage	Just full (i.e., no spill)	0.5 PMF or 10 000 year flood (take larger)	0.3 PMF or 1000 year flood (take larger)	Flood with probability that minimizes spillway plus damage costs (Fig. 1); inflow not to be less than minimum standard but may exceed general standard	
C. Reservoirs where a breach will pose negligible risk to life and cause limited damage	Just full (i.e., no spill)	0.3 PMF or 1000 year flood (take larger)	0.2 PMF or 150 year flood (take larger)		Average annual maximum hourly wind (Fig. 3) Wave surcharge allowance not less than 0.4 m
D. Special cases where no loss of life can be foreseen as a result of a breach and very limited additional flood damage will be caused	Spilling long term average daily inflow	0.2 PMF or 150 year flood	Not applicable	Not applicable	Average annual maximum hourly wind (Fig. 3) Wave surcharge allowance not less than 0.3 m

Notes
Where reservoir control procedure requires, and discharge capacities permit, operation at or below specified levels defined throughout the year, these may be adopted providing they are specified in the certificates or reports for the dam.
Where a proportion of PMF is specified it is intended that the PMF hydrograph should be computed and then all ordinates be multiplied by 0.5, 0.3 or 0.2 as indicated.

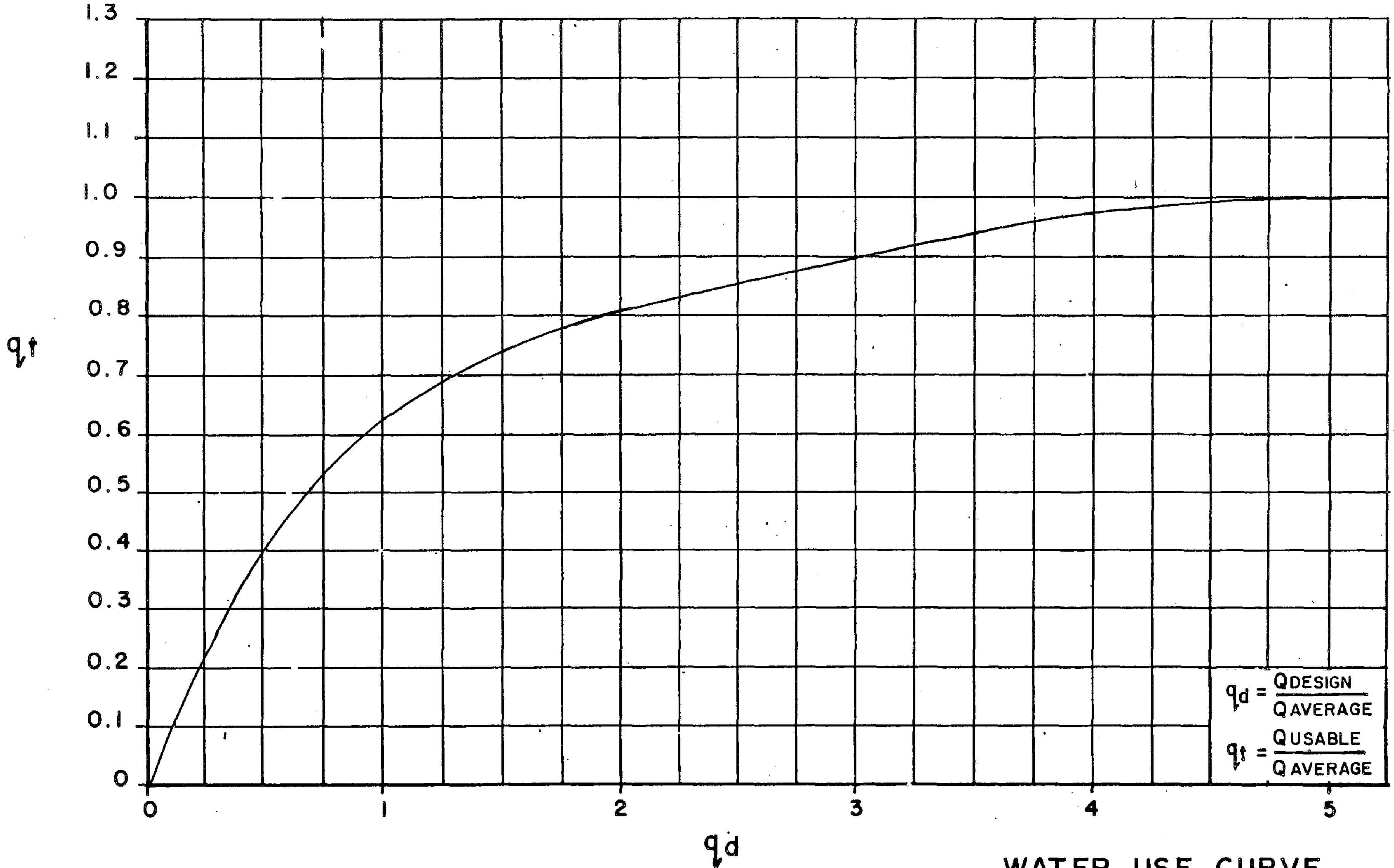
Source: "Floods and Reservoir Safety"

Institution of Civil Engineers,
London, 1978

TABLE 3.5

KITTY'S BROOK DEVELOPMENT SPILLWAY DESIGN FLOWS

Basin/Dam	Drainage Area (km^2)	I.C.E. Category and Design Criteria	Design Flood Peak (m^3/s)	Max Flood Surcharge (m)	Spillway Design Flow (m^3/s)
Barneys Brook	46.8	Category "C", Q150/0.2 PMF	39.5	1.0	39.5
Burnt Berry Brook	20.5	Category "C", Q150/0.2 PMF	20.8	1.0	20.8
Upper Sheffield Brook	64.9	Category "C", Q150/0.2 PMF	49.4	1.0	49.4
Chain Lakes					
- Diversion Dam	76.9	Category "C" Q1000/0.3 PMF	97.5 m^3/s	1.7	88.0
- Storage Dams	75.9	Category "B" Q10000/0.5 PMF	132.0 m^3/s	2.0	120.0
Kitty's Brook	15	Category "C", Q1000/0.3 PMF	150.5	1.5	150.5
Side Hill Canals		Q100	4.3m^3/s (per km^2)	---	---

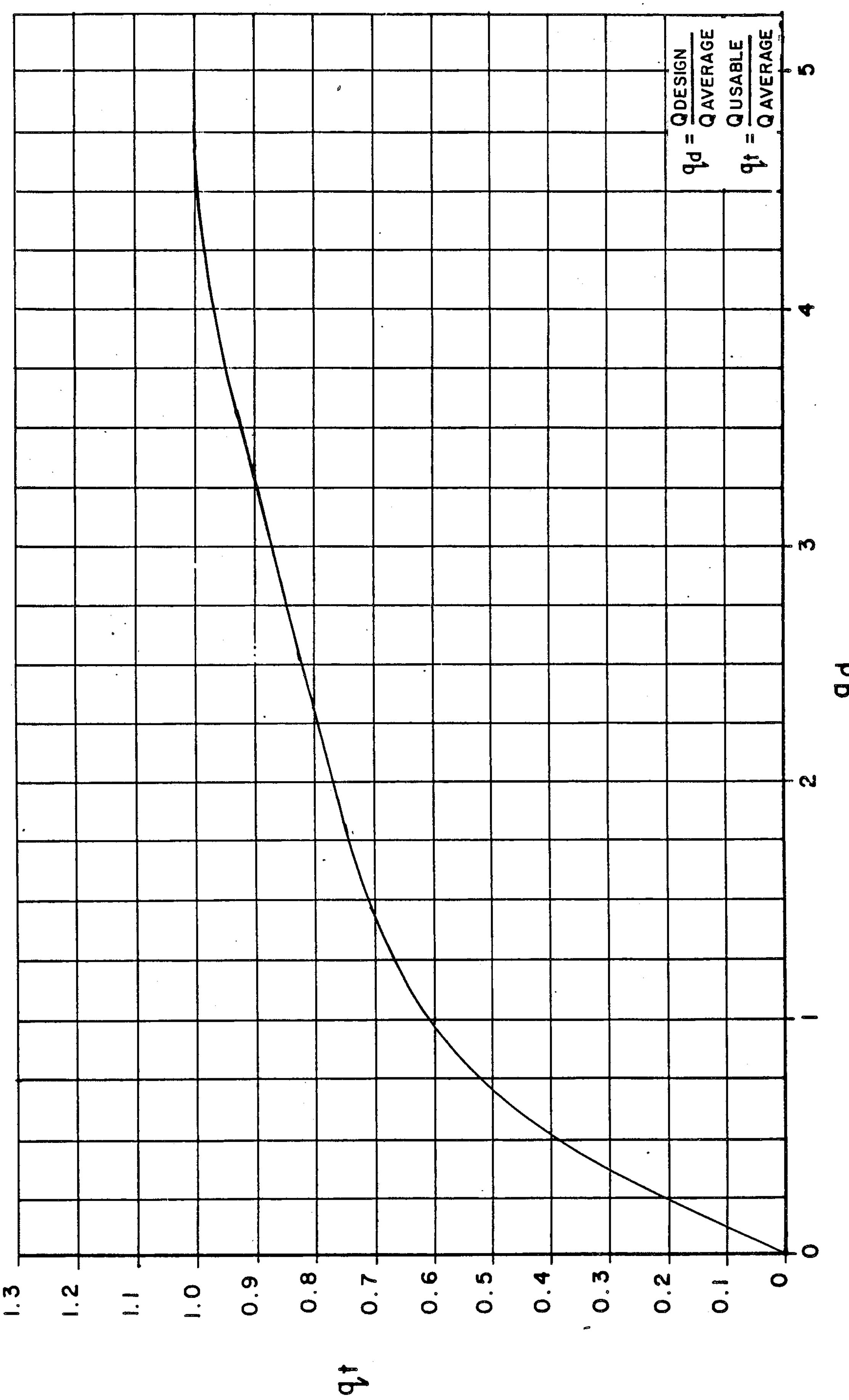


WATER USE CURVE
BARNEY'S BROOK

D.A. = 46.8 km^2
Qav. = 1.38 m^3/s

FIGURE 3.1

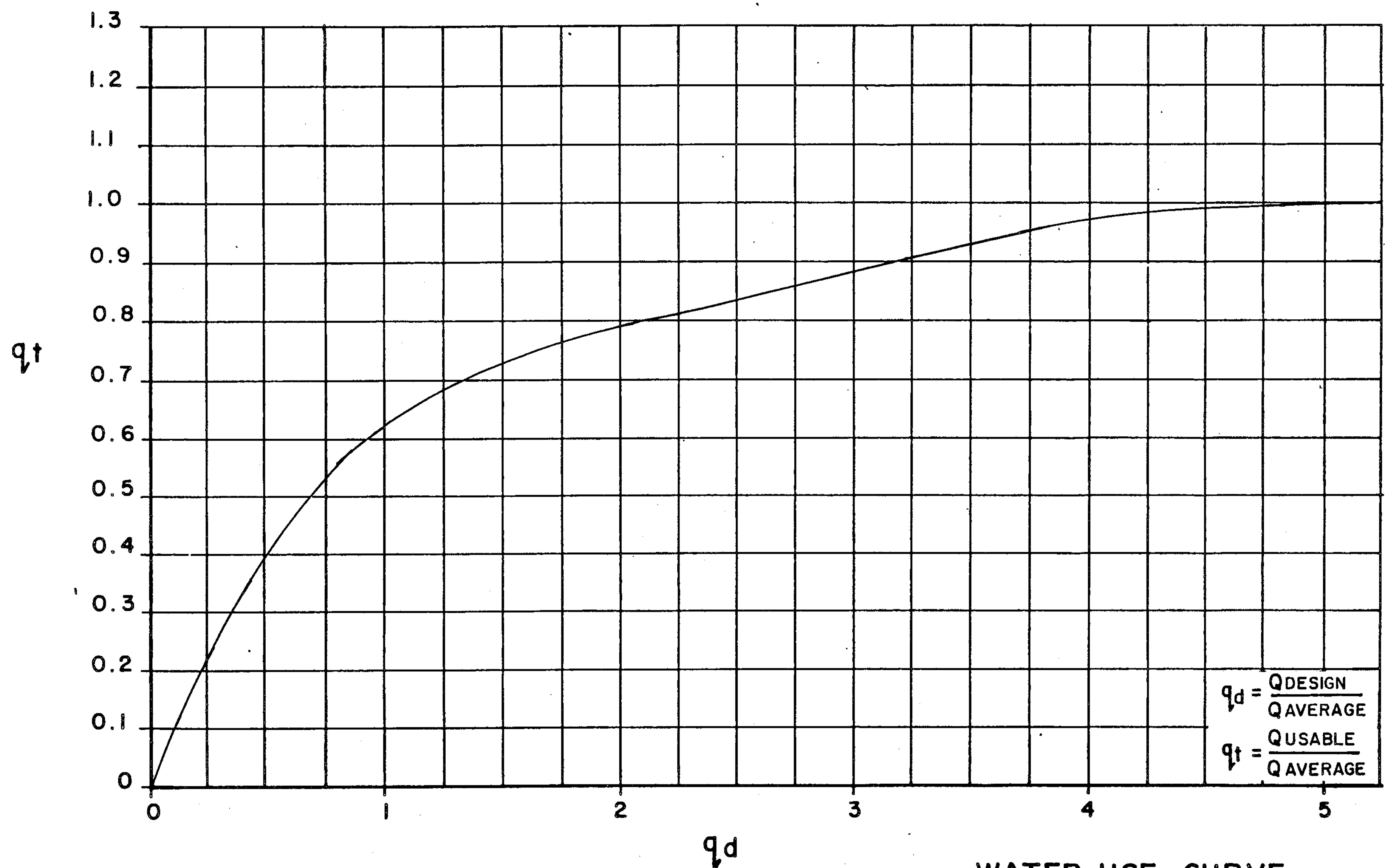
FIGURE 3.2



WATER USE CURVE
BURNT BERRY BROOK

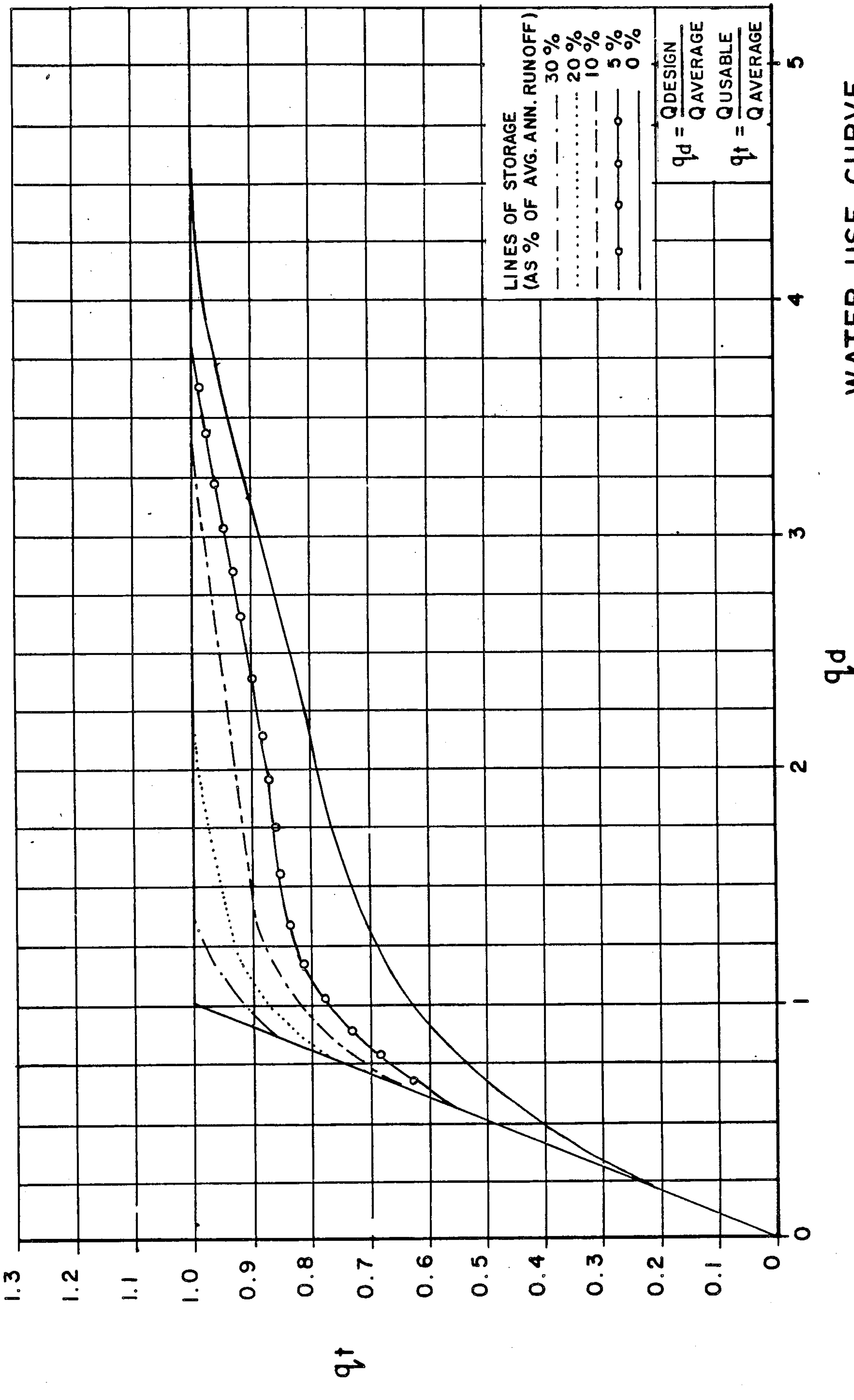
D.A. = 20.5 km^2
Q av. = 0.60 m^3/s

FIGURE 3.3



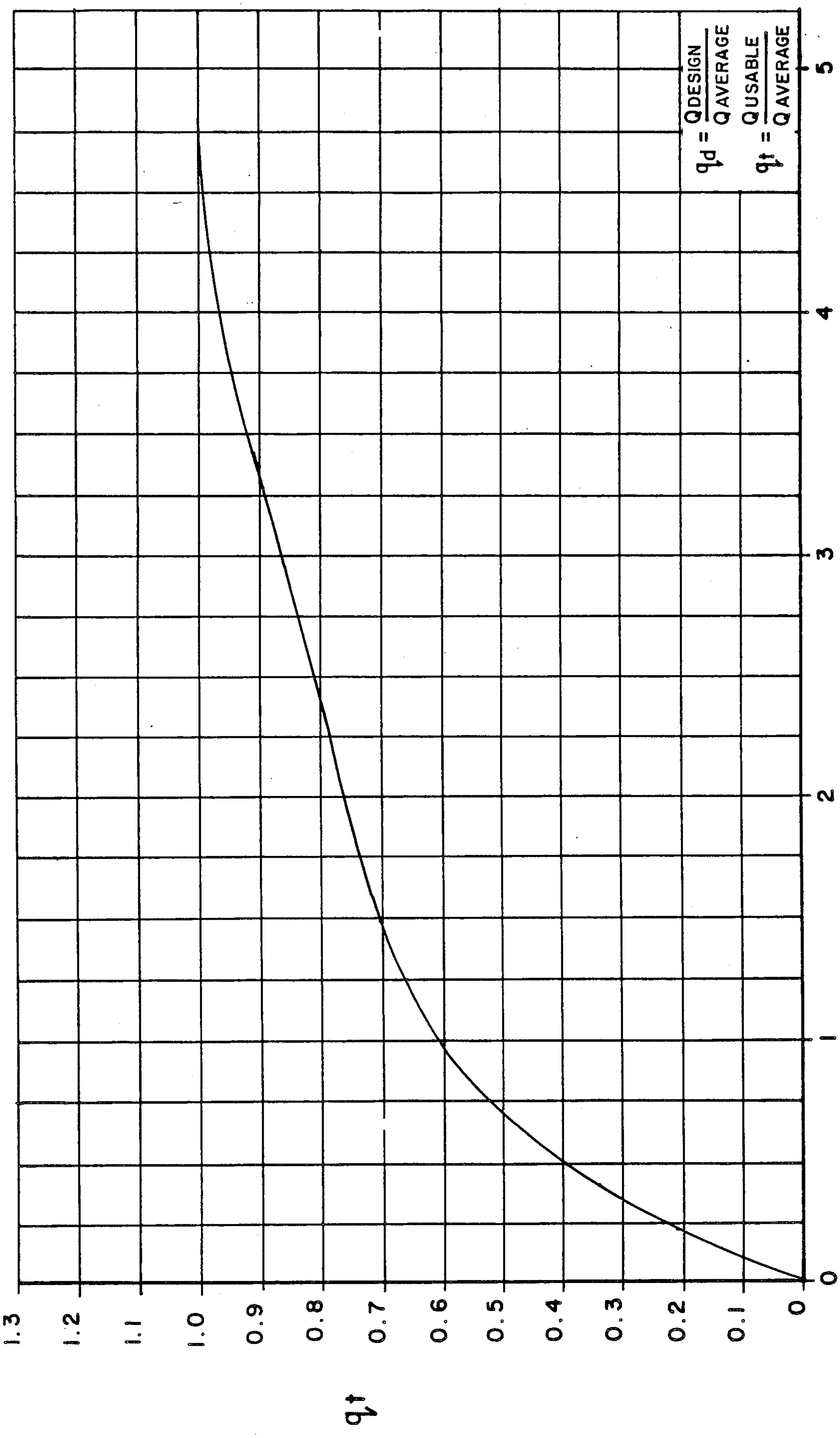
WATER USE CURVE
UPPER SHEFFIELD BROOK

D.A. = 80.9 km^2
Qav. = 2.38 m^3/s



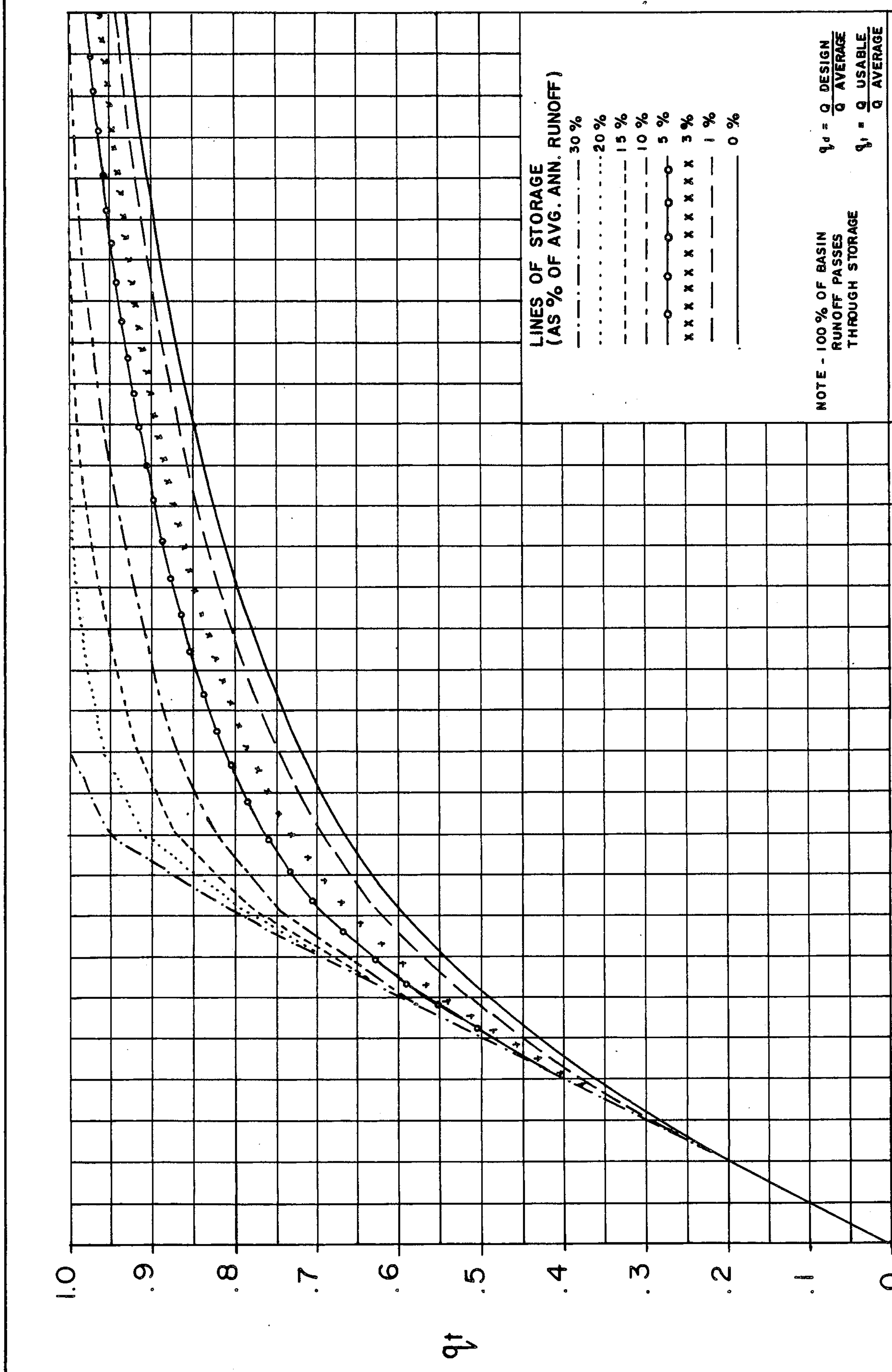
WATER USE CURVE
CHAIN LAKES

D.A. = 87.7 km^2
Qav. = 2.58 m^3/s



WATER USE CURVE
KITTY'S BROOK

D.A. = 186.2 km^2
Qav. = 5.47 m^3/s



WATER USE CURVE

HINDS LAKE

D.A. = 650.9 km^2
Q av. = 20.30 m^3/s

PART FOUR

DESCRIPTION OF DEVELOPMENT

4.1 ALTERNATIVES CONSIDERED

For each of the watershed diversions, several alternative schemes for diversion were considered. These are discussed in the following sections.

4.1.1 Kitty's Brook Diversion

The diversion of Kitty's Brook into Goose Pond would require the following structures:

-- a diversion dam on Kitty's Brook,

-- a spillway on Kitty's Brook,

-- an inlet to a water conveyance structure at the diversion dam, and

-- a water conveyance structure from Kitty's Brook to Goose Pond.

The different alternatives considered for the water conveyance structure are as follows:

1) a sidehill canal around the escarpment between Kitty's Brook and Goose Pond,

2) a surface woodstave pipeline around the escarpment, generally along the same route as the canal,

3) a single semi-buried fibreglass reinforced plastic (FRP) pipeline around the escarpment, generally along the same route as the canal,

4) a tunnel through the escarpment separating the Kitty's Brook and Goose Pond watersheds.

Alternative 1

The sidehill canal alternative (see Figure 4.1) would comprise a balanced excavation to fill with the material excavated from the uphill side being placed and compacted in a dyke embankment on the downhill side. The excavation and embankment would be provided with a filter and an impervious blanket to reduce leakage and the embankment would be topped off with a gravel surface to facilitate permanent access along the canal dyke. The canal invert slope was set at 1 in 10,000 and the invert width at 5m. A graph of capital cost versus design flow capacity (cost curve) was developed for the canal. Variation in flow was taken into account for varying the

4.1.1 Kitty's Brook Diversion (Cont'd)

total depth of the canal (invert to dyke crest), while maintaining the bottom width constant at 5.0 m. For this study, it was assumed that construction of the sidehill canal would be limited to where the sidehill cross-slope is less than 25%, ie. for a distance of 17,600m upstream of Goose Pond. Beyond this point, the sidehill cross-slope exceeds 25% and an 800m long semi-buried FRP pipeline would be provided. An energy dissipater would be provided at the downstream end of the pipeline to reduce the water velocity prior to entry into the canal. Allowances were made in the cost estimate of the canal for a spillway for every 6 km of canal length, for approximately 25 stream entries and for excavation through bedrock outcrops along the canal route.

Alternative 2

The surface woodstave pipeline alternative (see Figure 4.2) would require preparation of a right-of-way on which to erect the pipe. It would comprise excavation to fill with the fill being spread along the downhill side of the excavation. The pipeline would be constructed on a prepared bed with timber cradles, along the uphill side of the right-of-way. On the downhill side of this right-of-way, a gravel surface would facilitate the permanent access road alongside the pipeline. The pipeline was assumed to be continuous for the total distance of 18,400m between Goose Pond and the dam on Kitty's Brook. A cost curve was developed for the pipeline with variation in flow provided by varying pipe diameter. An allowance was made in the cost estimate for an energy dissipater at the downstream end of the pipe, to reduce the water velocity prior to its entry into Goose Pond, and for sidehill drainage and stream crossings across the pipeline route.

Alternative 3

The semi-buried FRP pipeline alternative (see Figure 4.3) would comprise a pipeline excavation in the original ground with the excavated material being spread along the downhill side of the excavation, thereby providing the subgrade for a permanent access road alongside the pipeline. The FRP pipe would be placed on a gravel bed in the excavated trench and backfilled with selected fill to the pipe haunches. This pipeline was assumed to be continuous for the total distance of 18,400m between Goose Pond and the dam on Kitty's Brook.

4.1.1 Kitty's Brook Diversion (Cont'd)

A cost curve was developed for the pipeline with variation in flow provided by varying pipe diameter. Allowances were made in the cost estimate for energy dissipation, sidehill drainage and stream crossings.

Alternative 4

The tunnel alternative (see Figure 4.4) would comprise an 11,500m long, tunnel through the escarpment from just upstream of the dam on Kitty's Brook to Goose Pond. The tunnel arrangement would have portals at each end, an inlet structure, a short section of FRP pipe at the downstream end to carry water from the tunnel outlet to Goose Pond, and an energy dissipater at the downstream end of the pipe.

Table 4.1 gives comparative capital costs for the four main alternative water conveyance systems considered based on a design flow of 20 m^3/s. The table includes the estimated costs for maintenance and the estimated value of lost energy resulting from lost drainage area, over a 60 year life, (using alternative 1 as the base).

In addition, various length/size/location combinations of the four alternatives were considered. However, no savings in costs were realized with these combinations, which were:

-- tunnel and canal: with different lengths of each component,

-- double FRP pipeline: two pipes of smaller diameter than single pipe, providing same flow capacity,

-- canal and pipe: with different lengths of each component, and

-- location of dam on Kitty's Brook: different dam locations for each of the four main alternatives and above combinations, varying the lengths of the conduits as required.

For each alternative considered, the cost of a dam on Kitty's Brook and an inlet for the water conveyance structure at the dam on Kitty's Brook was included in the comparative cost. The spillway, however, being a relatively minor and common cost, was not included.

Based on the analysis of alternative water conveyance systems for the Kitty's Brook Diversion, the apparent

4.1.1 Kitty's Brook Diversion (Cont'd)

economic choice* was the sidehill canal with the short section of FRP pipe along the steep hillside between the canal and the dam.

4.1.2 Chain Lakes Diversion

The diversion of Chain Lakes into Kitty's Brook would require the following structures, if no storage is provided in Chain Lakes:

-- a diversion dam on Chain Lakes Brook,

-- a spillway on Chain Lakes Brook,

-- an inlet/control structure to a water conveyance structure on Chain Lakes Brook,

-- a water conveyance structure from Chain Lakes to Kitty's Brook, and

-- a crossing of the CN railway line at Chain Lakes Brook.

If storage is provided in Chain Lakes, four additional structures would be requried, namely:

-- a storage dam at the outlet of Chain Lakes (west end),

-- a control outlet

-- a cut-off dam at the east end of Chain Lakes, and

-- a spillway at the east end of Chain Lakes.

Basically, two main alternative water conveyance structures were considered for this diversion, namely:

1) a sidehill canal, and

2) an excavated channel.

Each alternative would require a diversion dam on Chain Lakes Brook, an inlet/control structure at its upstream end for unwatering purposes, and a crossing of the CN line.

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* Although not included here, a review of bored tunnel costs received from a Tunnel Boring Machine (TBM) supplier late in the study indicated that a bored tunnel could be an economic alternative to the canal. This should be looked at in any future studies.

4.1.2 Chain Lakes Diversion (Cont'd)

Alternative 1

The sidehill canal alternative would comprise a 9500 m long sidehill canal around the hillside between Chain Lakes Brook and Kitty's Brook. It would be similar in design to the Kitty's Brook canal except that the average sidehill cross-slope is slightly less. At each end of the sidehill section, a 300 m long section of excavated channel would be required where the sidehill cross-slope is too steep for a sidehill canal. The diversion dam required for this alternative would be a relatively low dam; the maximum height is limited by the elevation of the adjacent railroad bed. At the downstream end, the canal would discharge into Kitty's Brook upstream of its diversion dam.

Alternative 2A

This alternative would comprise a 1900 m long excavation through a low saddle in the hill between Chain Lakes Brook and Kitty's Brook. The diversion dam, canal inlet/ control structure and railway crossing required at Chain Lakes Brook would be the same as for Alternative 1. To intercept the diverted water at Kitty's Brook, the Kitty's Brook diversion dam would have to be relocated farther downstream than the optimum location of that dam required for the Kitty's Brook diversion. This would result in a larger and more costly dam, the extra costs of which would be attributable to this alternative.

Alternative 2B

This alternative would be similar to alternative 2A except that the diversion dam on Chain Lakes Brook would be a little farther upstream, necessitating a little higher water level upstream of the dam but providng a shallower canal. The canal length, however, would be a little longer and the dam a little larger than those in Alternative 2A.

Alternative 2C

This alternative would be similar to alternatives 2A and 2B except that the diversion dam on Chain Lakes Brook would be even farther upstream. The location of the diversion dam would be at the site of Dam A identified in the earlier work by Newfoundland & Labrador Hydro. This alternative would require a higher water level and a larger volume of dead storage in Chain Lakes than either of Alternative 2A or 2B and would require another dam at the other end of Chain Lakes to contain the water. The canal length also would be longer than those required for Alternative 2A or 2B.

4.1.2 Chain Lakes Diversion (Cont'd)

Alternative 2D

This alternative would be a combination of Alternatives 1 and 2A. It would comprise an 1800 m long excavated channel through the low saddle in the hill between Chain Lakes Brook and Kitty's Brook and a 2700 m long sidehill canal from the downstream end of the excavated channel to the Kitty's Brook dam (at the location required for the Kitty's Brook diversion).

Table 4.2 gives comparative costs for the main alternatives considered based on a design flow of 11.5 m^3/s. The table includes the estimated value of lost energy resulting from lost drainage area (using the sidehill canal alternative as the base).

In addition to Alternative 1 and the various layouts for Alternative 2, other variations and alternatives were considered. No savings in costs were realized, however, with these variations and other alternatives which included the following:

-- FRP pipeline: this was rejected based on the comparative costs for Kitty's Brook,

-- woodstave pipeline: this was also rejected based on the comparative costs for Kitty's Brook,

-- tunnel: (i) with dam on Kitty's Brook in downstream location and no sidehill canal, and (ii) with original dam on Kitty's Brook and a sidehill canal,

-- sidehill canal: shorter sidehill canal (similar to Alternative 1) with dam on Kitty's Brook in downstream location, and

-- diversion dam: other locations for the diversion dam including (i) a location further downstream, in conjunction with a sidehill canal, and (ii) locations further upstream, including sites B and C identified in the earlier work by Newfoundland and Labrador Hydro.

The tunnel alternative was rejected after the site visit. Field observation indicated deep overburden in the area and a general lack of bedrock for portals and tunnelling.

Based on the analysis of alternatives for the diversion of Chain Lakes into Kitty's Brook, the obvious economic

4.1.2 Chain Lakes Diversion (Cont'd)

choice was Alternative 2D with the excavated channel through the low saddle and the associated sidehill canal between it and the Kitty's Brook dam.

4.1.3 Upper Sheffield Brook Diversion

The diversion of Upper Sheffield Brook into Chain Lakes would require the following structures:

-- a diversion dam on Upper Sheffield Brook,
-- a spillway on Upper Sheffield Brook, and
-- a water conveyance structure from Upper Sheffield Brook to Chain Lakes.

Several different alternatives were considered for the dam and spillway structures, including:

1) Timber crib
2) Concrete gravity
3) Earth fill

Each alternative required a canal for water conveyance to Chain Lakes. This canal could be one of two types.

1) a sidehill canal
2) an open cut canal.

Dam and Spillway Alternative 1

The timber crib alternative would consist of a standard 5 m wide rock filled crib of squared treated timber. The foundation would initially be levelled with concrete and be provided with rock anchors. The spillway would be formed by a depressed section of the dam crest.

Dam and Spillway Alternative 2

The concrete gravity alternative would be a reinforced concrete structure, 1m wide at the top with a 2V:1H sloping upstream face and a 1V:1H sloping downstream face. Rock anchors would be provided for stability. A spillway would be provided by depressing a section of the dam crest.

Dam and Spillway Alternative 3

The earthfill alternative would comprise a standard zoned earth/rock fill dam across the river with extensive foundation preparation and possible grouting. The spillway would comprise an excavated channel with a concrete overflow weir.

4.1.3 Upper Sheffield Brook Diversion (Cont'd)

Canal Alternative 1

The sidehill canal alternative would be a 7500 m long sidehill canal along the hillside between Upper Sheffield Brook and Chain Lakes. It would comprise a balanced excavation to fill with the excavated material being used to form a dyke on the downhill side. The canal invert slope was set at 1 in 1,000 and the invert width at 5m. An allowance was made for 1 major stream entry and one small spillway.

Canal Alternative 2

The open cut alternative would be a 7500 m long completely excavated canal along the same route as Alternative 1. All other parameters and allowances would be the same as Alternative 1.

It was obvious from field observations that the open cut canal would be the most practical alternative as a large portion of the excavation, which would be in rock, would be unsuitable for dyke construction. The simplicity of excavation versus dyke construction would shorten the construction period and would have advantages in construction logistics. It was also evident that the sidehill slope (only 3 - 4 %) would be inappropriate for a balanced cut-to-fill operation.

Based on preliminary cost estimates it was evident that the concrete dam/spillway alternative would be the most economic dam alternative. This alternative would also require less construction time and would likely experience fewer construction problems.

The preferred scheme would therefore be a concrete dam/spillway diversion structure with an open cut canal from Upper Sheffield Brook to Chain Lakes.

4.1.4 Burnt Berry Brook Diversion

The diversion of Burnt Berry Brook into Upper Sheffield Brook would require the following structures:

-- a diversion dam on Burnt Berry Brook,
-- a spillway on Burnt Berry Brook,
-- two small cut-off dykes, and
-- a canal from Burnt Berry Brook to Upper Sheffield Brook.

The dam alternatives considered for this diversion were the same as those for Upper Sheffield Brook. The choice

4.1.4 Burnt Berry Brook Diversion (Cont'd)

of alternative was also the same with the exception of the cut-off dykes. The diversion dam would be a concrete dam/spillway and the cut-off dykes would be simple earth/rockfill structures.

There were no alternatives for the canal which would comprise a series of short open cut canals between Burnt Berry Brook and Upper Sheffield Brook.

The preferred schemes would then comprise a concrete dam/spillway structure for the diversion dam, two earth/rockfill cut-off dykes and an open cut canal to convey water into Upper Sheffield Brook.

4.1.5 Barneys Brook Diversion

The diversion of Barneys Brook into Burnt Berry Brook would require the following structures:

-- a diversion dam on Barneys Brook,
-- a spillway on Barneys Brook, and
-- a canal from Barneys Brook to Burnt Berry Brook.

For this diversion, with bedrock in the area being close to the surface, the only practical structures would be a concrete dam with an integral overflow spillway and an open cut canal in rock. The canal would be approximately 2500 m long.

4.2 OPTIMIZATION OF DESIGN FLOWS

The available energy from the Kitty's Brook Development is directly proportional to the volume of water which can be delivered to the powerhouse of the Hinds Lake Development. The Kitty's Brook development comprises a maximum of five possible watershed diversions, all in series. The average flow of all five watersheds is potentially 12.41 m^3/s however, the actual average flow that could be obtained from the total development depends on the optimum design flow capacity of each diversion.

To determine the optimum design flow capacity of any particular diversion, all costs associated with conveying the water from that diversion to the next downstream watershed, plus all costs associated with delivering the water to Hinds Lake through each of the downstream diversions, was optimized against the value of the energy generated from that water in Hinds Lake. The final design flow for each diversion is the sum of its optimum design flow and the optimum design flow of each of the diversions upstream of it.

4.2 OPTIMIZATION OF DESIGN FLOWS (Cont'd)

The optimization procedure was to produce the maximum net benefit which is the difference between the value of the energy from an alternative source, such as thermal or other hydroelectric sources, and the cost of producing the same amount of energy from this development. The maximum net benefit occurs at the point where a small increase in energy could be provided as economically from the alternative source.

For this optimization, several assumptions were made as follows:

i) The turbinable flow calculated for each diversion could be delivered to Hinds Lake by increasing the structure sizes in the downstream diversions.

ii) The extra cost for increasing downstream structure sizes to pass extra flow would be attributable to the upstream diversion from which the extra flow originated.

iii) Additional flows delivered to Hinds Lake would not be spilled. In other words, the water utilization factor at Hinds Lake would be 1.00 (see Table 3.1).

iv) The capacity of the Goose Pond canal is sufficient to pass the design flow of Kitty's Brook canal without excessive increase in the original design water levels of Goose Pond. The Goose Pond canal will pass an additional average flow of 12.4 m^3/s from Kitty's Brook with an increase in the Goose Pond normal water level (NWL) of approximately 0.28m. It will pass an additional flood flow of 45 m^3/s with an increase in the Goose Pond maximum flood level (MFL) of approximately 0.25m.

v) The present worth of energy for the next 60 years was taken as $0.80 per kWh as agreed with Hydro. This value is consistent with that used for Paradise River.

The optimization of this development was completed for two cases: the first without storage on Chain Lakes, the second second with storage on Chain Lakes. The optimization procedure for each case began at the farthest downstream diversion, working progressively upstream optimizing each diversion enroute. For optimization, a range of design flows and associated costs were considered for each diversion and compared with the energy benefits derived from the usable flow corresponding to each value of design flow. Costs

4.2 OPTIMIZATION OF DESIGN FLOWS (Cont'd)

were taken from a cost curve* for the particular diversion. Table 4.3 summarizes the data required for optimization of each diversion.

Table 4.4 summarizes the results of the optimizations carried out with and without storage on Chain Lakes. The results indicate that for both cases, the cost of energy decreases as additional diversions are added. Also indicated is that the cost of energy is slightly higher for all five diversions when storage is provided on Chain Lakes. However, since only the major variable costs were included in the optimizations, these costs are not completely accurate. For this reason a detailed cost estimate was prepared for each case and these showed that provision of storage in Chain Lakes would result in a lower energy cost (see Section 6.4).

Figures 4.5 to 4.9 graphically illustrate the design flow optimization for each diversion. The present worth of costs and benefits are plotted against design flow to give a cost curve and a benefit curve from which the net benefit (benefits-cost) curve is derived.

4.3 DESCRIPTION OF SELECTED LAYOUT

It is apparent from the results of the optimization that the Development becomes more economic with each additional watershed diversion added. Based on (i) the selected alternatives for each diversion described in Section 4.1, (ii) the optimization described in Section 4.2 and (iii) the cost estimates for the two cases of storage and no storage in Chain Lakes (see Tables 6.1 and 6.2), the optimum layout of the Development was selected as shown on Plates 1 - 4. The required structures for each diversion are shown in more detail on Plates 5 - 13. The following sections describe the selected diversion scheme for each watershed diversion.

4.3.1 Kitty's Brook Diversion

In Section 4.1, the sidehill canal alternative was shown to be the most economic method for diverting Kitty's Brook into Goose Pond. This alternative, which is shown

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* Cost curves included the costs of structures required to deliver specific volumes of water from a particular watershed to the next downstream watershed, plus the costs associated with increasing the flow capacity of each of the downstream diversions to pass those volumes of water.

4.3.1 Kitty's Brook Diversion (Cont'd)

on Plate 2, would comprise a diversion dam on Kitty's Brook, a spillway, an intake at the dam, an 800 m long section of pipeline to carry water from the intake, along a steep hillside, to the 17,600 m long sidehill canal. At the downstream end of the pipe, an energy dissipater would be required to reduce the high velocity flow of the pipeline to an acceptable velocity for the canal.

The optimized design for this diversion results in the following criteria:

Design Flow Capacity (with storage in in Chain Lakes)	34.50 m^3/s
NWL - Kitty's Brook	326.00 m
Diversion Dam Crest El.	329.00 m
Spillway Capacity - Q_{1000}	150.50 m^3/s
FRP Pipe - diameter	3.0 m
- length	800 m
Canal - length	17,600 m
- velocity	0.60 m/s
- gradient	0.0001 m/m
- friction coefficient	Manning's n = 0.025
- side slopes	1V:2.5H

Diversion Dam

The dam as presently envisaged would be a zoned earth-fill structure with a thin impervious core and upstream blanket (see Plate 5). This design would reduce seepage through the deep pervious foundation which is anticipated at this and most other damsites in this development. The crest of the dam would be at El 329m, have a width of 5 m and a length of approximately 180 m. The maximum height of the dam above the river bed would be about 30m.

It is anticipated that the unwatering system for the dam site would be a conduit with an upstream cofferdam. Although not detailed for this study, an allowance for unwatering was made in the cost estimate.

Spillway

The spillway for this diversion would be located in a low saddle to the east of, and separated from the dam by, a high knoll (see Plate 5). It would comprise a 30 m long concrete overflow weir located near the downstream end of a 30 m wide and approximately 430 m long channel excavated through the low saddle. Water would spill into an existing small stream and return to Kitty's Brook approximately 400 m downstream of the dam.

4.3.1 Kitty's Brook Diversion (Cont'd)

Intake

The intake would be an unhoused, reinforced concrete structure located in the west abutment of the dam (see Plate 5). It would have a deck of steel grating at El. 329 located just upstream of the dam crest. Access to the deck would be provided by decked timber cribbing.

The structure would be equipped with trashracks and steel stoplogs which would be positioned behind the trashracks and installed and/or removed as required by mobile crane. The stoplogs would normally be stored near the intake and installed only when necessary to unwater the canal or pipe, or to clean the trashracks.

Pipe

An 800 m long pipeline would be required to carry water from the intake to the canal, around the steep hillside overlooking Kitty's Brook. The pipe would be a fibre-glass reinforced plastic (FRP) pipe semi-buried in a berm constructed of heavy rockfill (see Plate 5). The berm would be constructed wide enough to provide permanent access along the pipe.

Runoff from the hillside above the pipeline would be accomodated by ditching along the uphill side of the pipeline and providing catch-basins and cross-drains to discharge the water on the downhill slope of the heavy rockfill. To stabilize the uphill slope of the ditching, a rockfilled gabion wall has been allowed in the cost estimate.

To reduce the high water velocity of the pipe to an acceptable limit for the canal, an energy dissipating transition would be required at the downstream end of the pipeline. This would be a reinforced concrete structure with a flared transition and stilling basin.

Canal

The canal would be a 17,600 m long sidehill canal with a cut and fill cross-section (see Plate 6). Wherever possible, the quantity of material excavated on the uphill side would be balanced against the common fill requirements of the embankment on the downhill side. The excavated material suitable for use as common fill would be placed and compacted in the embankment.

To allow for the probable variation in gradation of local materials and to reduce potential leakage from the

4.3.1 Kitty's Brook Diversion (Cont'd)

canal, the preliminary canal design includes a 300 mm thick filter blanket between an impervious liner of glacial till and the excavated and common fill surfaces in the canal. In addition, to allow for changes in water depth in the canal (resulting from variation in canal flows) and to help stabilize the impervious liner, a 300 mm thick surface layer of coarse gravel was allowed for in the cost estimate for this structure. In fact, the most cost effective method of accomodating variation in water depth may be a flattening of the canal side slopes instead of adding the gravel surface. This item would be considered as a design detail at a later date.

A subsurface drainage system was also allowed for in the cost estimate. This would be required to relieve ground water pressure on the uphill side of the canal and prevent blow outs of the impervious liner. Ground water would be collected by the filter material and carried away by perforated pipes and cross-drains.

For this report it was assumed that construction of the sidehill canal would be impractical when the sidehill cross-slope exceeds 25%. This is because of the long downhill slope required on the embankment and the long uphill slope required for the excavation when the side-hill cross-slope becomes too steep. The average cross-slope over the 17,600 m length of the canal is 9.2%.

The depth of water in the canal for the design flow capacity of 34.50m^3/s (with storage in Chain Lakes) would be about 3.7 m and the freeboard to the embankment crest would be 1.5 m. To limit flood surcharge on the canal water level during flood flows, a spillway would be provided for approximately every 6 km of canal length. For this study it was assumed that the spillways would be low concrete overflow weirs placed on bedrock. The spillway crests would be set at the normal (design) water level at each location.

Stream entries would be designed at every significant stream crossing. These inlets would be provided with an impervious cut-off in the original stream bed, to prevent water passing under the canal, and a riprap lining to prevent erosion of the canal liner. Where possible, small streams would be collected by ditching along the uphill side of the canal to minimize the number of stream entries.

4.3.2 Chain Lakes Diversion

As indicated in Section 4.1, the alternative which consisted of the excavated channel/sidehill canal, a small diversion dam on Chain Lakes Brook and the Kitty's

4.3.2 Chain Lakes Diversion (Cont'd)

Brook dam in its upstream location, was the most economic method for diverting Chain Lakes into Kitty's Brook. This alternative is shown on Plate 2 and would comprise a diversion dam on Chain Lakes Brook, a spillway, an excavated channel, a containment dyke on the channel, a sidehill canal, relocation of a short section of railway road bed, a bridge, and an inlet/ control structure at the upstream end of the channel.

The optimization of the total development (all five diversions) indicated that provision of storage on Chain Lakes was marginally economic. However, the detailed cost estimate prepared subsequent to the optimization indicated that storage should be provided on Chain Lakes.

The optimized design for this diversion results in the following criteria:

Design Flow Capacity (with storage in Chain Lakes)	11.50 m^3/s
NWL - Chain Lakes Brook	331.50 m
Diversion Dam Crest El.	334.00 m
Spillway Capacity - Q_{1000}	88 m^3/s
Excavated Channel	
- length	1500 m
- velocity (maximum)	1.0 m/s
- gradient	0.0001 m/m
- friction coefficient	Manning's n = 0.025
- side slopes	1V:2H
Containment Dyke Crest El.	334.00 m
Sidehill Canal	
- length	2,700 m
- velocity	0.6 m/s
- gradient	0.0001 m/m
- friction coefficient	Manning's n = 0.025
- side slopes	1V:2.5H
Storage Dam Crest El.	350.00 m
Chain Lakes Reservoir - FSL	347.00 m
- LSL	334.00 m
Storage Volume	57 Mm^3
Spillway Capacity - Q_{10000}	120 m^3/s

Diversion Dam

The dam, which is envisaged to be a zoned earthfill structure similar to Kitty's Brook, would be located across a narrow section of Chain Lakes Brook adjacent to the CN railway and would tie into the railway roadbed. The foundation at this site is expected to comprise deep

4.3.2 Chain Lakes Diversion (Cont'd)

pervious material. To reduce seepage through the foundation, an upstream impervious blanket would be provided. The crest elevation of the dam would be El. 334 (the same as the railway roadbed) with a total length of two sections, separated by a small knoll, of approximately 330 m. The maximum height of the dam above the riverbed would be about 11m.

The unwatering envisaged for this site would comprise a conduit with an upstream cofferdam. The unwatering scheme was not detailed for this study but an allowance was made for it in the cost estimate.

Spillway

The spillway for this diversion would be located approximately in the middle of the north section of the dam which plugs a natural low saddle on the north side of Chain Lakes Brook (see Plate 7). It would comprise a 23.5m long overflow weir with a crest elevation of 331.5m, set in a 23.5m wide and approximately 170m long channel. The channel would be excavated horizontally through the low saddle and then at a 10% gradient down the slope of the hill behind the dam. The weir would be constructed of graded rockfill and filter materials which would be contiguous with a riprap and filter lining of the downstream channel. Such an arrangement would permit water to spill back into a natural pool on Chain Lakes Brook at a point approximately 150 m downstream of the dam.

Excavated Channel

The channel, which was assumed would be excavated entirely in earth through low saddles of the hill between Chain Lakes Brook and Kitty's Brook, would cross the existing railway right-of-way. The channel inlet would be located just upstream of the diversion dam, adjacent to the railway right-of-way.

To minimize disruption to the railway, the railway would have to be realigned and either a large culvert or bridge installed to facilitate the proposed channel. Such a realignment allows for the bridge and new roadbed and track to be installed without interruption of traffic until the connection between the old and new track at each end of the realigned section was required. With the new track in place, the channel excavation could be completed through the old roadbed. For the purpose of this report costs for a bridge and a 260m long realignment of the roadbed was included in the cost estimate.

4.3.2 Chain Lakes Diversion (Cont'd)

The excavated channel would be in two sections totalling 1500 m in length. The shorter upstream section would be 300 m long and would carry water from Chain Lakes Brook through high ground into a small stream valley which drains across the railway right-of-way approximately 850 m west of the channel crossing. Near the downstream end of the valley, but upstream of the railway, a dyke would be required to contain water in the valley that forms part of the water channel. This dyke would have a crest elevation of 334 m and would be in two sections separated by high ground. Its total length would be approximately 470 m and its maximum height would be approximately 15 m. The dyke construction would be similar to the diversion dam.

The downstream section of the channel would be approximately 1200 m long and would be excavated through two small ponds. The maximum depth of excavation would be about 16 m. Water flowing in the excavated channel would enter a sidehill canal as it flows toward Kitty's Brook.

The depth of water in the channel for the design flow capacity of 11.50 m^3/s would be about 2.3 m.

Inlet/Control Structure

This would be a simple reinforced concrete structure upstream of, and integral with, the bridge. Its purpose would be to allow unwatering of the canal if required. It would comprise two water passages separated by a center pier with slots in the concrete for installation of timber stoplogs when required. A simple access from the railway roadbed and a deck would be provided to facilitate stoplog installation/removal.

Sidehill Canal

The canal would be a 2,700 m long sidehill canal with a cut and fill cross-section similar to the Kitty's Brook canal (see Plate 9). The canal would be constructed along the sidehill of the valley of a small tributary of Kitty's Brook.

The depth of water in this canal for the design flow capacity of 11.50 m^3/s would be about 2.2 m and the freeboard to the dyke crest would be 1.5 m. This canal is relatively short and no spillways would be required.

As for the Kitty's Brook canal, stream entries would be provided at all stream crossings except where ditching along the uphill side is practical for collecting adjacent small streams to minimize the number of stream entries.

4.3.2 Chain Lakes Diversion (Cont'd)

The downstream end of this sidehill canal would exit into a short excavated channel that extends to the spillway channel of the Kitty's Brook diversion, just upstream of the spillway weir (see Plate 5). The water would then flow through the spillway channel to Kitty's Brook.

Storage Dam

This dam, located across the outlet at the west end of Chain Lakes is envisaged to be a zoned earthfill structure with an upstream impervious blanket similar to Kitty's Brook. The crest elevation would be El. 350 and the length of the crest would be approximately 850 m. The maximum height above the riverbed would be about 20m.

Control Outlet

The outlet would be located in the storage dam and on the north side of the original stream outlet of Chain Lakes. It is envisaged to be a 1200 mm x 1200 mm concrete box conduit approximately 74 m long. The upstream inlet would be provided with a set of trashracks. Near the centreline of the dam, a vertical concrete shaft extending up to the dam crest would contain a sliding gate and provisions for installing stoplogs. The top of the shaft would be enclosed by a wooden housing.

Cut-off Dam

This dam would be located along the top of the narrow strip of land separating the east end of Chain Lakes from the headwaters of Sheffield Brook. It would have a crest at El. 350 and a crest length of approximately 1800 m. The maximum height would be about 8m.

Spillway

This spillway would be very similar to the spillway at the diversion dam and would be located in the cut-off dam at a low saddle in the existing ground. It would comprise a 24m long overflow weir at a crest elevation of 347 m. The weir would be of graded rockfill construction with a riprap lined downstream channel which would spill water into a small pond in the headwaters of Sheffield Brook.

4.3.3 Upper Sheffield Brook Diversion

In Secion 4.1, the concrete dam/spillway structure and the open cut canal was shown to be the preferred method for diverting Upper Sheffield Brook into Chain Lakes. This scheme is shown on Plates 3 and 11.

The optimized design for the diversion results in the following criteria:

Design Flow Capacity		15.39 m^3/s
NWL - Upper Sheffield Dam		361.0 m
Diversion Dam crest		362.5 m
Spillway crest		361.0 m
Spillway capacity/Q_{150}		49.4 m^3/s
Canal - length		7500 m
- velocity (maximum)		1.14 m/s
- gradient		.001 m/m
- friction coefficient	Mannings n =	.030
- side slopes		1V:2H Gravel
		6V:1H Rock

Diversion Dam and Spillway

The dam as presently envisaged would be a reinforced concrete structure with a depressed section of crest forming an overflow weir. Both abutments of the dam would be constructed of earth/rock fill spoil to provide freeboard and cut-off any perimeter low spots adjacent to the dam.

The crest of the concrete weir would be at El. 361. The concrete weir would have a top width of 1m with 1V:1H upstream sloping face and 2V:1H downstream sloping face. This section of the dam would be founded on bedrock and would have rock anchors provided for stability against overturning and sliding.

The crest of the freeboard abutments would be at El. 362.5 with a top width of 6m and a length of 400 m. These abutments would be constructed of earthfill spoil to El. 362.0 and would be capped with 0.5 m of rockfill spoil. Foundation preparation would consist only of stripping and a small core trench excavated by backhoe as the total maximum height would be only 2 m.

4.3.3 Upper Sheffield Brook Diversion (Cont'd)

It is anticipated that the unwatering system for the dam site would consist of simple low cofferdams to facilitate sectional construction of the dam, with the last section being completed after the construction of the canal. Although not detailed for this study, an allowance for unwatering was made in the cost estimate.

Canal

The canal would be a 7,500m long open cut canal, excavated along a sidehill with a slight cross-slope. It would be partly in rock and partly in earth.

At sites where the freeboard on the downhill side of the canal is insufficient due to undulating topography, varying stripping depth or material quality, small freeboard dykes would be constructed of earthfill spoil from the canal excavation.

The depth of water in the canal for the design flow capacity of 15.39 m^3/s would be 2.5m and the freeboard on the downhill side would be 2.0m. To limit flood surcharge on the canal water level due to runoff from the sidehill and the major stream entry, a simple overflow spillway structure would be constructed at the stream location. The spillway would be founded on rock and its crest would be set at the normal water level at that location.

4.3.4 Burnt Berry Brook Diversion

In section 4.1, the concrete dam/spillway structure, earth fill dykes and the open cut canal was shown to be the preferred method for diverting Burnt Berry Brook into Upper Sheffield Brook. This scheme is shown on Plates 3 and 12.

The optimized design for this diversion results in the following criteria:

Design Flow Capacity		5.87 m^3/s
NWL - Burnt Berry Dam		366.0 m
Diversion Dam and Dyke crests		367.5 m
Spillway crest		366.0 m
Spillway capacity - Q_{150}		20.8 m^3/s
Canal - length		400 m
- velocity (maximum)		1.14 m/s
- gradient		.001 m/m
- friction coefficient	Manning's n =	.030
- side slopes		1V:2H

4.3.4 Burnt Berry Brook Diversion (Cont'd)

Diversion Dam and Spillway

The dam as presently envisaged would be a reinforced concrete structure with a depressed section of crest forming an overflow weir. The north abutment of the dam would require a rockfill berm to provide freeboard at a low spot on the flooded perimeter.

The crest of the concrete weir would be at El. 366.0 with the abutment crests being El. 367.5. The total crest length would be 60m. The concrete weir would have a top width of 1m with a 1V:1H upstream sloping face and a 2V:1H downstream sloping face. This section of dam would be founded on bedrock and would have rock anchors provided for stability against overturning and sliding.

The freeboard berm crest would be at El. 367.5 and would have a top width of 6m and a crest length of 100m. This berm would be constructed of rockfill spoil. Foundation preparation would not be necessary as the total maximum height would only be 1 m.

The two small dykes required on the perimeter of the flooded area would have a crest at El. 367.5, a top width of 6 m and a combined crest length of 300 m. These dykes would be constructed of local glacial till and/or earthfill spoil to El. 367.0 and would be capped with 0.5 m of rockfill spoil. Foundation preparation would consist only of stripping and a small core trench excavated by backhoe as the total maximum height would be only 3 m.

It was anticipated that the unwatering scheme would be similar to that for Upper Sheffield Brook.

Canal

The canal would comprise a short open cut excavation of the high ground between Burnt Berry Brook and Upper Sheffield Brook. The total length of this canal would be 400m and the excavation would be entirely in earth.

The depth of water in the canal for the design flow capacity of 5.87 m^3/s would be 1 m.

4.3.5 Barneys Brook Diversion

In Section 4.1, the concrete dam/spillway structure and the open cut canal was shown to be the preferred method for diverting Barneys Brook into Burnt Berry Brook. This scheme is shown on Plates 4 and 13.

4.3.5 Barneys Brook Diversion (Cont'd)

The optimized design for this diversion results in the following criteria:

Design Flow Capacity		3.77 m^3/s
NWL - Barneys Brook Dam		404.0 m
Diversion Dam crest		405.5 m
Spillway crest		404.0 m
Spillway capacity - Q_{150}		38.0 m^3/s
Canal - length		2500 m
- velocity (maximum)		1.14 m/s
- gradient		.001 m/m
- friction coefficient	Manning's n =	.030
- side slopes		6V:1H

Diversion Dam & Spillway

The dam as presently envisaged would be a reinforced concrete structure with a depressed section of crest forming an overflow weir. Both abutments of the dam would require a rockfill berm to provide freeboard at low spots on the flood perimeter.

The crest of the concrete weir would be at El. 404.0 with the abutment crests being 405.5. The total crest length would be 50m. The concrete weir would have a top width of 1m with a 1V:1H upstream sloping face and a 2V:1H downstream sloping face. This section of dam would be founded on bedrock and would have rock anchors provided for stability against overturning and sliding.

The freeboard berm crest would be at El. 405.5 and would have a top width of 6m and a crest length of 200 m. This berm would be constructed of rockfill spoil. Foundation preparation would not be necessary as the total maximum height would only be 1 m.

It was anticipated that the unwatering scheme would be similar to that for Upper Sheffield Brook.

Canal

The canal would be a 2,500 m long open cut canal entirely in rock.

To avoid possible backwater effects, an allowance was made for some downstream channel improvements as the diverted water would pass through a small existing streambed.

The depth of water in the canal for the design flow capacity of 3.77 m^3/s would be 1.0 m. As the canal would be excavated entirely in rock, no allowance was made for stream entry points.

TABLE 4.1

ALTERNATIVE WATER CONVEYANCE SYSTEMS BETWEEN KITTY'S BROOK

AND GOOSE POND FOR Qd = 20 m3/s

Alternative / Item	1 (Canal)	2 (FRP Pipeline)	3 (Wood Stave Pipeline)	4 (Tunnel)
Direct Cost (1986 Dollars)	$31,095,000	$44,070,000	$56,759,000	$40,482,000*
Present Worth of Maintenance (60 years)	$ 350,000	120,000	900,000	200,000
Present Worth of lost drainage area (60 years)	0	9,891,000	9,891,000	10,652,000
TOTAL COST	$31,445,000	$54,081,000	$67,550,000	$51,334,000

ALTERNATIVES:

1 - Canal: 17,600 m of sidehill canal between Goose Pond and Kitty's Brook to point where sidehill cross-slope is greater than 25%. From this point to Dam on Kitty's Brook 800 m of 3 m diameter semi-buried FRP pipe.

2 - FRP Pipeline: 18,400 m of semi-buried FRP pipe along sidehill between Goose Pond and Dam on Kitty's Brook.

3 - W.S. Pipeline: 18,400 m of surface wood stave pipe along sidehill between Goose Pond and Dam on Kitty's Brook.

4 - Tunnel: 11,500 m of 4.5 m x 4.5 m tunnel through hill from Kitty's Brook. Inlet of tunnel located upstream of Dam on Kitty's Brook. From outlet of tunnel to Goose Pond a 700 m long, 2 m diameter semi-buried FRP pipe.

Note: Costs are comparative only and do not include all fixed costs.

* A review of bored tunnel costs which was received from a Tunnel Boring Machine (TBM) supplier late in the Study, indicated this could be approximately $24,000,000 for a total of $34,852,000 and could be competitive with Alternative 1.

TABLE 4.2

ALTERNATIVE WATER CONVEYANCE SYSTEMS BETWEEN

CHAIN LAKES AND KITTY'S BROOK FOR Qd = 11.5 m3/s

Item \ Alternative	1 Sidehill Canal	2 (Excavated Channel) A	B	C	D
Capital Cost (i) (1986 Dollars)	$14,833,000	$ 5,867,000	$11,774,000	$11,074,000	$ 9,820,000
Change in Kitty's Brook dam costs to accommodate Chain Lake diversion (ii)	0	+ 9,064,000	+ 9,064,000	+ 9,064,000	0
Change to Kitty's Brook pipeline costs to accommodate Chain Lake diversion (iii)	0	- 864,000	- 864,000	- 864,000	0
Present Worth of lost drainage area (60 years)	0	+ 541,000	+ 541,000	+ 811,000	+ 541,000
TOTAL COST	$14,833,000	$14,608,000	$20,515,000	$20,085,000	$10,361,000

NOTES:

(i) Does not include costs included in Kitty's Brook Diversion (with dam in upstream location)
(ii) Additional cost to relocate Kitty's Brook dam downstream.
(iii) Cost saving in pipeline with Kitty's Brook dam located downstream.
(iv) Costs are comparative only and do not include all fixed costs.

TABLE 4.2 (Cont'd)

ALTERNATIVES:

1	-	Sidehill Canal: 9,500 m of sidehill canal and 750 m of excavated channel around the hill between Chain Lakes and Kitty's Brook. Kitty's Brook dam in upstream location.
2A	-	Excavated Channel - Total of 1,500 m of excavated channel in two excavated sections through low saddles in hill between Chain Lakes and Kitty's Brook. Kitty's Brook dam in downstream location.
2B	-	Excavated Channel: Similar to 2A except larger dam on Chain Lakes Brook further upstream. Excavated channel longer than in 2A. Kitty's Brook dam in downstream location.
2C	-	Excavated Channel: 2A and 2B except dam even further upstream at site of Dam A identified by NLH. Excavated channel longer than in 2A or 2B. Dam required at other end of Chain Lakes to contain water in Chain Lakes. Kitty's Brook dam in downstream location.
2D	-	Excavated Channel: Similar to Scheme A except dam on Kitty's Brook in upstream location and downstream end of excavated channel from Chain Lakes Brook would dump into sidehill canal which would carry water to Kitty's Brook dam.

TABLE 4.3

COST AND BENEFIT DATA REQUIRED

FOR OPTIMIZATION OF A DIVERSION

COSTS	BENEFITS
i) Cost associated with diverting each of a range of design flows from the diversion being optimized, to the next downstream watershed, plus	i) Value of energy generated from each design flow.
ii) Extra costs attributable to increasing flow capacity of each downstream diversion to pass each design flow from the diversion being optimized, plus	

TABLE 4.4

SUMMARY OF OPTIMIZATION

CASE	WATERSHED DIVERSION*	DESIGN FLOW (m^3/s)					COST ($1000)						ENERGY (gWh)	(2) ENERGY COST (mils/kWh)
		KB	CL	US	BB	B	KB	CL	US	BB	B	TOTAL		
No Storage in Chain Lakes	KB	23.00					60,720					60,720	90	81.9
	KB+CL	33.00	10.00				66,240	14,680				80,920	129	76.1
	KB+CL+US	42.52	19.52	9.52			71,190	16,140	5,950			93,280	169	67.0
	KB+CL+US+BB	44.62	21.62	11.62	2.10		72,280	16,400	6,830	1,120		96,630	179	65.5
	KB+CL+US+BB+B	48.39	25.39	15.39	5.87	3.77	74,240	16,860	8,400	1,190	1,000	101,690	200	61.7
With Storage in Chain Lakes	KB	23.00	(1)				60,720					60,720	90	81.9
	KB+CL	26.61	3.61				62,720	16,930				79,650	129	74.9
	KB+CL+US	29.94	6.94	9.52			64,560	21,210	5,950			91,720	169	65.8
	KB+CL+US+BB	30.78	7.78	11.62	2.10		65,040	22,430	6,830	1,120		95,420	179	64.7
	KB+CL+US+BB+B	34.50	11.50	15.39	5.87	3.77	66,960	24,800	8,400	1,190	1,000	102,350	200	62.1 (3)

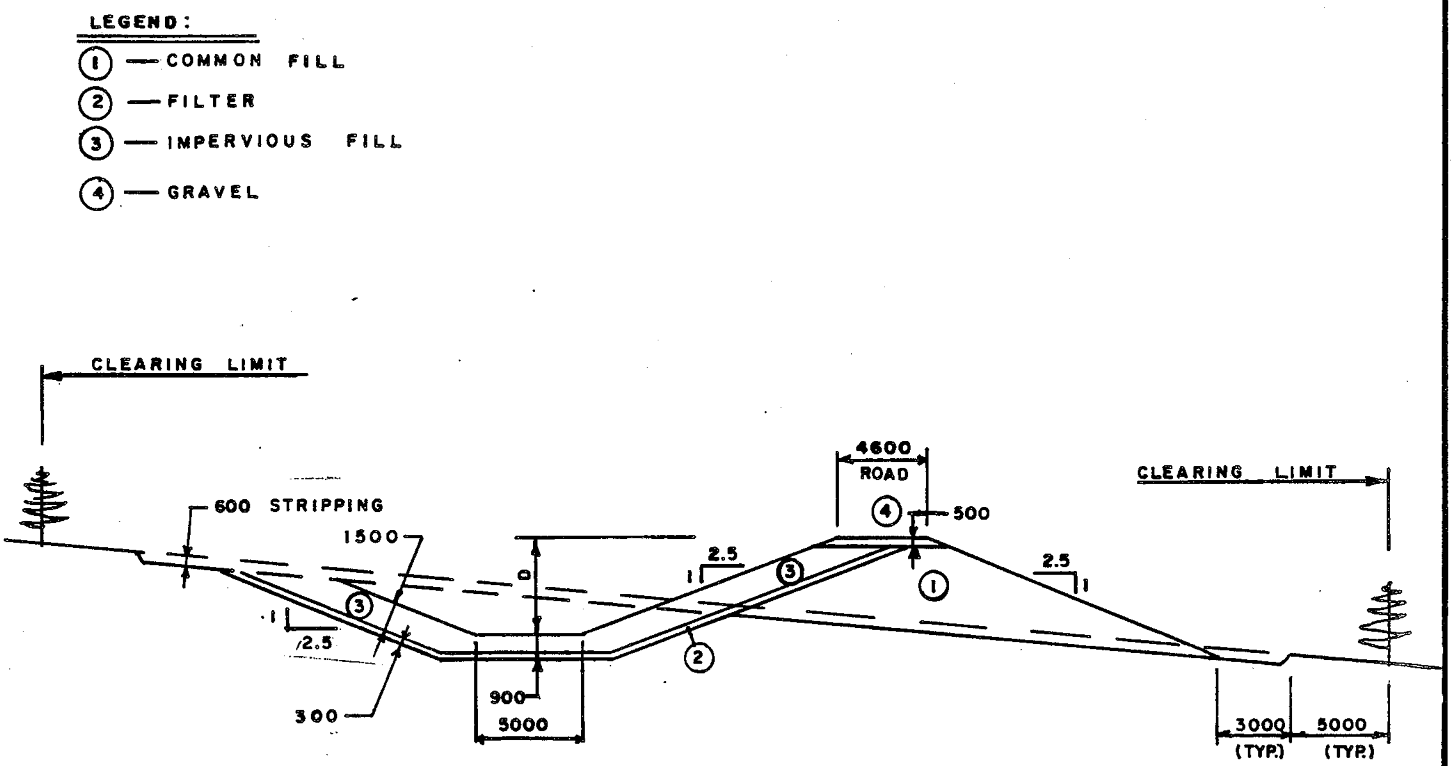
* KB = Kitty's Brook
CL = Chain Lakes
US = Upper Sheffield Brook
BB = Burnt Berry Brook
B = Barneys Brook

NOTES:

1. Design flows with storage in Chain Lakes (see Table 3.3)

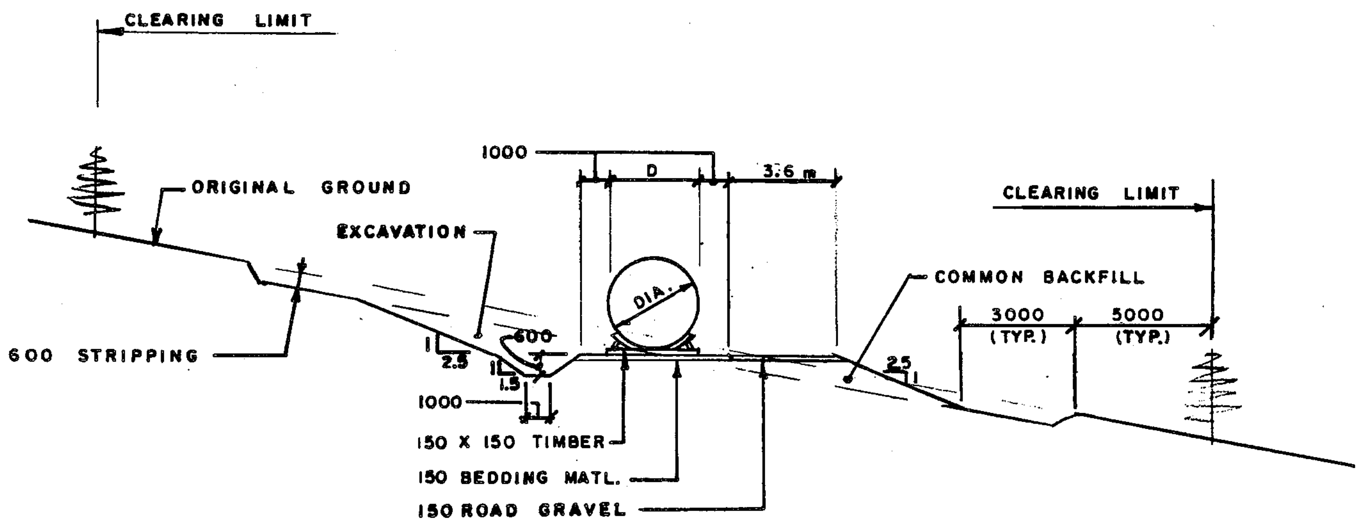
2. Energy costs are comparative only since costs do not include all fixed cost items.

3. Cost of energy for all five diversions and storage on Chain Lakes is slightly greater than without storage on Chain Lakes. The cost difference (0.4 mils/kWh) is well within the accuracy of the cost estimates. A detailed cost estimate for the two cases (with and without storage) was therefore prepared (see Part 6).



TYPICAL SECTION

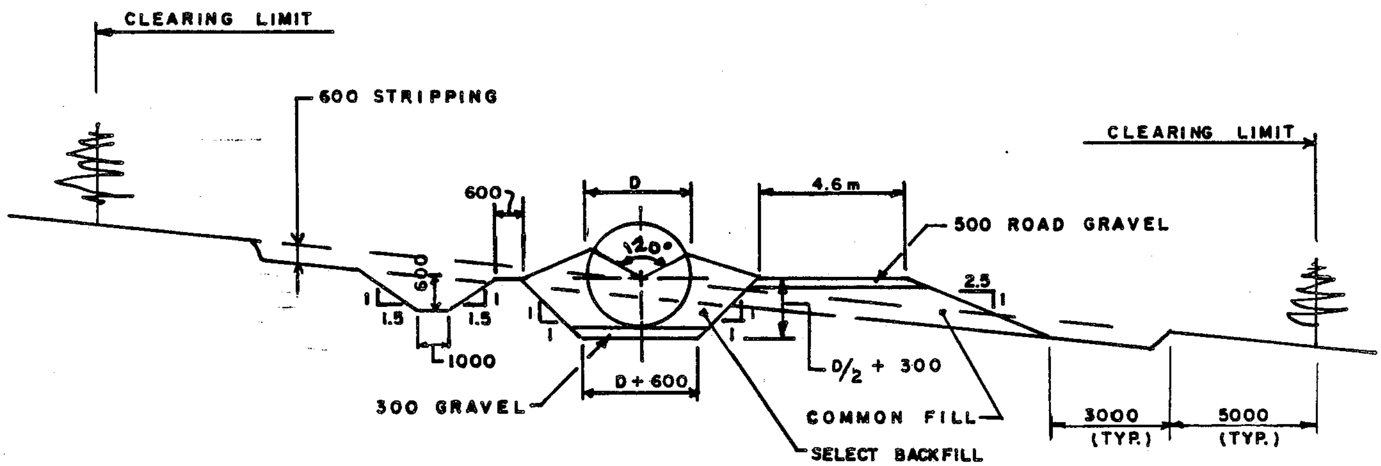
KITTYS BROOK DIVERSION
SIDEHILL CANAL
ALTERNATIVE



TYPICAL SECTION

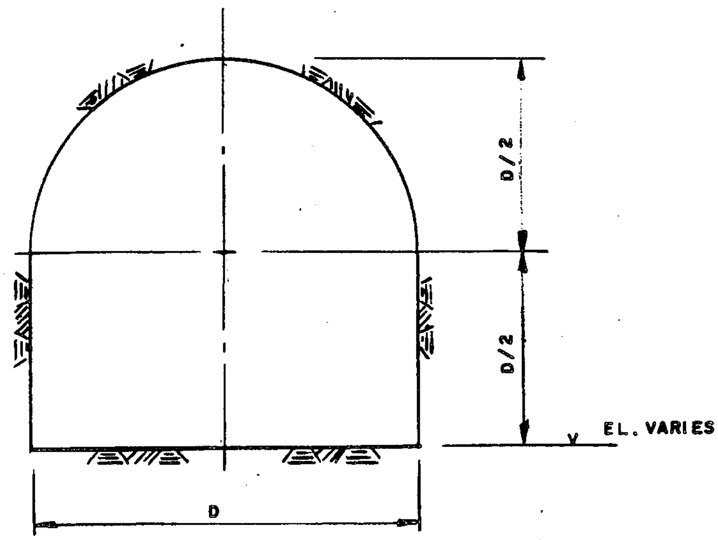
KITTYS BROOK DIVERSION
WOODSTAVE PIPE
ALTERNATIVE

FIGURE 4.3



TYPICAL SECTION

KITTYS BROOK DIVERSION
SEMI-BURIED FRP PIPE
ALTERNATIVE

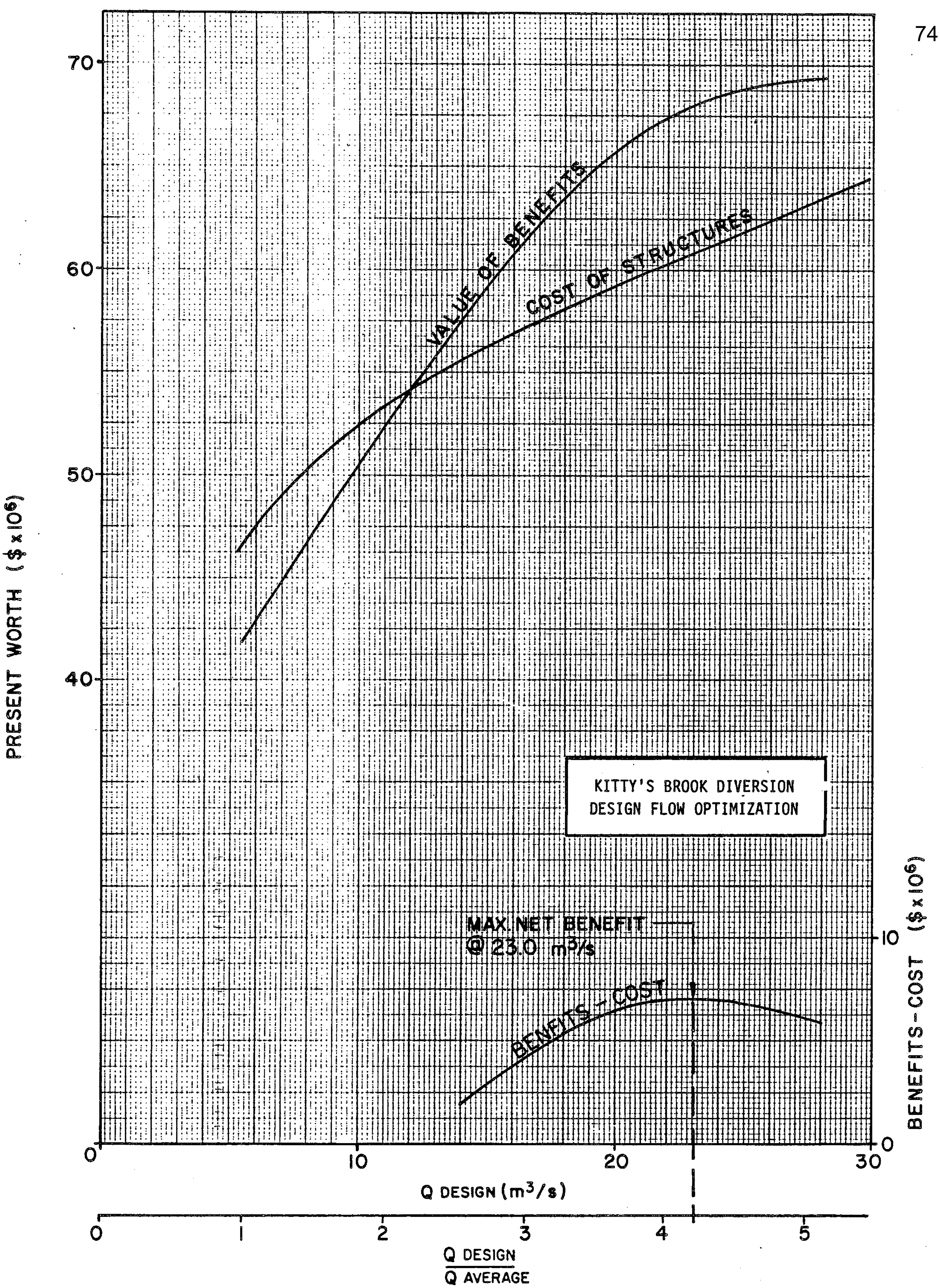


TYPICAL SECTION

KITTYS BROOK DIVERSION

TUNNEL
ALTERNATIVE

FIGURE 4.5



PRESENT WORTH ($ x 10^6)
70
60
50
40
0
VALUE OF BENEFITS
COST OF STRUCTURES
KITTY'S BROOK DIVERSION
DESIGN FLOW OPTIMIZATION
MAX. NET BENEFIT
@ 23.0 m³/s
BENEFITS - COST
BENEFITS - COST ($ x 10^6)
10
0
10
20
30
Q DESIGN (m³/s)
0
1
2
3
4
5
Q DESIGN / Q AVERAGE

FIGURE 4.6

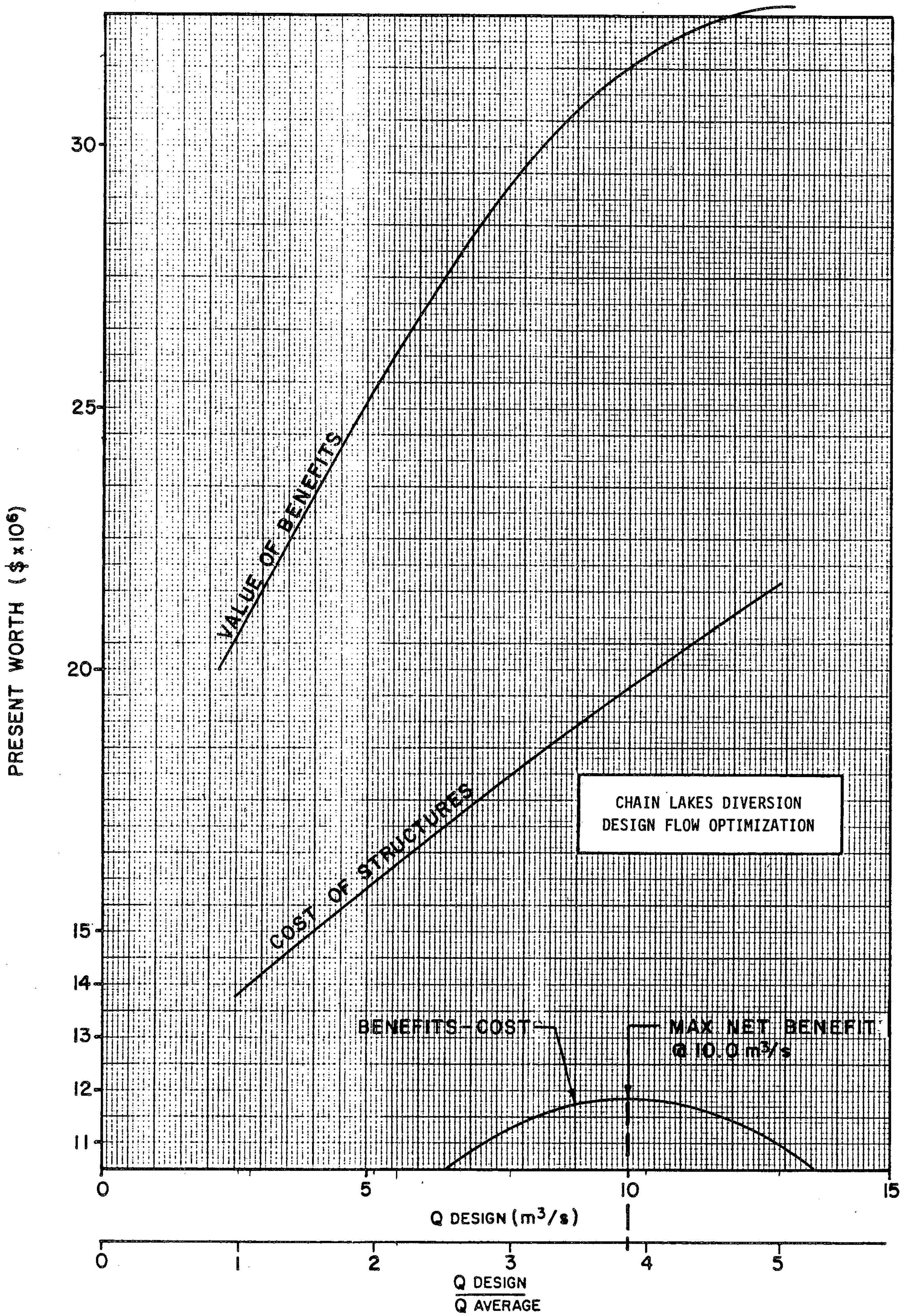
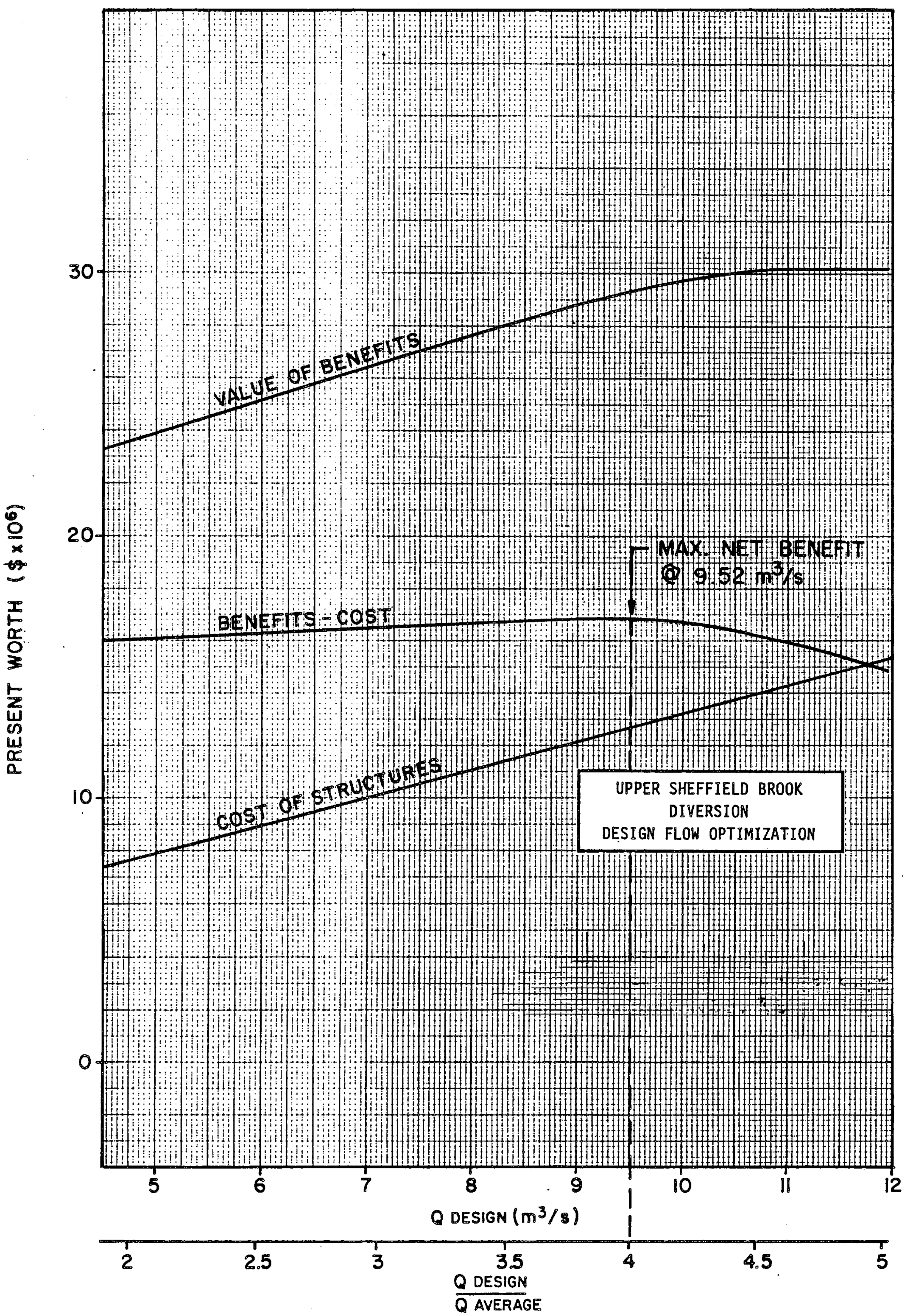
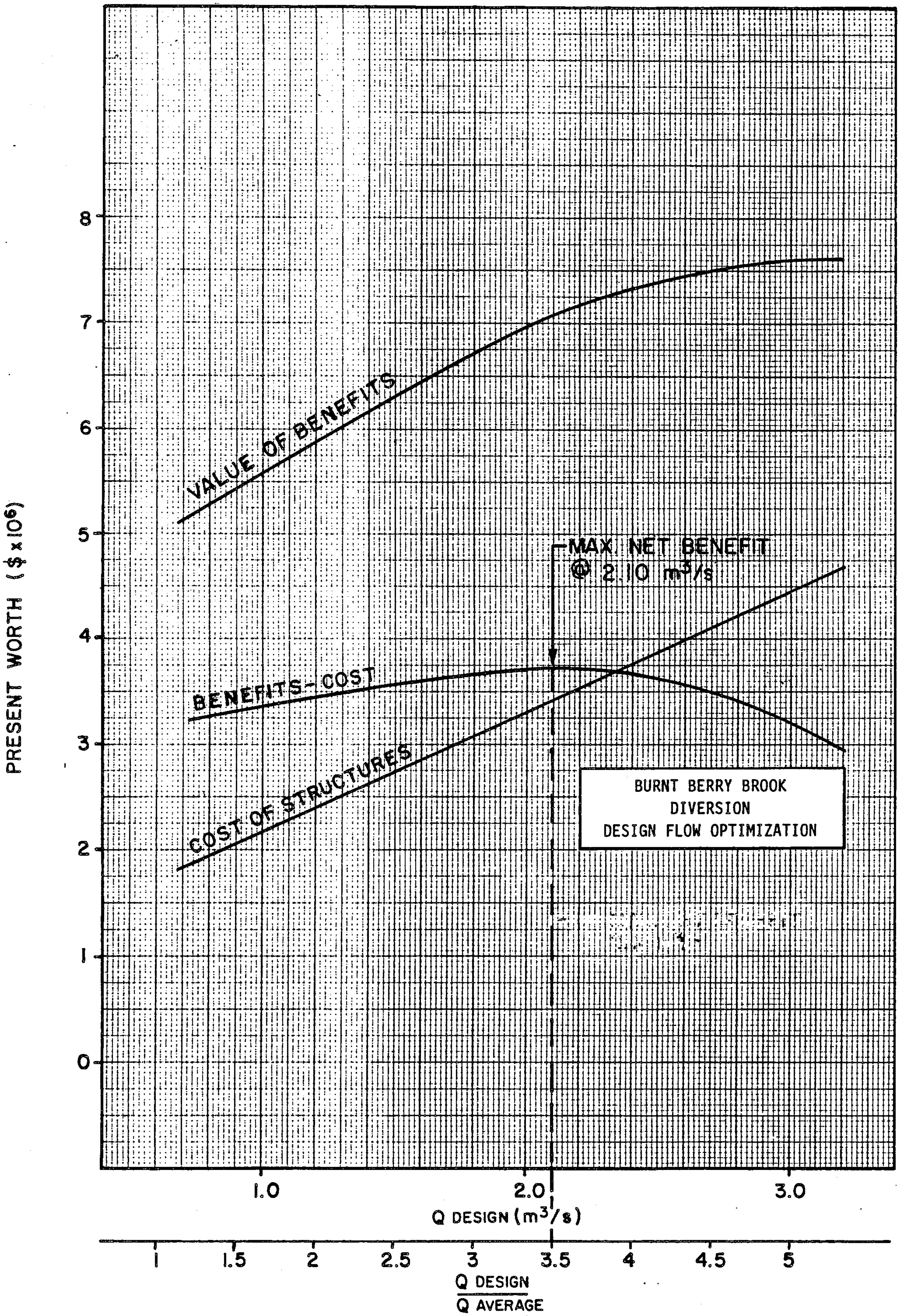
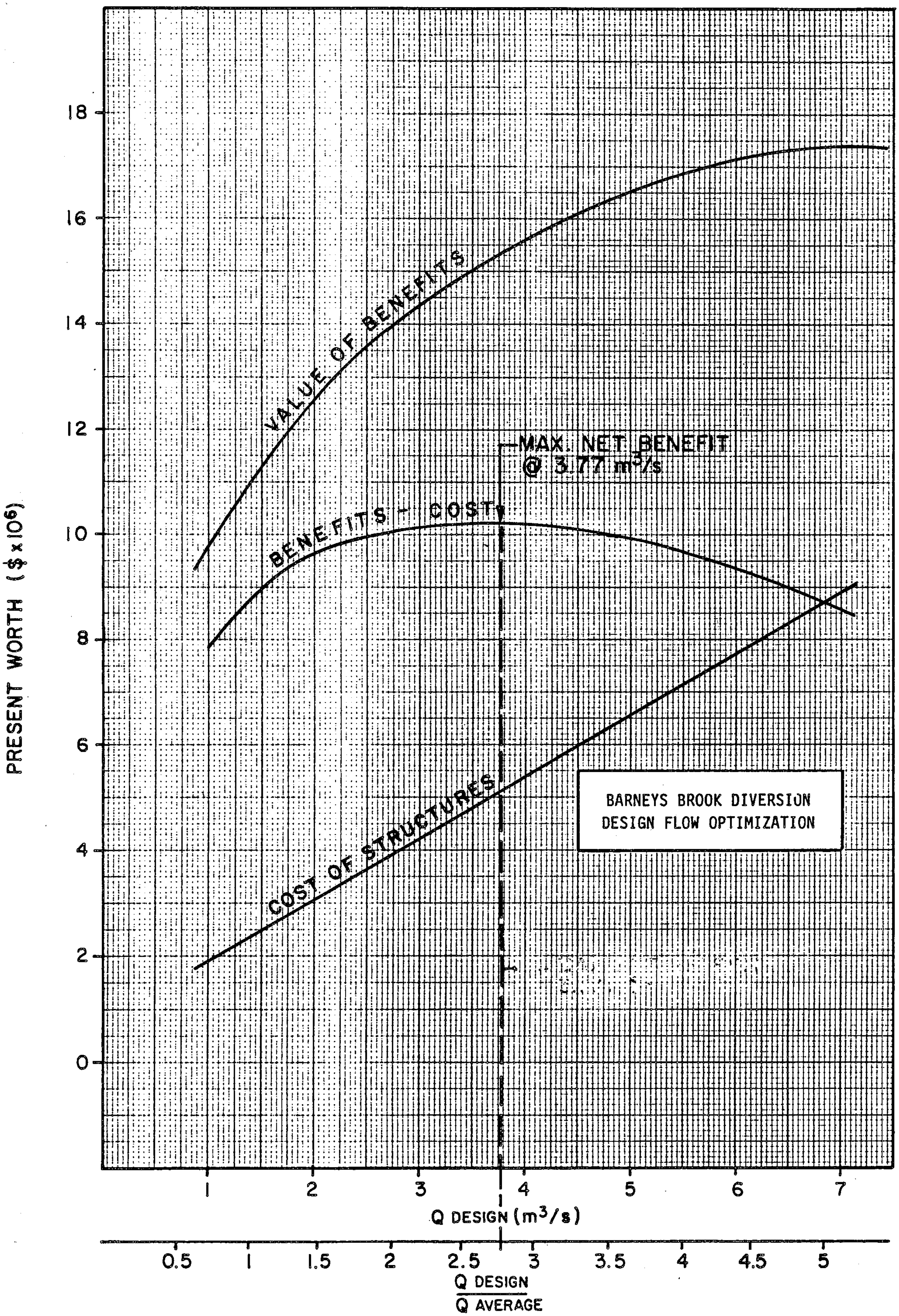

PRESENT WORTH ($ x 10^6)
30
25
20
15
14
13
12
11
VALUE OF BENEFITS
COST OF STRUCTURES
CHAIN LAKES DIVERSION
DESIGN FLOW OPTIMIZATION
BENEFITS - COST
MAX NET BENEFIT
@ 10.0 m³/s
0
5
10
15
Q DESIGN (m³/s)
0
1
2
3
4
5
Q DESIGN / Q AVERAGE

FIGURE 4.7


PRESENT WORTH ($ x 10^6)
30
20
10
0
VALUE OF BENEFITS
BENEFITS - COST
COST OF STRUCTURES
MAX. NET BENEFIT
@ 9.52 m³/s
UPPER SHEFFIELD BROOK
DIVERSION
DESIGN FLOW OPTIMIZATION
5
6
7
8
9
10
11
12
Q DESIGN (m³/s)
2
2.5
3
3.5
4
4.5
5
Q DESIGN / Q AVERAGE



PRESENT WORTH ($ x 10^6)
8
7
6
5
4
3
2
1
0
VALUE OF BENEFITS
MAX. NET BENEFIT @ 2.10 m³/s
BENEFITS - COST
COST OF STRUCTURES
BURNT BERRY BROOK
DIVERSION
DESIGN FLOW OPTIMIZATION
1.0
2.0
3.0
Q DESIGN (m³/s)
1
1.5
2
2.5
3
3.5
4
4.5
5
Q DESIGN / Q AVERAGE


PRESENT WORTH ($ x 10^6)
18
16
14
12
10
8
6
4
2
0
VALUE OF BENEFITS
BENEFITS - COST
MAX. NET BENEFIT @ 3.77 m^3/s
COST OF STRUCTURES
BARNEYS BROOK DIVERSION
DESIGN FLOW OPTIMIZATION
1
2
3
4
5
6
7
Q DESIGN (m^3/s)
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Q DESIGN / Q AVERAGE

PART FIVE

CONSTRUCTION SCHEDULE

5.1 CIVIL WORKS CONTRACT PACKAGES

It is proposed to carry out the civil works contracts for the Kitty's Brook Development under the following contract packages:

Contract No. 1

General

-- upgrading of an existing access road (extending from Route 401 to Goose Pond) and construction of a permanent access road along the top of Kitty's Brook sidehill canal dyke/pipeline berm;

-- supply and installation of a 200 man construction camp in the vicinity of Goose Pond Dam;

Kitty's Brook Diversion

-- Construction of a 17.6 km long sidehill canal and 0.8 km long pipeline;

-- construction of an energy dissipater structure at the upstream end of the sidehill canal;

Contract No. 2

General

-- upgrade existing woods access road (off Trans Canada Highway at Birchy Narrows) and construction of approximately 10 km of access road to Kitty's Brook Dam via Chain Lakes Dam;

-- supply and installation of a 175 man construction camp in the vicinity of Chain Lakes diversion dam;

Kitty's Brook Diversion

-- construction of diversion dam, intake, spillway channel and overflow weir;

Chain Lakes Diversion

-- construction of 2.7 km sidehill canal, 1.9 km excavated channel, dykes, diversion dam and spillway, and railway re-alignment and bridge;

-- construction of storage dam and control outlet structure at the outlet of Chain Lakes;

5.1 CIVIL WORKS CONTRACT PACKAGES (Cont'd)

It should be noted that the interface of Contract No. 1 with Contract No. 2 would be at the downstream side of the intake where the FRP pipeline joins the embedded steel pipe.

Contract No. 3

General

-- construction of 26 km of access road to three diversions;

-- supply and installation of a 175 man construction camp to serve a workforce for three diversions. Camp to be located 2 km east of Upper Sheffield dam and spillway;

Chain Lakes Diversion

-- construction of storage dam at the eastern end of Chain Lakes;

Upper Sheffield Diversion

-- construction of diversion dam and spillway;

-- excavation of an 8 km long canal;

Burnt Berry Diversion

-- construction of dykes, diversion dam and spillway;

-- excavation of canal;

Barneys Brook Diversion

-- construction of diversion dam and spillway;

-- excavation of canal.

5.2 CONSTRUCTION SCHEDULE

A construction schedule has been developed for the project and is shown on Plate 14. It reflects a 3 1/2 year program starting with a feasibility study in the summer of year 1. The schedule reflects construction work carried out under the three civil works contracts with Contract No. 1 and No. 2 to be awarded simultaneously.

5.2 CONSTRUCTION SCHEDULE (Cont'd)

Contract No. 1

Work on the Kitty's Brook sidehill canal would commence in year 2 as soon as the access is provided to the outlet of the canal. The construction camp would be completed as early as possible in year 2 to provide housing for the workforce in the following years on the sidehill canal, pipeline and energy dissipater. Work is scheduled to be completed by the end of year 4.

Contract No. 2

Work on Kitty's Brook dam and intake would be started early in year 3 after an alternate access has been completed to this site. The dam is scheduled for completion by the end of October, year 4.

During years 3 and 4, work would also be carried out on the Chain Lakes structures to allow completion of the sidehill canal and excavated channel in the fall of year 4. Closure of the unwatering conduit through the Chain Lakes diversion dam would not be scheduled until all downstream works are completed. Construction of Chain Lakes storage dam is scheduled to ensure that the dam is above existing water levels before closure of the Chain Lakes diversion dam unwatering conduit.

Contract No. 3

The construction of access roads (extending from Contract No. 2 access) as well as partial completion of the construction camp are scheduled to be carried out in year 3. Work on the three upstream diversions of Upper Sheffield, Burnt Berry and Barneys Brook are scheduled to be carried out in year 4. The final closure of each of the dams of these diversions is scheduled to be carried out upon completion of the canals and downstream works.

For this study no construction activities were scheduled for the winter seasons. In any future studies on this project, the economics of winter construction versus an earlier completion date and subsequent reduction in IDC costs should be reviewed.

Also, since downstream diversions must be completed in sequence, to allow the next upstream diversion to be added to the system, scheduling completion of downstream diversions in the earliest possible sequence would result in early benefits through energy generation and reduced IDC.

PART SIX

COST ESTIMATE

6.1 BASIS OF COST ESTIMATE

Cost estimates were prepared at three key points in this study. The first estimate was carried out to select the preferred layout for each diversion and included estimates for each structure and some preliminary optimization within each diversion to determine the most cost effective structure layouts. The second estimate was prepared for the preferred layout for each diversion. This included more detailed estimates for various design flows and preparation of cost curves for each structure and each diversion. These were then used for the optimization of each diversion and the development as a whole. The third estimate was then prepared for the optimized development.

The three estimates were prepared based on quantities calculated from 1:2,500 mapping where this was available and from 1:50,000 mapping otherwise. Assumptions were made for the depth of stripping and the rock/overburden interface.

6.2 UNIT RATES

The unit rates used in estimating the civil works have been derived from experience on similar works carried out in recent years and updated to anticipated current prices for the work.

Budget prices received from suppliers of FRP and woodstave pipes were used in estimating costs for these items in alternative water conveyance systems in the first estimate.

The costs are provided in mid-1986 dollars and escalation and interest during construction have been calculated separately. The construction schedule has been shown from year 1 to year 4 however, for purposes of this report, the escalation and IDC have been based on the project being carried out between June, 1987 and December, 1990.

6.3 CAPITAL COST ESTIMATE

A summary of the capital cost estimates for the selected development layout with and without storage in Chain Lakes are provided in Tables 6.1 and 6.2 respectively.

The following points are noted:

6.3 CAPITAL COST ESTIMATE (Cont'd)

-- No costs have been allowed for fish compensation flows or structures or for any special environmental considerations.

-- No costs have been included to bring the project to feasibility level status.

Temporary support costs have been included as 10.5% of direct costs and would include the costs for construction camp and services, road maintenance, vehicles and supplies, field office and laboratory expenses, field board and lodging, and site communication.

Engineering and project management costs have been included as 15% of direct costs and would include the cost of management, office design and field supervision of construction including office expenses.

Owner's costs have been included as 3% of direct costs.

Cost and cash flows are included in Appendix I. These have been prepared on a quarterly basis and provide the data on which escalation and interest during construction have been calculated.

Although we have included for a reasonable coverage for unforseens in the contingency, we consider that this pre-feasibility estimate has an accuracy of $\pm$ 25% for the layout presented.

6.4 COST OF ENERGY

Table 6.3 provides a summary of the costs for diversion of Kitty's Brook alone and for each scheme of development as additional drainage areas are added. The costs are presented for two cases: with and without storage in Chain Lakes.

The annual charge rate for calculating annual operating costs was assumed to be 12.133%. This includes interest, insurance and interim replacement costs.

TABLE. 6.1

COST ESTIMATE (X $1000)

(WITH STORAGE IN CHAIN LAKES)

DIVERSION	STRUCTURE	COSTS STRUCTURE	COSTS DIVERSION
Kitty's Brook	Access Road	345	
	Reservoir Clearing	100	
	Diversion Dam	4,194	
	Spillway/Channel	2,365	
	Intake	656	
	FRP Pipe	2,670	
	Energy Dissipater	250	
	Canal	32,090	
	Subtotal		42,670
Chain Lakes	Access Road	1,450	
	Reservoir Clearing	50	
	Diversion Dam/Spillway	872	
	Dykes	1,683	
	R R. Realignment/ Bridge	550	
	Excavated Channel	2,580	
	Sidehill Canal	3,740	
	Storage Dams	5,430	
	Control Outlet	1,000	
	Subtotal		17,355
Upper Sheffield	Access Road	330	
	Diversion Dam	986	
	Canal	4,519	
	Subtotal		5,835
Burnt Berry	Access Road	270	
	Diversion Dam	645	
	Dykes	55	
	Canals	120	
	Subtotal		1,090
Barneys	Access Road	330	
	Diversion Dam	70	
	Canal	610	
	Subtotal		1,010

TABLE 6.1 (Cont'd)

COST ESTIMATE (X $1000)

(WITH STORAGE IN CHAIN LAKES)

Subtotal - Direct Costs	67,960
Temporary Support	7,136
Management and Engineering	10,194
Owner's Costs	2,039
Contingencies	18,280
Subtotal	105,609
Escalation	14,410
Interest During Construction	15,331
TOTAL	135,350

TABLE. 6.2

COST ESTIMATE (X $1000)

(NO STORAGE IN CHAIN LAKES)

DIVERSION	STRUCTURE	COSTS STRUCTURE	COSTS DIVERSION
Kitty's Brook	Access Road	345	
	Reservoir Clearing	100	
	Diversion Dam	4,194	
	Spillway/Channel	2,365	
	Intake	915	
	FRP Pipe	3,290	
	Energy Dissipater	300	
	Canal	36,100	
	Subtotal		47,609
Chain Lakes	Access Road	1,450	
	Reservoir Clearing	50	
	Diversion Dam/Spillway	872	
	Dykes	1,683	
	R R. Realignment/ Bridge	600	
	Excavated Channel	3,080	
	Sidehill Canal	4,360	
	Storage Dams	--	
	Control Outlet	--	
	Subtotal		12,095
Upper Sheffield	Access Road	330	
	Diversion Dam	986	
	Canal	4,519	
	Subtotal		5,835
Burnt Berry	Access Road	270	
	Diversion Dam	645	
	Dykes	55	
	Canals	120	
	Subtotal		1,090
Barneys	Access Road	330	
	Diversion Dam	70	
	Canal	610	
	Subtotal		1,010

TABLE 6.2 (Cont'd)

COST ESTIMATE (X $1000)

(NO STORAGE IN CHAIN LAKES)

Subtotal - Direct Costs	67,639
Temporary Support	7,102
Management and Engineering	10,146
Owner's Costs	2,029
Contingencies	18,796
Subtotal	105,711
Escalation	14,360
Interest During Construction	15,628
TOTAL	135,700

TABLE 6.3

SUMMARY OF COSTS

CASE	SCHEME	TOTAL CAPITAL COST (1)	AVERAGE ANNUAL ENERGY (gWh)	ENERGY COST (mils/kWh)
No Storage in Chain Lakes	KB	$ 78,960,000	90	106.4
	KB+CL	107,008,000	129	100.6
	KB+CL+US	123,437,000	169	88.6
	KB+CL+US+BB	128,364,000	179	87.0
	KB+CL+US+BB+B	135,700,000	200	82.3
With Storage in Chain Lakes	KB	$ 78,960,000	90	106.4
	KB+CL	100,746,000	129	94.8
	KB+CL+US	119,000,000	169	85.4
	KB+CL+US+BB	124,850,000	179	84.6
	KB+CL+US+BB+B	135,350,000	200	82.1

NOTES:

1. Escalation and IDC were originally calculated for the schemes with all five diversions. However, for schemes with less diversions, the escalation and IDC were pro-rated from the original figures.

REFERENCES

REFERENCES

(1) "Hydrological Design Methodologies for Small Scale Hydro at Ungaged Sites - Phase I - Prefeasibility Level" by Acres Consulting Services Limited for Department of Environment - IWD, and Department of Mines and Energy. Ottawa (1984).

(2) "Floods and Reservoir Safety, An Engineerng Guide". Institution of Civil Engineers, London. 1978

(3) "Regional Flood Frequency Analysis for the Island of Newfoundland" by U. S. Panu, D. A. Smith and D. C. Ambler, Canada-Newfoundland Flood Damage Reduction Program, St. John's (Nfld) 1984.

APPENDIX

Prepared by: L. Matchim

Capital Cost Estimate & Cash Flow Requirements
Kitty's Brook Diversion (w/o Chain Lakes)

Prepared: 86-10-31

Escln %	86/87=	.0451	87/88=	.0395	88/89=	.0310
	89/90=	.0330	90/91=	.0335	(Est.Base: Jul/86)	
I.D.C.% =	12.682	Annual	.0100	Mthly	.0303	Qtrly

Period	Direct Cost	Escln	Total Cost	I.D.C.	Total Proj.	Cash Flow	Accum Cash
1988 Jan	120	10	130	0	130	0	0
Feb	360	30	390	0	390	130	130
Mar	460	30	490	0	490	390	520
Apr	520	40	560	10	570	490	1,020
May	520	40	560	10	570	560	1,580
Jun	390	30	420	20	440	560	2,170
Jul	2,605	220	2,825	20	2,855	420	2,610
Aug	4,660	430	5,090	30	5,120	2,825	5,475
Sep	4,610	440	5,050	60	5,110	5,090	10,525
Oct	4,410	430	4,840	110	4,950	5,050	15,785
Nov	2,570	260	2,830	160	2,990	4,840	20,735
Dec	1,120	120	1,240	210	1,450	2,830	23,825
Total 1988	22,345	2,090	24,435	630	25,065	23,195	
1989 Jan	120	10	130	240	370	1,240	25,305
Feb	120	10	130	250	380	130	25,685
Mar	120	10	130	260	390	130	26,075
Apr	120	10	130	260	390	130	26,465
May	2,390	280	2,670	270	2,940	130	26,865
Jun	4,360	520	4,880	270	5,150	2,670	29,805
Jul	4,595	570	5,165	300	5,465	4,380	34,985
Aug	6,450	810	7,260	350	7,610	5,165	40.500
Sep	6,950	900	7,850	410	8,260	7,260	48,170
Oct	6,930	350	7,280	490	7,770	7,850	56,510
Nov	4,935	550	5,485	570	6,055	7,280	64,360
Dec	1,320	180	1,500	640	2,140	5,485	70,185
Total 1989	37,810	4,800	42,610	4,310	46,920	42,350	
1990 Jan	60	10	70	700	770	1 500	72,685
Feb	60	10	70	720	800	70	73,485
Mar	60	10	70	730	800	70	74,285
Apr	120	20	140	740	880	70	75,095
May	2,060	320	2,380	750	3,130	140	75,985
Jun	7,350	1,150	8,500	770	9,270	2,380	79,135
Jul	8,136	1,300	9,436	800	10,236	8,500	88,425
Aug	8,270	1,330	9,600	890	10,490	9,436	98,761
Sep	8,323	1,390	9,713	990	10,703	9 600	109,351
Oct	5,651	960	6,611	1,098	7,709	9,713	120,162
Nov	4,220	730	4,950	1,210	6,160	6,611	127,983
Dec	1,042	180	1,222	1,280	2,502	4,950	134,213
Total 1990	45,332	7,430	52,762	10,688	63,450	53,040	
1991 Jan	120	20	140	0	140	1,222	135,435
Feb	60	10	70	0	70	140	135,575
Mar	45	10	55	0	55	125	135,700
Total 1991	225	40	265	0	265	1,487	
Total Proj	105,712	14,360	120,072	15,628	135,700	120,072	

Prepared by: L. Matchim

Capital Cost Estimate & Cash Flow Requirements
Kitty's Brook Diversion (incl. Chain Lakes)
Prepared: 86-10-31

Escln %	86/87= .0451	87/88= .0395	88/89= .0310		
	89/90= .0330	90/91= .0335	(Est.Base: Jul/86)		
I.D.C.% =	12.683 Annual	.0100 Mthly	.0303 Qtrly		

Period	Direct Cost	Escln	Total Cost	I.D.C.	Total Proj.	Cash Flow	Accum Cash
1988 Jan	120	10	130	0	130	0	0
Feb	360	30	390	0	390	130	130
Mar	460	30	490	0	490	390	520
Apr	520	40	560	10	570	490	1,020
May	520	40	560	10	570	560	1,590
Jun	390	30	420	20	440	560	2,170
Jul	2,605	230	2,835	20	2,855	420	2,610
Aug	4,260	390	4,650	30	4,680	2,835	5,475
Sep	4,510	430	4,940	60	5,000	4,650	10,185
Oct	4,210	410	4,620	100	4,720	4,940	15,225
Nov	2,370	240	2,610	150	2,760	4,620	19,995
Dec	1,220	130	1,350	200	1,550	2,610	22,805
Total 1988	21,545	2,010	23,555	600	24,155	22,205	
1989 Jan	120	10	130	230	360	1,350	24,385
Feb	120	10	130	240	370	130	24,755
Mar	120	10	130	250	380	130	25,135
Apr	120	10	130	250	380	130	25,515
May	2,490	290	2,780	260	3,040	130	25,905
Jun	4,260	510	4,770	260	5,030	2,780	28,945
Jul	4,886	600	5,486	290	5,776	4,770	34,005
Aug	6,250	790	7,040	340	7,380	5,486	39,331
Sep	6,975	900	7,875	400	8,275	7,040	47,271
Oct	6,405	850	7,255	480	7,735	7,875	55,626
Nov	4,655	630	5,285	560	5,845	7,255	63,141
Dec	1,320	180	1,500	640	2,140	5,285	69,366
Total 1989	37,721	4,790	42,511	4,200	46,711	42,361	
1990 Jan	60	10	70	690	760	1,500	71,556
Feb	60	10	70	720	790	70	72,346
Mar	60	10	70	720	790	70	73,136
Apr	120	20	140	730	870	70	73,936
May	1,980	300	2,280	740	3,020	140	74,816
Jun	6,280	990	7,270	750	8,020	2,280	77,846
Jul	8,931	1,430	10,361	790	11,151	7,270	85,906
Aug	8,450	1,380	9,830	870	10,700	10,361	97,137
Sep	8,623	1,440	10,063	980	11,043	9,830	107,947
Oct	6,026	1,020	7,046	1,080	8,126	10,063	119,090
Nov	4,090	710	4,800	1,190	5,990	7,046	127,326
Dec	1,370	240	1,610	1,271	2,881	4,800	133,397
Total 1990	46,050	7,560	53,610	10,531	64,141	53,500	
1991 Jan	120	20	140	0	140	1,610	135,007
Feb	60	10	70	0	70	140	135,147
Mar	113	20	133	0	133	203	135,350
Total 1991	293	50	343	0	343	1,953	
Total Proj	105,609	14,410	120,019	15,331	135,350	120,019	

June 30, 1966

File: 70

Mr. F.E. Newbury
Deputy Chairman
Newfoundland and Labrador
Power Commission
P.O. Box 396
St. John's, Newfoundland

Hydro Power Studies
Upper Lloyds River
Diversion

Dear Mr. Newbury,

In accordance with the instructions contained in your letter of March 25th, 1966, we have completed our studies of the Upper Lloyds River Diversion on a similar basis to the previous study of the Victoria Lake Diversion. We are submitting this letter report to you now in order that you will have information on this Diversion in advance of the main report on Stage II of the Bay D'Espoir Development, which, of course, will include the Upper Lloyds Diversion.

1. GENERAL DESCRIPTION

(a) Physical Features

The Upper Lloyds River Diversion would direct the flow of 184 sq. miles of the Lloyds River drainage basin into the Victoria River drainage basin and ultimately into the Salmon River basin for utilization at the Bay D'Espoir Development.

..... 2.

Appendix I entitled "General Plan" is enclosed. The principal structures, shown on the drawing, are:

- A dam 70 feet high, located on the Lloyds River about one mile below King George IV Lake which would raise the natural level of the lake from its present elevation of 1134 to a Full Supply Level of 1165 before spilling would occur.

- A diversion canal, extending along a depression and cutting through the height of land between King George IV Lake and Wood Lake in the Victoria River drainage basin.

(b) Schedule

Diversion of flow from Lloyds River would aid in filling the dead storage portion of the Victoria Lake reservoir and the two projects should be finished in the same construction year.

Road construction should be carried out during 1968, if the dam construction and canal excavation are to be completed in 1969, and diversion would begin as soon as the reservoir level reached the level of the uncontrolled canal invert.

2. INFORMATION USED

(a) Structure locations have been mapped to a scale of 1" = 400' with 10 foot contours. Vertical control of these maps was established by altimetry during April 1966. Horizontal control was derived from the 1:50,000 maps published by the Department of Mines and Technical Surveys.

(b) Photo interpretation of probable depths to bedrock, overburden types and possible sources of construction materials at each structure site was provided by Mr. L.A. Rivard of British Newfoundland Exploration Ltd.

3. HYDROLOGY

No streamflow gauging has been carried out on the Upper Lloyds River and no precipitation records have been kept on the drainage basin above Red Indian Lake.

The upper basin borders the western end of the Victoria River basin and is between the Lewaseechjeech Brook and Grey River basins. The Shawinigan Engineering Company studied the records available in neighbouring water sheds and determined that the probable long term average flow of the Victoria Lake basin is 3.0 cfs/sq. mile and provided synthesized monthly run-off figures. In this study it has been assumed that the run-off of the Upper Lloyds River drainage basin is similar to that of the Victoria Lake basin.

4. FLOWS

The drainage area above the proposed dam is 184 sq. miles. Using the long term average flow calculated for Victoria Lake, the diversion flow would be 550 cfs. With no storage provided, studies of combinations of canal depths and dam heights indicate that for minimum capital cost, structures settings would be:

Dam Crest	1175	Full Supply Level 1165
Canal Invert	1150	

Dead storage (between present lake level of 1134 and 1150) would be 4.4 BCF.

Studies indicate that if the diversion culvert in the cofferdam were closed in September, in a minimum flow year the dead storage would be filled by the end of January, or in an average year by the end of December. At this time the Upper Lloyds flow would begin passing into the Victoria Lake basin.

The surface area (and storage capacity) of the lake is small in relation to the Victoria Lake or the Grey Reservoirs and the provision of storage would be uneconomic at this location.

5. DESCRIPTION OF STRUCTURES

(a) Roads

At present an existing logging road extends from Lake Ambrose slightly beyond the west end of Victoria Lake. This road will be used for the construction of the White Bear and Victoria Lake Diversions to a point about 7 miles from its western end. An access road would be built from the existing road to the damsite, with an additional short section of road leading to the canal. The approximate length of the access roads would be 19.5 miles.

The Department of Highways is building a highway from Bottom Brook to Buchans north of the site at the present time. An alternative access road could be extended from the highway if it is completed in time, and the length would be approximately the same as the road from Victoria Lake.

(b) Lloyds River Dam

The Lloyds River Dam would have a crest length of 1350 feet at crest elevation of 1175. Maximum height above the river bed would be 70 feet.

Appendix II "Lloyds River Dam" shows a plan and cross section of the dam at maximum height, and includes the cofferdam.

Photo interpretation indicates that overburden depth should be a maximum of 8 feet on the south bank, but may be as much as 20 feet on the north bank, and these depths have been used for estimating.

..... 5.

The dam would be of rock fill with a vertical central impervious core and a cut-off trench to rock, except in the upper portions of the north bank abutment where deep overburden was assumed.

The total volume of the dam would be 150,000 cu. yds. made up as follows:

Compacted rock fill	74,000 cu. yds.
Dumped rock fill (cofferdam)	8,000 cu. yds.
Impervious core-rolled till	21,000 cu. yds.
Transitions and Filters	47,000 cu. yds.

No difficulty is anticipated in finding suitable rock quarry sites in the vicinity of the dam and a source of impervious till is located nearby. However, no adequate source of pervious material is indicated near the dam site and allowance has been made in the estimate for overhaul from a long esker at the upper end of the diversion canal with a construction road across an arm of King George IV Lake.

The cofferdam would be of rock fill which is later incorporated in the main dam upstream face. A 9 foot diameter diversion conduit would bypass flow during construction and would be sealed after the dam has reached elevation 1150.

(c) Lloyds River Spillway

The spillway would be located on the south bank. It would be a concrete overflow structure 250 feet long with crest elevation at 1165. At maximum flood level of 1171, the discharge would be 10,500 cfs.

..... 6.

The required discharge capacity is made up as follows:

Peak flood inflow (run-off)	17,500 cfs.
Flood routing effect (approx.)	2,000 cfs.
Diversion canal	5,000 cfs.
Required spillway capacity	10,500 cfs.

The peak flood inflow has been derived from Graph 4 of Report No. SM-4-65 on the Bay D'Espoir Development.

The entire spillway structure of overflow weir, wing walls and downstream apron would require 1350 cu. yds. of concrete.

No provision has been made for passing flow down the Lloyds River after construction, and if this is required a gated spillway or low level gated conduit would be required.

(d) <u>King George IV Lake - Wood Lake Diversion Canal</u>

The diversion canal would extend from the south-east side of King George IV Lake along a valley in a south easterly direction, cut through the height of land and drop into Wood Lake on the upper reaches of the Victoria River.

Topography at 1" = 400' does not extend to the upper end of the canal, but the estimate assumed that at least 2300 feet of channel improvement would be required between a series of small lakes. The length of the canal proper would be 1720 feet and would involve the excavation of about 41,000 cu. yds. of material, about 90% being rock excavation. The total excavation including channel improvement, would be about 114,000 cu. yds.

..... 7.

The canal has been designed to discharge 2800 cfs at Full Supply Level of 1165. This flow represents the maximum mean five day flow in the dry period, using the synthesized run-offs for Victoria Lake from the SECo Hydrology Report. As indicated in Section 4, the long term average flow would be 550 cfs.

No estimate has been made of the cost for the enlargement of the Victoria Lake, Granite Lake or Ebbegunbaeg canals to accommodate the increased flow due to the Lloyds River diversion.

6. CONSTRUCTION SCHEDULE

The access roads to the structure sites would have to be built in 1968, if the structures are to be completed during the summer of 1969. Work could begin on the long south bank section, including the spillway, which is well above river level, as soon as weather permitted. The cofferdam and diversion conduit would be started in late May or early June as soon as the spring flood was passed. Even if no work were done before mid-June, a four month placing schedule would require less than 40,000 cu. yds. per month. A seven month excavation schedule for the canal would require about 16,000 cu. yds. per month, both operations to end in October 1969.

7. ENERGY

The results of power studies to determine the energy available at Bay D'Espoir from the Lloyds River Diversion are presented in Table 1.

These studies were completed some months ago at which time it was necessary to make an assumption of the total volume of storage in the system when the Upper Lloyds River is diverted based on conditions as they existed at that time. It now seems likely that rather more storage than 93 BCF will be provided and consequently the estimate of firm energy may be low. The estimate of average total energy, however, will only be slightly affected.

It should also be noted that information now at hand indicates that the installed capacity at Bay D'Espoir with Units 1 to 5 may be 420 MW rather than the 412 MW assumed in the power studies. This will result in slightly less spill. Furthermore, it should be mentioned that when Unit 6 is added, the average annual spill will be virtually eliminated (less than 20 cfs.)

The studies were made for the 10 year period, October 1st, 1955 to September 30th, 1965, which embraces the minimum one year, two year and three year sequences of flow recorded on relevant rivers in Newfoundland and also a period of high run-off from 1963 - 1965. The average flow of the contributing drainage areas in the 10 year period was 97% of the long term average.

The following assumptions and conditions were assumed in the power studies:

- An overall plant efficiency of 84% which includes an operating efficiency of 95%.

- During periods of secondary energy generation, the full plant capability can be utilized.

- No allowance has been made for possible storage releases in connection with fish conservation, logging operations and compensation water or for dam leakage.

- No allowance has been made for compensation to Price (Newfoundland) Ltd., for loss in generation at their Grand Falls and Bishops Falls plants as a result of the Diversion.

- The total storage in the system would be at the Operating Rule Curve level at the commencement of the period.

If an arbitrary reduction of 2% (10 cfs) is allowed for secondary water uses and dam leakage, the long term benefit of the Lloyds River Diversion at the Bay D'Espoir Development would be:

..... 9.

Mr. F.E. Newbury June 30, 1966

Annual firm energy	152×10^6 Kwh
Average annual secondary energy	14×10^6 Kwh
Average annual total energy	166×10^6 Kwh

8. ESTIMATE OF CAPITAL COST

The Lloyds River Diversion is estimated to cost $3,900,000. as follows:

Structures at King George IV Lake	$3,735,000.
Escalation if constructed in 1969	$ 165,000.
	$3,900,000.

A detailed breakdown of the cost of the structures is enclosed with this letter. Contingencies to cover increases in quantities and unforeseen construction difficulties have been applied to the individual structures. In this estimate no allowance has been made for the following:

- Clearing of flooded areas or loss of merchantable timber.
- Additional capacity required in the downstream canals to accommodate the Lloyds River flow.
- Facilities which might be required at the dam for log driving.

9. COST OF ENERGY

The annual fixed charges of the Lloyds River diversion are estimated to be 0.0755 x $3,900,000 = $295,000 based on the following rates:

..... 10.

Mr. F.E. Newbury June 30, 1966

Cost of Capital	7.00%
Depreciation (50 years)	0.25%
Interim Replacement	0.20%
Insurance	0.10%
	7.55%

Direct operating and maintenance costs are estimated to be about $20,000 which does not include system operation, maintenance and administration. The total annual charges are estimated to be $315,000.

The cost of energy made available by the Lloyds River Diversion at the Bay D'Espoir Terminal Station would be:

Gross average annual energy	166×10^6 Kwh
Annual charges	$315,000.
Cost of energy	1.90 mills/Kwh

As indicated in Section 7, about 90% of this energy would be firm.

We trust that the information contained in this letter report will be sufficient for your present needs. We shall, of course, be pleased to discuss it with you whenever you wish.

Yours very truly,

D R Nancarrow

DRN/GSW/lm

D.R. Nancarrow
Project Engineer

BENEFIT OF THE LLOYDS RIVER DIVERSION - ENERGY GENERATION AT BAY D'ESPOIR DEVELOPMENT

TABLE 1.

- Drainage areas utilized : Salmon, Grey, White Bear, Victoria Lake and Lloyds River
- Bay D'Espoir Capacity Units 1 - 5 - (412 MW)

Power Study	Period of Study	Duration of Critical Low flow period Months	Total Storage Utilized BCF	Total Storage Available BCF	Bay D'Espoir Development Annual Firm Energy Kwh x 10^6	Average Annual Secondary Energy Kwh x 10^6	Average Annual Total Energy Kwh x 10^6	Average Spill cfs
1. Prior to the Lloyds River Diversion	Oct. 1/55 to Sept. 30/65	34	86.6	93.0	2120	231	2351	70
2. After the Lloyds River Diversion	Oct. 1/55 to Sept. 30/65	34	86.6	93.0	2275	245	2520	135
BENEFIT OF THE LLOYDS RIVER DIVERSION					155	14	169	

NOTES:

1. No allowance has been made for storage releases for fish conservation, logging,operations, compensation water, or dam leakage
2. No compensation to Price (Nfld) Ltd. for loss of generation at their Hydro Electric Stations resulting from the Lloyds River Diversion
3. The total storage in the system is at the Operating Rule Curve Level at the commencement of the period

ShawMont Engineering Newfoundland Limited

ESTIMATE

CUSTOMER: NEWFOUNDLAND AND LABRADOR POWER COMMISSION

FILE: 70.

PROJECT: UPPER LLOYDS RIVER DIVERSION

DATE 29 June 1966.

ESTIMATED BY: G.S.W. R.W.N

CHECKED BY: G.S.W.

TYPE OF ESTIMATE:

(1) PRELIMINARY APPRAISAL TO INDICATE GENERAL MAGNITUDE OF COST:		
(2) PRELIMINARY ESTIMATE BASED ON PARTIAL FIELD INFORMATION:	x	
(3) CONSTRUCTION ESTIMATE BASED ON FULL FIELD INFORMATION:		

DESCRIPTION OF PROJECT:

Diversion of 184 sq. miles of the Lloyds River drainage area into the Bay D'Espoir Development via Victoria Lake, White Bear and Grey River Diversions by means of a 70 ft. high dam with a crest length of 1350 ft. located on the Lloyds river about one mile below King George IV Lake and a 3920 ft. long diversion canal from King George IV Lake to Wood Lake. Dam crest elevation 1175, F.S.L. 1165, L.S.L. 1050. 250 ft. long overflow spillway at damsite to discharge 10,500 cfs. at MSL 1171. Diversion canal uncontrolled designed to pass 2800 cfs at F.S.L. 1165, long term average flow 550 cfs.

AVAILABLE INFORMATION: (INCLUDE REFERENCE TO ADDITIONAL INFORMATION REQUIRED)

1:50,000 maps published by the Department of Mines and Technical Surveys.

1:" 400' topography of structure locations with 10' contours.

Photo interpretation to ascertain probable depth to bedrock and possible sources of construction materials.

APPROVED

JCBrooking per TKNancarrow

COST

DOLLARS:

LOCAL CURRENCY AT ____: DOLLARS:

TOTAL ESTIMATED ---- 3,735,000

Notes

1. Estimate based on 1966 prices without escalation.

2. Contingencies are included in the individual structures.

3. Estimate does not include the cost of enlarging the Victoria Lake, Granite Lake and Ebbegunbaeg canals to accommodate the Lloyds River flow.

4. Facilities for log driving and fish conservation have not been included.

5. Clearing of reservoir flooded area has not been included.

6. No allowance made for loss of merchantable timber.

7. No compensation to Price Nfld. Ltd., for loss of generation at their Grand Falls and Bishops Falls Hydro-Electric Stations is included.

SHAWMONT ESTIMATE OF LLOYDS RIVER DIVERSION

FILE 70 DATE JUNE 20/66 ESTIMATED BY G.S.W. R.W.N. PAGE 2 OF 4

ACCOUNT No.	DESCRIPTION	UNIT COST	TOTALS IN:	
7-11	LAND PURCHASE			
7-12	ROADS AND BRIDGES			
.1	Main access road from Victoria Lake Road		858,000	
	to Lloyds River Dam (19.5 mi.)			
.2	Access roads on site (3.5 mi.)		154,000	
.3	Road improvement and bridge strengthening (7.5 mi.)		193,000	
	TOTAL ACCOUNT 7-12			1,205,000
7-13	RAILWAYS AND DIVERSION OF POWER LINES		-	-
7-14	DAMS, SPILLWAYS AND RESERVOIRS			
.1	Lloyds River Dam			
	- Cofferdam and unwatering		121,000	
	- Main embankment		645,000	
			766,000	

SHAWMONT ESTIMATE OF LLOYDS RIVER DIVERSION

FILE 70 DATE JUNE 29/66 ESTIMATED BY G.S.W. R.W.N. PAGE 3 OF 4

ACCOUNT NO.			DESCRIPTION	UNIT COST	TOTALS IN: 1,205,00(	
-2			Spillway		198,000	
			TOTAL ACCOUNT 7-14			964,000
7-15			CANALS			
-1			Diversion Canal - King George IV Lake to Wood Lake		394,000	
			TOTAL ACCOUNT 7-15			394,000
			SUB TOTAL A			2,563,000

SHAWMONT ESTIMATE OF LLOYDS RIVER DIVERSION

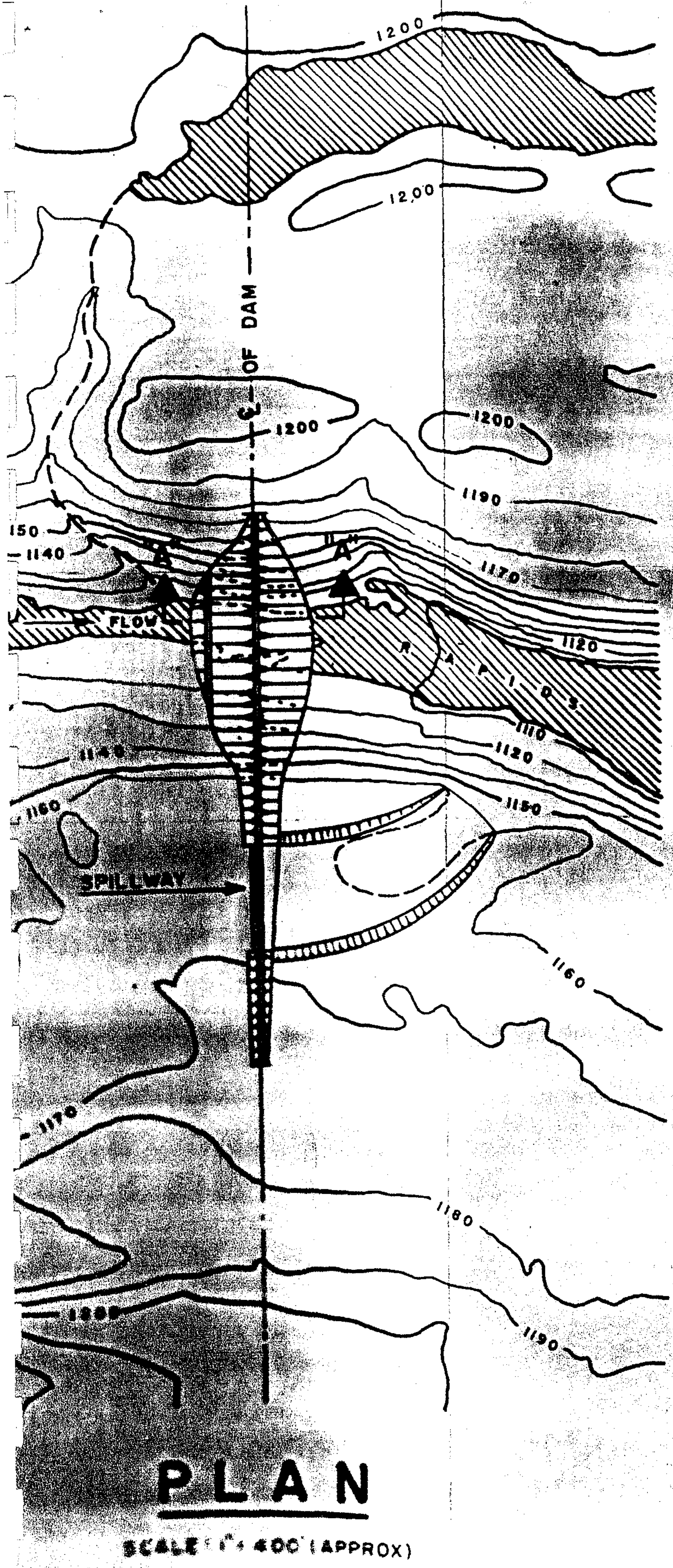
FILE 70 DATE JUNE 29/66 ESTIMATED BY G.S.W. R.W.N. PAGE 4 OF 4

ACCOUNT No.	DESCRIPTION	UNIT COST	TOTALS IN:
			2,563,000
7-19	INDIRECT COSTS		641,000
	Including : Camp Costs		
	Mobilization		
	Power Supply		
	Transportation		
	Administration		
	Aircraft Rental		
	On Site Road Maintenance		
	Fee		
7-20	ENGINEERING AND PROJECT MANAGEMENT		256,000
	SUB TOTAL B		3,460,000
7-21	OWNER'S COST		50,000
	INTEREST DURING CONSTRUCTION		225,000
	TOTAL PROJECT COST		3,735,000

APPENDIX II

NEWFOUNDLAND POWER STUDIES

LLOYDS RIVER DAM



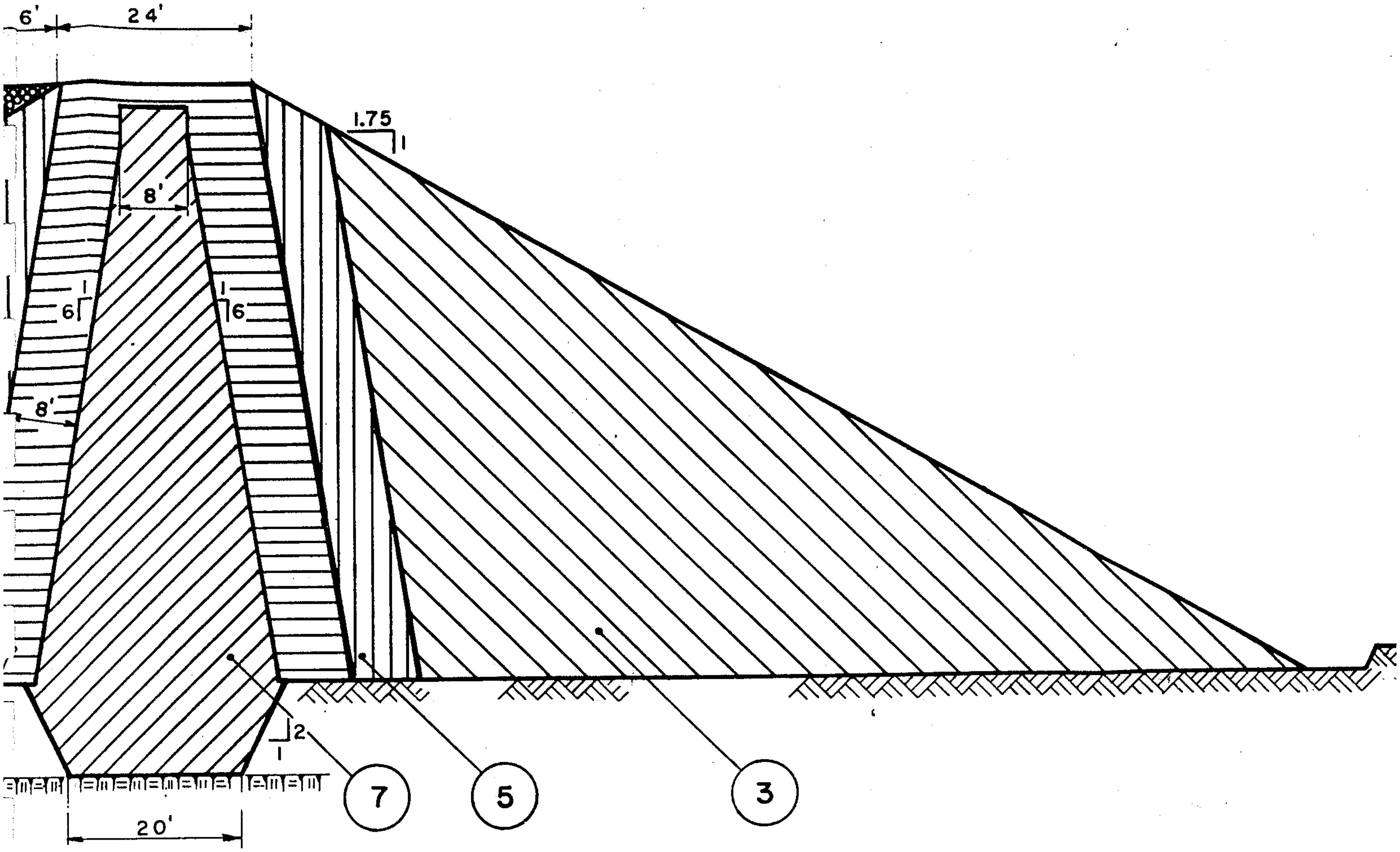
SHAWMONT NEWFOUNDLAND LIMITED

NEWFOUNDLAND & LABRADOR POWER COMMISSION

HYDRO POWER STUDIES

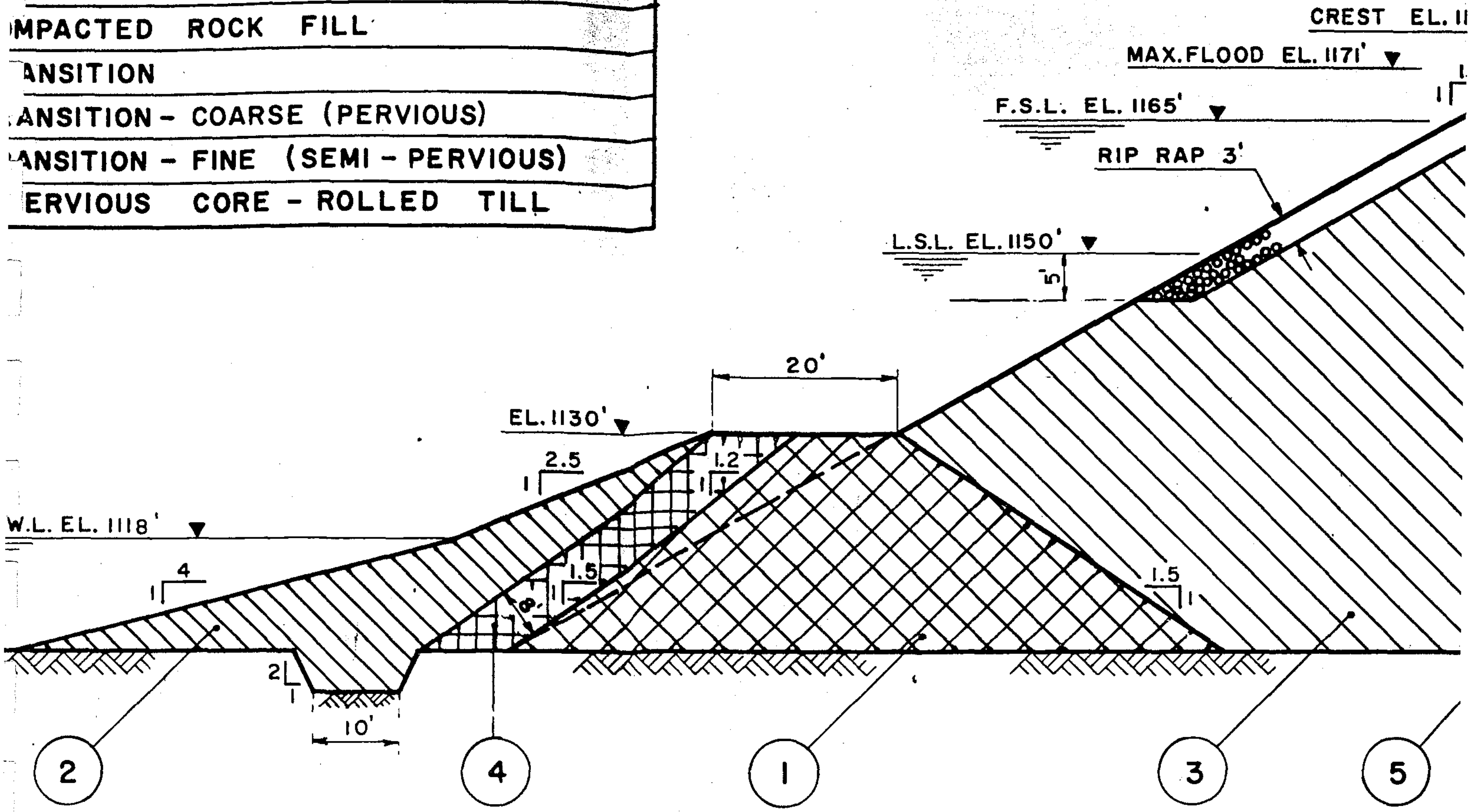
LLOYDS RIVER DAM
SECTION AND PLAN

DESIGNED G.S.W.	DRAWN EGYED	SCALE AS NOTED	DATE JUNE 30 - 1956
FILE No 70		APPROVED	



A - A"

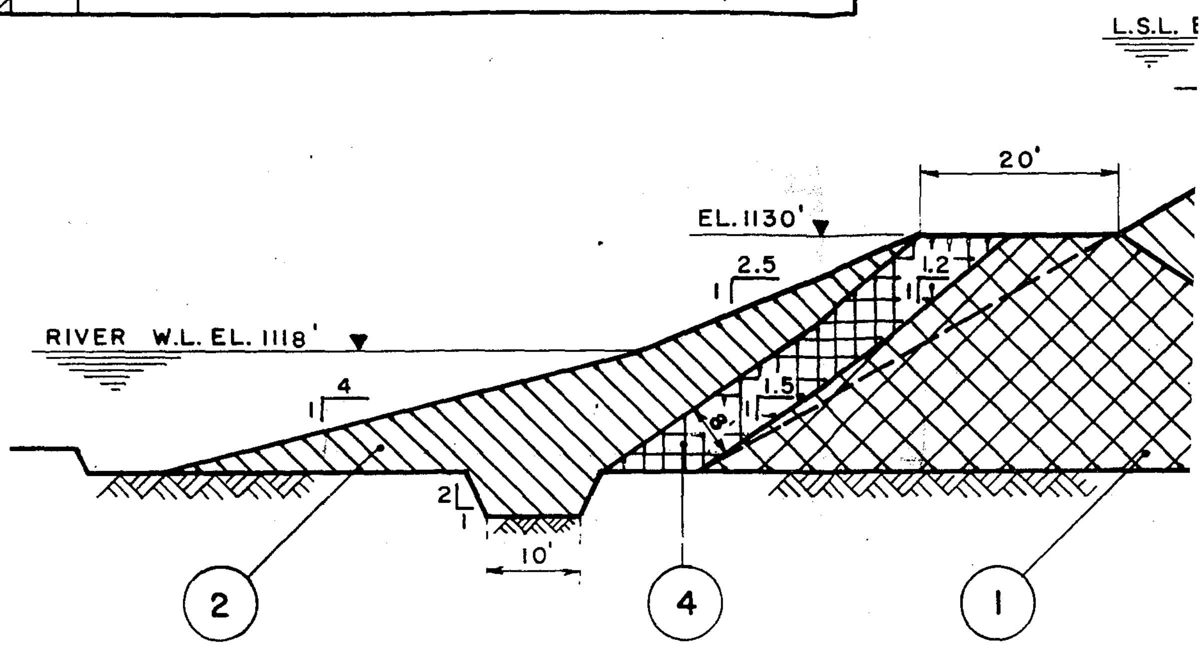
MATERIAL
MPED ROCK FILL
L BLANKET
MPACTED ROCK FILL
ANSITION
ANSITION - COARSE (PERVIOUS)
ANSITION - FINE (SEMI - PERVIOUS)
ERVIOUS CORE - ROLLED TILL



SECTIO

SCAL

ZONE		MATERIAL
	1	DUMPED ROCK FILL
	2	TILL BLANKET
	3	COMPACTED ROCK FILL
	4	TRANSITION
	5	TRANSITION - COARSE (PERVIOUS)
	6	TRANSITION - FINE (SEMI - PERVIOUS)
	7	IMPERVIOUS CORE - ROLLED TILL



CAPITAL BUDGET PROPOSAL

Division System Planning Project No. 2-15-06

Project Title Lloyds River Diversion

Description The Diversion of the flow of 184 square miles of drainage area which lie upstream of the outlet of King George IV Lake into the Victoria Lake drainage area and therefore into the Bay D'Espoir watershed, thus increasing the energy capability of the Bay D'Espoir plant.

Justification Increase energy capability of Bay D'Espoir in order to enable the Power Corporation to meet the Island's energy needs until the arrival of power from Labrador.

Estimated Starting Date ______ Estimated Completion Date Mid 1978

ESTIMATED COST *	
Engineering and Supervision (including preliminary Engin.)	$ 821,000.00
Materials Labour and Expense	4,971,400.00
Labour and Expense	
XXXX Owners cost	162,000.00
Interest during Construction	776,000.00
Contingency	596,600.00
Escalation	1,318,000.00
Reservoir Clearing	965,000.00
Total Direct Cost	9,610,000.00
Capitalized Overhead	
Total Project Cost	$9,610,000.00

CASH FLOW **	
Amount Authorized to Date	$2,210,000.00
1975 Authorization Requested	3,200,000.00**
Future Expenditures 1976	3,200,000.00**
1977	1,000,000.00**
1978	
Total Project Cost	$9,610,000.00

* See attached note ** N&LPC estimate

This estimate is based upon information supplied by ShawMont in their report on the extension of Bay D'Espoir (SM-12-74) and in their letter to Mr. D. Collett, dated May 1, 1975.

In their letter they state that if normal bidding must be carried out, mid 1978 is the earliest we could receive outflow from the Lloyds Diversion.

However, since the government intends to construct a road from Red Indian Lake to the south coast in any event, ShawMont recommends that a contract be initiated immediately to start work on the road. It is on this basis that the figure of $2,210,000. is derived for 1975.

CAPITAL BUDGET PROPOSAL

Division System Planning Project No. ______

Project Title Lloyds River Diversion

Description The diversion of the flow of 184 square miles of drainage area which lie upstream of the outlet of King George IV Lake into the Victoria Lake drainage area and therefore into the Bay D'Espoir watershed, thus increasing the energy capability of the Bay D'Espoir plant by 204 Gwh.

Justification Increase energy capability of Bay D'Espoir in order to enable the Power Corporation to meet the Island's energy needs until the arrival of power from Labrador.

Estimated Starting Date ______ Estimated Completion Date November 1, 1977

ESTIMATED COST	
Engineering and Supervision	
Materials	
Labour and Expense	6,655,600.00
Land	
Interest during Construction	
Contingency	
~~XXXXXX~~ ..Indirects	2,870,500.00
Total Direct Cost	
Capitalized Overhead	
Total Project Cost	9,526,100.00

CASH FLOW	
Amount Authorized to Date	1,661,800.00
1975-76 Authorization Requested	4,714,400.00
Future Expenditures 1976-77	3,149,900.00
1977-78	
Total Project Cost	9,526,100.00

ShawMont Newfoundland Limited

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May 1, 1975

File: 1350

Mr. David Collett,
Assistant Chief Engineer Planning,
Newfoundland & Labrador Power Corporation,
P.O. Box 9100,
St. John's, Newfoundland.

LLOYDS RIVER DIVERSION

Dear Mr. Collett:-

Your verbal enquiries of last evening have been considered and we report as follows:-

1. CONSTRUCTION SCHEDULE

There are two main elements in the construction schedule which govern completion, namely; access road construction which takes virtually one year; and construction of the diversion itself which must be completed in the period following the spring flood and before the onset of winter in the same calendar year. The latter criteria is fixed by the unwatering scheme which is designed to pass normal autumn flows only. Hence, it is doubtful that a two year construction programme could be achieved at this late date, especially if normal competitive bidding must be carried out. We would suspect that mid - 1978 is the earliest you could expect to receive outflow from the Lloyds Diversion.

In view of the fact the government intends to construct a road from Red Indian Lake to the south coast in any event we strongly recommend that a contract be initiated immediately on a crash basis to start work on the road. This road will be useful whether or not Lloyds goes ahead and considering present escalation rates there would probably be no additional cost involved to do this.

2. COST ESTIMATE

The overall cost of Lloyds was estimated at $7,327,000 in 1974 dollars in our report SM-12-74, with escalation estimated at 18%. We still believe this to be a realistic assessment of costs for the basic diversion but you should note that environmental costs should be added to this. In summary we estimate overall cost to be:

1974 Basic Cost		$ 7,327,000
Escalation @ 18%		1,318,000
	Sub-Total	$ 8,645,000
Reservoir Clearing		965,000
	Sub-Total	$ 9,610,000
Reservoir Grubbing		1,930,000
		$11,540,000

The latter two items are discussed in our letter of March 24, 1975. We would agree the clearing to be a reasonable undertaking to create a neat site but we strongly question the necessity or even the advisability of grubbing (removal of stumps). We would think rather than grubbing some consideration be given to the construction of a beach for public swimming or some other useful facility.

Please note that no other environmental costs have been included. We would agree that a weir at the outlet of Lloyds Lake would be advisable to maintain water levels and possibly some other small projects could be incorporated to enhance the environmental situation.

3. 1975 BUDGET

Our best, quick estimate for a 1975 budget should roadwork and engineering investigations go ahead is:

Site Clearing		$ 60,000
Access Road		1,450,000
Engineering and Field Investigations		250,000
	Sub-Total	$1,760,000
Reservoir Clearing		450,000
	Sub-Total	$2,210,000
Reservoir Grubbing		NIL
		$2,210,000

We trust you will find the foregoing sufficient for your immediate requirements and would be pleased to examine the items in more detail given additional time.

We have not had an opportunity to examine the environmental report on the Lloyds Diversion and would appreciate receiving a copy of it. Should you so wish we would be pleased to prepare a review of the report and offer some concrete proposals with respect to mitigating environmental damage.

We are presently assembling data on the Cat Arm, Upper Salmon and Terra Nova Projects.

Yours very truly,

R. A. Robertson,
Manager.

RAR/jaw

c.c. Mr. L. J. Cole