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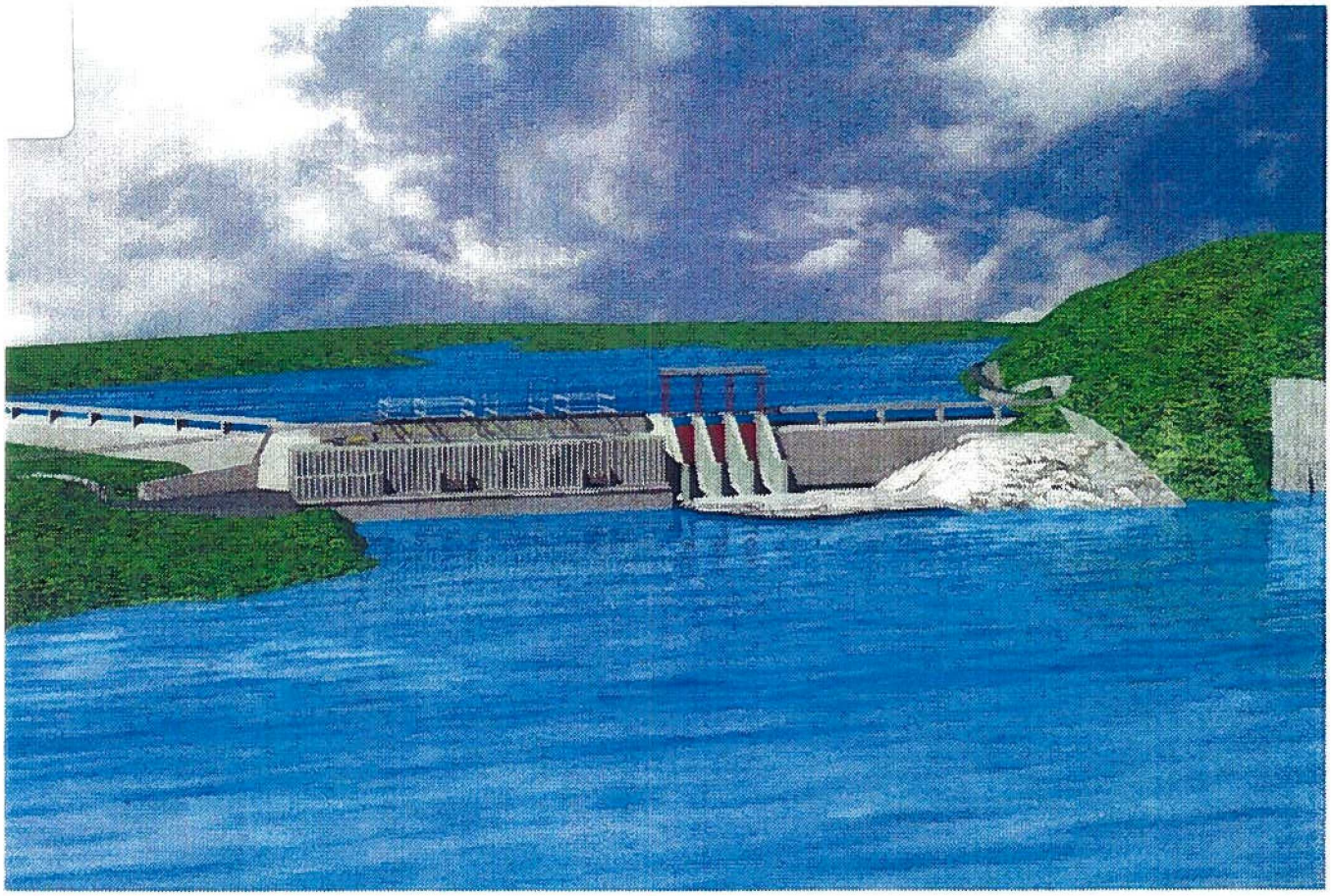
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Muskrat Falls Final Feasibility Study

Volume 1 - Engineering Report

HYDRO NEWFOUNDLAND AND LABRADOR HYDRO

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT



FINAL FEASIBILITY STUDY

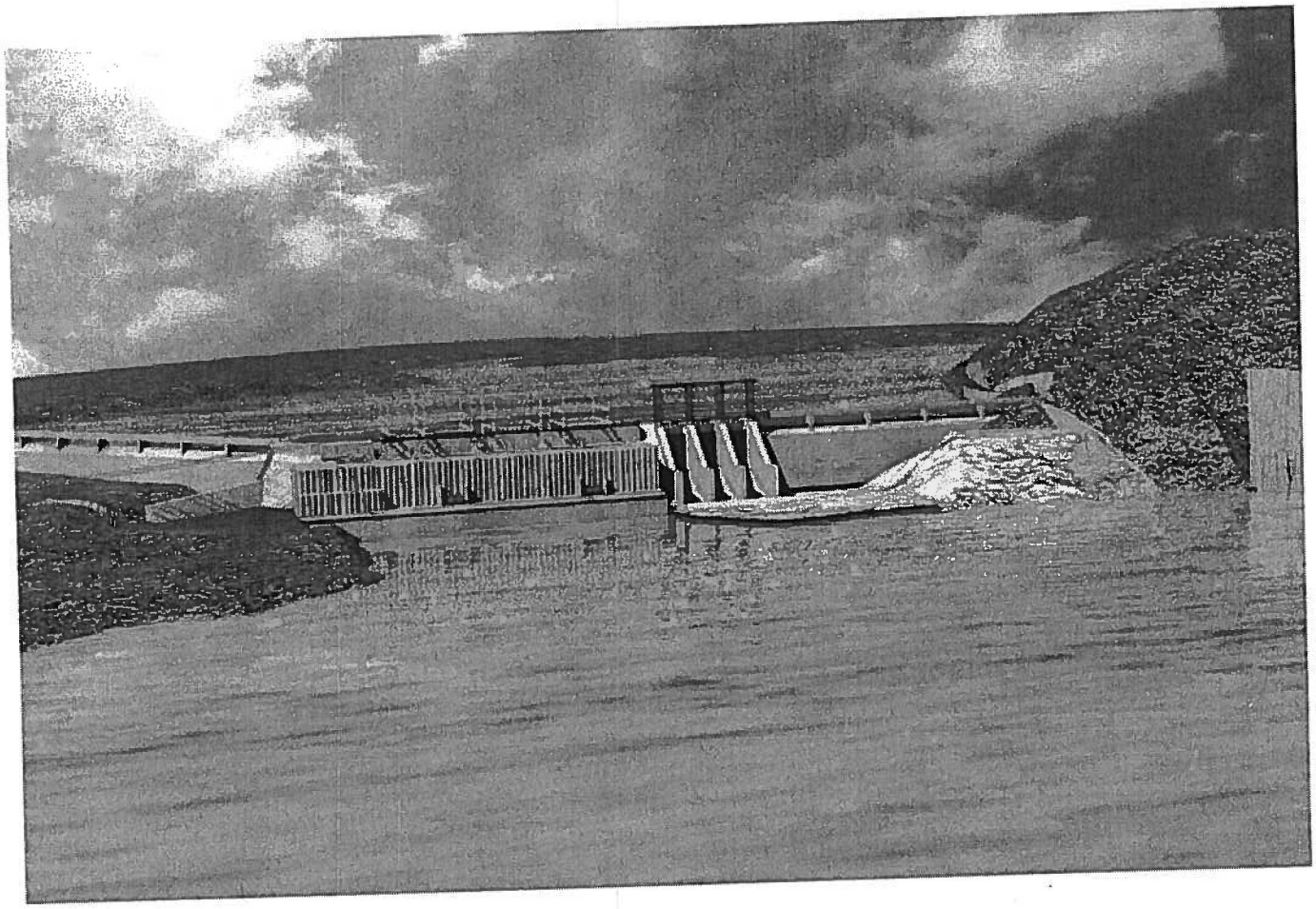
VOLUME 1 - ENGINEERING REPORT

JANUARY, 1999

SNC-AGRA

HYDRO NEWFOUNDLAND AND LABRADOR HYDRO

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT



FINAL FEASIBILITY STUDY

VOLUME 1 - ENGINEERING REPORT

JANUARY, 1999

SNC-AGRA

FOREWORD

This report is presented in three volumes:

Volume 1: Engineering Report

Volume 2: 1998 Geotechnical Investigations

Volume 3: Detailed Backup of Capital Cost Estimate

Volume 1 presents the results of the engineering study and the capital cost estimate as of June 1998, together will all relevant Plates (drawings) and appendices incorporated after the text of the report.

Volume 2 contains the results of the geotechnical investigations carried out in the summer of 1998 together with the relevant plates and appendices incorporated after the text of the report. This volume is presented in limited copies to be distributed per request only.

Volume 3 contains backup calculations for the detailed cost estimate, and is presented in limited copies to be distributed per request only.

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PLATES

APPENDICES

Appendix A **Detailed Capital Estimate**

Appendix B **Design Criteria**

Appendix C **North Spur Stabilization**

Appendix D **Construction Flood Study**

Appendix E **Optimization of Number of Generating Units**

Appendix F **La Salle Consulting Group Inc. - Ice Study**

Appendix G **Roller Compacted Concrete (RCC) Dam Background and Application and Rubber Dam Experience Lists and Technical Literature**

LIST OF ABBREVIATIONS

LIST OF ABBREVIATIONS

BBS	Budget Breakdown Schedule
BM	Bench Mark
CCE	Capital Cost Estimate
CFLCo	Churchill Falls (Labrador) Corporation
CPIF	Cost Plus Incentive Fee
CPFF	Cost Plus Fixed Fee
CPPF	Cost Plus Percentage Fee
CSP	Corrugated Steel Pipe
El.	Elevation
EWP	Engineering Work Packaging
FFP	Firm Fixed Price
FP	Fixed Price
FPPI	Fixed Price Plus Incentive
FS	Factor of Safety
GPR	Ground Probing Radar
GPS	Global Positioning System
GST	Goods and Services Tax
HST	Harmonized Sales Tax
IDC	Interest During Construction
LCC	Lower Churchill Consultants
LCDC	Lower Churchill Development Corporation
LCG	LaSalle Consulting Group
LS	Lump Sum
NLH	Newfoundland and Labrador Hydro
OBS	Organization Breakdown Structure
O&M	Operation and Maintenance
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RCC	Roller Compacted Concrete

RQD	Rock Quality Designation
TP	Target Price
UP	Unit Price
VP	Variable Price
WBS	Work Breakdown Schedule

ELECTRICAL

AC	Alternating Current
ADSS	Attached Dielectric Self Supporting
AGC	Automatic Generation Control
ANSI	American National Standards Institute
AVR	Automatic Voltage Regulator
BIL	Basic Insulation Level
CEMA	Canadian Electrical Manufacturers Association
CT	Current Transformer
CVT	Capacitor Voltage Transformer
DAU	Data Acquisition Unit
DC	Direct Current
DCS	Distributed Control System
DPU	Data Processing Unit
DTT	Direct Transfer Trip
GPR	Ground Potential Rise
HOA	Hand-Off-Auto
HPS	High Pressure Sodium
HV	High Voltage
HVAC	High Voltage Alternative Current
HVDC	High Voltage Direct Current
IEC	International Electromechanical Commission
IEEE	Institute of Electrical and Electronic Engineers
IESNA	Illumination Engineering Society of North America
LAN	Local Area Network
LRCC	Labrador Regional Control Center

LV	Low Voltage
MH	Metal Halide
NPCC	Northern Pool Coordinating Council
ODF	Optic Fiber Distribution Frame
ONAF	Oil Natural Air Forced
ONAN	Oil Natural Air Natural
OPGW	Fibre Optic Ground Wire
PD	Partial Discharge
PID	Proportionate, Integral, Derivative (governor)
PLC	Programmable Logic Controller
PLC/DCS	Programmable Logic Control/Distributed Control System
PT	Potential Transformer
RTD	Resistive Temperature Device
RTU	Remote Terminal Unit
SCR	Silicon Control Rectifier
SF_6	Sulphur Hexafluoride
SONET	Synchronous Optical Network
UPS	Uninterruptible Power Supply
UV	Ultra Violet
VDC	Visual Display Control
VDU	Visual Display Unit
VT	Voltage Transformer
WAN	Wide Area Network

LIST OF UNITS

LIST OF UNITS

A	Ampere
Amps	Ampere second
cfs	cubic feet per second
cm	centimetre
cm/s	centimetre per second
^{0}C	degrees Celsius
c^1	Cohesion based on effective stresses
dbA	decibel A
DC	direct current
ft (')	feet
g	acceleration
GHz	GigaHertz
GPa	GigaPascal
GWh	GigaWatt hour
H	horizontal
hm^3	cubic hecto metre = 10^6 m^3
hrs	hours
Hz	Hertz
kA	kiloAmpere
kg	kilogram
km	kilometre
km^2	square kilometre
kN	kiloNewton
kN/m	kiloNewton per metre
kN/m^2	kiloNewton per square metre
kN/m^3	kiloNewton per cubic metre
kPa	kiloPascal
kV	kiloVolt
kVA	kiloVoltsAmperes
kVAr	kiloVoltAmperes reactive
kW	kiloWatt
kWh	kiloWatt hour

L	Litre
L/s	Litre per second
m	metre
m^2	square metre
m^3	cubic metre
m/min	metre per minute
M$	million dollars
m/s	metre per second
m^3/s	cubic metre per second
mA	milliAmpere
min	minute
mm	millimetre
ms	millisecond
MPa	MegaPascal
MVA	MegaVolt Amperes
MVAr	MegaVolt Amperes reactive
MW	MegaWatt
MWh	MegaWatt hour
Q	Flow in Litres per second
Q_{40}	1 in 40 year diversion flood
$Q_{10,000}$	10,000 year return period design flood
Q_p	Plant flow capacity
rpm	revolutions per minute
s	second
t	metric ton (1000 kg)
TWh	TeraWatt hour
V	Volt
VAr	Volt Amperes reactive
Vdc	Volt direct current
yr	year
$\varnothing^1$	effective angle of internal friction
%	Percent

LIST OF PLATES

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SECTION 1.0

EXECUTIVE SUMMARY

SECTION 1.0

EXECUTIVE SUMMARY

1.0 EXECUTIVE SUMMARY

1.1 INTRODUCTION

On June 29, 1998, Newfoundland and Labrador Hydro (NLH) retained SNC-AGRA Joint Venture, in partnership with the SGE Group Inc., Newfoundland Design Associates Limited, and Quadratec Inc. to provide engineering services to carry out a Final Feasibility Study for the Muskrat Falls Hydroelectric Development. The services included coordination with others for optimization of the project energy output and capacity.

This final feasibility study is intended to be the basis of a decision to proceed with project implementation. With this in mind, a goal for the study was to minimize the cost of energy through structure and equipment optimization.

The following sections summarize the findings of the study and include the recommendations for project development as set out in the following descriptions of the project and component layouts. A summary of the capital cost estimate and the quarterly "Incurred Cost Flow" are provided in Section 1.7. The recommended Project schedule is described in Section 1.8. A summary of the study conclusions is provided in Section 1.9 and recommendations for further work are presented in Section 1.11.

1.2 PROJECT DESCRIPTION

1.2.1 General

The Churchill River flows eastward from the Churchill Falls Power Plant, on the central plateau of Labrador, to Lake Melville just east of the town of Happy Valley-Goose Bay. The river flows in a relatively narrow valley from elevation 126 m at the Churchill Falls tailrace tunnel outlets to sea level in Lake Melville, through a series of rapids. The Muskrat Falls Hydroelectric Development will be located approximately 32.5 km upstream from Happy Valley-Goose Bay.

The general arrangement of the Muskrat Falls Hydroelectric Development is shown on Plate 10 and the principal elements of the Project are described in detail in Sections 8 to 12.

These principal elements are:

- diversion facilities;
- reservoir control structures (overflow concrete dams and a three bay, gated spillway);
- stabilization works for the north spur, which connects the rock knoll to the north bank of the original river valley; and
- power and 230 kV switchyard facilities.

1.2.2 Drainage Basins

The drainage basin of the Upper Churchill River, regulated by the reservoir at Churchill Falls, is 78,850 km^2, including the St-Jean and Romaine River diversions. The unregulated Lower Churchill River basin at Muskrat Falls has a drainage area of 23,088 km^2.

This Lower Churchill drainage basin area is subdivided as follows:

Churchill Falls to Gull Island: 19,832 km^2
Gull Island to Muskrat Falls: 3,256 km^2

Plate 1 shows the project location and the division of the drainage basin.

1.2.3 Diversion

During construction of the project the river flow will be diverted by an upstream cofferdam, located just above the upper falls, into two diversion tunnels through the rock knoll, as shown on Plate 12. The flow will be returned to the river, downstream of the lower falls, from the outlet portal of the tunnels. The inlet structures of the diversion tunnels will be gated to permit regulation of flow through the tunnels in order to raise the upstream water level for the purpose of winter ice control. A downstream cofferdam at the crest of the lower falls will prevent flooding of the main construction area from downstream.

The diversion facilities, cofferdam and diversion tunnels were designed to handle a 1 in 40 year flood of 5,300 m^3/s, assuming flow control would be provided by Churchill Falls Generating Station. The dimensions of the diversion works, the tunnel size and the crest elevation of the upstream cofferdam were determined on the basis of an economic optimization, giving a tunnel size of 14.0 m wide x 17.0 m high and a maximum upstream diversion flood level of 30.5 m.

The salient data of each structure are presented in Section 7.3.

1.2.4 Reservoir

The maximum reservoir operating level is at El. 39.0 m. The reservoir area covers about 106.8 km^2 and extends 60.0 km upstream to the proposed Gull Island Hydroelectric Development.

The reservoir area will be totally cleared up to El. 42.0 m, 3 m above the maximum operating level.

Commercial timber from the cleared area will be piled for salvage outside the reservoir limits and will be accessible from the Trans-Labrador Highway.

Further salient data of the reservoir are presented in Section 7.3.2.

1.2.5 Plant Capacity and Average Energy

Determination of plant capacity and energy was done under a separate contract by Acres International Limited in 1998 (Reference 54).

The findings from their study were:

Optimum Plant Capacity at HV bus = 824 MW
Average Annual Energy at HV bus = 5.53 TWh

1.2.6 North Spur

The north spur forms a natural earthfill dam, with a crest elevation of about 60 m, and about one km long, which connects the rock knoll to the north bank of the valley as shown on Plates 10 and 18. The crest width varies from about 1,000 m at its north end to about 70 m at its south end where it has been narrowed by erosion and landslide activity in the past. The head across the spur is presently 16 m from river level upstream to downstream. The impounding of the reservoir to El. 39 m will increase the hydraulic head across the spur to 36 m and stabilization measures are then necessary to ensure its long-term stability under both normal and extreme water levels.

The soils forming the spur consist of a complex interbedded sequence of relatively low permeability silty sands and sands, and sensitive marine clays. Investigations in the 1979-80 feasibility study suggested that the most effective method of stabilization of the spur was with a line of pumped wells to lower the groundwater table. Additional stabilization measures necessary included upstream rock erosion protection and stabilizing berm, downstream rock toe erosion protection and relief drains, and local unloading of high, oversteepened slopes along the downstream side of the spur.

In 1982, an interim line of pumpwells was installed to prevent ongoing sliding which could have breached the spur. This system has performed well since 1982. No landslides have occurred and the performance data validates the efficiency of this approach. The layout of stabilization measures necessary to ensure spur stability under operating conditions has been optimized from the performance data available since 1982. The final layout of the stabilization measures is similar in scope and cost to the 1979-80 layout, except at the south end of the spur where a drainage trench and downstream granular fill was found to be more cost-effective than pumped drainage, due to local soil conditions.

Therefore, the stabilization of the north spur can be done both economically and reliably with the expansion of the existing pumpwell system, along with provision of downstream erosion protection and downstream stabilizing fill; local topcutting and unloading of portions of the high and steep side slopes, provision of drainage trenches and erosion protection and a stabilizing upstream berm.

1.2.7 Spillway

The three gate spillway structure plan and sections are shown on Plate 15. It is located in the river channel above the lower falls, abutting an RCC gravity overflow dam on the north side and the intake/powerhouse on the south side.

Section 9.6 presents in detail the description of the spillway structure, the stability analysis and the spillway gates and stoplogs.

The salient data of the spillway structure are presented in Section 7.3.6.1.

1.2.8 Dams

Two concrete gravity overflow dams with roller compacted concrete (RCC) interiors, together with the spillway and the integrated intake/powerhouse structures, are located in the river channel, upstream of the lower falls, to create a sealed closure of the river valley and the reservoir for the development, as shown on Plate 11. The north RCC dam closes to the north side of the valley and the south RCC dam closes to the south side of the valley with a small freeboard dyke.

At the PMF (22,100 m^3/s), the RCC dams will discharge higher flow (11,300 m^3/s) than the gated spillway (10,800 m^3/s).

The north RCC dam includes five sections of pneumatically controlled adjustable spillway crest(widely used in the industry and referred to as "rubber dam"), each 2.4 m high and 34.5 m long, secured to the top of the concrete dam between the bridge piers with a crest elevation (fully inflated) of 39.5 m.

RCC dams have been constructed world wide and are becoming more popular in North America. They are particularly suitable in projects where overtopping of the dam is a possibility, since these are essentially concrete structures with excellent resistance to erosion. A background and application description, with a list of RCC dams constructed between 1980 and 1989, is provided in the report in Appendix G. The list includes 46 RCC dams, ranging in height from 12 m to about 100 m.

Rubber dams, also called "inflated weirs", "inflatable gates" have proven experience. More than 1,000 rubber dams are in operation in the world. They are widely used in Japan, in the United States and, in a lesser degree, in Canada. They are mostly used for irrigation, tidal barriers and more and more for hydro power plant rehabilitation. Technical literature, lists of Canadian, American and World experience, with project contact names, are provided in the report in Appendix G.

The spillway and the north RCC dam with the rubber dam fully deflated (crest elevation 37.1 m) will pass the 1 in 1,000 year flood at a reservoir water level of 40.0 m. Any floods greater than 1 in 1,000 year flood, up to the probable maximum flood (PMF), will pass over the north RCC overflow dam, the south RCC overflow dam and through the spillway.

Section 9.0 presents in detail the description of the north and south RCC dams, the rubber dam, the freeboard dyke, the stability analysis and the pertinent geotechnical field investigations and results at each structure.

The salient data of the north and south RCC dams together with the rubber dam and the freeboard dyke are presented in Sections 7.3.7 and 9.7.

1.2.9 Integrated Intake/Powerhouse

The intake/powerhouse structure is located approximately in the centre of the river channel, upstream of the lower falls, abutting the spillway to the north, and the south RCC dam to the south as shown on Plates 11 and 17.

The powerhouse includes four generating units, three of which have Propeller turbines and one with a Kaplan turbine, for a total installed capacity of 824 MW.

Two power transformers are located on the downstream deck of the powerhouse. These transformers will be connected to an open ring bus switchyard located on the roof of the powerhouse.

Section 11.0 presents in detail the description of the power facilities, including the field explorations and results, the stability analysis, the turbines and generators, the plant electrical and the plant mechanical.

The salient data for the power facilities are presented in Section 7.3.5.

1.2.10 Transmission Lines, Switchyard and Converter Station

In the present study, two 230 kV AC transmission lines will connect the Muskrat Falls generating plant to the HVDC Converter Station located near Gull Island, approximately 60 km upstream of Muskrat Falls.

The Muskrat Falls switchyard is located on the roof of the powerhouse as shown on Plates 22 and 23 and includes all the 230 kV equipment necessary to connect the two 230 kV transmission lines and the four generating units into the transmission system. It also includes space for a possible future third line to Goose Bay.

The two 230 kV AC transmission line interties to the HVDC Converter Station are excluded from this study.

1.3 CONSTRUCTION CAMP

The camp located at the Muskrat Falls site, will accommodate 1215 persons, based on single occupancy. The general arrangement of the camp and the accommodation units are shown on Plates 25 to 27. Amenities will be provided for comfortable living conditions, recreation, security and safety. Family accommodations will be provided in Happy Valley-Goose Bay where 100

existing housing units will be rented. Bus transportation for management and engineering personnel will be provided between Happy Valley-Goose Bay and the site.

1.4 CONSTRUCTION POWER

An existing 138/25 kV, 2 MVA, substation supplies power to the area and this supply will be used for early mobilization at site. A new 6/8 MVA transformer is required for full construction power, estimated to peak at 6 MVA and will supply 95 GWh of energy over the 5 year construction period. The construction of 5 km of 25 kV distribution facilities are required to supply power to the camp and the construction sites. Capacitors are required to support the 138 kV transmission system from Churchill Falls.

1.5 TRANSPORTATION AND LOGISTICS

The following modes of transportation are available to the project site.

1.5.1 Marine

Marine service by ship and barge is available to the port of Goose Bay. The dock at Goose Bay is approximately 40 km from the Muskrat Falls site via local roads in Happy Valley-Goose Bay and the Trans-Labrador Highway. This port is ice free for a maximum of seven months a year. The port of Goose Bay will be utilized as the destination point for heavy lift and bulk cargo items. Items arriving via Goose Bay will be trucked to the Project site. A small marshalling yard and dry storage building will be utilized at dockside in Goose Bay as a receiving area for material arriving by ship. It is assumed that these facilities will be constructed as part of the Gull Island Project and hence no costs for these facilities are included in this estimate.

1.5.2 Road

Transportation by road from Goose Bay, or Central Canada, to the Project site via the Trans-Labrador Highway is available year round. This mode of transport will be the primary method of moving materials and equipment to the Project site.

The section of road from Goose Bay to Muskrat Falls will require localized widening or passing areas for traffic during those times that bulk or heavy lift items are being transported to the site. These widened areas will be required for the Gull Island Project and hence no cost for this item of work is included in this estimate.

1.5.3 Airport

A modern airport exists at Goose Bay and is capable of handling the largest civilian aircraft available. Presently, Goose Bay is serviced on a daily basis by Dash 8 aircraft for cargo and passenger travel from St. John's and Halifax.
The airport at Goose Bay will be utilized for all aircraft passenger travel and miscellaneous air freight service.

1.5.4 Rail

Rail service is available from dockside at Sept-Iles to Ross Bay near Esker, at Mile 286, on the Quebec North Shore and Labrador Railway where freight would be transferred for truck transport to Muskrat Falls.

This route would only be used as an alternate for heavy load items during the winter months. The cost of freight by this route would be more expensive than by ship to Goose Bay and would only be utilized for critical delivery items as required by the Project schedule.

1.6 CONTRACT PACKAGING

The proposed strategy for contract packaging for Muskrat Falls is to utilize large contracts for all work of a similar type in a given area. The major contracts would be:

- Preliminary Work - Supply and erect camps and temporary services
- Clearing - Clear reservoir and site
- North Spur and Diversion Works - North spur stabilization, diversion tunnels and cofferdam
- Main Civil Works - Main Civil Structures
- Balance of Plant Mechanical/Electrical - Supply and installation of all secondary systems & hydro-mechanical systems
- Turbine-generators - Supply and installation of the turbines and generators
- Camp Operations - Operation and maintenance of the construction camp, and catering
- Switchyard - Supply and installation of electrical equipment and transformers

In addition to the construction contracts, supply contracts could be awarded for the procurement of large equipment where the Owner wishes to have more direct control of the type of equipment supplied to the Project. A detailed description of the strategy and of the contract packages is contained in Section 15.3.

1.7 CAPITAL COST ESTIMATE

The Capital Cost Estimate (CCE) is based on an evaluation of the resources required to undertake the work and the costs of these resources, i.e. labour, equipment and materials as of June of 1998.

The CCE and the Project schedule have been developed based on current labour policies, occupational health and safety standards, transportation methods, logistics, procurement policies and management systems.

The CCE was prepared in a detailed manner based on the preliminary design developed for this Final Feasibility Study. It is assessed as having an accuracy of ± 10%. Costs were estimated as a "contractor's" type estimate in that the work was broken down into identifiable components with the necessary labour and equipment crews to undertake the work.

The CCE takes into account changes in infrastructure from the 1980 report. In particular, the improvements to the Trans-Labrador Highway have reduced the cost of transportation and greatly improved efficiency of moving materials and equipment to the site.

An assessment of modern construction technology was made to ensure the CCE is realistic and up-to-date. All amounts stated in this report are in Canadian dollars as of June, 1998. All amounts are exclusive of GST and HST.

A summary of the CCE, and the incurred cost flow and labour force curves for the Project are shown in Table 1.1 and Figures 1.1 and 1.2, respectively.

The CCE, the incurred cost flow and labour force are presented in greater detail in Appendix A. Volume 3 presents the "Detailed Backup of the Capital Cost Estimate".

1.7.1 Contingencies

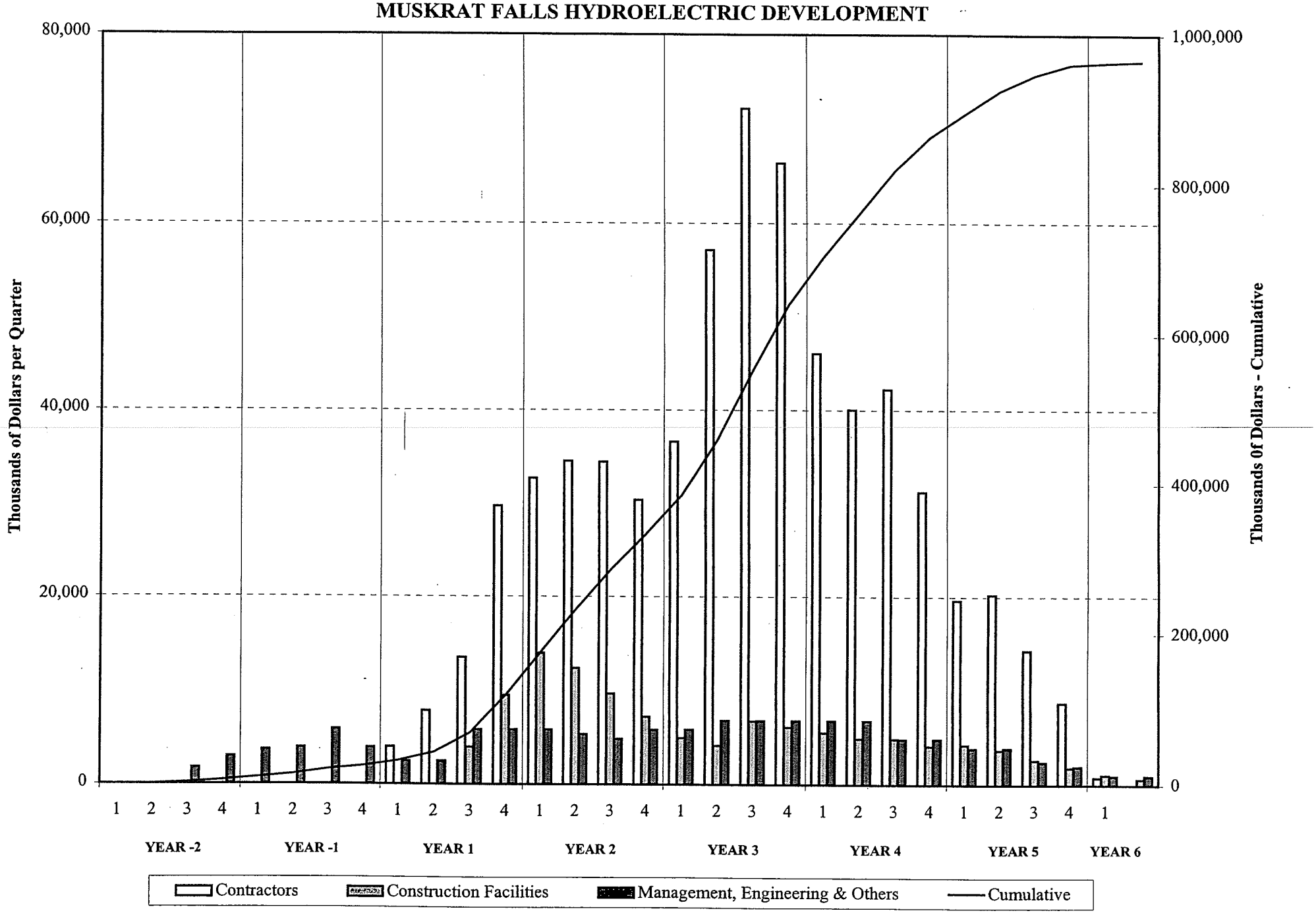
The contingency carried in Cost Division 9800 of the CCE is a Project contingency and is additional to contractors' contingencies for which allowance has been made in the detailed construction cost elements of the estimate. The Project contingency is based on consideration of the level of information available for the present study.

1.7.2 Costs Excluded

A list of costs excluded from the estimate is presented in Section 16.1.1. These are, escalation after June 30, 1998; interest during construction; construction "all risk" and other insurances which are provided by Owner; any applicable sales taxes including HST and GST; land acquisition, easements, rights-of-way, compensation to land owners or lease holders and other similar items of cost; the Owner's operational and pre-operational expenses including supervision, operator training, staff build-up for the operation phase of the Project, operating supplies and plant start-up; cost reduction due to further optimization studies; the Owner's general and administrative contingency and expense items, including the procurement of

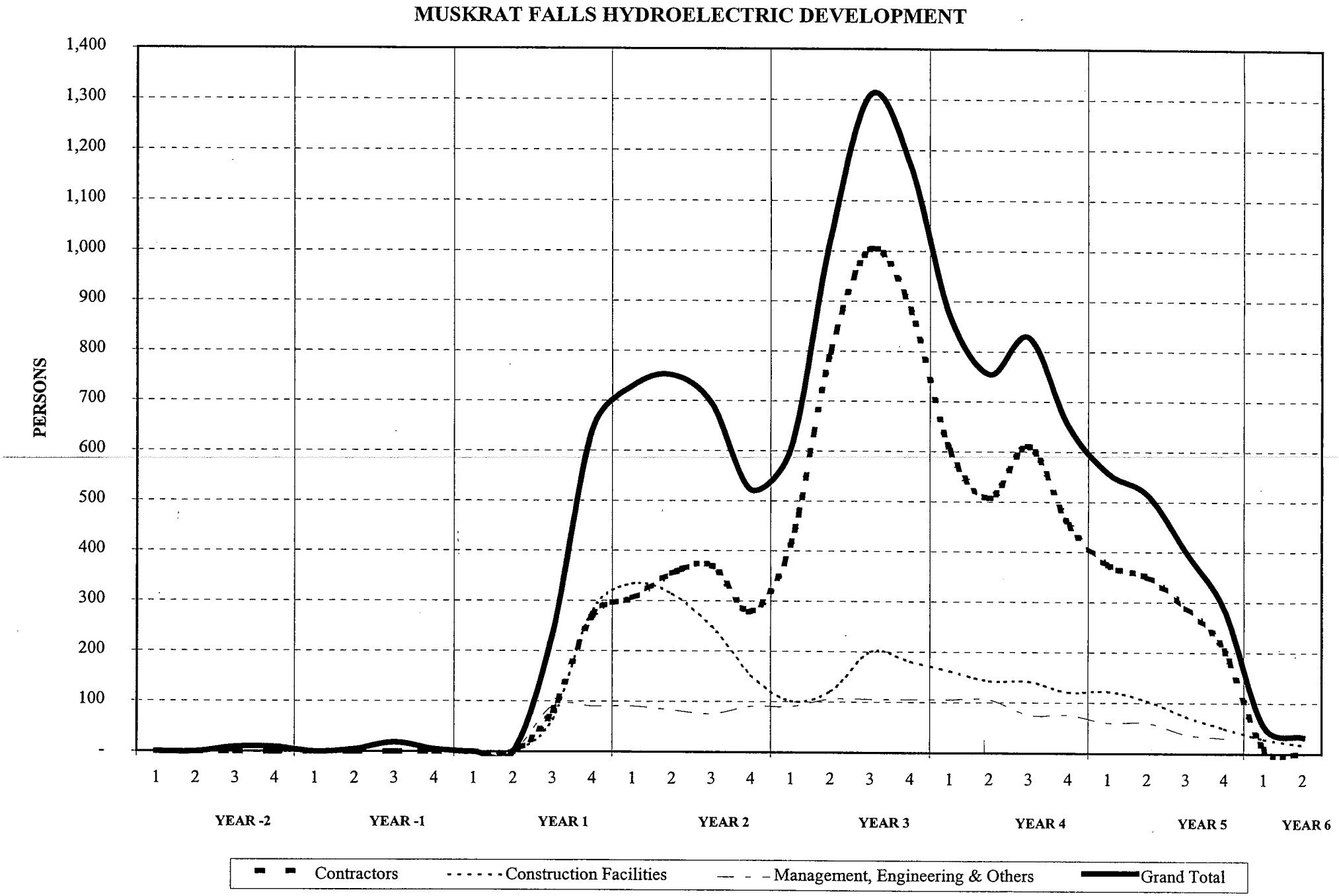
TABLE 1.1
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
SUMMARY OF CAPITAL COST ESTIMATE

Code	Description	June 1998 Canadian Dollars
1000	Permanent Support Facilities	6,171,000 $
2000	Reservoir, Diversion, Dam, Spillway	
	2100 Reservoir	37,600,000 $
	2200 Diversion	45,416,000 $
	2300 RCC Dam and Cofferdams	50,996,000 $
	2400 Spillway	36,548,000 $
	2800 North Spur Stabilisation	18,016,000 $
	Total 2000	188,576,000 $
3000	Powerhouse and Related Facilities	
	3300 Powerhouse/Intake/Tailrace	254,263,000 $
	3400 Turbines and Generators	145,000,000 $
	3500 Electrical Ancillary Equipment	11,076,000 $
	3600 Mechanical Ancillary Equipment	12,016,000 $
	3700 Generator Transformers and HV Switchyard	16,180,000 $
	3800 Spare Parts and Special Tools	7,000,000 $
	Total 3000	445,535,000 $
6000	Communications	
	6500 Communication Systems	1,965,000 $
	Total 6000	1,965,000 $
8000	Construction Facilities and Support	
	8200 Muskrat Falls Site	104,411,000 $
	8300 Happy Valley - Goose Bay	7,320,000 $
	Total 8000	111,731,000 $
	Sub-Total (Codes 1000 to 8000)	753,978,000 $
9000	Management, Engineering and Others	
	9200 Management	79,860,000 $
	9250 Engineering	41,140,000 $
	9300 Field Investigations and Hydraulic Model Studies	2,500,000 $
	9800 Contingencies	87,748,000 $
	Total 9000	211,248,000 $
	GRAND TOTAL	**965,226,000 $**



Quarterly Incurred Cost Flow

FIGURE 1.1



Labour Force

Figure 1.2

financing and working capital requirements; permits and licenses; engineering, field exploration and construction performed prior to the preparatory activities (previous studies, etc.); environmental and/or fisheries requirements that are more stringent than those described in the 1980 report; Environmental Baseline Studies and Environmental Impact Statement (EIS); archeological study; changes and/or additions to laws and in the regulations by governments and/or their agencies after June 30, 1998; 230 kV transmission lines including OPGW; and aboriginal land claims.

1.8 PROJECT MASTER SCHEDULE

The recommended Project Master Schedule is based on a Project release date of January 1 of Year 1 and a Project start date of June 1 of Year 1. Prior to Project release there would be two years which would include engineering, model testing and site investigations. The contracts for the turbine-generators and the construction power transformers would be tendered prior to Project release. All other contracts would be tendered and ready for award following Project release.

Following Project release there is one year of construction on the diversion works. River closure and diversion would be achieved by the end of July Year 2. Work on the main civil structures would start immediately after diversion. Approximately 38 months would be required from the start of civil works to first power on October 1 of Year 5. First power would be available by October 1 of Year 5, or 57 months from Project release. Full commercial operation would be achieved by mid-April of Year 6 or 63 ½ months from Project release.

The duration from the award of the turbine-generator contract to first power would be 54½ months and to full commercial operation would be 61 months.

The Project Master Schedule is shown on Plate 28 and is described in detail in Section 15.0.

1.9 CONCLUSIONS

- The Project layout was derived from Variant 7 in the 1989 SNC Report (Reference 35) and was further optimized to minimize capital cost. The layout includes:

  - Tunnel diversion, based on a construction flood of 5,300 m^3/s with modified flood handling procedures at Churchill Falls and regulation of the upstream water level during winter to El. 24.0 m for ice control. The volume of ice passing through the diversion will be < 1% of the volume occurring under existing conditions.

  - Flood handling structures, including a three gate spillway and two roller compacted concrete (RCC) overflow dams, one of which is fitted with a rubber dam crest, will discharge the PMF with a maximum reservoir elevation of 44.0 m.

  - An integrated intake/powerhouse with concrete semi-spiral cases and four generating units - one Kaplan and three Propeller turbines for optimum plant energy output of 5.53 TWh/y and capacity of 824 MW, at the HV bus. The derived plant capacity factor

for the optimum plant output is 0.77.

- Permanent access by road from either Happy Valley-Goose Bay, or Labrador City, via the Trans-Labrador Highway and a site access road.

- Stabilization of the north spur is required and can be done both economically and reliably with the expansion of the existing pumpwell system, along with:
  - provision of downstream erosion protection and downstream stabilizing fill;
  - local topcutting and unloading of portions of the high, steep side slopes;
  - provision of erosion protection and a stabilizing upstream berm at the upstream slopes;
- Protection of other areas of the reservoir rim from slope failures is not necessary.
- Downstream of the site, during and after construction, water level fluctuations will be reduced by the effective elimination of ice damming in the River below the falls.
- The probability of seismic activity resulting from impoundment of the reservoir is very remote.
- The amount of sediments deposited into the future Muskrat Falls reservoir is unlikely to result in significant loss of storage or other problems and will be significantly reduced if the Gull Island Development is constructed first.
- The switchyard is an open ring bus arrangement, located on the roof of the powerhouse. It includes equipment for two-230 kV transmission lines to the HVDC Converter Station near Gull Island, and sufficient space for a future third line to Goose Bay.
- The generator operating voltage has been selected as 15 kV.
- A 1215 person construction camp is provided at the Muskrat Falls site. One hundred existing housing units in the town of Happy Valley-Goose Bay, currently owned by the Department of National Defense, will be upgraded to provide family accommodations and this cost has been included in the capital cost estimate of the project.
- Construction power is provided from the existing 138 kV transmission line and by upgrading the existing substation, located near the site, to 6 MVA for the peak construction load.
- The Project Master Schedule has a Project release of January 1 of Year 1 with a Project start of June 1 of Year 1 which will provide first power on October 1 of Year 5, ie. 57 months after Project release and 52 months after Project start. With this schedule, river closure would be after the spring flood of Year 2.

- To achieve first power on October 1 of Year 5, based on Project release on January 1 of Year 1 and Project start on June 1 of Year 1:
  - the turbine-generator contract and the construction power transformer contract must be awarded 2 ½ months following Project release;
  - the clearing contract must be awarded 3 ½ months following Project release; and
  - the diversion contract must be awarded 5 months following Project release.
- The total duration of the Project schedule, from award of the turbine-generator contract to first power would be 54½ months and to full commercial operation would be 61 months.

1.10 PROJECT FEASIBILITY

Based on the studies and on the design development work described in this report it is concluded that, the Muskrat Falls Hydroelectric Development is technically and economically feasible. Further site investigations, studies and hydraulic model studies are recommended to firm up certain aspects of the design and reduce risks and contingencies in the tendered prices.

1.10.1 Technical Feasibility

The following highlights the technical feasibility aspects of the Project:

- Based on the results of previous and recent geotechnical investigations, it is concluded that the foundations are suitable for the structures described in the report.
- All proposed structures and electrical and mechanical equipment are of conventional design.
- Proposed large concrete semi-spiral cases are within precedent in the industry and existing operational experience.
- North spur stabilization layout is technically feasible and practical and more reliable than previous layout.
- Project release date January 1 of Year 1 and Project start date June 1 of Year 1 results in an effective realistic construction project schedule for the type and volume of work to be performed within the duration shown, taking into consideration the season and the weather conditions expected.

For the reasons mentioned above and further discussed in detail in the report, plus other reasons less outstanding, it is concluded that the Project recommended layout, as presented, is technically feasible, sound and reliable.

1.10.2 Economic Feasibility

The following highlights the economic feasibility aspects of the Project:

- Selected Project layout chosen as the most economical attractive development is based on the following elements:
  - Cost of gated spillway to pass PMF is minimized with both the North RCC dam topped with rubber dam and the South RCC dam used as overflow dams;
  - North spur stabilization recommended layout is more reliable and cost effective than the previous 1980 layout;
  - Project release date of January 1 of Year 1 and Project start date June 1 of Year 1 are best suited for construction of the various structure components, cost and schedule effective, considering the seasons and the weather expected for the type and volume of work to be done within the duration shown on the Project construction schedule.
- Further optimization of Project components is possible during final design.
- Annual energy output of Project is 5.53 TWh/y which results in a plant capacity factor of [redacted]
- For a plant capacity of 824 MW, the capital cost of approximately [redacted] makes the Muskrat Falls Hydroelectric Development economically attractive.

Therefore, for the reasons mentioned above and described in more detail in the report, including other reasons less outstanding, it is concluded that the Muskrat Falls Project is economically feasible, cost efficient and attractive.

1.11 RECOMMENDATIONS FOR FURTHER WORK

- It is recommended that a detailed field program be undertaken, prior to Project release, to further define the topographic and subsurface conditions at each structure location.
- It is recommended that hydraulic modelling be carried out to refine the design of the gated spillway and the diversion facilities.
- It is recommended that pre-engineering be carried out in Project Years –1 and –2 for the following elements:
  - Camps and infrastructure including access roads;
  - Field investigations, including subsurface investigations and surveying at structure

locations;

- Diversion inlet channel, diversion tunnels, inlet gates, guides and hoists;
- Main civil works;
- Mechanical - Electrical work; and
- Turbines and generators.

- It is recommended that preliminary work be completed for:
  - Tender and award of contracts for the turbines-generators and the construction power transformer 2 ½ months following Project release.
  - Tender and award a contract for the diversion and north spur stabilization immediately after Project start, June 1 of Year 1.
  - Tender and award a contract for preliminary works immediately after Project start, June 1 of Year 1.

- It is suggested that some investigations and studies could be undertaken in Year -3 to provide more precise information so as to minimize some risks with the Project. Such investigations and studies are outlined as follows and could be further detailed at a later date if NLH so wishes:
  - Geotechnical field investigation to confirm the quality and quantity of required aggregates from borrow areas and quarry material used for the construction of the Project;
  - Schedule modifications and variant studies;
  - Cost savings through planned phasing of Muskrat Falls and Gull Island hydroelectric developments;
  - Environmental review taking into consideration the environmental impact assessment and the development of resulting actions or mitigation works;
  - PMF study for the total Churchill River basins (upper and lower basins);
  - Power system study;
  - Churchill River system control strategy;
  - Determination of cement type for the project;
  - Alkali aggregate reactivity of aggregates;

- RCC dams concrete mix proportions;
- Rubber dam performance evaluation; and
- Optimization of the number of Kaplan turbines to be installed in the four unit powerplant.

Although not all the above work is absolutely required to be done in Year -3, it would be advisable to have all the parameters known and results available at the beginning of the detailed engineering in Year -2.

SECTION 2.0

INTRODUCTION AND BACKGROUND

SECTION 2.0

INTRODUCTION AND BACKGROUND

2.0 INTRODUCTION AND BACKGROUND

2.1 GENERAL

On June 29, 1998, Newfoundland and Labrador Hydro (NLH) retained the Consultant (SNC-AGRA Joint Venture, in partnership with the SGE Group Inc., Newfoundland Design Associates Limited, and Quadratec Inc.) to provide engineering services for preparation of a Final Feasibility Study Report to verify the viability of developing the hydroelectric potential of Muskrat Falls. The services are to include coordination with Acres International Ltd. (Acres) for optimization of the project energy output and capacity.

This "Final Feasibility Study" is intended to be the last extensive study prior to final design and implementation of the Project.

2.2 TERMS OF REFERENCE

The following terms of reference were established at the beginning of the study, for the provision of engineering services:

- to work with NLH and Acres personnel in the completion of an energy and capacity optimization study;
- to verify the Project parameters (head, layout and quantities), based on a new geodetic elevation survey (to be provided by NLH);
- to determine the bathymetric contours of the riverbed at all structure locations;
- to carry out geotechnical investigations, including aerial photo interpretation, drilling, testpitting, geological mapping and seismic surveys to confirm foundation conditions and sources of construction materials;
- establish survey controls and produce mapping of the structure locations at a scale of 1:1,000 with contours at 1.0 m intervals, based on black and white 1:8,000 scale aerial photographs provided by NLH. Also, produce reservoir mapping at a scale of 1:10,000 with contours at 2.0 m intervals to identify all flooded zones and reservoir clearing limits, based on coloured 1:12,500 scale aerial photographs provided by NLH;
- to establish design criteria and preliminary design of all structures;
- to carry out final optimization of all Project parameters, including the number and size of generating units;
- to evaluate the effects of winter conditions, including ice cover and frazil ice conditions;
- to review the spillway design capacity, based on the new PMF, provided by others;

- to determine the logistics of the Project;
- to prepare a Project Capital Cost Estimate and quarterly cash flow in June 1998 Canadian dollars, within an accuracy of ± 10%, excluding interest during construction;
- to prepare an estimate of Project annual operating and maintenance costs;
- to prepare an optimum Project schedule, based on Project release on January 1 of Project Year 1; and
- to prepare a Final Feasibility Study Report.

The 230 kV AC transmission lines from Muskrat Falls switchyard to the HVDC converter station near Gull Island are not included in the scope of this study.

The following changes to the Terms of Reference were implemented during the progress of the work:

- extend the scope of the telecommunications system to include:
  - microwave link from Muskrat Falls to the new microwave system between Gull Island and Happy Valley-Goose Bay; and
  - termination for the Fibre Optic line on the transmission line at Muskrat Falls. A cost allowance is included in the CCE for termination of fibre optics for the lines at the HVDC Converter Station, based on the Muskrat Falls cost.
- twenty-four copies of the draft report shall be submitted to the Client by November 24, 1998, and a presentation of the draft report shall be scheduled for December 3, 1998. Twenty-four copies of the final report shall be submitted by end of January, 1999.

2.3 REPORT ORGANIZATION

This report is presented in three volumes.

a) Volume 1 - Engineering Report

This volume presents relevant background investigations and engineering study details arranged in Sections and Appendices as follows:

- Section 1 is the Executive Summary which includes a brief project description, a Project Master Schedule, a summary of the capital cost estimate and a list of conclusions and recommendations for further work.

- Section 2 presents the Terms of Reference for the present study and the project background.
- Section 3 presents a description of the project area and existing surveys and mapping.
- Section 4 presents the flood and ice studies and recommendations for flood handling.
- Section 5 presents the power and energy studies.
- Section 6 presents the comparison of alternative project layouts.
- Section 7 presents a description of the selected scheme for project development.
- Section 8 presents a description of the river diversion.
- Section 9 presents a description of the dams and spillway.
- Section 10 presents a description of the stabilization of the north spur.
- Section 11 presents a description of the power facilities, including intake and powerhouse.
- Section 12 presents the 230 kV switchyard, control and protection arrangements.
- Section 13 presents the construction support facilities, including access and communications.
- Section 14 presents the construction materials and sources.
- Section 15 presents the construction program, including scheduling, contract packaging, and owner furnished equipment and materials.
- Section 16 presents the capital cost estimate, including assumptions, pricing data, methodology, operation and maintenance costs, potential cost savings, and management and engineering.
- Section 17 presents conclusions and recommendations.
- Appendix A presents the detailed capital cost estimate.
- Appendix B presents the Design Criteria.
- Appendix C presents the Interim Report on North Spur Stabilization.
- Appendix D presents the Construction Flood Study.
- Appendix E presents the Interim Report on Optimization of Number of Generating Units.

- Appendix F presents the LaSalle Consulting Group Inc. - Ice Study.
- Appendix G presents rubber dam experience lists and technical literature.

b) Volume 2 - 1998 Geotechnical Investigations

c) Volume 3 - Detailed Backup of Capital Cost Estimate.

2.4 PROJECT BACKGROUND

The first assessment of potential power sites on the Lower Churchill River, including Muskrat Falls, was prepared for British Newfoundland Corporation Limited (BRINCO) in a report dated April 1965 by Acres Canadian Bechtel (Reference 4). This preliminary assessment was based on information from existing maps and aerial photographs. No field reconnaissance was carried out.

In May 1965, further work was authorized including reconnaissance surveys and field investigations comprising test pits, test trenches and two boreholes in the north spur, a narrow neck of land located between the north bank of the river valley and the rock knoll immediately north of the falls. Preliminary engineering, structure layouts, quantities and cost estimates were prepared at that time, which identified Muskrat Falls as a viable site The interpretation of the field observations raised certain questions regarding seepage and stability in the north spur connecting the rock knoll with the north bank. No further work was carried out until 1975, when a geotechnical review of the 1965 layout was commissioned. The results of this study for Gull Island Power Company Limited by Lower Churchill Consultants suggested changes to the 1965 layouts and identified a program of more detailed field investigations (Reference 11).

In 1977, Newfoundland and Labrador Hydro engaged Acres Consulting Services to undertake further field work, including a study of the landslide areas in the vicinity of the site and an update of previous preliminary layouts and cost estimates (Reference 14 and 15).

With increasing interest to carry out full engineering report studies for the Lower Churchill projects, other preparatory engineering work was carried out in the spring of 1979 by Acres (Reference 17). This work, for Newfoundland and Labrador Hydro, included definition and preparation of specifications for a detailed exploration program at Muskrat Falls, study of alternative intake and powerhouse designs, review of ice studies and diversion works, review of the design floods and additional studies on schedule and support requirements. The results of these activities were presented in a report dated May 1979.

Subsequently, in 1979, SNC-Lavalin Newfoundland Ltd. was appointed as engineering advisor to the Lower Churchill Development Corporation (LCDC). Their initial assignment included a detailed field investigation program and engineering study for the potential development of Muskrat Falls. As a result of this study, it was established that development of the Muskrat Falls site, for a capacity of 618 MW, was technically feasible.

The study culminated in an engineering report, dated March, 1980 (Reference 21). Six main alternative project layouts, called Variants, were developed, each comprising 3 units of 206 MW capacity. Several variations of each alternative were considered and the six Variants were ultimately selected through a process of optimization. Variant 5 was ultimately accepted by the Board of Consultants and the Client, LCDC, as the preferred option for a 618 MW development.

The 1979 study also included an investigation into the stabilization requirements for the north spur. The recommendations made to resolve this concern included stabilizing rockfill berms, a pumped drainage curtain along the axis of the spur and erosion protection and relief wells downstream.

In 1989, SNC carried out a supplemental development layout study (Reference 35) of the Muskrat Falls site, for a total installed capacity of 824 MW. Four alternative layouts, designated as Variants 7 to 10, inclusive, were evaluated, each comprising 4 units of 206 MW capacity. This Study did not include Variant 5 from the 1980 Study because Variant 5 was considered to have a developmental limit of three units, due to space limitations and the presence of a fault zone located at the northern extremity of its three unit powerhouse. However, Variant 3 of the 1980 Study was revisited in this Study, and was renamed Variant 7, because in 1980, it was determined to have a similar capital cost to Variant 5, and only a slightly longer construction schedule. Variant 7 would not have this limitation on the number of units.

In accordance with a proposal dated June 1, 1998, SNC-AGRA were retained by Newfoundland and Labrador Hydro to carry out the current Final Feasibility Study which comprises a review of the previous studies, conducting further site investigations and, ultimately, the determination of the most practical and economic solution for development of the Muskrat Falls site.

SECTION 3.0

PROJECT AREA

SECTION 3.0

PROJECT AREA

3.0 PROJECT AREA

3.1 LOCATION, ACCESS AND SERVICES

The Muskrat Falls Hydroelectric Development is located on the lower reaches of the Churchill River in Labrador approximately 32.5 km upstream of the town of Happy Valley-Goose Bay as shown on Plates 1 and 2. Presently at Muskrat Falls, the river has an immense drainage area in central Labrador totaling 92,355 km^2. Approximately 75% of this drainage is currently regulated by the Smallwood and Ossokmanuan reservoirs of the Churchill Falls Project. After the diversion of the St-Jean and Romaine Rivers, the total drainage area at Muskrat Falls will be 101,938 km^2 of which 77% will be regulated by the Ossokmanuan and Smallwood reservoirs. Completion of the two lower Churchill sites, Muskrat Falls and Gull Island, would develop the available remaining head between Churchill Falls and Lake Melville.

Access to the site is available from Central Canada and Happy Valley-Goose Bay via the newly upgraded all weather Trans-Labrador Highway. Goose Bay provides an ice free shipping port for seven months a year, an international class airport and the usual amenities of a small town.

No means is currently available for crossing the Churchill River other than by helicopter or boat either upstream or downstream of the falls. Temporary access to the south shore during construction will be obtained by barge, downstream of the falls, and over the cofferdam. Subsequent permanent access to the powerhouse will be over the completed dams, spillway and intake/powerhouse structures.

Camp facilities are not currently available at Muskrat Falls. For fast initial mobilization, accommodation could be provided by renovation of existing facilities in Goose Bay. This, however, should only be considered as a temporary measure.

An existing 138 kV transmission line from Churchill Falls provides power to Happy Valley-Goose Bay. An existing substation approximately 3 km from the site will initially provide up to 2.6 MW of construction power. These facilities will be upgraded to provide additional construction power as construction progresses.

3.2 TOPOGRAPHY AND VEGETATION

The Churchill River between Gull Island and its mouth at Charles Point in Goose Bay has eroded down through in a broad estuarine and marine fluvial terrace which reaches a maximum elevation of about 80 m. This terrace overlies an older rock channel some 2 km wide with its thalweg more than 100 m below sea level. The new existing river channel occupies a bed approximately 1 km wide in these sedimentary deposits.

At the falls location, the river flows through a broad valley containing a variety of landforms and surficial sediments. The site is dominated by a well rounded hill on the north side of the river and by thick sediments that infill a pre-existing valley. The river flows within a well defined bedrock channel. Two falls, separated by approximately 0.8 km, create the feature known as Muskrat Falls. Distinct fluvial terraces dominate both sides of the river.

Vegetation adjacent to the river comprises a mature boreal forest with spruce and fir trees and abundant moss. On top of the Spur the vegetation is an open black spruce forest with abundant moss and reindeer lichen.

The main features at the Muskrat Falls site are:

a rock knoll 800 m long x 400 m wide, rising with steep sides to a maximum elevation of about 140 m;

- a spur of the northern terrace connecting to the rock knoll. This terrace has a flat crest at El. 60 m with relatively steep side slopes both upstream and downstream;

- the south shore slopes gradually upward from the riverbed, indicating the presence of underlying bedrock.

On the south bank about 85% of the area is wooded with a mature spruce forest. The remaining 15% comprises open bog and scrub spruce. The knoll area is heavily wooded with mature spruce and fir except along the shoreline and top of the hill. The ground surface of the Spur is relatively flat and covered by an open black spruce forest with abundant sphagnum moss and reindeer lichen. A chain of kettle lakes are drained by a gully at the north boundaries of the Spur.

3.3 CLIMATE, HYDROLOGY AND SEDIMENTATION

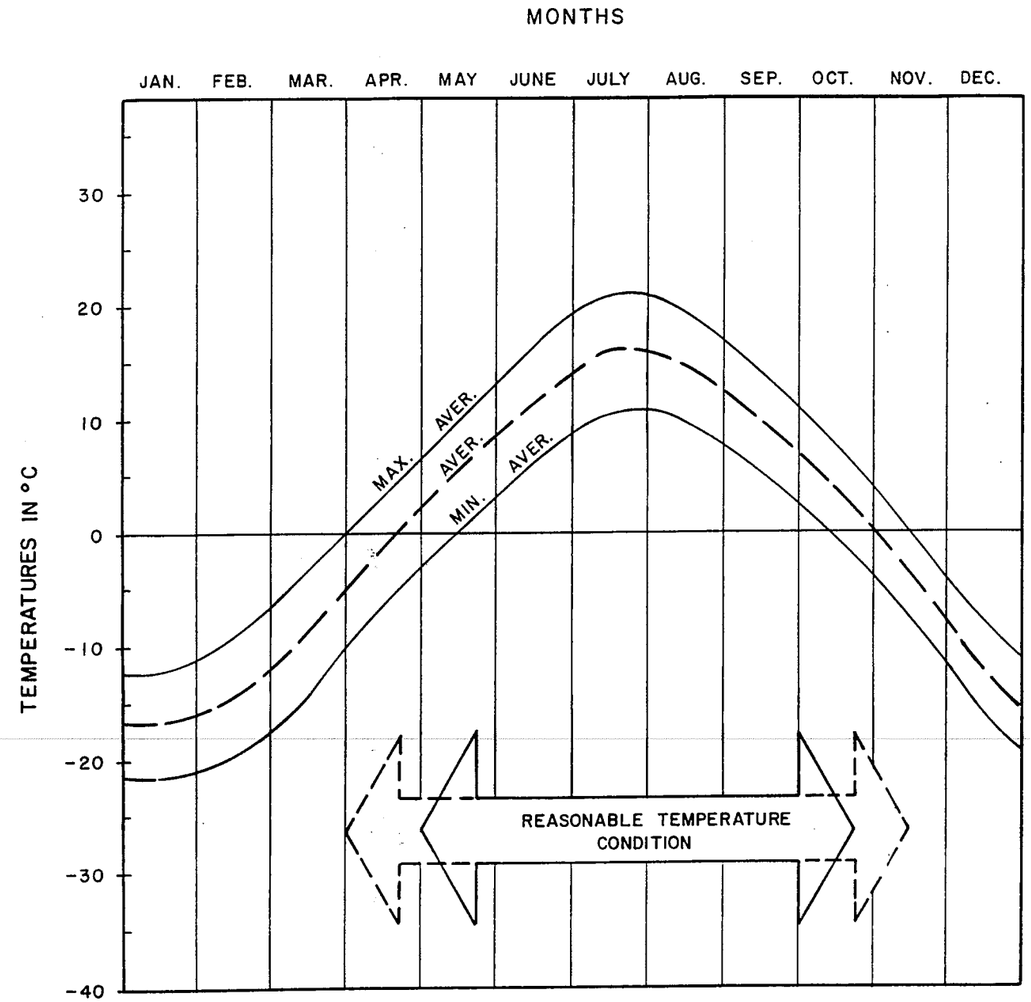
The Labrador plateau experiences a continental type of climate with a wide range of temperatures. In the Upper Churchill River basin, the recorded extremes are -48^0 C and +30^{0}C. The average annual temperature is -4^0 C. The Lower Churchill River basin climate is moderated by the Atlantic Ocean and experiences more of a maritime climate. The recorded extremes at Goose Bay are -40^0 C and +38^0 C with an average annual temperature of 0^0 C. Typical weather information by month including wind speeds are shown on Figure 3.1.

The average annual precipitation in the Upper Churchill basin is 764 mm, approximately equally divided between snow and rain. In the lower basin, as measured at Goose Bay, the average annual precipitation is 960 mm, of which 55% occurs as rainfall and 45% as snowfall. The frost free season in the Goose Bay area is about four months duration (June through September).

The total drainage basin area of the Churchill River at Muskrat Falls is 101,938 km^2 (including the Romaine and St-Jean river diversion) of which 78,850 km^2 or approximately 77% of the total are regulated by the Churchill Falls Power Project. The remaining 23,088 km^2 drainage of the Lower Churchill River basin at Muskrat Falls is unregulated.

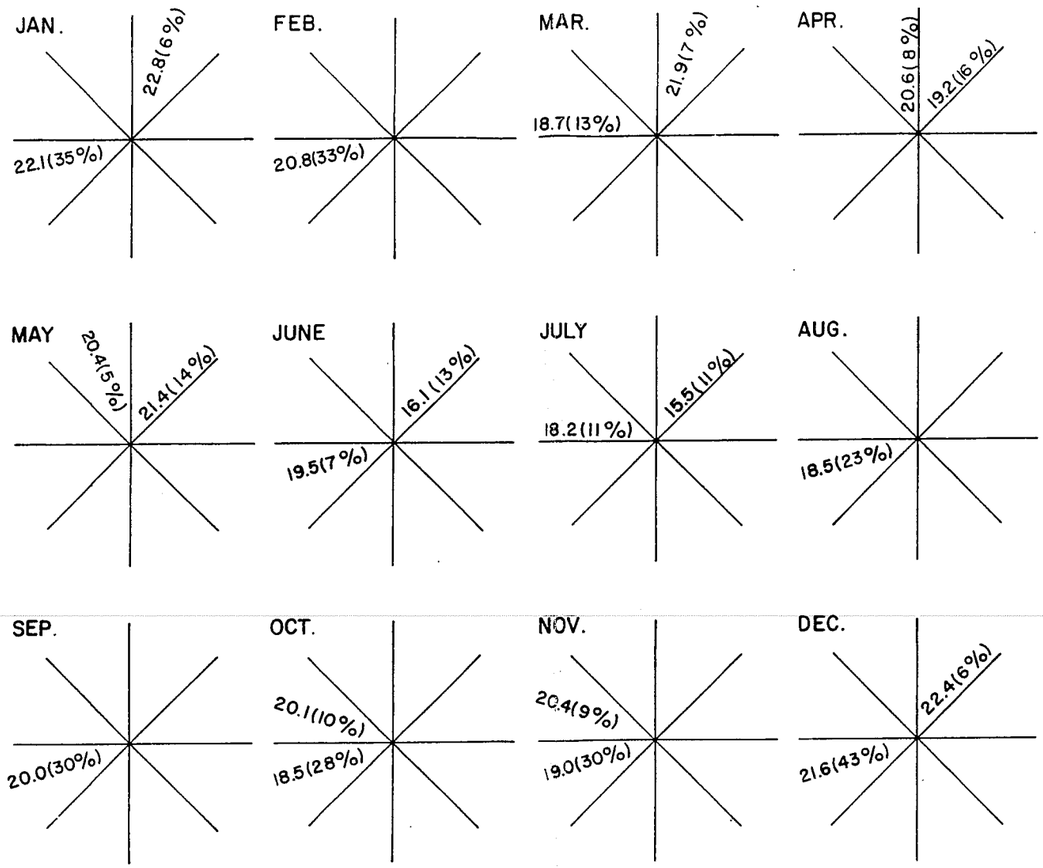
Flow at Churchill Falls is very well regulated by the large live storage of 31,200 x 10^6 m^3 provided by the massive Smallwood and Ossokmanuan reservoirs.

The long term average flow at Muskrat Falls is 1,841 m^3/s (2,026 m^3/s with the Romaine and St-



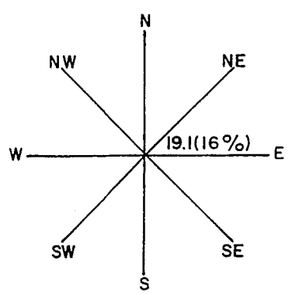
	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.
DAYS OF FROST	31	28	30	27	17	2	0	0	3	18	27	31
DAYS OF RAIN	1	1	2	4	11	15	16	16	13	11	5	2
DAYS OF SNOW	16	13	14	11	6	1	0	0	1	5	12	15
RAIN (MM)	2.5	5.1	5.1	10.2	43.2	78.7	101.6	94.0	73.7	45.7	43.2	5.1
SNOW (MM)	711	584	711	432	178	25	0	0	25	254	533	660

WIND



MAX. HOURLY : 77.2 km / h
MAX. GUST : 128.7 km / h

WINDS FROM DOMINANT DIRECTION AND HIGHEST AVERAGE VELOCITY WINDS ARE SHOWN.



19.1 = AVERAGE VELOCITY km/h
16 % = DIRECTION FREQUENCY

LOWER CHURCHILL RIVER
WEATHER INFORMATION

Figure 3.1

Reference: SNC Lavalin 1980 Engineering Report Volume 1, Figure 2.1 Weather Information

Jean diversions).

The Lower Churchill River flows through a relatively straight channel, from west to east. The River maintains a balance between its flow, the sediment discharge, slope, and cross section, as evidenced by the regular shape of the valley and the numerous sand bars in the channel. The main source of sediment is from bank erosion.

The construction of the Muskrat Falls Development will result in only minor sedimentation of the reservoir and negligible degradation in the downstream channel, limited by the small natural drop in elevation of approximately 2.0 m between Muskrat Falls and Lake Melville, 46 km downstream of Muskrat Falls.

Lake Melville is a tidal water body having a tidal range of about 0.75 m. An inspection of gauge charts from the Water Survey of Canada's station D3ØE007, at the base of Muskrat Falls, indicated that this tidal effect does not reach as far up-river as Muskrat Falls.

3.4 GEOLOGY

3.4.1 Regional Geology

The Muskrat Falls area is located within the Grenville Structural Province of the Canadian Shield. This Province extends from Hudson Bay in the west to the Atlantic Ocean in the east. Local rocks are most often northeast trending, southeast dipping structures with radiometric K/Ar dates of $905 \pm 150 \times 10^6$ years. Rock types are typically quartzofeldspathic to amphibolite gneiss. These rocks occur within a relatively high grade metamorphic facies, generally upper amphibolite to granulite range. The rocks within the Grenville are subdivided into seven major subdivisions and a number of subsidiary segments. The Muskrat Falls site lies within the Eastern Grenville Province.

The Muskrat Falls site is located in a zone of low seismicity (National Research Council, Zone 0).

3.4.2 Bedrock Geology At The Falls

Bedrock occurrence at the site is limited with outcrops exposed along the river channel and on the knoll. Quartzofeldspathic gneiss, of probable sedimentary origin in upper amphibolite facies, is the dominant country rock in the area. Locally, rocks are granite gneiss interbanded with amphibolite gneiss. Intrusions of megacrystic pegmatite of granitic composition parallel to the gneissic bands and occasionally cut them. The gneiss is well to semi-foliated, whereas the pegmatite is fresh and generally unfoliated. All rocks encountered were relatively hard and crystalline with a well-developed foliation that generally runs parallel to the river in an east west direction and dip moderately to steeply towards the south. On a local scale the gneissic structure orientation can vary considerably. Details of the local surficial and bedrock geology are found in Volume 2 under each structure heading.

3.4.3 Surficial Geology and Geomorphology

The proposed Muskrat Falls Hydroelectric site is located along the Churchill River approximately 46 km west of its termination at tidewater in Goose Bay. At the falls location, the river flows through a broad valley containing a variety of landforms and surficial sediments. The site is dominated by a well rounded hill on the north side of the river and by thick sediments that infill a pre-existing valley. The river flows within a well defined bedrock channel comprised mainly of gneiss and minor pegmatite. Two falls, separated by approximately 0.8 km, create the feature known as Muskrat Falls. Distinct fluvial terraces dominate both sides of the river.

The area has been glaciated. Bedrock in the area has been scoured and polished by the ice action. Hilltops are rounded and the sides of many valleys are steep and U-shaped. Subsequent to the glaciation, the valley was submerged and then re-emerged with various episodes of sedimentation and erosion. Glacial polish, striae, fluting and chattermarks can be observed on exposed unweathered outcrop near the river.

3.5 SURVEYS AND MAPPING

Detailed information on surveys and mapping available in the Muskrat Falls area is described in the following sections.

3.5.1 Aerial Photography

In addition to the high level aerial photography carried out by the Canadian Government, four air photo surveys have been made of the project area. The first, at a scale of 1:40,000, was taken in September 1963 by Lockwood Survey Corp. In July 1965, additional surveys were carried at a scale of 1:12,000 by Lockwood Survey Corp. In 1973, aerial photography at a scale of 1:36,000 was taken by Photosur Inc. In July 1979, Atlantic Air Survey Ltd. completed black-and-white aerial photography at a scale of 1:25,000 and color air photography at a scale of 1:12,000 in the immediate project area.

In July 1998, Airborne Sensing Corporation completed black and white aerial photography of the site at Muskrat Falls and colour photography of the reservoir between Muskrat Falls and Gull Island. The site photography was at a scale of 1:8,000 and the reservoir photography was at a scale of 1:12,500. The limits of the site photography are shown on Plate 4. The reservoir photography covered the area between Muskrat Falls and Gull Island Rapids, and is extended as required to include all areas up to the 42 m contour.

3.5.2 Mapping

Existing topographic maps of the Muskrat Falls area are available at a scale of 1:1,000,000, 1:500,000, 1:250,000 and 1:50,000 from the Department of Government Services and Lands, Lands Branch. Elevations are presented at a maximum detail of 50 feet contour intervals.

Detailed topographic mapping at a scale of 1 in = 200 ft. with a contour interval of 5 ft. was

prepared in 1965 for the site area. This mapping by Lockwood Survey Corp. was supplemented by underwater soundings with 10 ft contours in the area downstream of Muskrat Falls.

General mapping and site specific mapping were carried out in 1979 by Atlantic Air Survey Ltd. The general mapping was at a scale of 1:25,000, covering the area from below Muskrat Falls to above Gull Island. The site specific mapping was at a scale of 1:2,000 with 2 m contour intervals. The topographic mapping was supplemented by bathymetric surveys of the river channel upstream and downstream of the falls, also with 2 m contour intervals.

In 1998, Geodata Ltd. completed new topographic mapping for both the Muskrat Falls site and the reservoir from Muskrat Falls to Gull Island. The site mapping was prepared with a contour interval of 1 m and the reservoir mapping was prepared with a contour interval of 2 m.

Also in 1998, GeoScott Ltd. carried out a bathymetric survey using the ground probing radar (GPR) technique, in the River between a point just upstream of the upper falls and a point just downstream of the lower falls. Bathymetric mapping was prepared with a 1 m contour interval. The extent of this mapping is shown on Plate 4.

The topographic and bathymetric mapping were referenced to three-dimensional datum established on site by Parrott Surveys Ltd., under contract to NLH. The horizontal coordinates of the reference datum were based on NAD 83 and the vertical elevation was based on geodetic datum.

3.5.3 General Surveys

The level traverse surveys in the Muskrat Falls project area are as follows:

- *Desbarats - 1947*
  - A first level traverse survey was carried out by Commander Desbarats in 1947 and established a bench mark with an elevation of 72.04 ft., located above the Muskrat Falls. This BM was the reference for the 1965 mapping discussed above.
- *Geodetic Survey of Canada (GSC) Survey Line 66-B*
  - In 1966, a 3rd order level traverse, line 66-B, was carried by the Surveys and Mapping Branch of the Department of Energy, Mines and Resources Canada along the Churchill River between Churchill Falls and Goose Bay.
- *Geodetic Survey of Canada (GSC) Survey Line IF-72*
  - In 1972, a 1st order level traverse, line IF-72, was completed along the Trans-Labrador Highway from BM-136, located on the survey line between Sept-Iles and Schefferville, and Goose Bay.

- *Adjustment to Survey Lines IF-72 and 66-B*

  - In 1977, Geodetic Survey of Canada adjusted the elevation of BM-136 mentioned above.

In May 1978, Geodetic Survey of Canada officially issued Rev. 1 for the survey line 1F-72 for the purpose of bench mark adjustments and conversion to metric system.

In the summer of 1979, during the field survey program, it was discovered that the traverse line 66-B along the Churchill River should be adjusted by approximately -0.97 m. This was confirmed by Geodetic Survey of Canada in a letter dated November 20, 1979. Therefore, for topographical maps done in July 1979 covering the Muskrat Falls area all vertical detail must be adjusted by -0.97 m (i.e. all elevations must be lowered by 0.97 m).

The 1979 vertical control survey program (3rd order of accuracy) is based on the geodetic datum station BM-72-F-005 (El. 103.766 m, as per Ret. 1 issued in May 1978 by Geodetic Survey of Canada, Surveys and Mapping Branch, Department of Energy, Mines and Resources Canada).

All reference points at the site are referred to coordinates according to Universal Traverse Mercator (U.T.M.) grid system.

The 1979 bathymetry survey program covered a portion of the Churchill River upstream and downstream of the Muskrat Falls site. The bathymetric survey of the upstream portion was done by Newfoundland and Labrador Hydro personnel in July 1979, and the downstream portion of Muskrat Falls by Geophysique France-Quebec.

3.5.4 Detailed Site Surveys

1998 Survey Program

The 1998 topographic survey program was based on the GPS survey control point established on site by Parrott Surveys Limited and the NAD 83 coordinate system. This control point, designated as GPS98G9101 is located at coordinates 5902179.64 N and 648906.30 E has geodetic elevation of 20.219 m.

All reference points at the site have been referenced to the NAD 83 coordinate system and the geodetic datum.

The 1998 bathymetric survey (underwater contour mapping) covered an area of the River from just upstream of the upper rapids to just downstream of the lower rapids. The survey was carried out by GeoScott Exploration Consultants Inc. using a helicopter-mounted ground probing radar (GPR) system. The details, description and results of the bathymetric survey are presented in Volume 2, Appendix C.

All elevations given in this report refer to the 1998 GPS survey, based on geodetic datum.

SECTION 4.0

FLOOD AND ICE STUDIES

SECTION 4.0

FLOOD AND ICE STUDIES

4.0 FLOOD AND ICE STUDIES

4.1 AVAILABLE FLOW DATA

Data on daily and monthly flows are required for the flood studies and for the power and energy studies (Section 5.0). Flow data for the Churchill River Basin are available from the following sources:

- period 1956 to 1971, from Water Survey of Canada flow gauging station 03ØB002 at Flour Lake, just upstream of Churchill Falls Forebay Reservoir;
- period 1955 to 1971, from Water Survey of Canada flow gauging station 03ØD003 at Lake 51 on the Unknown (Atikonak)River;
- period 1954 to 1998, from Water Survey of Canada flow gauging station 03ØE001 located at Muskrat Falls;
- period 1971 to 1998, as derived from Churchill Falls plant records (plant and spillway flows plus daily reservoir levels); and
- period 1973 to 1998, from Water Survey of Canada flow gauging station 03ØC006 at Gabbro Lake on the Atikonak River.

Some derived data, notably reconstituted daily inflows to Ossokmanuan and Smallwood Reservoirs, are also available from consultant studies (Reference 54).

For the Upper Churchill, this data base has been extended back to January 1956 by correlation, to produce a 41 year long series of monthly flows for energy studies (Reference 54). Flows originating on the Lower Churchill were obtained by subtracting Upper Churchill flows from flows observed at Muskrat Falls, to produce a 41 year sequence of monthly flows for energy studies (Reference 54).

Accurate daily flow data required for flood analyses on the Lower Churchill is limited to the period 1956 to 1998 when flows were measured simultaneously at both the Upper and Lower Churchill.

Data on the Churchill basin is summarized in Table 4.1 and on Plate 1.

Table 4.1
Churchill River Drainage Areas

Basin	Sub-Basin/River	Local Drainage Area (km^2)	Cumulative Drainage Area (km^2)	
			Without Diversion	With Diversion
Diversion from the Province of Quebec	St-Jean River above diversion	1,164	-	1,164
	Romaine River above diversion	8,419	-	9,583
Upper Churchill	Lake Joseph	7,070	7,070	16,653
	Ossokmanuan Reservoir	15,318	22,388	31,971
	Smallwood Reservoir	45,170	67,558	77,141
	West Forebay	1,098	68,656	78,239
	East Forebay	611	69,267	78,850
Lower Churchill	Gull Island	19,832	89,099	98,682
	Muskrat Falls	3,256	92,355	101,938

4.2 DERIVATION OF CONSTRUCTION FLOOD

In order to facilitate dam construction at Muskrat Falls, it will be necessary to divert the Churchill River around the dam site for a period of two years. Canadian practice is to accept a 5% risk for such temporary works; therefore requiring the diversion works (tunnel and cofferdams) for a two year risk period to be designed for the 1 in 40 year flood (Q_{40}).

Floods occurring at Muskrat Falls are made up of two components:

- unregulated flows from the Lower Churchill (the sub-basin below Churchill Falls); and
- flows released through the Churchill Falls Development.

Churchill Falls Development has two large reservoirs, Ossokmanuan and Smallwood Reservoirs, that permit significant control of flows discharged downstream to Muskrat Falls. It should be possible, therefore, to reduce the peak flows for construction floods at Muskrat Falls by cutting back on flow releases from Churchill Falls during critical peak flow periods on the Lower Churchill. This proposition was evaluated in a construction flood study that undertook the following tasks:

- to estimate construction flood magnitudes for river diversion facilities at Gull Island and Muskrat Falls;
- to develop reservoir operating procedures to reduce peak flows at Gull Island and Muskrat Falls, while avoiding or minimizing water (and energy) losses at Churchill Falls; and

- to test these procedures to demonstrate that the safety of Churchill Falls dams would not be jeopardized.

The major points of this study are summarized below. Further details are provided in Appendix D.

This investigation initially disclosed that the desired reductions in flow releases from Churchill Falls would not be permitted under the current (Reference 54) flood handling rules at Churchill Falls that were designed to protect against the probable maximum flood (PMF). Additionally, it was found that river diversion works at Muskrat Falls would have to be designed for a Q_{40} flood of 8,800 m^3/s, some 30% higher than recommended in previous studies. It was, therefore, desirable to investigate modifications to these rules.

The key to the proposed modifications is the ability to distinguish between conditions conducive to a possible PMF from conditions conducive to a less severe flood. Preliminary analyses show that the May 1 snow accumulation could be used as an indicator and that a PMF would require a May 1st snow accumulation with greater than 600 mm water equivalent, whereas the 1 in 10,000 year flood would require a snow accumulation with between 400 and 520 mm water equivalent.

The proposed modifications are as follows:

- If mid April snow accumulation is greater than 500 mm water equivalent, current operating rules apply.

- If mid April snow accumulation is less than 500 mm water equivalent, modified rules apply.

Modified Rules

Ossokmanuan Reservoir (see Figure 4.1 for Structure and Reservoir Locations)

- If the snow accumulation in Upper Churchill exceeds 300 mm water equivalent, no water transfer from St-Jean and Romaine Rivers is allowed during the flood season, throughout the diversion period at Gull Island or Muskrat Falls.

- Maintain the water level in Ossokmanuan reservoir at or below the current rule curve by continuously transferring water to Smallwood through the Gabbro control structure.

- Do not use the Ossokmanuan control structure to release water into Churchill River.

- Allow the Ossokmanuan reservoir level to rise up to the maximum flood surcharge level of 480.40[1] m on Ossokmanuan Lake for the construction period.

[1] Customary water levels are cited in this section for consistency with current operating practice at Churchill Falls; to obtain geodetic levels, 1.85 m must be subtracted from the levels cited.

N

BIRCH LAKE

OVERFLOW LAKE

MICHIKAMAU LAKE

LOBSTICK LAKE

LOBSTICK CONTROL STRUCTURE

SANDGIRT LAKE

SANDGIRT-LOBSTICK DIKES

TO ESKER

195 KILOMETRES FROM POWER INSTALLATIONS AREA TO ESKER

GABBRO CONTROL STRUCTURE

FLOUR LAKE

CHURCHILL RIVER

WEST FOREBAY

JACOPIE LAKE

WHITEFISH FALLS CONTROL STRUCTURE

CHURCHILL FALLS ROAD

JACOPIE SPILLWAY

TWIN FALLS AIR STRIP

EAST FOREBAY

GABBRO LAKE

CHURCHILL FALLS

AIRPORT

FOREBAY SPILLWAY

TRANS LABRADOR HIGHWAY

OSSOKMANUAN CONTROL STRUCTURE

UNKNOWN RIVER

TWIN FALLS POWER DEVELOPMENT

OSSOKMANUAN LAKE

CHURCHILL FALLS POWER INSTALLATIONS

230 KV TRANSMISSION LINES TO WABUSH AND LABRADOR CITY

735 KV TRANSMISSION LINES

FOR CLARITY OF PRESENTATION, ONLY MAJOR LAKES AND RIVERS ARE SHOWN

SAIL DIKES

SAIL LAKE

KASHESHIBAW LAKE

ORMA ACCESS ROAD

ORMA LAKE

ORMA DIKES

MACKENZIE LAKE

LEGEND

- SMALLWOOD RESERVOIR MAXIMUM LEVEL EL. 472.7
- OSSOKMANUAN RESERVOIR MAXIMUM LEVEL EL. 479.1
- WEST FOREBAY MAXIMUM LEVEL EL. 452.9
- EAST FOREBAY MAXIMUM LEVEL EL. 448.5
- CONTROL STRUCTURES
- DIKES

0 10 20 30
KILOMETRES

NOTE:
Elevations shown are based on local (Churchill Falls) datum. For Geodetic datum subtract 1.85m from these elevations.

Reference: Acres Consulting Services LTD. Churchill River Power and Energy Studies 1978 Report

CHURCHILL FALLS DEVELOPMENT
PLAN OF DEVELOPMENT

FIGURE 4.1

Smallwood Reservoir

- Lower the target reservoir level on May 1 if snow accumulation in Upper Churchill exceeds 300 mm water equivalent. Preliminary evaluation shows that a starting reservoir level set as 468.50 m is adequate to pass Upper Churchill floods up to the 1 in 10,000 year flood without spilling; however, further investigation may show that the same objective can be achieved with a higher starting level (forced production[2] or prespilling[3] could be necessary to reach the target level on May 1).

- Allow continuous water transfer from Ossokmanuan to Smallwood through the Gabbro control structure for the duration of the flood.

- Only the flow requested for power generation will be released from Lobstick control structure.

- The turbinable flow for the entire flood season will be 2,300 m^3/s, which is about 95% of the maximum turbinable discharge at Churchill Falls after Churchill Falls extension has been completed.

- The turbinable flow at Churchill Falls will be reduced stepwise to a minimum of 500 m^3/s, based on water levels observed at Muskrat Falls, to reduce the flood peak. Maximum generation at Churchill Falls will resume after flood recession has started.

- Allow the Smallwood reservoir level to rise up to the maximum flood surcharge level of 473.23 m during the construction period.

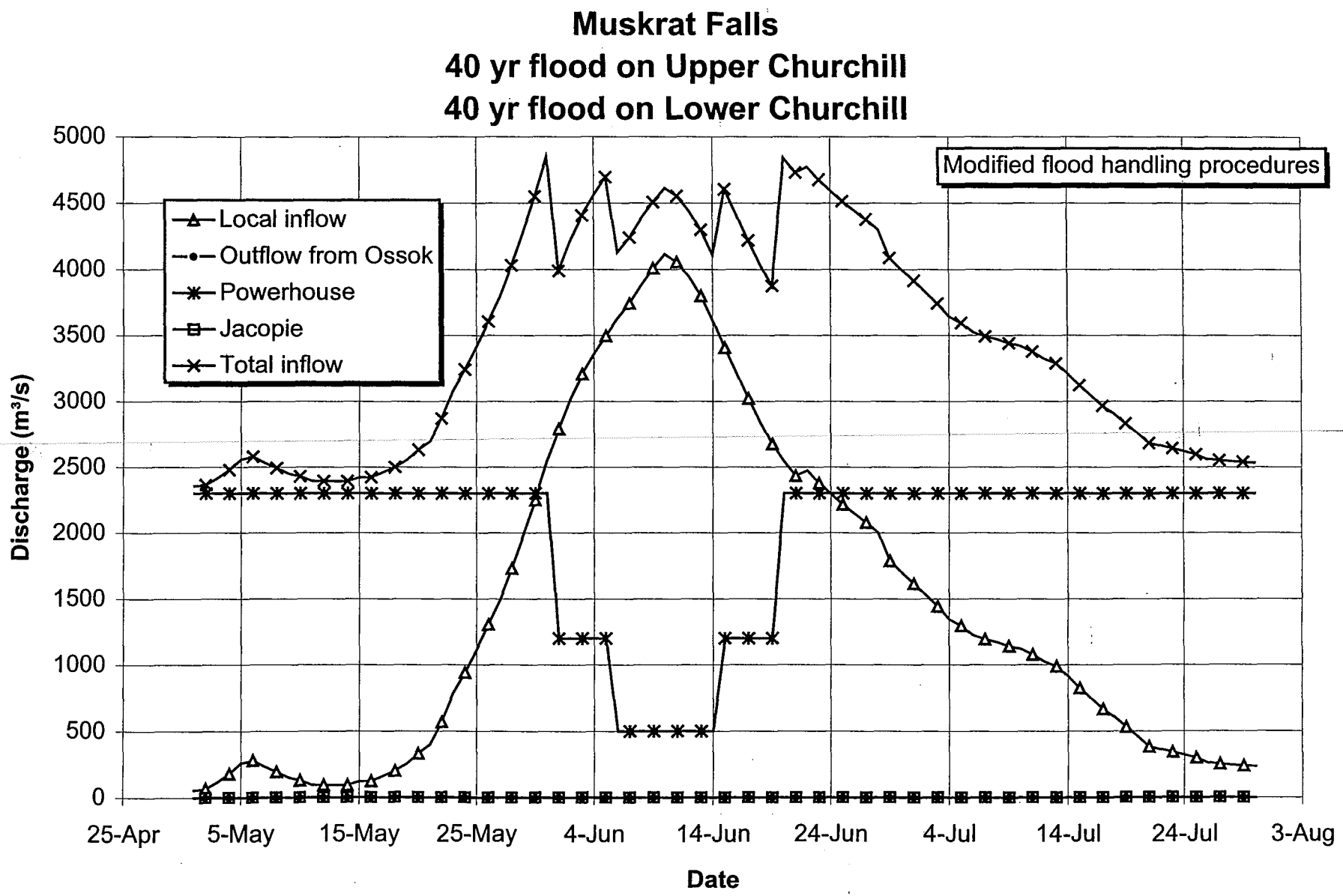
Figure 4.2 shows the application of these procedures for the 1 in 40 year flood, which is limited to a maximum flow of 5,300 m^3/s at Muskrat Falls with these procedures.

The procedures were also tested for more severe floods, up to $Q_{10,000}$, to verify that the safety of Churchill Falls dams/dykes would not be jeopardized; while maintaining the same level of flow control on downstream releases (see Figures 4.3 and 4.4).

The simulations also demonstrate that the flood handling procedures do not result in loss of energy production, on the assumption that the energy production from forced production and subsequent reduction in energy production due to cutbacks in flow discharges from Churchill Falls at peak flood periods on the Lower Churchill can be "rebalanced" by interconnected hydro plants in the Hydro Quebec System. It should also be noted that this exercise will be required in late May to Mid-June where the loads on the Labrador/Hydro Quebec System are relatively low and subject to fewer constraints than in higher load periods.

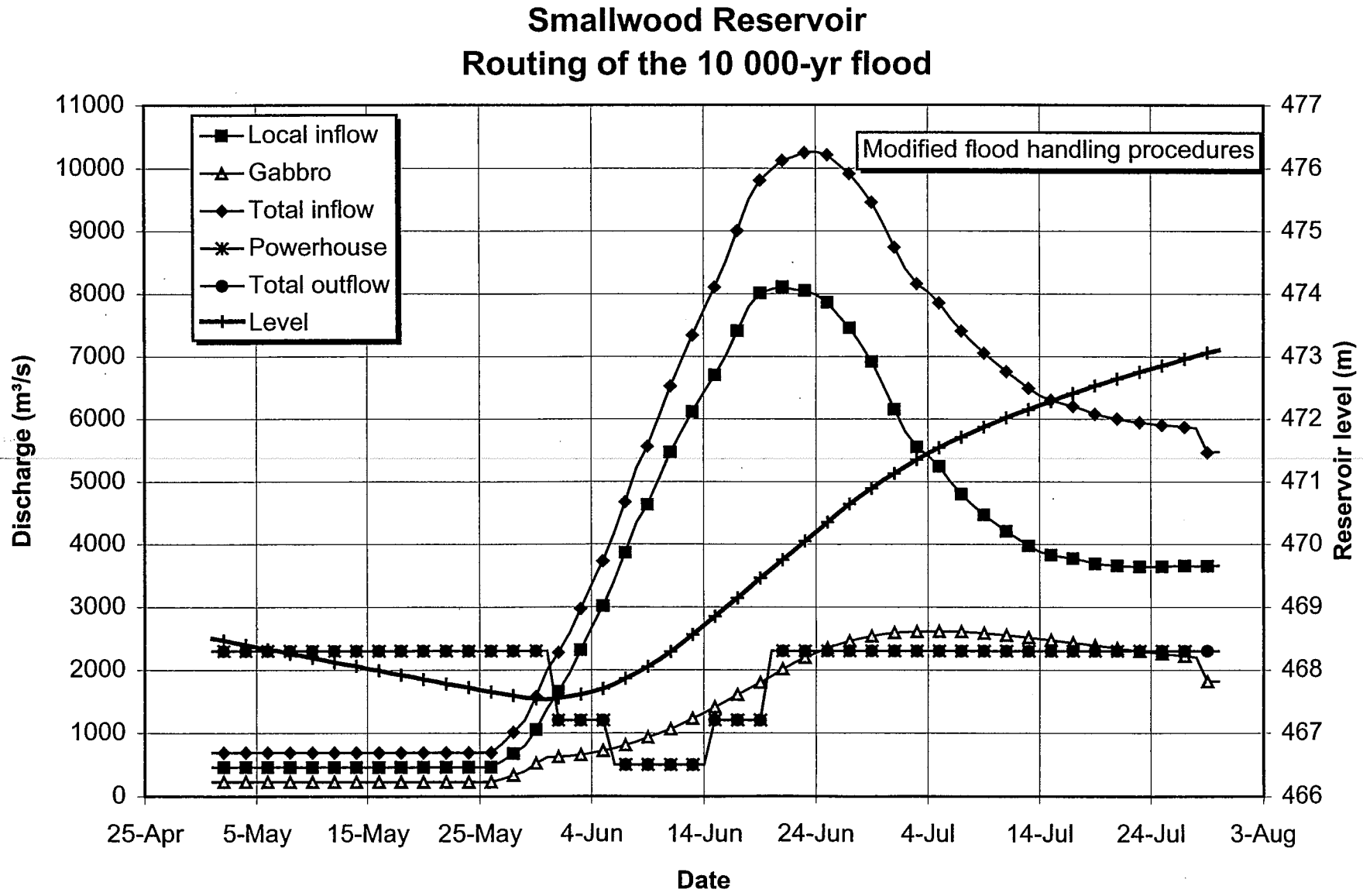
[2] "Forced Production" means operating Churchill Falls at the maximum plant output so as to lower the water level of Smallwood Reservoir as quickly as possible.

[3] "Prespilling" means opening Jacopie Spillway to release flow in a controlled fashion to lower the water level of Smallwood Reservoir, if forced production is inadequate to provide the desired reservoir level adjustment.



SNC-AGRA
Joint Venture

Figure 4.2

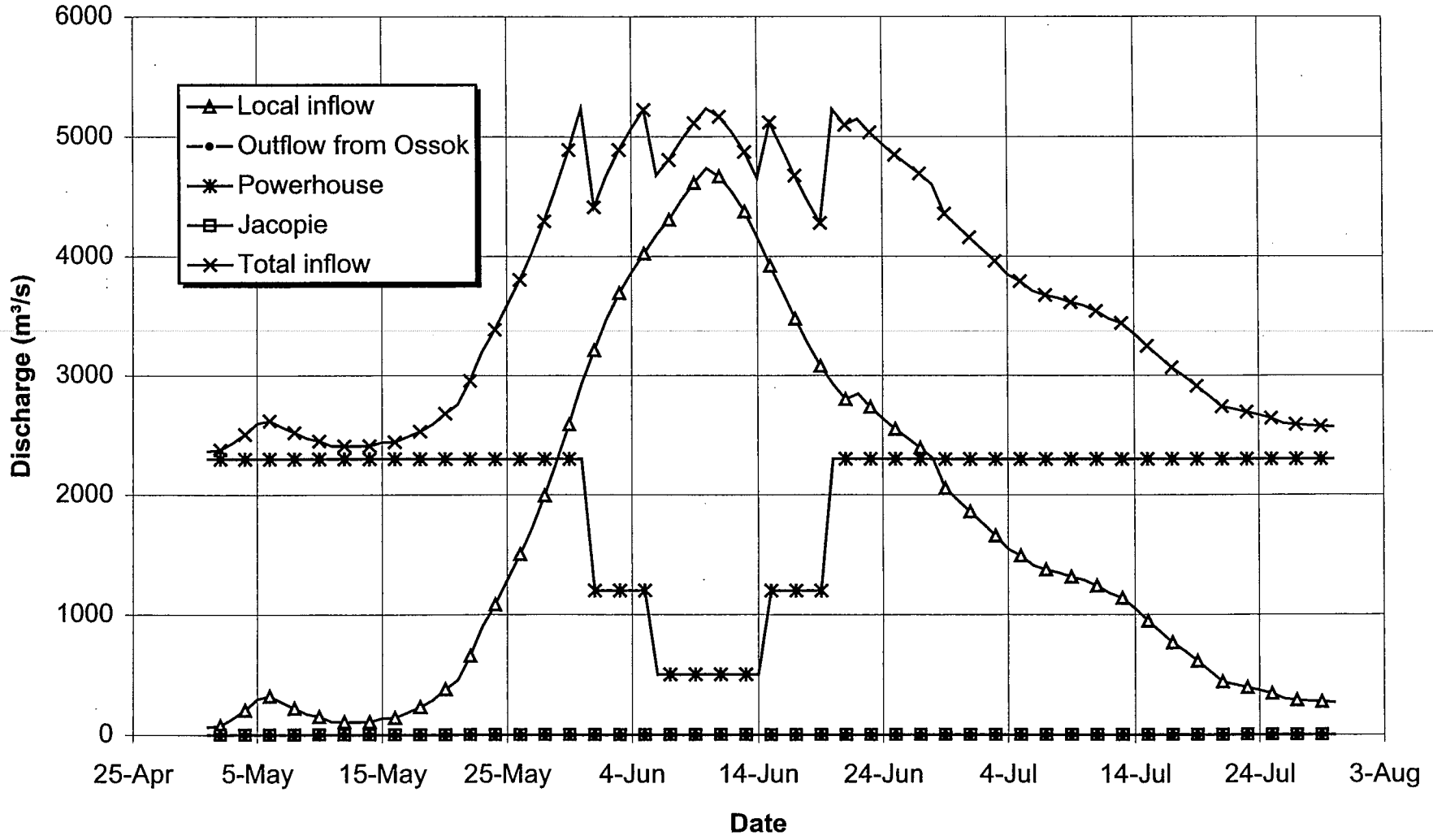


SNC-AGRA
Joint Venture

Figure 4.3

Muskrat Falls

40 yr flood on Upper and Lower Churchill with the modified flood handling procedures



SNC-AGRA
Joint Venture

Figure 4.4

The minimum release from Churchill Falls of 500 m^3/s is similar to the naturally occurring pre-development minimum flow and is believed to be sufficient to protect river habitat.

Energy losses by restrictions of diverted flows from Romaine and St-Jean rivers were not evaluated as the characteristics of the proposed diversion project were not known. However, it is believed that with minimal refinements to the modified flood handling procedures energy losses can be avoided or minimized to an acceptably small value.

Finally, flood routing calculations show that Gull Island's reservoir is too small to provide significant flood peak reduction during major floods.

Recommendations:

a) Muskrat Falls, Q_{40} = 5,300 m^3/s.

b) The proposed procedures be refined, as an extension to an updating of the PMF on the Upper Churchill, including an assessment of the impact of the Romaine and St-Jean river diversion on flood handling at Churchill Falls.

c) A suitable protocol should be negotiated between NLH and Hydro Quebec to facilitate application of the modified procedures during construction of Muskrat Falls.

(In the event that Churchill Falls Extension (CF2) is not constructed prior to Muskrat Falls, the recommended diversion design flood would remain unchanged. However, without CF2, the forced production flow capacity would be reduced and the required water level adjustments for Smallwood Reservoir may require some prespilling and consequent loss in energy production at Churchill Falls. Assessment of this issue should be undertaken during the final design stage.)

4.3 PROBABLE MAXIMUM FLOOD

Determination of the PMF for the Gull Island and Muskrat Falls Hydroelectric Developments was done by Acres International Ltd. under a separate contract with NLH (Reference 55). SNC-AGRA's role was to review the findings. This section summarizes the main points of this study.

In accordance with the Canadian Dam Safety Association Guidelines, Muskrat Falls dams and reservoir are classified in the high risk category for which the required project design flood is the PMF.

The PMF is defined as the flood that would be produced by the most adverse combination of flood producing factors that is meteorologically and hydrologically possible for a given region and season of the year.

For the Lower Churchill Basin, Acres selected a spring flood scenario to produce the maximum flood in the basin; that is, a flood comprising runoff from both snowmelt and rainfall. By following the recommendations provided by the Canadian Dam Safety Association, they developed a PMF based on the combination of a 1 in 100 year snow accumulation, the probable maximum precipitation (PMP) and critical temperature sequences to prime and melt the snow pack. The PMP was developed by maximizing storm precipitation over the Lower Churchill Basin (intermediate drainage area between Churchill Falls Generating Station and Muskrat Falls). Typical (but conservative) releases through Churchill Falls plant and spillway were assumed, consistent with a severe storm affecting the Upper Churchill Basin, but no detailed flood analyses were done for the Upper Churchill basin.

Acres recommended the following value for spillway design:

PMF, Muskrat Falls Development = 22,100 m^3/s

SNC-AGRA's validation of the PMF study consisted of a review of the Acres basic assumptions, notably the value of the PMP, snowpack, antecedent precipitation and temperature sequences. This review drew on recent SNC-AGRA and Hydro Quebec PMF studies for adjacent areas of Quebec having similar climatic and hydrologic regimes as the Lower Churchill Basin. (Only preliminary results were available at the time of this review). The most important flood producing factor is the PMP. It is therefore interesting to compare the consultants' (Acres) 66 hr PMP value with comparable determinations in Northern Quebec, on rivers with large drainage areas, >20,000 km^2:

Lower Churchill (Acres 1998) = 189 mm

Quebec Studies:

- Manicouagan River = 200 mm
- Ste. Marguerite River = 225 mm
- St. Maurice River = 190 mm
- Lake St-Jean Basin = 145 mm
- Great Whale River = 160 mm

Following this review, it was concluded that the methodology, assumptions and findings of the PMF study, relative to the Lower Churchill were satisfactory.

Earlier studies by SNC-Lavalin in 1980, suggest that the Acres assumed release from Churchill Falls, coincident with the peak on the Lower Churchill PMF may be excessive. Substantial savings could be achieved, if this release can be reduced, as the cost of spill capacity is about $4,800 per m^3/s at Muskrat Falls.

In view of the above, SNC-AGRA strongly recommend that the PMF studies be extended to include the entire Churchill basin. SNC-AGRA also recommend, as part of the same study program, that the flood handling procedures for Churchill Falls be reviewed and updated to take into account the effects on future downstream developments at Gull Island and Muskrat Falls.

For completeness, the PMF produced during summer and fall seasons should also be determined, since the possibility of a more severe PMF cannot be discounted "a priori". SNC-AGRA understands that this work is being done by Acres but was not available at the time of this review.

4.4 ICE STUDY

The 56 km section of the Lower Churchill River, upstream of Muskrat Falls to Gull Island, is fast flowing and, under existing conditions, remains open all winter with the exception of a few slow flowing stretches at Sandy Island Lake and Gull Lake. As a consequence of this situation, very large quantities of frazil and pan ice are produced which are then carried downstream to the pool below Muskrat Falls. This drift ice then accumulates beneath the thermal ice cover that forms over this pool with the onset of winter, to create a very large "hanging dam". As the hanging dam increases in size, the water level and ice surface rise to as much as 17 m above normal water levels, sufficient even to "drown-out" the Upper Muskrat Falls. This has occurred on two occasions in the last twenty four years, 1978 and 1979, see Table 4.2.

A key concern, during the construction of the Muskrat Falls Development, will be to control the ice problem so that the size of the downstream cofferdam and its cost can be kept as low as possible. Preliminary calculations done in an earlier study (Reference 53), suggested that the ice problem could be controlled by maintaining a suitable water level in the pool above the upper Muskrat Falls to develop and maintain an upstream ice cover.

The task of assessing the ice problem and recommending suitable means of control was done under subcontract, by LaSalle Consulting Group (LCG) (Reference 53). They were provided with the following information.

- river bathymetry in the form of 43 river cross-sections, describing the reach between Muskrat Falls and Gull Lake;
- reports from NLH's winter ice observer program for 1974/75 to 1991/92 (References 12, 18, 27, and 40), complete with videotapes for the most recent winters (1989 to 1992);
- flow data at Muskrat Falls (Reference 54, 1998); and
- temperature data from Goose Bay (Environment-Canada) and Churchill Falls (NLH).

The following paragraphs summarize LCG's study. Further details may be found in their report in Appendix F.

LCG's analyses were carried out using their proprietary freeze-up ice jam computer model FRASIL. This model utilizes a one-dimensional procedure for backwater calculation, but in addition takes into account explicitly the presence of ice, including anchor ice, shore ice, thermal ice cover and hanging dams. The program accounts for both the hydro-mechanical and thermal processes involved in river ice problems.

LCG first investigated the existing situation and adjusted their model to reproduce conditions observed in the river. From these analyses LCG concluded that the volume of ice transported downstream of Muskrat Falls during a winter would range between 200 million to 400 million m^3, depending on whether the winter was mild or severe. LCG acknowledge that their computation method is conservative and over states the amount of ice generated. After they adjusted their model to better represent the extent of border ice (and consequent reduction in open leads), they found ice production was reduced to a range of 145 - 170 million m^3. This is of the same order of magnitude as the underwater volume of the pool below the Falls (estimated as about 70 million m^3) and is considered more realistic.

Table 4.2
Summary of Ice Observations by NLH

Winter	Max. Stage Above Upper Falls (m)	Water Levels Below Falls[3]
1974/75	16.12	
1975/76	17.00	
1976/77	17.11	
1977/78	19.67[1]	
1978/79	20.13[2]	
1979/80	17.22	
1980/81	17.57	
1981/82	17.64	
1982/83	16.90	
1983/84	17.21	13.6
1984/85	17.19	10.63
1985/86	17.17	11.6
1986/87	17.10	10.2
1988/89	16.83	11.7
1989/90	18.28	14.46
1990/91	18.13	13.07
1991/92	17.77	9.67 record incomplete

(1) & (2) - Water levels affected by ice
(3) - Normal water level ≈ 3.0 m

For the purposes of this investigation, where control of ice is the main concern, it is considered appropriate to assume conservative values.

LCG then investigated the benefits of controlling the water level in the pool upstream of the diversion tunnels. They considered a range of water levels, flow and temperature scenarios, as summarized in Table 4.3. Net ice volumes reaching Muskrat Falls are also noted in this table. These detailed analyses were limited to the reach downstream of Gull Island Development and assume that this Development will be in operation and will intercept all ice produced upstream of Gull Island.

Table 4.3
Ice Scenarios Studied

Case	Water Level	November/December Flow[1] (m^3/s)	Freezing Index [2] (°C.days)	Volume of Ice Reaching Muskrat Falls (million m^3)
# 1	21 m	2,000	1,916	2.09
# 2		2,000	2,551	2.34
# 3		2,200	1,916	35.0
# 4		2,200	2,551	36.0
# 5		2,500 (up to Jan 10)	1,916	88.0
# 6	22 m	2,200	2,551	2.46
# 7		2,500 (up to Jan 10)	2,551	33.0
# 8	23 m	2,200	1,916	1.17
# 9		2,200	2,551	2.31
# 10		2,500	2,551	2.18

Notes:

1. *Flow sequences were based on the monthly flow sequence for 1969 (the year with highest flows) obtained from Acres energy study. Sequences of daily flows were obtained by interpolating between monthly means. Two other sequences were obtained by modifying the 1969 sequence, one assuming constant November and December flows of 2,000 m^3/s and another assuming constant November and December flows of 2,500 m^3/s. These sequences were chosen in consultation with SNC-AGRA, to cover a representative range of winter flows.*

2. *An average winter with 1,916 °C. days is represented by 1963/64 while a severe winter (approximately 1 in 40 occurrence), is represented by 1971/72 with 2,551 °C. days.*

Figure 4.5, gives the results for Case #9 and shows that an ice cover would be formed for about 40 km up to the inlet to Sandy Island Lake, except at a couple of short open water sections where flow velocities are very high. Their report also notes that this ice cover would form quickly after the onset of winter (by early to mid November) and thereby greatly reduce the production of frazil ice.

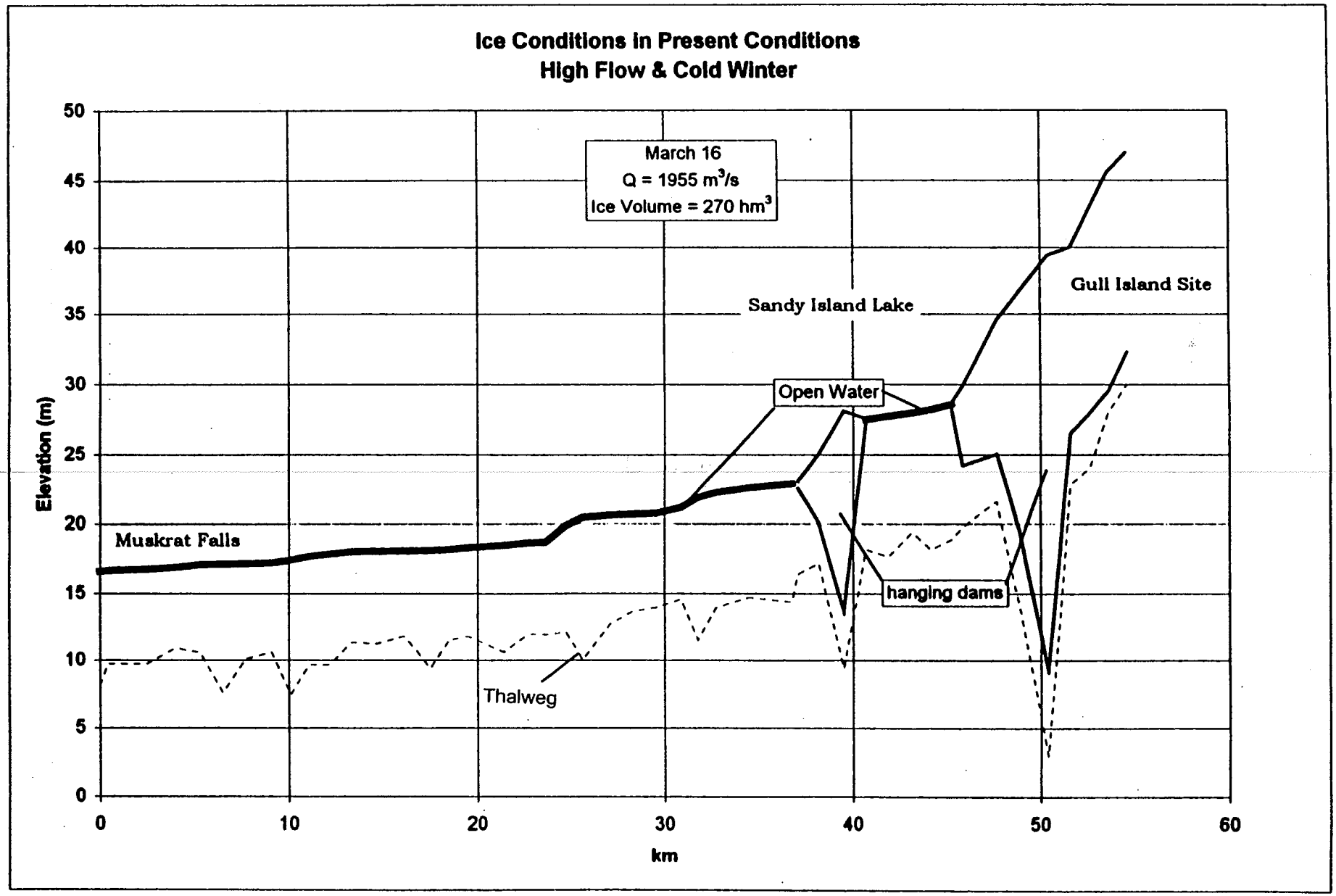


Figure 4.5
(Ice Study - LaSalle Consulting Group)

LCG further investigated the following issues:

- Effect of air temperature: They found that the maximum rate of ice production would occur with temperatures of -20 °C. For temperatures below -20°C, increased ice production is offset by reduction in open area.

- Effect on ice control without Gull Island: If Gull Island were not in place, incoming ice would not be completely trapped in Gull and Sandy Island Lakes but would accumulate as smaller hanging dams downstream of those points. The increase in ice volume reaching Muskrat Falls, would be minimal - estimated at 0.17×10^6 m^3 (versus 2.3×10^6 m^3, shown in Case 9 - Table 4.3).

- Stability analysis: Stability analysis was carried out to assess the risk of generalized breakup and loss of cover occurring during winter due to increased flow releases from upstream, up to the maximum capacity of the diversion works (5,300 m^3/s). LCG's analysis concluded that the cover would always stabilize by a process of shoving and thickening - and that there would be no significant risk of generalized mid-winter breakup.

Conclusions:

LCG concluded that ice control could be reliably obtained by maintaining water level at a minimum of El. 23 m, regardless of flow (up to 2,500 m^3/s) and air temperatures throughout the winter and will result in reducing the amount of ice reaching Muskrat Falls to less than 1% of the amount under existing conditions. However, LCG recommends a level of El. 24 m be adopted, given uncertainties in the accuracy of available bathymetric data.

SECTION 5.0

POWER AND ENERGY STUDIES

SECTION 5.0

POWER AND ENERGY STUDIES

5.0 POWER AND ENERGY STUDIES

5.1 GENERAL

The main objectives of the power and energy studies is to determine the optimum plant capacity and energy output. Given that Muskrat Falls will be part of a complex of several interconnected hydropower plants, the preferred approach is to model the performance of Muskrat Falls within a system context. This task, hereinafter referred to as the Energy Study, was carried out under a separate contract for Newfoundland and Labrador Hydro by Acres International Limited.

This section describes the essentials of the methodology and records the conclusions on the capacity and energy capability of the Muskrat Falls Development. For more detailed information, the reader should refer to the consultant's report (Reference 54).

5.2 METHODOLOGY

The consultant has used its in-house computer model to simulate the operation of the Churchill River Complex. This model is a regulation model that accounts for inflows, outflows, changes in reservoir elevations, power flows and energy production through a hydro system over discrete time intervals, by month in the case of the present study. The model represents a hydro system as a simplified network of channels, reservoirs, nodes (connecting points for channels), flow control structures and power stations. Operation of the system is then simulated respecting, physical constraints and operating rules and priorities.

The input data required to set up the model include:

- monthly flow sequences for the study period;
- reservoir characteristics;
- power station characteristics;
- structure data; and
- operational data.

The Churchill River Complex includes four hydro stations, as below:

- Churchill Falls 1 .....existing
- Churchill Falls 2 .....proposed
- Gull Island.....proposed
- Muskrat Falls .....proposed

The complex also includes two large storage reservoirs, Ossokmanuan and Smallwood Reservoirs. It is also proposed to divert flows from the headwaters of the Romaine and St-Jean Rivers from Quebec into Ossokmanuan Reservoir for the benefit of all four plants.

As the reservoirs at Gull Island and Muskrat Falls are relatively small, both plants are treated as run-of-river plants and their reservoir levels held constant over the monthly time step.

Demand on the system is determined by monthly demand factors - provided by Newfoundland and Labrador Hydro. Capacity availability was based on experience at Churchill Falls and recognizes seasonal variations in capacity. Maintenance is assumed to be scheduled in low demand low flow periods and as such there are no deductions for units out of service for maintenance. This is consistent with operating experience at Churchill Falls.

A schematic of the system is given in Figure 5.1.

The period adopted for this study was from January 1956 to December 1996, a total of forty-one (41) years.

5.3 PROJECT OPTIMIZATION

The Project optimization process assessed a variety of system configurations and carried out cost/benefit analyses for each. Input on plant characteristics and capital costs were supplied to Acres by the responsible feasibility consultants.

For Muskrat Falls, Acres' responsibility was limited to determining the optimal plant capacity, since the reservoir maximum operating level (the full supply level) had been previously determined at El. 39.0 m.

Initially, SNC-AGRA supplied Acres with a set of plant data, comprising plant characteristics and capital costs for plants of varying sizes (618 MW to 1236 MW).

The economic factors assumed for evaluating energy and capacity benefits, as supplied by Newfoundland and Labrador Hydro were:

- present value of 1 TWh per year for fifty year project life and 10% interest rate

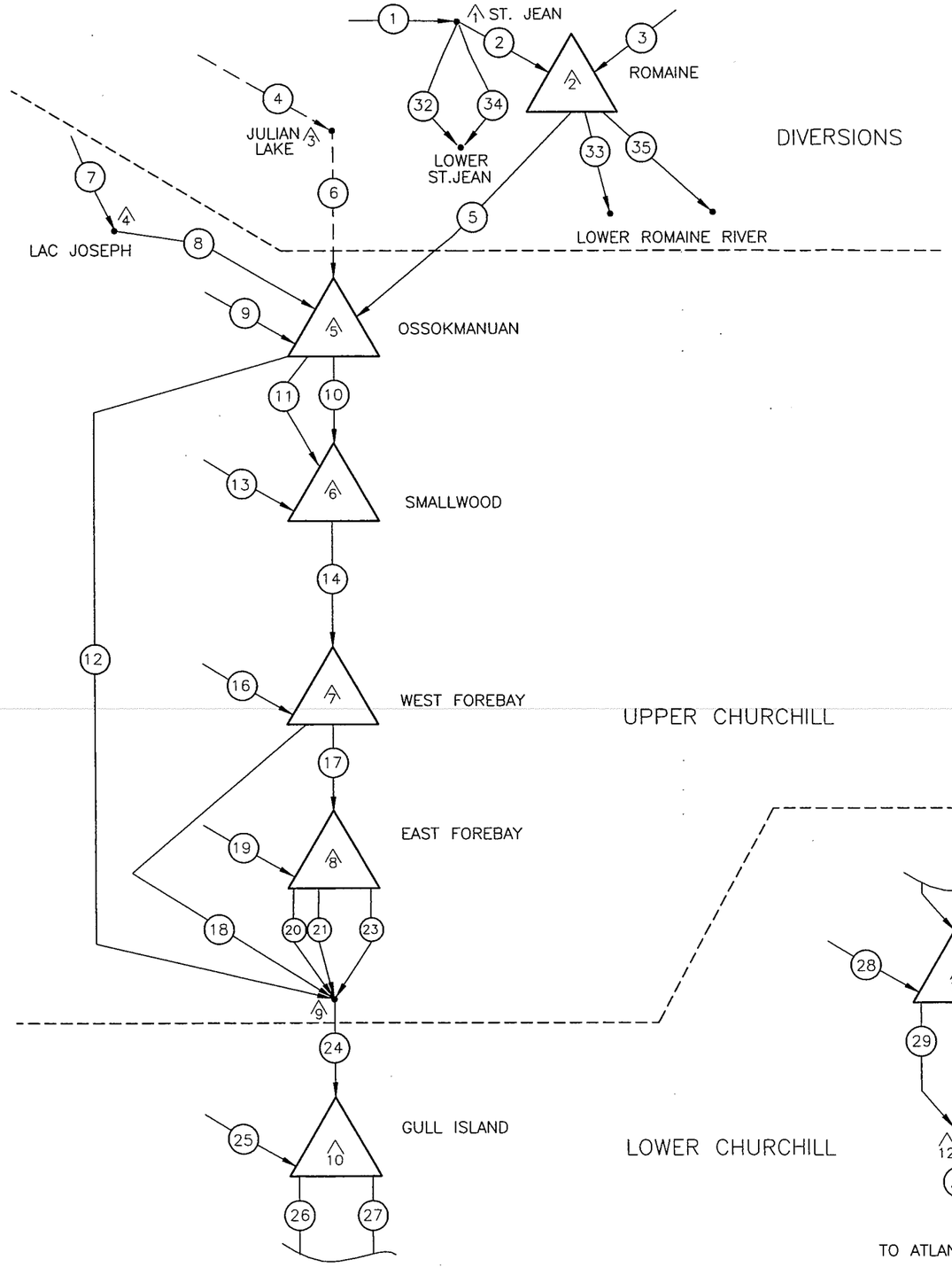
  = $225,000,000

- value of 1 MW capacity

  = $550,000

On the basis of their analyses Acres concluded that the optimal plant capacity of Muskrat Falls is 824 MW.

Further details on these analyses are provided in Acres Optimization Study Report (Reference 54).



RESERVOIRS/NODES

1 — ST. JEAN
2 — ROMAINE
3 — JULIAN LAKE
4 — LAC JOSEPH
5 — OSSOKMANUAN
6 — SMALLWOOD
7 — WEST FOREBAY
8 — EAST FOREBAY
9 — DOWNSTREAM OF CHURCHILL FALLS STATION (CONTRIBUTING TO TAILWATER LEVELS)
10 — GULL ISLAND
11 — MUSKRAT FALLS
12 — POWER CONTROL NODE

CHANNELS

1 — ST. JEAN LOCAL INFLOW
2 — DIVERSION CHANNEL ST. JEAN TO ROMAINE
3 — ROMAINE LOCAL INFLOW
4 — JULIAN LOCAL INFLOW
5 — REGULATED FLOW ROMAINE TO OSSOKMANUAN
6 — GENERAL FLOW JULIAN TO OSSOKMANUAN
7 — LAC JOSEPH LOCAL INFLOW
8 — GENERAL FLOW LAC JOSEPH TO OSSOKMANUAN
9 — OSSOKMANUAN LOCAL INFLOW
10 — GABBRO CONTROL STRUCTURE
11 — FLOW OVER GABBRO CONTROL STRUCTURE
12 — OSSKMANUAN CONTROL STRUCTURE
13 — SMALLWOOD LOCAL INFLOW
14 — LOBSTICK CONTROL STRUCTURE
16 — WEST FOREBAY LOCAL INFLOW
17 — WHITEFISH FALLS CONTROL STRUCTURE
18 — JACOPIE SPILLWAY
19 — EAST FOREBAY LOCAL INFLOW
20 — CHURCHILL FALLS POWER FLOW
21 — CF2 STATION POWER FLOW
23 — EAST FOREBAY SPILL
24 — GENERAL FLOW CF STATION TO GULL ISLAND
25 — GULL ISLAND LOCAL INFLOW
26 — GULL ISLAND POWER FLOW
27 — GULL ISLAND SPILL
28 — MUSKRAT FALLS LOCAL INFLOW
29 — MUSKRAT FALLS POWER FLOW
30 — MUSKRAT FALLS SPILL
31 — POWER CONTROL CHANNEL
32 — ST. JEAN FISHERY RELEASE
33 — SPILL TO LOWER ROMAINE
34 — SPILL TO LOWER ST. JEAN
35 — ROMAINE FISHERY RELEASE

("FROM CHURCHILL RIVER COMPLEX OPTIMIZATION STUDY" ACRES, 1998)

NOTE: JULIAN LAKE INCLUDED IN MODEL SETUP, NOT USED IN CURRENT MODEL SIMULATIONS.

MODEL SCHEMATIC – CHURCHILL RIVER SYSTEM FIGURE 5.1

5.4 OPTIMIZATION OF NUMBER AND SIZE OF UNITS

The optimization of the number and size of units was the subject of an interim report carried out as part of this study. The interim report is included in Appendix E.

The optimum plant capacity as determined by the Energy Study is 824 MW. Comparative costs and benefits were calculated for alternative power plant arrangements with 4, 5 and 6 units to determine the optimum number of units, as explained in Section 4.0 of the interim report in Appendix E.

Comparative costs, based on all propeller turbines, were compiled for the civil work, as shown in Table 5.1, and the electro-mechanical equipment, as shown in Table 5.2, for each of the three alternatives. The comparative energy gain (benefit), associated with increasing the number of units, was calculated (Table 5.3) based on the flow duration curve, resulting from the Energy Study (Reference 54), and the efficiency curves for 4, 5 and 6 Propeller unit installations (see interim report in Appendix E). The net total costs were then determined as total costs less the present value of benefits, as summarized in Table 5.4. The comparison of net total costs showed that a 4-unit installation is the most economical, approximately $59 million less than 5-units and approximately $118 million less than 6-units.

Comparative costs and benefits were also compiled for an installation with one of the Propeller turbines in the 4-unit and the 5-unit installations replaced with a Kaplan turbine, in order to improve the operational plant efficiency. The purpose of this was to ascertain if additional benefits could be obtained economically. The additional cost for a Kaplan turbine, approximately $4.5 million, was added based on a Kaplan turbine being 25% more expensive than a Propeller turbine. It was determined that with one Kaplan turbine, together with either three or four Propeller turbines, would provide approximately the same energy benefit as six propeller turbines (approximately $24.3 million - see Table 5.3). Therefore, there would be no benefit gained by increasing the total number of units from four to five or from five to six. The total costs less the present worth of the benefits for this case were compared in note 1 of Table 5.4 which shows that a 4-unit installation, with three Propellers and one Kaplan would be the most economical of the 4, 5 or 6-unit alternatives (approximately $73 million less than 5-units). With the additional energy benefit of approximately $24.3 million, this alternative would have a net total benefit of approximately $19.8 million as compared with a four Propeller arrangement, see Table 5.4 - Note 1.

In addition, comparative costs and benefits were compiled for an installation with all Kaplan turbines. Note 2 in Table 5.4 shows that there is no economical advantage of having all Kaplan turbines.

This study concluded that, a 4-unit installation with three propeller turbines and one Kaplan turbine is the most economical arrangement at Muskrat Falls. Furthermore, with an annual benefit of approximately $2.45 million (based on i = 10%, n = 50 years), for the additional energy obtained with one Kaplan turbine, the additional cost of the Kaplan would be recovered in about two years. It is recommended that this selection be studied further during the final design.

The four unit arrangement employs 206 MW units with a close coupled intake/powerhouse and concrete semi-spiral cases. The combination of a 9.0 m turbine diameter and 35.0 m head will put these units among the largest in the world. SNC-AGRA inquired about the experience with similar size units, notably Iron Gate (Danube River) and Gezouba (China). Information received on these projects confirmed satisfactory performance with the concrete semi-spiral cases. SNC-AGRA also carried out preliminary structural analyses of the concrete semi-spiral which showed that the loads could be handled using conventional reinforced concrete construction techniques.

5.5 FINAL CAPACITY AND ENERGY DETERMINATION

Following the selection of the number and type of units, the final data on plant characteristics were provided as input to the Energy study to determine the final values of capacity and energy benefits. The tailrace rating curve (T.W.L. vs Q) was also updated using water levels measured during the 1998 field program, points measured at Water Survey of Canada gauge 03OE007 and flows measured at the W.S.C. gauge 03OE001, above Muskrat Falls. This curve was further extrapolated up to the PMF value (22,100 m^3/s) assuming typical downstream channel geometry. Table 5.5 summarizes the final (selected) plant characteristics; while the tailrace (T.W.L. vs. Q) curve is shown in Figure 5.2.

Muskrat Falls Hydroelectric Development
Tailrace Water Level vs. Flow

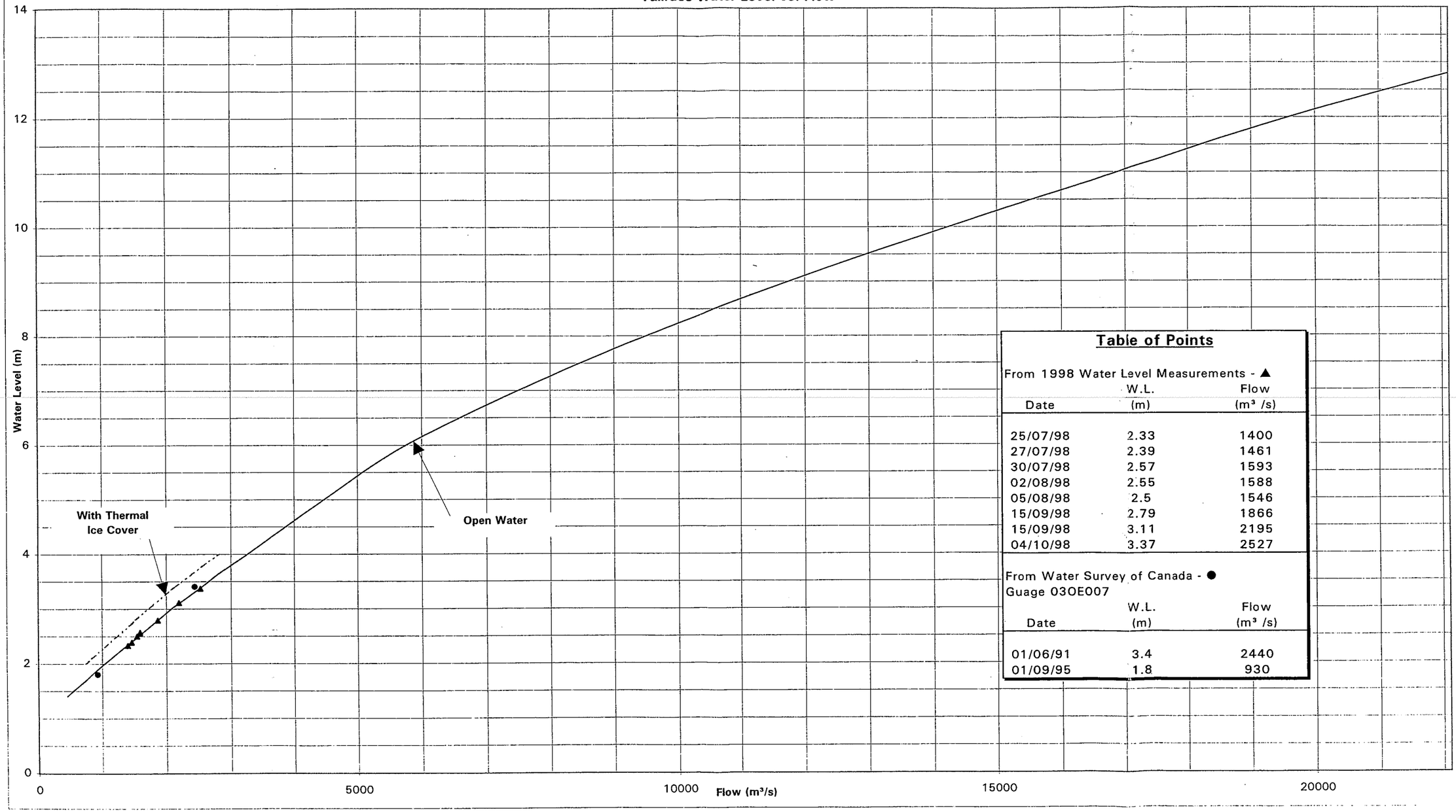


Table of Points

From 1998 Water Level Measurements - ▲

Date	W.L. (m)	Flow (m³ /s)
25/07/98	2.33	1400
27/07/98	2.39	1461
30/07/98	2.57	1593
02/08/98	2.55	1588
05/08/98	2.5	1546
15/09/98	2.79	1866
15/09/98	3.11	2195
04/10/98	3.37	2527

From Water Survey of Canada - ●
Guage 03OE007

Date	W.L. (m)	Flow (m³ /s)
01/06/91	3.4	2440
01/09/95	1.8	930

Figure 5.2

Table 5.1
Comparative Civil Costs

Description	Unit	4 units 665 m^3/s 81.8 rpm 212 MW, ea.	5 units 532 m^3/s 92.3 rpm 169.6 MW, ea.	6 units 443 m^3/s 100 rpm 141.3 MW, ea.
Parameters:				
N = number of units		4	5	6
Ne = equivalent number of units		4.45	5.47	6.48
R = repair bay length	m	29.70	27.30	25.70
S = unit spacing	m	32.97	28.90	26.87
d = turbine throat diameter	m	9.18	8.17	7.50
T = horizontal distance between u/s face of intake & vertical plane drawn at midway between the d/s face of the dt piers & the dt exit.	m	63.80	59.70	57.30
H = height of intake deck above foundation rock	m	46.00	45.50	45.00
h = gross head on the turbine	m	36.00	36.00	36.00
Concrete volume 1 unit (intake)	m^3	39,530	34,790	31,590
Concrete volume 1 unit (powerhouse)	m^3	26,590	20,100	16,370
Concrete volume 1 unit (total)	m^3	66,120	54,890	47,960
Concrete volume total	m^3	294,260	300,380	310,700
Cost @ $700/$m^3$		$205,982,000	$210,266,000	$217,490,000
$ difference in concrete			$4,284,000	$11,508,000
Powerhouse excavation	m^3	245,100	242,400	238,400
Powerhouse excavation difference	m^3	-	-2,700	-6,700
$ difference in excavation @ 30 $/$m^3$		-	-$81,000	-$201,000
Upstream approach channel excavation - dry	m^3	9,200	10,500	11,800
Tailrace channel excavation - dry	m^3	261,300	266,300	264,200
Tailrace channel excavation - wet	m^3	98,600	100,400	100,000
Channel excavation difference - dry	m^3	-	6,300	5,500
Channel excavation difference - wet	m^3	-	1,800	1,400
$ difference in excavation @ $30/$m^3$ dry & $60/$m^3$ wet		-	$297,000	$249,000
Total $ difference in civil		-	$4,500,000	$11,556,000

Table 5.2
Comparative Electro - Mechanical Equipment Costs
(All Propeller Turbines)

	Description	4 units 665 m³/s 81.8 rpm 212 MW, ea.	5 units 532 m³/s 92.3 rpm 169.6 MW, ea.	6 units 443 m³/s 100 rpm 141.3 MW, ea.
1	Propeller turbines	$72,992,000	$85,526,000	$91,962,000
2	Generators	62,557,000	71,639,000	79,658,000
3	Powerhouse cranes	1,274,000	965,000	843,000
4	Transformers	11,196,000	11,844,000	12,400,000
5	Switchyard equipment	15,698,000	17,026,000	18,194,000
6	Aux. Mechanical equipment	12,015,000	12,543,000	13,002,000
7	Aux. Electrical equipment	11,077,000	11,077,000	11,076,000
8	Intake equipment	35,401,000	36,230,000	38,573,000
9	Draft tube equipment	4,549,000	4,098,000	5,016,000
	Total Costs	$226,759,000	$250,948,000	$270,724,000
	Total $ difference in E / M	**-**	**$24,189,000**	**$43,965,000**

Table 5.3
Comparative Energy Gains
(All Propeller Turbines)

	Description	4 units 665 m³/s 81.8 rpm 212 MW, ea.	5 units 532 m³/s 92.3 rpm 169.6 MW, ea.	6 units 443 m³/s 100 rpm 141.3 MW, ea.
	Average Annual Energy (GWh)	5,436	5,500	5,544
	Avg. Annual Energy gain (GWh)	-	64	108
	Present Value of Energy Gain (based on $225,000/GWh/yr.)	-	$14,400,000	$24,300,000

Table 5.4
Comparative Total Costs
(All Propeller Turbines)

	Description	4 units 665 m^3/s 81.8 rpm 212 MW, ea.	5 units 532 m^3/s 92.3 rpm 169.6 MW, ea.	6 units 443 m^3/s 100 rpm 141.3 MW, ea.
1	Direct Costs:			
	- Civil costs (ref. page 2)	-	$4,500,000	$11,556,000
	- Electro mechanical costs (ref. page 3)	-	24,189,000	43,965,000
	Sub-Total difference - Directs	0	$28,689,000	$55,521,000
2	Indirect Costs:			
	- IDC @ 10%	0	7,172,000	13,880,000
	Sub-Total difference - Indirects	0	$7,172,000	$13,880,000
	Total $ difference - Capital Cost	0	$35,861,000	$69,401,000
3	Annual Financial Costs:			
	- Interest @ 10.00%	0	3,586,000	6,940,000
	- Sinking fund @ 0.026%	0	9,000	18,000
	- Insurance @ 0.10%	0	29,000	56,000
	- Interim replacement @ 0.20%	0	57,000	111,000
4	Annual O & M @ 0.30%	0	$108,000	$208,000
	Total $ difference - Annual Costs	0	3,789,000	7,333,000
5	PV of Annual Costs (i = 10%, n = 50 yrs)	0	$37,567,000	$72,705,000
	Total $ Difference in Costs	0	$73,428,000	$142,106,000
6	PV credit for Energy Gain (ref. page 4)	-	-$14,400,000	-$24,300,000
	Total $ Difference	-	$59,028,000	$117,806,000
	Note 1: With the substitution of 1 Kaplan in the 4 unit and 5 unit installations, and the same energy credit for all three installations:			
	Total $ Difference in Costs (compared to 4 propellers)	*$4,500,000*	*$77,928,000*	*$142,106,000*
	PV credit for Energy Gain	*-$24,300,000*	*-$24,300,000*	*-$24,300,000*
	Total $ Difference	*-$19,800,000*	*$53,628,000*	*$117,806,000*
	Note 2: With all Kaplan turbines and the same energy credit for all three installations:			
	Total $ Difference in Costs (compared to 4 propellers)	*$18,000,000*	*$95,928,000*	*$169,106,000*
	PV credit for Energy Gain	*-$24,300,000*	*-$24,300,000*	*-$24,300,000*
	Total $ Difference	*-$6,300,000*	*$71,628,000*	*$144,806,000*

Table 5.5
Final Plant Characteristics

Characteristic	Value
Normal operating level	39.0 m
Installed Capacity: -at the HV bus	824 MW
-at generator terminals.	832 MW
Weighted efficiency (turbine/generator)	92.5%
Flow and best efficiency	2,437 m^3/s
Efficiency at maximum plant flow	90.9%
Maximum plant flow	2,667 m^3/s
Head losses: -at best efficiency flow	0.43 m
-at maximum flow	0.50 m
Tailwater at best efficiency	3.3 m
Tailwater at maximum flow	3.5 m
Tailwater at mean flow	3.0 m
Net head at best efficiency	35.27 m
Net head at maximum flow	35.0 m
Tailrace Curve:	

Flow (m^3/s)	1,000	1,500	2,000	2,500	3,000	4,000
T.W.L. (m)	2.0	2.4	2.9	3.4	3.8	4.6

The final evaluation of benefits are:

- Optimal plant capacity: 824 MW (at the HV bus)
- Average annual energy: 5.53 TWh (at the HV bus)
- Plant flow capacity: 2,667 m^3/s
- Average plant flow: 1,980 m^3/s
- Average spill: 46 m^3/s
- Plant capacity factor 77%

The plant flow duration curve, illustrating the plant operating pattern is shown in Figure 5.3.

SECTION 6.0

ALTERNATIVE PROJECT LAYOUTS

SECTION 6.0

ALTERNATIVE PROJECT LAYOUTS

6.0 ALTERNATIVE PROJECT LAYOUTS

6.1 GENERAL

SNC-AGRA has reviewed the ten layouts previously studied by SNC-Lavalin in 1979-80 (Reference 21) and by SNC in 1989 (Reference 35), together with one other (Variant 11), at the request of Hydro Quebec. It was thought that Variant 11 might offer a more economic development and a shorter construction schedule than the Variants previously studied.

The following sections describe each of the eleven Variants. The Variants are described in order of River Schemes, Tunnel Schemes and Right Bank Scheme. For reasons explained later, Variants 5, 7, 10 and 11 were selected for a more detailed quantitative analysis.

Rock foundations are necessary for the key structures such as powerhouse, intake, spillway and RCC dams. At the Muskrat Falls site these occur in the river channel, in the rock knoll to the north bank and to the south bank.

During the early engineering studies, options involving the use of diversion tunnels or structures with concentrated discharges were rejected. Concentrated discharges raised concerns that increased back eddy erosion activity downstream, could trigger additional slope failures in the spur. Information obtained during the 1979 field investigations indicates that this concern was not valid. The prime cause of slope instability was found to be excessive internal pore water pressure. Toe erosion of the banks due to back eddies was found to be insignificant. This finding reopened the possibilities for site optimization.

The layouts are grouped into three categories:

- River schemes with the spillway and an intake/powerhouse forming a concrete dam in the river channel at the lower falls;
- Tunnel schemes with a powerhouse located on the west or east side of the rock knoll and large diameter tunnels connecting the forebay with the tailwater; and
- Right bank scheme with the spillway and an intake/powerhouse block located on the right (south) bank, adjacent to the lower falls, and a main rockfill dam across the river above the lower falls.

Generally, several variations of each of the basic layouts were considered and the basic Variants described in the following sections, and shown schematically on Plates 5 and 6 present the features of the optimized layout for each variant.

6.2 RIVER SCHEMES

The river schemes include developments with an integrated intake/powerhouse block located in the river channel at the lower falls. During construction, the river would be diverted around the construction site either by diversion tunnels (Variant 3), through a partially completed spillway

(Variants 1, 2 and 9), or through a diversion channel (Variant 11). In each case, river closure for diversion would be accomplished by means of a multi-stage cofferdam arrangement. In each of the river schemes, the intake/powerhouse would be constructed in the unwatered riverbed above the lower falls. In all, except Variant 9, the intake/powerhouse would be adjacent to the spillway. In Variant 9 the spillway would be located in the right bank. Abutment dams on the left and right banks would provide closure of the river valley.

6.2.1 Variant 1

In this layout, shown on Plate 5, the basic structures of spillway, intake and powerhouse would be located in the river above the lower falls. Diversion would be carried out using a two-stage cofferdam arrangement located upstream of the lower falls. Initially, the river would be diverted along the north side of the river with rockfill and cellular cofferdams. Cofferdams adjacent to high velocity flow, and in areas of restricted space, would be cellular steel sheet-pile structures. The rockfill cofferdam would be constructed from the south bank and connected to the cellular cofferdam which would be located along the center of the river channel, along the crest of the lower falls and closing to the south bank, below the falls. An integrated intake/powerhouse, and a partial spillway, would be constructed behind this cofferdam with the river diverted through the north side of the channel at the lower falls. During this stage there would be no means of control over frazil ice generation in the river upstream of the site.

During the second stage of construction, work would proceed behind the second stage cofferdam and diversion would be through operable gates of the partially completed spillway. These gates would provide a means of controlling the upstream water level during the winter so as to form a stable ice cover, which would reduce the amount of frazil ice generated upriver from the site. However, frazil ice would still be generated through the spillway and would still be a potential problem downstream. A higher downstream cofferdam would therefore be required than in either of the tunnel schemes.

Finally, the cofferdams connecting to the north bank would be removed and the spillway rollways would then be completed using temporary upstream and downstream stop logs.

Access across the river for construction and ultimately for plant access would be provided by a permanent bridge located near the upper falls.

6.2.2 Variant 2

This layout, shown on Plate 5, is a modification of Variant 1, requiring only a single stage cofferdam. A portion of the spillway would initially be constructed in the dry on the left bank of the river. River diversion during the next stage of construction would take place through the partially completed spillway structure, while the remaining structures are built in the river, protected by upstream and downstream cofferdams. Cofferdams adjacent to high velocity flow through the spillway, and in areas of restricted space, would be cellular steel sheet-pile structures. Operable gates in the partially completed spillway on the left bank, being used for river diversion, would provide control of the upstream water level during the winter to reduce generation of frazil ice; however, the frazil ice problem is not expected to be completely

eliminated, and would require a higher downstream cofferdam than in either of the tunnel schemes.

Access for construction would be from the north, via a road around the rock knoll. Access to cross the river would be via a temporary bridge, in front of the partially completed spillway, and thence via the cofferdams, during construction.

6.2.3 Variant 3

This layout, shown on Plate 5, is a further development of Variants 1 and 2. Diversion of the river during construction would be provided by a cofferdam located upstream of the upper falls with river discharge through two medium span diversion tunnels through the rock knoll. The main structures of the spillway and an integrated intake/powerhouse would be constructed in the unwatered riverbed at the lower falls. The use of diversion tunnels, with operable inlet gates, would provide more positive control of the upstream water level during the winter and, therefore, eliminate the frazil ice problem downstream and the risk associated with flooding of the construction site. This would enable a lower cofferdam to be used downstream.

Access for construction would be from the north, via a temporary road along the shoreline of the rock knoll. Access across the river would be over the upstream cofferdam and via a temporary construction road along the right bank and/or the riverbed. Permanent access would be via a road around the rock knoll and across the dam.

6.2.4 Variant 7

This layout, shown on Plate 6, is very similar to Variant 3 described above except it would have four units of 206 MW. It would comprise an integrated intake/powerhouse and an adjacent spillway located in the river channel, above the lower falls. In this scheme, the powerhouse would be located south of the spillway (toward the right bank), in the deeper part of the riverbed, to minimize excavation.

Diversion of the river during construction would be provided by a cofferdam located upstream of the upper falls and diversion tunnels through the rock knoll. Positive control of the upstream water level would eliminate the frazil ice problem downstream and the risk associated with flooding of the construction site, enabling a lower cofferdam to be used downstream.

Access for construction would be from the north, via a temporary road along the shoreline of the rock knoll. Access across the river would be over the upstream cofferdam and via a temporary construction road along the right bank and/or the riverbed. Permanent access would be via a road around the rock knoll and across the dam.

6.2.5 Variant 8

This layout, shown on Plate 6, would be basically the same as Variant 7, except that the locations of the spillway and the intake/powerhouse would be interchanged. Although this would increase the cost, it would provide direct access from the main road system to the

powerhouse, at the left bank, without having to cross the spillway and the powerhouse. The advantage of Variant 8, over Variant 7, would be the possibility of earlier access to the powerhouse and a consequential improvement in the construction schedule. The disadvantage of the direct access would be a steeper and more costly access to the powerhouse.

6.2.6 Variant 9

In this layout, shown on Plate 6, the spillway would be partially constructed in the dry on the right bank and would include the approach and discharge channels for river diversion. An integrated intake/powerhouse would then be constructed in the river, above the lower falls, between upstream and downstream cofferdams. Diversion of the river for construction of the intake/powerhouse would be carried out using a two-stage rockfilled cofferdam to divert the water through the partially completed spillway. Operable gates in the partially completed spillway on the right bank would provide control of the upstream water level during the winter to reduce the frazil ice problem downstream; however, the frazil ice problem is not expected to be completely eliminated, and would require a higher downstream cofferdam than in either of the alternative tunnel schemes.

Access for construction would be from the north, via a long span construction bridge across the river at the upper falls and a temporary access road along the right bank. Construction access to the left bank would be via a road around the rock knoll.

6.2.7 Variant 11

As presented, this layout, shown on Plate 6, is similar to Variant 8 except that diversion of the river would be via an open channel, excavated through the point of land on the left bank, adjacent to the lower falls.

The layout would comprise an integrated intake/powerhouse and a spillway located in the river channel, above the lower falls. The spillway would be located to the north of the intake/powerhouse. Closure of the river valley would be provided by a rockfill dam on the right bank and a gravity concrete dam on the left bank, through the diversion channel.

Diversion of the river during construction would be provided by a two-stage, zoned rockfill cofferdam, located upstream of the lower falls and the intake/powerhouse and spillway construction areas, to divert flow through the diversion channel on the left bank. Unwatering of the construction areas would be provided by the upstream cofferdam and a downstream cofferdam located at the lower falls.

In this layout, there would be limited control of the upstream water level, provided by constriction of the river flow by an armoured rockfill spur constructed above the upper falls. The limited control of the upstream water level would not eliminate frazil ice generation and, therefore, frazil ice would still be a concern. The risk of flooding of the construction areas from downstream, due to ice-damming in the downstream river channel would necessitate a downstream cofferdam at an elevation of at least eight metres.

Access for construction would be from the north, via a road around the rock knoll. Access across the river would be via a construction bridge over the diversion channel and over the upstream cofferdam.

6.3 TUNNEL SCHEMES

The tunnel schemes include developments with power or tailrace tunnels through the rock knoll. During construction, two tunnels would be used for diversion of the river around the construction site. River closure at diversion would be accomplished by means of a cofferdam located just above the upper falls. In each of the tunnel schemes, the spillway would be constructed in the unwatered riverbed at the upper falls, and abutment dams on the left and right banks would provide closure of the narrow river valley.

6.3.1 Variant 4

This layout, shown on Plate 5, would utilize an integrated intake and surface powerhouse, located on the left bank, at the upper falls. Two tailrace tunnels, through the rock knoll, would carry the discharge from the powerhouse to below the lower falls. A temporary diversion inlet located to the north of the powerhouse, would provide connection into the tailrace tunnels during the diversion phase. The use of diversion tunnels, with operable inlet gates, would provide control of the upstream water level during the winter and, therefore, eliminate the frazil ice problem downstream and the risk associated with flooding of the construction site from downstream.

In this layout, the close proximity of the intake/powerhouse, the diversion cofferdam and the diversion inlet, would require a longer cofferdam, located farther upstream, and a closure to the north bank using cellular steel sheet-pile construction would be required.

Access for construction would be from the north, via a road around the rock knoll. Access across the river, for construction, would be provided over the cofferdam.

6.3.2 Variant 5

In this layout, shown on Plate 5, a surface powerhouse would be located on the east (downstream) side of the rock knoll, with power tunnels through the rock knoll. Diversion of the river during construction would be provided by a cofferdam located upstream of the upper falls and two of the power tunnels would initially be used as diversion tunnels. A diversion outlet would be provided on the east side of the rock knoll, north of the powerhouse. Temporary tunnels connecting the power tunnels and the diversion outlet would be plugged when the diversion is no longer required. The spillway would be constructed in the unwatered riverbed downstream of the cofferdam, in the middle of the river channel at the upper falls. Diversion of the river through the power intake and tunnels, using the intake gates to control the upstream water level, during the winter, would eliminate the frazil ice problem downstream and the risk associated with flooding of the construction areas from downstream, enabling a lower cofferdam and/or a rock plug to be used at the powerhouse site.

Access for construction would be from the north, via a road around the rock knoll. Access across the river for construction of the right bank closure dam would be via the upstream cofferdam.

6.3.3 Variant 6

This layout, shown on Plate 5, is similar to Variant 5, with power tunnels through the rock knoll, but with an underground powerhouse located in the east (downstream) side of the rock knoll. Although an underground powerhouse is uncommon for such a low head development as Muskrat Falls, this layout was considered because of the possible benefits of achieving early access to the powerhouse via an access tunnel, and year-round construction in a controlled environment. This layout would also utilize two of the power tunnels for diversion, with a diversion outlet located just north of the tailrace tunnel outlet. As in Variant 5, the spillway would be located in the river channel at the upper falls and access for construction would be from the north, via the permanent road over the intake. Access to the powerhouse, would be by tunnel from the north. Temporary access across the river would be via the cofferdam.

6.4 RIGHT BANK SCHEME

The right bank scheme would have the spillway and an intake/powerhouse block located on the right (south) bank, adjacent to the lower falls, and a main rockfill dam across the river above the lower falls.

6.4.1 Variant 10

In this layout, shown on Plate 6, excavation for the spillway and an integrated intake/powerhouse would be carried out in the dry on the right bank, including large portions of the approach and discharge channels. The structures would be constructed behind cofferdams and water control structures. A two-stage cofferdam would then be constructed across the river, to divert the river through the partially completed spillway. A permanent zoned earth and rockfill dam would then be constructed across the entire width of the river, between the upstream and downstream cofferdams, above the lower falls. Operable gates in the partially completed spillway on the right bank would provide control of the upstream water level during the winter to reduce the frazil ice problem downstream; however, the frazil ice problem is not expected to be completely eliminated, and would require a higher downstream cofferdam than in either of the alternative tunnel schemes.

Access for construction would be from the north, via a long span construction bridge across the river at the upper falls and a temporary access road along the right bank. Construction access to the left bank would be via a road around the rock knoll which would become the permanent access road to cross on the main dam and spillway to the powerhouse.

6.5 COMPARISON OF ALTERNATIVES

All eleven Variants were compared relatively, on a qualitative basis. The qualitative factors include advantages, disadvantages and risks associated with each Variant. These are discussed in Section 6.5.1, and are summarized in Tables 6.1 and 6.2.

A quantitative comparison was undertaken on four selected Variants 5, 7, 10 and 11. The quantitative comparison was expressed as relative costs which are summarized in Table 6.3. The comparison was based on the previous capital cost estimates, escalated to June 1998 dollars.

6.5.1 Qualitative Comparison

The eleven Variants were compared on a qualitative basis which is summarized in Table 6.1 which:

- lists the Variants from 1 – 11;
- identifies the source for the Variant, i.e. from the 1980 SNC-Lavalin Report, the 1989 SNC Report or Hydro Quebec;
- provides an abbreviated description of each variant;
- lists the identified advantages of each variant;
- lists the identified disadvantages of each variant;
- lists the identified risks of each variant; and
- shows the comparative construction schedule for each variant.

The qualitative factors are discussed in the following sections and the construction schedules are discussed in Section 6.5.1.4.

6.5.1.1 *Relative Advantages*

There are clear advantages offered by some Variants. They are discussed, in order of importance, in the following sections.

a) Upstream Water Level and Frazil Ice Control

Frazil ice has been identified as a significant concern for development of Muskrat Falls. During the winter, significant lengths of the river upstream of Muskrat Falls, and through the Falls, have open water areas as a result of the flow velocity being too high to enable an ice cover to form. Within the open water areas large amounts of frazil ice are generated. The frazil ice is carried downstream and accumulates in the river, resulting in ice dam formation downstream of the Muskrat Falls site. The ice dam causes the water to back-up, resulting in higher than normal water levels at the site. The highest water level witnessed at the site resulted in flooding out of the upper falls, at El. 19 m.

The consequence of high water levels at the site, during construction, is flooding of the construction areas and the resulting delay for cleanup and replacement of completed work

Table 6.1

MUSKRAT FALLS

Comparison of Alternative Layouts (Variants)

Variant	Source	Description	Advantages	Disadvantages	Risk[1]	Schedule[2]
1	SNC-Lavalin Report 1980	- Intake, PH & spillway in the river at the lower falls. - Two stage rockfill & cellular cofferdam. - Constr'n bridge across river at upper falls.	1. Layout suitable for 4, 5 or 6 unit development. 2. Least hydraulic headloss during operation.	1. Limited frazil ice control. 2. Difficult unwatering with cellular cofferdams. 3. Construction schedule about 12 months longer than Variant 5, due to requirement for a long span construction bridge and two stage cofferdam before powerhouse excavation.	1. Increased cost and schedule due to cellular cofferdams. 2. Flooding of construction due to ice dam downstream and ice accumulation at spillway structure. **Relative Risk = 9.0**	66 months
2	SNC-Lavalin Report 1980	- Modified version of Variant 1. - Intake, PH & spillway in the river at the lower falls. - Two stage rockfill & cellular cofferdam. - Diversion through partially completed spillway.	1. Layout suitable for 4, 5 or 6 unit development. 2. Least hydraulic headloss during operation.	1. Limited frazil ice control. 2. Difficult unwatering with cellular cofferdam. 3. Two stages of spillway construction. 4. Late start on PH excavation.	1. Six to 12 months longer schedule than variant 5, due to requirement for stg 1 spillway & cofferdam before PH excavation. 2. Increased cost and schedule due to cellular cofferdams. 3. Flooding of construction due to ice dam downstream and ice accumulation at spillway structure. **Relative Risk = 9.0**	66 months
3	SNC-Lavalin Report 1980	- Intake, PH & spillway in the river at the lower falls. - Diversion cofferdam upstream of upper falls with discharge via tunnels through rock knoll.	1. Layout suitable for 4, 5 or 6 unit development. 2. Frazil ice control with tunnel diversion. 3. Simple diversion cofferdam & tunnel arrangement. 4. PD works remote from diversion discharge. 5. Limited unwatering closures req'd for PD constr'n. 6. Complete unwatering of PD works throughout construction. 7. Concurrent construction of PH & spillway. 8. Least hydraulic headloss during operation. 9. Simpler sequence of construction than for Variant 5. 10. Direct access to PD works across riverbed.	1. Rock plug required at tunnel outlet for construction.	**Relative Risk = 6.0**	55 months
4	SNC-Lavalin Report 1980	- Power structures all on the left bank. - U/S surface powerhouse. - U/S diversion cofferdam, intake & spillway above the upper falls, with spillway in the river. - Rockfill & cellular cofferdam. - Diversion via a temporary inlet and the tailrace tunnels.	1. Frazil ice control with tunnel diversion. 2. Spillway & diversion cofferdam are above upper falls. 3. Earlier access to PH site is possible. 4. Limited unwatering closures req'd for PD constr'n.	1. Difficult unwatering with cellular cofferdam. 2. Complex tunnel arrangement. 3. Rock plug required at tunnel outlet for construction. 4. High hydraulic headloss due to tailrace tunnels. 5. Space limited for development of 4, 5 or 6 units.	1. Increased cost and schedule due to cellular cofferdams. 2. Very large tunnels. **Relative Risk = 7.0**	60 months
5	SNC-Lavalin Report 1980	- Power structures all on left bank. - D/S surface powerhouse. - U/S diversion cofferdam, intake & spillway above the upper falls, with spillway in the river. - Diversion via the intake and the power tunnels.	1. Accepted by Client & Board of Consultants in 1980. 2. Frazil ice control with tunnel diversion. 3. All schedule-critical work is on the north (left) bank. 4. Spillway & diversion cofferdam are above upper falls. 5. Earlier access to PH site may be possible.	1. Four-unit PH & tunnels located in fault zone. 2. Extensive underground work with complex tunnel arrangement, surge shafts, transitions & plugs. 3. Cofferdam, intake & spillway in very close proximity. 4. Space limited for development of 4, 5 or 6 units. 5. Precarious PH access along floodway. 6. Flood discharge adjacent to PH. 7. High hydraulic headloss due to power tunnels. 8. Rock plug required at tunnel outlet for construction.	1. PH site close to deep pool may present dewatering problems. 2. Complexity of underground works. **Relative Risk = 6.5**	58 months

Table 6.1
MUSKRAT FALLS
Comparison of Alternative Layouts (Variants)

Variant	Source	Description	Advantages	Disadvantages	Risk[1]	Schedule[2]
6	SNC-Lavalin Report 1980	- Layout similar to Variant 5, except with D/S underground powerhouse. - Power structures all on left bank. - U/S diversion cofferdam, intake & spillway above the upper falls, with spillway in the river. - Diversion via the intake and the power tunnels.	1. Frazil ice control with tunnel diversion. 2. Spillway & diversion cofferdam are above upper falls. 3. Earlier access to PH site may be possible. 4. Year-round construction in a controlled environment.	1. Underground PH is unusual for a low head development (i.e. requirement for large openings for low head & high flow). 2. Higher in cost than for Variants 1 - 5. 3. Extensive underground work & confined work area. 4. Rock plug required at tunnel outlet for construction. 5. High hydraulic headloss due to tunnels. 6. Space limited for development of 4, 5 or 6 units.	1. PH site close to deep pool may present dewatering problems. 2. Complexity of underground works. **Relative Risk = 8.0**	66 months
7 (5)	SNC-Lavalin Report 1989	- Similar to Variant 3 in 1980.	1. Layout suitable for 4, 5 or 6 unit development. 2. Frazil ice control with tunnel diversion. 3. Simple diversion cofferdam & tunnel arrangement. 4. PD works remote from diversion discharge. 5. Limited unwatering closures req'd for PD constr'n. 6. Complete unwatering of PD works throughout construction. 7. Concurrent construction of PH & spillway. 8. Least hydraulic headloss during operation. 9. Direct access to PD works across riverbed. 10. Earlier access to PH site may be possible. 11. Possibility for rock quarry in unwatered riverbed.	1. Rock plug required at tunnel outlet for construction. 2. Variant 8 requires difficult cofferdam for 1st phase PH.	**Relative Risk = 6.0**	56 months
9	SNC-Lavalin Report 1989	- Intake & PH in the river at the lower falls. - Spillway on the right bank at the lower falls. - Cofferdam to divert river through partially completed spillway. - D/S cofferdams at PH & spillway. - Constr'n bridge across river at upper falls.	1. Layout suitable for 4, 5 or 6 unit development. 2. Spillway excavation in the dry on the right bank. 3. Least hydraulic headloss during operation.	1. Limited frazil ice control. 2. Long span construction bridge required. 3. River diversion through partially completed spillway.	1. Flooding of construction due to ice dam downstream and ice accumulation at spillway structure. **Relative Risk = 9.0**	60 months
10	SNC-Lavalin Report 1989	- Intake, PH & spillway on the right bank at the lower falls. - Main dam across the river above the lower falls. - Two stage cofferdam to divert river through partially completed spillway. - D/S cofferdams at PH, spillway & main dam. - Constr'n bridge across river at upper falls.	1. Layout suitable for 4, 5 or 6 unit development. 2. PH & spillway excavation in the dry on right bank. 3. Concurrent construction of PH & spillway. 4. Less hydraulic headloss than Variants 4, 5 or 6. 5. Limited unwatering closures req'd for PD constr'n.	1. Limited frazil ice control. 2. Two stage U/S cofferdam required. 3. Long span construction bridge required. 4. Uncertain foundation conditions for dam. 5. Water control works required between spillway & PH excavations. 6. River diversion through partially completed spillway. 7. Completion of spillway rollways may not be possible before watering up of reservoir. 8. More headloss than Variants 1, 2, 3, 7, 8 & 9.	1. Flooding of construction due to ice dam downstream and ice accumulation at spillway structure. **Relative Risk = 8.0**	59 months
11	Hydro Quebec	- Intake, PH & spillway in the river at the lower falls. - Diversion cofferdam upstream of lower falls with discharge via open channel in left bank. - D/S cofferdam below the lower falls. - Constriction of river flow above upper falls with rockfilled spur, for upstream water level and ice control.	1. Layout suitable for 4, 5 or 6 unit development. 2. Diversion channel excavation in the dry on left bank. 3. Concurrent construction of PH & spillway. 4. Least hydraulic headloss during operation. 5. Similar construction schedule as Variant 7.	1. Limited U/S wi or frazil ice control with rockfill spur and river diversion through open channel. 2. Two-stage U/S cofferdam required. 3. Construction bridge required. 4. Conjested work area at left bank. 5. Difficult access for PH excavation. 6. Requirement for, and difficulty with placement of, large armour rock for spur.	1. Flooding of construction due to ice dam downstream. 2. Increased cost and schedule for flooding of powerhouse due to ice jam at rock spur or diversion channel. 3. Increased cost and schedule for construction and potential replacement of rock spur. 4. Erosion of right bank of river. **Relative Risk = 14.0**	55 months

Legend: PH = Powerhouse
PD = Power Development Structures, i.e. intake/powerhouse and spillway.

Notes:
1. Relative Risk is measured as the potential delay, in months, if the worst probable conditions occur for each component of the work. Refer to Table 6.2 for calculation of Relative Risk.
2. Schedules for Variants 5, 7 and 10 are based on review of previous schedules, adjusted for same basis and project start in January. Schedules for variants 1, 2, 3, 4, 6 and 9 are estimated. Schedule for Variant 11 is based on a project release in January (Variant 11a - Figure 6.4a)

Table 6.2
MUSKRAT FALLS
Comparison of Relative Risk (months)

Variant	Description	Cofferdams	Tunnels	Powerhouse	Spillway	Dams	Spur	Total
1	• PH & Spillway in river at lower falls • Two-stage rockfill and cells through spillway	3.0	-	4.0	1.0	1.0	-	9.0
2	• PH & Spillway in river at lower falls • Two-stage rockfill and cells through spillway	3.0	-	4.0	1.0	1.0	-	9.0
3	• PH & Spillway in river at lower falls • Tunnel diversion with single stage u/s cofferdam	1.0	1.0	2.0	1.0	1.0	-	6.0
4	• Surface PH on left bank upstream with large tailrace tunnels • Tunnel diversion with single stage rockfill and cell cofferdam • Spillway in channel upstream	3.0	2.0	1.0	0.5	0.5	-	7.0
5	• Surface PH left bank D/S with large diversion/power tunnels • Single stage with rockfill cofferdam • Spillway in channel upstream	1.0	3.0	1.5	0.5	0.5	-	6.5
6	• Similar to Variant 5, with underground powerhouse	1.0	3.0	3.0	0.5	0.5	-	8.0
7	• Similar to Variant 3, modified for 4 units	1.0	1.0	2.0	1.0	1.0	-	6.0
8	• Slight modification of Variant 7 with powerhouse/spillway location switched	1.0	1.0	2.0	1.0	1.0	-	6.0
9	• PH in river at lower falls • Spillway construction in dry on right bank • Two-stage diversion through spillway.	2.0	1.0	4.0	1.0	1.0	-	9.0
10	• Spillway construction right bank in dry, powerhouse in river • Two-stage diversion through Spillway	2.0	-	4.0	1.0	1.0		8.0
11	• PH & Spillway in river at lower falls • Two-stage cofferdam & open channel diversion • U/S Spur for water level and ice control	2.0	-	4.0	1.0	1.0	6.0	14.0

Table 6.3

MUSKRAT FALLS

Relative Cost Comparison of Variants 5, 7, 10 & 11

Item	Description	Unit	Variant 5 (Diversion & Power Tunnels) Qty	Unit $	Cost	Variant 7 (Div. Tunnels, PH & Spillway in River) Qty	Unit $	Cost	Variant 10 (PH & Spillway in Right Bank) Qty	Unit $	Cost	Variant ## 11 (Div. Channel Left Bank, PH & Sp'way in River) Qty	Unit $	Cost
1	**CONSTRUCTION COSTS**													
1000	**Permanent Support Facilities**				$ 1,900,000	+0.5km		2,200,000	+ 0.3 km		$ 2,100,000	+0.3 km		2,100,000
2000	**Reservoir, Div., Dam & Spillway**													
	- 2100 - Reservoir Clearing (Partial)	Ha	421	$ 10,000	$ 4,210,000	421	$ 10,000	$ 4,210,000	421	$ 10,000	$ 4,210,000	421	$ 10,000	$ 4,210,000
	- 2200 - Diversion / Water Control													
	- Portals/Div.Channel-Overburden Exc.	m3	57,700	$ 12.00	$ 692,400	10,000	$ 15.00	$ 150,000	- n/a			4,000	$ 15.00	$ 60,000
	- Portals/Div.Channel-Rock Exc	m3	525,000	$ 35.00	$ 18,375,000	230,000	$ 35.00	$ 8,050,000	- n/a			635,000	$ 20.00	$ 12,700,000
	- Inlet/Outlet/Div Chan. Conc incl rebar	m3	38,800	$ 700.00	$ 27,160,000	11,000	$ 720.00	$ 7,920,000	- n/a			14,000	$ 665.00	$ 9,310,000
	- Lining/Plug Conc. Incl rebar, ribs, etc	m3	49,100	$ 665.00	$ 32,651,500	20,000	$ 483.00	$ 9,660,000	- n/a			- n/a		
	- Tunnels/Adit/Shafts incl services, etc	m3	(3) 764,000	(4) $ 70.00	$ 53,480,000	430,000	(4) $ 70.00	$ 30,100,000	- n/a			- n/a		
	- Gates, Guides etc	L.S.		L.S.	$ 18,740,232	-	L.S.	$ 8,286,000	- n/a			- n/a		
	- Div. Channel/ Portal Plug Exc. - Rock	m3	50,000	$ 50.00	$ 2,500,000	50,000	$ 50.00	$ 2,500,000	- n/a			25,000	$ 35.00	$ 875,000
	- Overflow Rock Groin-incl Armor,R.Ra	m3	- n/a			- n/a			- n/a			80,000	$ 25.00	$ 2,000,000
	- Water Control/Div.Channel C'dam	m3	- n/a			- n/a			85,000	$ 25.00	$ 2,125,000	165,000	$ 25.00	$ 4,125,000
	Sub-Total - 2200 - Diversion				$ 153,600,000			$ 66,670,000			$ 2,130,000			$ 29,070,000
	- 2300 - Dam & Cofferdam							$ -						
	- Cofferdam - Fill (Rock,Till, Filter etc)	m3	218,700	$ 21.00	$ 4,592,700	255,000	$ 25.00	$ 6,375,000	(5) 725,000	$ 25.00	$ 18,125,000	(5) 1,025,000	$ 25.00	$ 25,625,000
	- Dam Fill - (Rock, Till, Clean Up etc)	m3	78,700	$ 30.00	$ 2,361,000	340,000	$ 30.00	$ 10,200,000	(5) 1,200,000	$ 30.00	$ 36,000,000	340,000	$ 30.00	$ 10,200,000
	- Concrete Gravity Dam	m3	- n/a			22,000	$ 450.00	$ 9,900,000	- n/a			75,000	$ 400.00	$ 30,000,000
	- Overburden Exc./Cofferdam Removal	m3	115,000	$ 11.00	$ 1,265,000	(7) 115,000	$ 15.00	$ 1,725,000	85,000	$ 11.00	$ 935,000	725,000	$ 15.00	$ 10,875,000
	Sub-Total - 2300 - Dam & C'Dam				$ 8,220,000			$ 28,200,000			$ 55,060,000			$ 76,700,000
	- 2400 - Spillway													
	- Overburden Excavation	m3	300	$ 11.00	$ 3,300	30,000	$ 11.00	$ 330,000	230,000	$ 10.00	$ 2,300,000	30,000	$ 11.00	$ 330,000
	- Rock Excavation	m3	68,000	$ 25.00	$ 1,850,000	75,000	$ 25.00	$ 1,875,000	1,270,000	$ 25.00	$ 31,750,000	75,000	$ 25.00	$ 1,875,000
	- Concrete incl rebar, cleanup, etc	m3	98,000	$ 650.00	$ 63,700,000	120,000	$ 665.00	$ 79,800,000	128,000	$ 665.00	$ 85,120,000	(8) 120,000	$ 665.00	$ 79,800,000
	- Spillway Bridge	t	155	$ 5,500	$ 852,500	- Incl in Concrete		-	- Incl in Concrete			- Incl in Concrete		
	- Gates, Guides & Misc Steel	L.S.			$ 23,959,836			$ 22,096,000			(9) $ 26,500,000			$ 22,096,000
	Sub-Total - 2400 - Spillway				$ 90,170,000			$ 104,100,000			$ 145,670,000			$ 104,100,000
	- 2800 - North Spur Stabilization	L.S.			$ 23,000,000			$ 23,000,000			$ 23,000,000			$ 23,000,000
3000	**Powerhouse & Related Facilities**													
	- 3200 - Penstocks/Power Tunnels	L.S.			$ 70,751,095	- n/a			- n/a			- n/a		
	- 3300 - Civil Works Incl Intake	L.S.			## $ 182,865,000			$ 218,050,000			$ 219,330,000			$ 218,050,000
	- 3400 - Turbines & Generators	L.S.			## $ 163,000,000			$ 150,000,000			$ 150,000,000			$ 150,000,000
	- 3500 - Electrical Equipment	L.S.			$ 23,000,000			$ 23,000,000			$ 23,000,000			$ 23,000,000
	- 3600 - Mechanical Equipment	L.S.			$ 10,000,000			$ 10,000,000			$ 10,000,000			$ 10,000,000
	- 3700 - Gen. Transformers/H.V. Switch.	L.S.			$ 34,000,000			$ 34,000,000			$ 34,000,000			$ 34,000,000
	Sub-Total - 3000				$ 483,620,000			$ 435,050,000			$ 436,330,000			$ 435,050,000
8000	**Construction Facilities & Support**	L.S.			$ 264,000,000			$ 264,000,000			$ 264,000,000			$ 264,000,000
	- Construction Bridge	L.S.			$ -			$ -			$ 3,000,000			$ 2,000,000
	Sub-Total - 8000				$ 264,000,000			$ 264,000,000			$ 267,000,000			$ 266,000,000
	SUB-TOTAL - DIRECT COSTS				$ 1,028,720,000			$ 927,430,000			$ 935,500,000			$ 940,230,000

Table 6.3

MUSKRAT FALLS

Relative Cost Comparison of Variants 5, 7, 10 & 11

Item	Description	Unit	Variant 5 (Diversion & Power Tunnels) Qty	Unit $	Cost	Variant 7 (Div. Tunnels, PH & Spillway in River) Qty	Unit $	Cost	Variant 10 (PH & Spillway in Right Bank) Qty	Unit $	Cost	Variant ## 11 (Div. Channel Left Bank, PH & Sp'way in River) Qty	Unit $	Cost
9000	**Management, Engineering & Others**													
	- Management & Engineering	L.S.	15%		$ 154,308,000	15%		$ 139,114,500	15%		$ 140,325,000	15%		$ 141,034,500
	- Field Investigations	L.S.			$ 1,500,000			$ 1,500,000			$ 1,500,000			$ 1,500,000
	- Contingencies	L.S.	10%		$ 116,452,800	13%		$ 138,845,785	13%		$ 140,052,250	13%		$ 140,759,385
	- Taxes	L.S.			$ -			$ -			$ -			$ -
	Sub-Total - Manage/Engineer & Others				**$ 274,260,000**			**$ 279,460,000**			**$ 281,880,000**			**$ 283,290,000**
TOTAL - DIRECT COSTS & MANAGE/ENGINEER/OTHERS - JUNE,1998$					**$ 1,302,980,000**			**$ 1,206,890,000**			**$ 1,217,380,000**			**$ 1,223,520,000**
2	**SCHEDULE COSTS**													
	- Duration	Months	58			56			59			## 55		
	- Interest During Construction @ 8%/yr	LS			$ 221,571,156			$ 199,379,144			$ 210,912,176			$ 198,132,714
	(Assume Payment of Total Construction Cost @ halfway point)													
3	**EFFICIENCY/LOST ENERGY COSTS**													
	- Due to Increased Schedule	kwh	1,300,000,000	$ 0.04	$ 52,000,000	433,000,000	$ 0.04	$ 17,320,000	1,733,000,000	$ 0.04	$ 69,320,000	-	$ 0.04	$ -
	- Due to Increased Headloss	m	0.55	$ 46,900,000	$ 25,795,000	-	$ -	$ -	-		$ -	-		$ -
	- Due to Increased Efficiency Turbine [14]	%			$ -	0.3%		$ (4,920,000)	0.3%		$ (4,920,000)	0.3%		$ (4,920,000)
4	**RELATIVE RISKS [15]**													
	- Months of Potential Delay	Months	6.5			6.0			8			14		
	- Interest During Construction @ 8%	L.S.			$ 29,232,772			$ 31,365,904			$ 28,298,064			$ 35,791,820
	- Lost energy sales	kwh	2,818,400,000	$ 0.04	$ 112,736,000	2,600,000,000	$ 0.04	$ 104,000,000	3,468,400,000	$ 0.04	$ 138,736,000	6,067,000,000	$ 0.04	$ 242,680,000
TOTAL RELATIVE PROJECT COSTS - JUNE,1998$					**$ 1,744,310,000**			**$ 1,554,040,000**			**$ 1,659,730,000**			**$ 1,695,200,000**

NOTES:

1 - Estimate for Variant 5 based upon 1980 SNC-Lavalin report. Cost estimate in 1980 Report pro-rated upwards to account for 4th Unit and escalated by 167% (avg.) to arrive at $1998.
2 - Estimate for Variants 7 & 10 based upon 1989 SNC report. Cost estimates in 1989 Report escalated by 133% (avg.) to arrive at $1998.
3 - Volume for Surge Shafts increased for Variant 5 to account for change in design from 1980 Report.
4 - Tunnel excavation unit price as per 1997 Gull Island Cost Estimate.
5 - Fill quantities for Cofferdam and Dam in Variant 10 increased based on recent bathymetric data.
6 - Variant 11 top of Downstream Cofferdam taken as El. 8.0 and top of Upstream Cofferdam taken as El. 24.0.
7 - Costs for removal of upstream cofferdam added to Variant 7 which were not included in 1989 Report.
8 - Concrete quantity for Variant 11 Spillway increased by 60,000 m3 based on recent bathymetric data.
9 - Spillway Gates for Variant 10 Spillway increased by 20% to account for increased height of travel for diverting river prior to pouring rollways.
10 - Powerhouse Civil Works (Division 3300) for Variant 5 increased by 9.4 % to account for greater unit spacing for larger Turbine spiral.
11- Variant 5 utilizes steel spiral scroll case, while Variants 7, 10 & 11 use a conc. semi-spiral scroll case. Cost of Turbine with steel spiral is assumed 15% more expensive than Turbine with concrete semi-spiral.
12 - Variant 11 as per Hydro Quebec correspondence to N&L Hydro, 01 Sept,1998. Layout optimized by SNC-AGRA based on recent bathymetric data. In particular the Powerhouse/Intake and Cofferdams moved upstream to locate D/S C'dam on top of Lower Falls.
13 - The scheduled duration of Variant 11 is based on a January start date (for comparison with the other variants). Schedule for Variant 11 is shown on Figure 4.4(a).
14 - Variants 7, 10 & 11 Turbines with concrete semi-spiral scroll case are assumed to be 0.3% more efficient than Variant 5 Turbine with steel spiral scroll case.
15 - The Relative Risks in Item 4 above are a qualitative assessment of the risks of significant delays associated with all Variants. These are quantitatively assessed to represent the potential cost of delay.
16 - Totals & Sub-totals rounded to nearest $10,000.
17 - Zero escalation taken for duration of project.

damaged by the flooding. The risk of damage and delay of construction is greater for construction areas located at or near the downstream water level, which is normally at an elevation of about 3 m. Cofferdams required downstream of construction areas will have to be built to a higher elevation to protect against frazil ice induced flooding, unless appropriate means of controlling the generation of frazil ice and the consequent ice damming of the river are implemented.

The most appropriate means of controlling the generation of frazil ice upstream of the site would be to raise the winter upstream water level to, and maintain it at, an elevation of about 24 m. With the exception of two short reaches of the river, this would provide a stable ice cover on slow moving water, for approximately 40 km upstream, where any frazil ice generated farther upstream would be trapped. This ice cover would also prevent the generation of frazil ice, immediately upstream of the site. This could be accomplished by:

- diverting the river around the site with an upstream cofferdam and diversion tunnels through the rock knoll;

- providing water control using operable gates in a partially completed spillway structure at the lower falls; or

- constricting the river flow above the upper falls with an armoured rockfill spur constructed partially across the river.

The first option is expected to eliminate the frazil ice problem completely. The second option would, most likely, partly eliminate the problem, since frazil ice would still be generated through the narrow river channel below the upper falls and downstream of the flow control structure. The second option would likely still result in some ice damming in the river downstream, with the consequence of higher water levels at the lower falls, than with the first option. The third option would, most likely, partly eliminate the frazil ice problem and, with the potential for fluctuation of the upstream water level, breakup of the ice cover could result. This would likely result in development of frazil ice, or ice-jams, as a result of moving ice, at the constriction at the spur and/or at the diversion channel. The spur would most likely be damaged by the spring flood and require substantial repair, or replacement, during the second year of diversion.

Variants 3, 4, 5, 6, 7 and 8 include diversion tunnels and, therefore, offer maximum frazil ice control as an advantage.

Variants 1, 2, 9 and 10 have the disadvantage of limited frazil ice control, with diversion through partially completed spillways.

Variant 11, with a rockfill spur and with diversion through an open diversion channel, has the disadvantage of limited frazil ice control, and risk of ice-jams, due to limited upstream water level control. Also, there is risk of damage to the spur during the spring floods.

b) Diversion Tunnel

Diversion of the river completely around the construction site, with an upstream cofferdam and diversion tunnel arrangement, would:

- simplify the unwatering of the construction areas;
- limit the number of unwatering closures; and
- eliminate access constraints for intake/powerhouse and spillway construction.

This offers the advantage of completely separating the major construction areas from the river and requires the construction of a relatively simple downstream cofferdam in addition to the tunnel diversion.

Variants 3, 4, 5, 6, 7 and 8 offer this advantage.

In Variants 4, 5 and 6 the spillway can be constructed in the dry, above the upper falls, and the intakes and powerhouses can be constructed behind relatively lower cofferdams or rock plugs.

In Variants 3, 7 and 8, which are similar in layout, all the major power development structures, i.e. the spillway and integrated intake/powerhouse, can be constructed in an unwatered riverbed throughout the entire construction period, behind a downstream cofferdam.

c) Hydraulic Headloss

Less hydraulic headloss can be anticipated in Variants with an integrated intake/powerhouse arrangement than Variants with power or tailrace tunnels.

Variants 1, 2, 3, 7, 8, 9, 10 and 11 offer this advantage and would have minimum headloss. However, in Variant 10 the approach flow would be less than ideal which could adversely affect turbine performance, and would result in slightly more headloss than others.

Variants 4, 5 and 6 have the disadvantage of higher hydraulic headloss due to the requirement for power and/or tailrace tunnels.

d) Early Access to Powerhouse

In some cases, early access may be advantageous, allowing an earlier start on excavation for the powerhouse. This is particularly true if the powerhouse construction is on the critical path.

Variants 4, 5 and 6 offer this advantage because, in these Variants, the powerhouses are located on the north (left) bank.

e) Construction in the Dry

Construction of major structures in the dry can be a significant cost advantage because this can reduce the cost of unwatering and improve the access and the construction schedule.

Variants 4, 5 and 6 offer the advantage of construction of the spillway in the dry, above the upper falls. Also, the intakes and powerhouses could be constructed, essentially in the dry, behind small cofferdams or rock plugs.

Variants 3, 7 and 8, which are similar in layout, offer the advantage of construction of all the major power development structures, i.e. the spillway and integrated intake/powerhouse, in an unwatered riverbed throughout the entire construction period, behind a downstream cofferdam. In addition, these Variants offer the advantage of providing an area for a rock quarry in the unwatered riverbed between the upper and lower falls.

Variant 9 offers the advantage of excavation for the spillway in the dry on the right bank.

Variant 10 offers the advantage of excavation for the spillway and the integrated intake/powerhouse in the dry on the right bank.

Variant 11 offers the advantage of excavation for the diversion channel essentially in the dry on the left bank.

6.5.1.2 *Relative Disadvantages*

There are a few distinct disadvantages associated with some Variants. They are discussed, in order of importance, in the following sections.

a) No, or Limited, Frazil Ice Control

As noted in 6.5.1.1(a), above, Variants 1, 2, 9 and 10 have the disadvantage of limited frazil ice control, with water level control provided by operable gates in partially completed spillways. Variant 11 has the disadvantage of limited upstream water level control and, therefore, limited frazil ice control.

b) Cellular Cofferdams

Variants 1, 2 and 4 each require the construction of steel sheet-pile cellular cofferdams in areas of the riverbed that would be expected to have exposed and irregular bedrock surfaces and high velocity flow. This type of cofferdam would be slow, difficult and costly to build in such areas. With the absence of riverbed deposits there would be no lateral support for the sheet-piling and the irregular, exposed, bedrock surface could make sealing of the bottom of the sheet-pile walls difficult.

c) Hydraulic Headloss

As noted in 6.5.1.1(c), above, Variant 10 would have unfavourable approach flow, which could affect turbine performance and increase headloss.

Also, as noted in 6.5.1.1(c), Variants 4, 5 and 6 have the disadvantage of higher hydraulic headloss due to friction losses in power and/or tailrace tunnels.

d) Construction Bridge

The requirement for a construction bridge would necessitate a longer schedule, especially if the bridge is a long span structure.

Variants 1, 9 and 10, and possibly Variant 2, have the disadvantage of requiring an early, long span, construction bridge across the river at the upper falls.

Variant 11 has the disadvantage of requiring a construction bridge across the diversion channel for construction of the upstream cofferdam.

e) Rockfill Cofferdams

Variants 1, 2, 9 and 10 have the disadvantage of requiring two-stage cofferdam construction in the river to divert the river around the construction areas and through partially completed spillways.

Variant 8 has the disadvantage of requiring a cofferdam in the river to divert the river around a first phase intake/powerhouse construction area.

Variant 11 has the disadvantage of requiring a large two-stage cofferdam to divert the river around the construction area and through the diversion channel. It also has the disadvantage of requiring a higher downstream cofferdam than other Variants, to prevent flooding of the construction area as a result of high water level due to ice-damming.

f) Tunneling Complexity

Variants 3, 4, 5, 6, 7 and 8 would require the construction of diversion, power and/or tailrace tunnels, with different degrees of complexity.

Variant 3 would have two medium span diversion tunnels between an inlet and an outlet on the west and east sides of the rock knoll, respectively. These tunnels would be large but straight and relatively easy to construct.

Variant 4 would have two large span tailrace tunnels between a surge chamber at the upstream powerhouse and the tunnel outlet on the east side of the rock knoll. These tunnels would be relatively simple and straight and would be initially used for diversion. A temporary diversion inlet and temporary, short, diversion tunnels would connect the inlet to the tailrace tunnels.

Variant 5 would have four large span power tunnels, two of which would be initially used as diversion tunnels. These would require a complex arrangement of multi-level interconnections, surge shafts, surge chambers and plugs, in limited available space within the rock knoll.

Variant 6 would be similar to Variant 5 but with an underground powerhouse and comparable complexity of tunnels.

Variants 7 and 8 would be similar to Variant 3, with two medium span straight, and relatively easy to construct, diversion tunnels between an inlet and an outlet on the west and east sides of the rock knoll, respectively.

Based on the foregoing, Variant 5 would have the most complex tunneling arrangement while Variants 3, 7 and 8 would have the least complex arrangements.

g) Rockfill Spur

Variant 11 requires the construction of an armoured rockfill spur above the upper falls to control upstream water level. This spur would constrict and contain the river flow to the south side of the river channel. It would require very large protective armour rock, not readily available at, or near, the site. Construction of the spur, and the constriction of the river, would likely result in erosion of overburden at the south bank of the river. The spur could be damaged or destroyed by spring floods during the diversion period.

6.5.1.3 Relative Risks

A qualitative assessment of the risks of significant delays and consequential extra costs is shown in Table 6.2 for the Variants discussed above, with potential delay in months shown for each component as discussed below. These provide for the worse probable conditions in each area, although problems and delays would naturally occur for each component of each Variant.

a) Cofferdams — The potential for delay is larger for a two-stage cofferdam than a one-stage cofferdam and increased further in high velocity flows where steel cellular cofferdams must be used.

Assume one month delay possible with single-stage fill cofferdam, two months with two-stage fill cofferdam and three months with cellular cofferdams, due to problems with closure, seepage, dewatering.

b) Tunnels — The potential for delays increases with increasing tunnel span due to increasing support problems in zones of poor rock quality.

Assume one month delay for tunnels with two medium span diversion tunnels, two months delay for large diversion/power tunnels, and three months for complex arrangement.

c) Powerhouse — The potential for delay for foundation treatment, seepage control, and additional rockslope support is estimated at one month for bank locations, two months for riverbed locations and three months for underground powerhouse.

Potential for cofferdam overtopping is higher with spillway diversion variants than with tunnel diversion due to winter staging from frazil

ice buildup downstream which could result in a four month delay.

d) Spillway — The potential for delay for foundation treatment, seepage control and dental concrete, etc. is estimated at one month for downstream riverbed locations and 0.5 month for upstream riverbed locations.

e) Dam — The potential for delay for foundation treatment is estimated at one month for downstream locations and 0.5 months for upstream locations.

f) Rockfill Spur — The potential for ice jamming at the spur and/or the diversion channel which could result in at least a six month delay if the construction area is flooded as a result of an ice jam in the diversion channel, or by sudden release of an ice jam at the spur.

The total delay in months for each Variant provides an approximate indication of the magnitude of relative delays and extra costs with each variant, although problems would not be expected in all components of any scheme.

Although the potential delays are judgmental, the assessment shows Variants 1, 2, 4, 5, 6, 9, 10 and 11 have a higher potential risk for delays than Variants 3, 7 and 8.

6.5.1.4 Comparative Construction Schedules

Comparative construction schedules were prepared for Variants 5, 7, 10 and 11 to illustrate the relative construction duration of each Variant, from project release to first unit on power. The schedules for Variants 5, 7 and 10 were all based on a project release in January. Whereas, the schedule for Variant 11 (Hydro Quebec scheme) was based on a project release in June. The construction schedules for Variants 5, 7, 10 and 11 are shown on Figures 6.1, 6.2, 6.3, and 6.4, respectively. For comparison, Variant 11 with a project release in January (called Variant 11a) is shown on Figure 6.4a.

The following summarizes the Variants, the project release date and the comparative duration for each. The schedule for each Variant is described below.

Variant	Project Release	Duration
5	January	4 years, 10 months (58 months)
7	January	4 years, 8 months (56 months)
10	January	4 years, 11 months (59 months)
11	June	4 years, 7 months (55 months)
11a	January	4 years, 11 months (59 months)

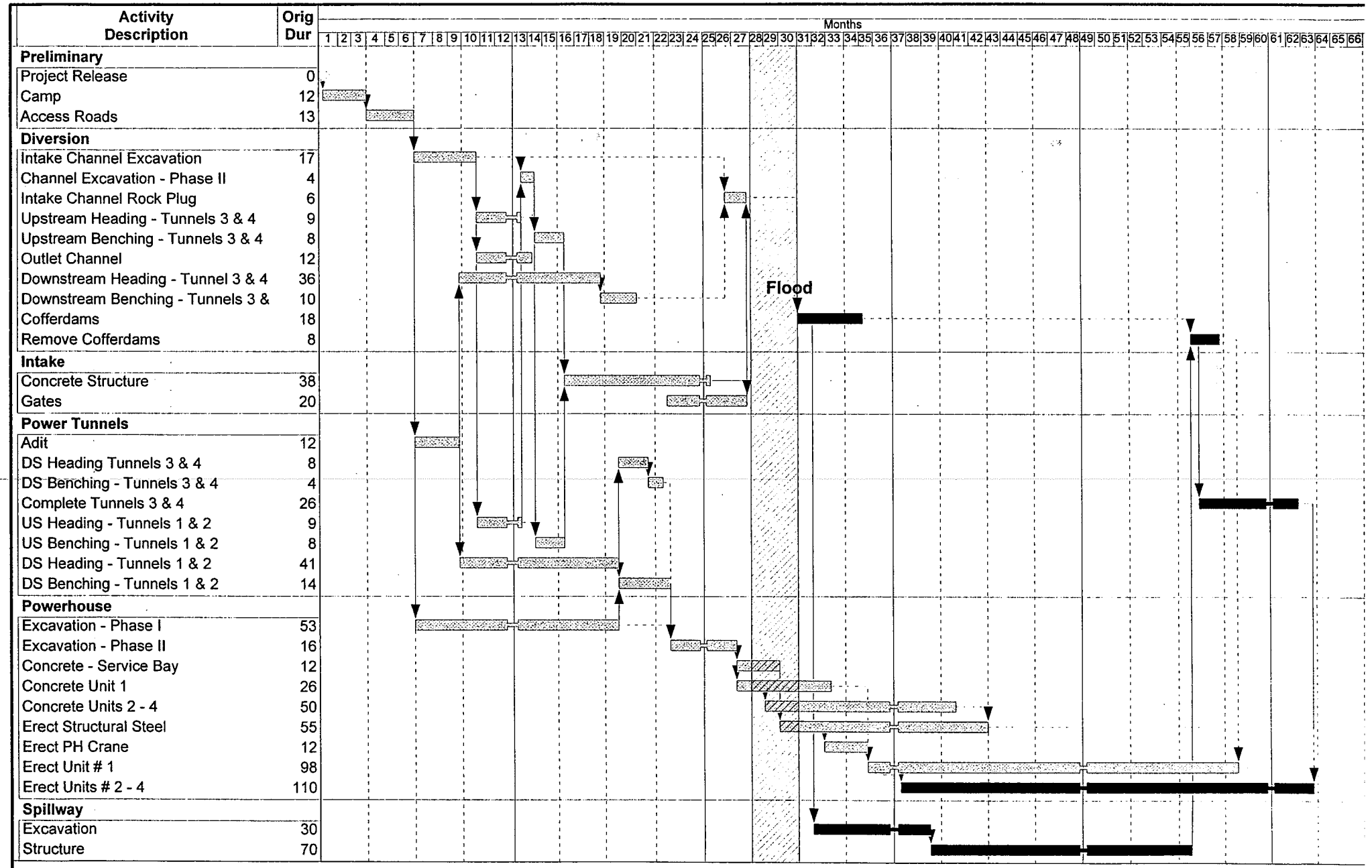

Activity Description
Orig Dur
Months
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66
Preliminary
Project Release 0
Camp 12
Access Roads 13
Diversion
Intake Channel Excavation 17
Channel Excavation - Phase II 4
Intake Channel Rock Plug 6
Upstream Heading - Tunnels 3 & 4 9
Upstream Benching - Tunnels 3 & 4 8
Outlet Channel 12
Downstream Heading - Tunnel 3 & 4 36
Downstream Benching - Tunnels 3 & 10
Cofferdams 18
Remove Cofferdams 8
Intake
Concrete Structure 38
Gates 20
Power Tunnels
Adit 12
DS Heading Tunnels 3 & 4 8
DS Benching - Tunnels 3 & 4 4
Complete Tunnels 3 & 4 26
US Heading - Tunnels 1 & 2 9
US Benching - Tunnels 1 & 2 8
DS Heading - Tunnels 1 & 2 41
DS Benching - Tunnels 1 & 2 14
Powerhouse
Excavation - Phase I 53
Excavation - Phase II 16
Concrete - Service Bay 12
Concrete Unit 1 26
Concrete Units 2 - 4 50
Erect Structural Steel 55
Erect PH Crane 12
Erect Unit # 1 98
Erect Units # 2 - 4 110
Spillway
Excavation 30
Structure 70
Flood
Run Date 29JAN99
Early Bar
Progress Bar
Critical Activity
Sheet 1 of 1
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Muskrat Falls - Variant 05
Master Schedule

Figure 6.1

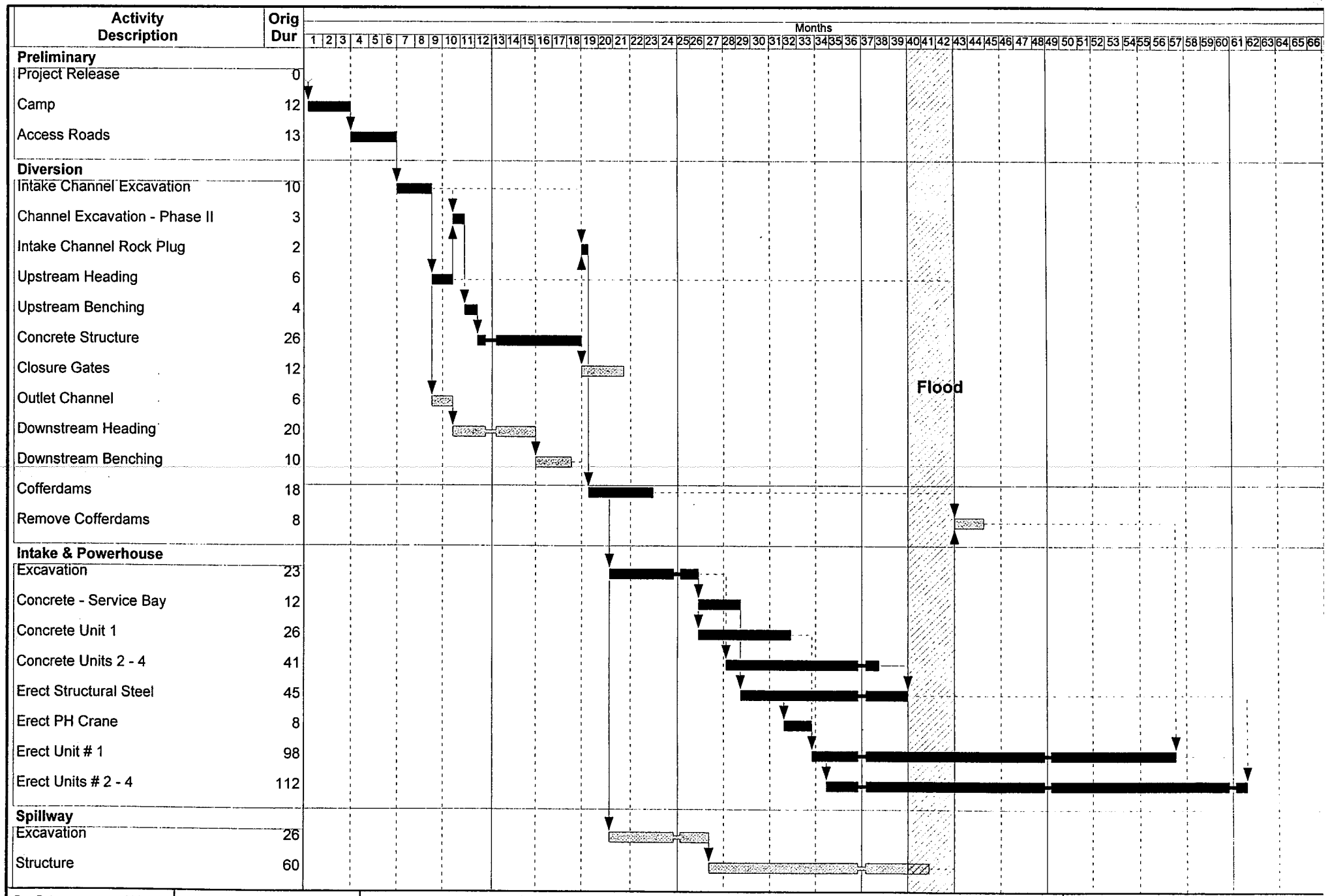
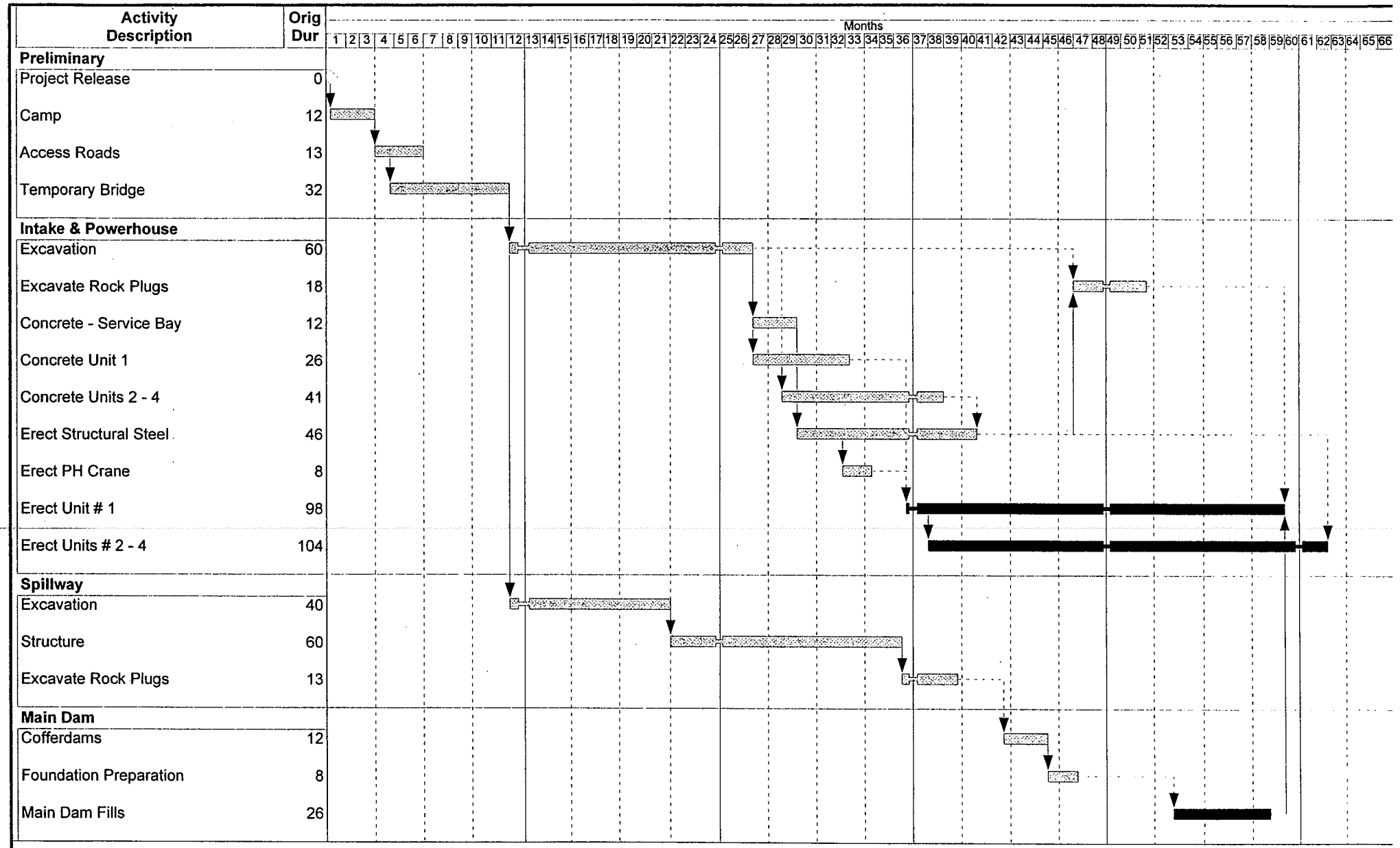

Activity Description
Orig Dur
Months
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66
Preliminary
Project Release 0
Camp 12
Access Roads 13
Diversion
Intake Channel Excavation 10
Channel Excavation - Phase II 3
Intake Channel Rock Plug 2
Upstream Heading 6
Upstream Benching 4
Concrete Structure 26
Closure Gates 12
Outlet Channel 6
Downstream Heading 20
Downstream Benching 10
Cofferdams 18
Remove Cofferdams 8
Intake & Powerhouse
Excavation 23
Concrete - Service Bay 12
Concrete Unit 1 26
Concrete Units 2 - 4 41
Erect Structural Steel 45
Erect PH Crane 8
Erect Unit # 1 98
Erect Units # 2 - 4 112
Spillway
Excavation 26
Structure 60
Flood
Run Date 29JAN99
Early Bar
Progress Bar
Critical Activity
Sheet 1 of 1
Muskrat Falls - Variant 07
Master Schedule
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Figure 6.2



Run Date 29JAN99

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Early Bar
Progress Bar
Critical Activity

Sheet 1 of 1

Muskrat Falls - Variant 10
Master Schedule

Figure 6.3

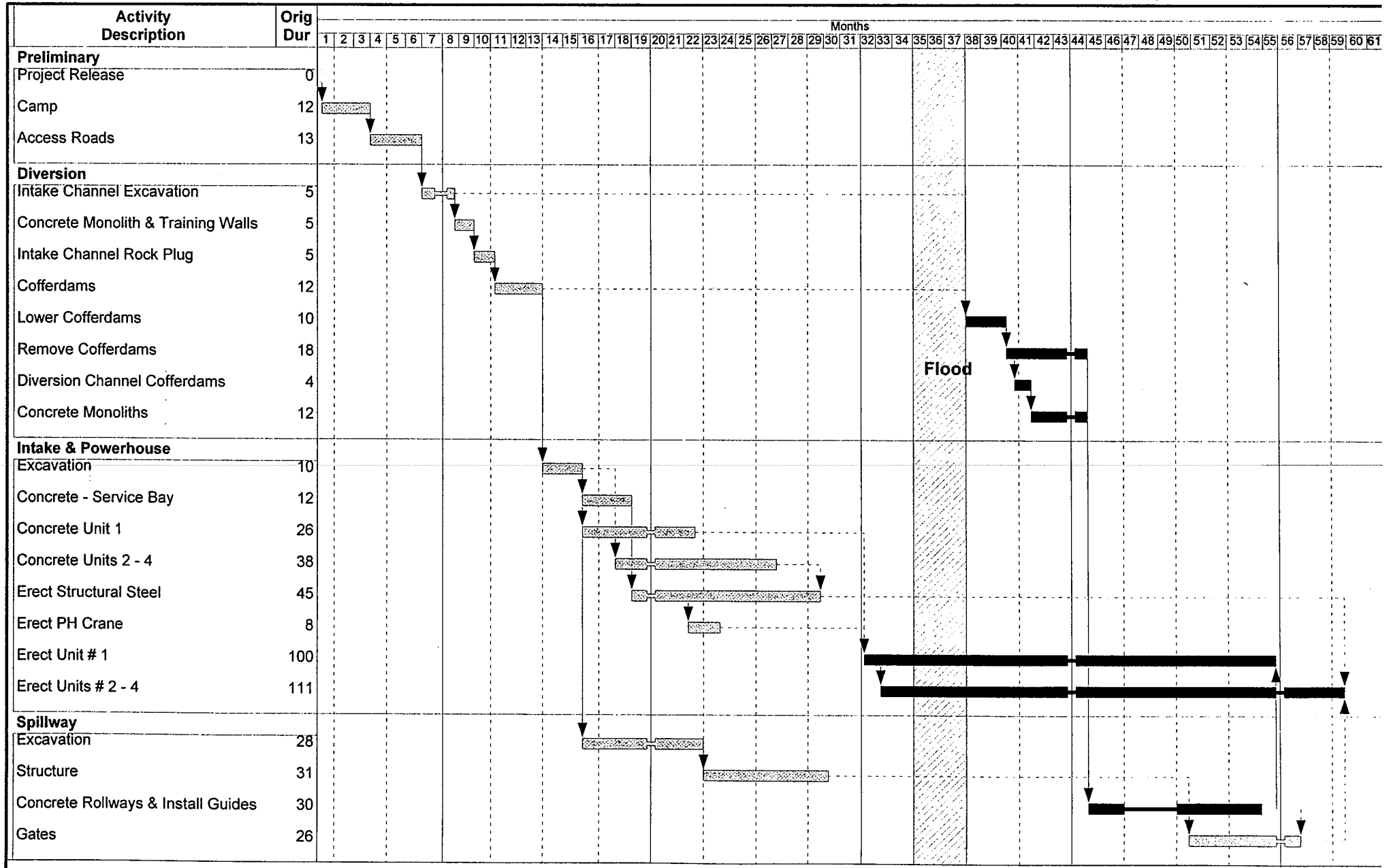
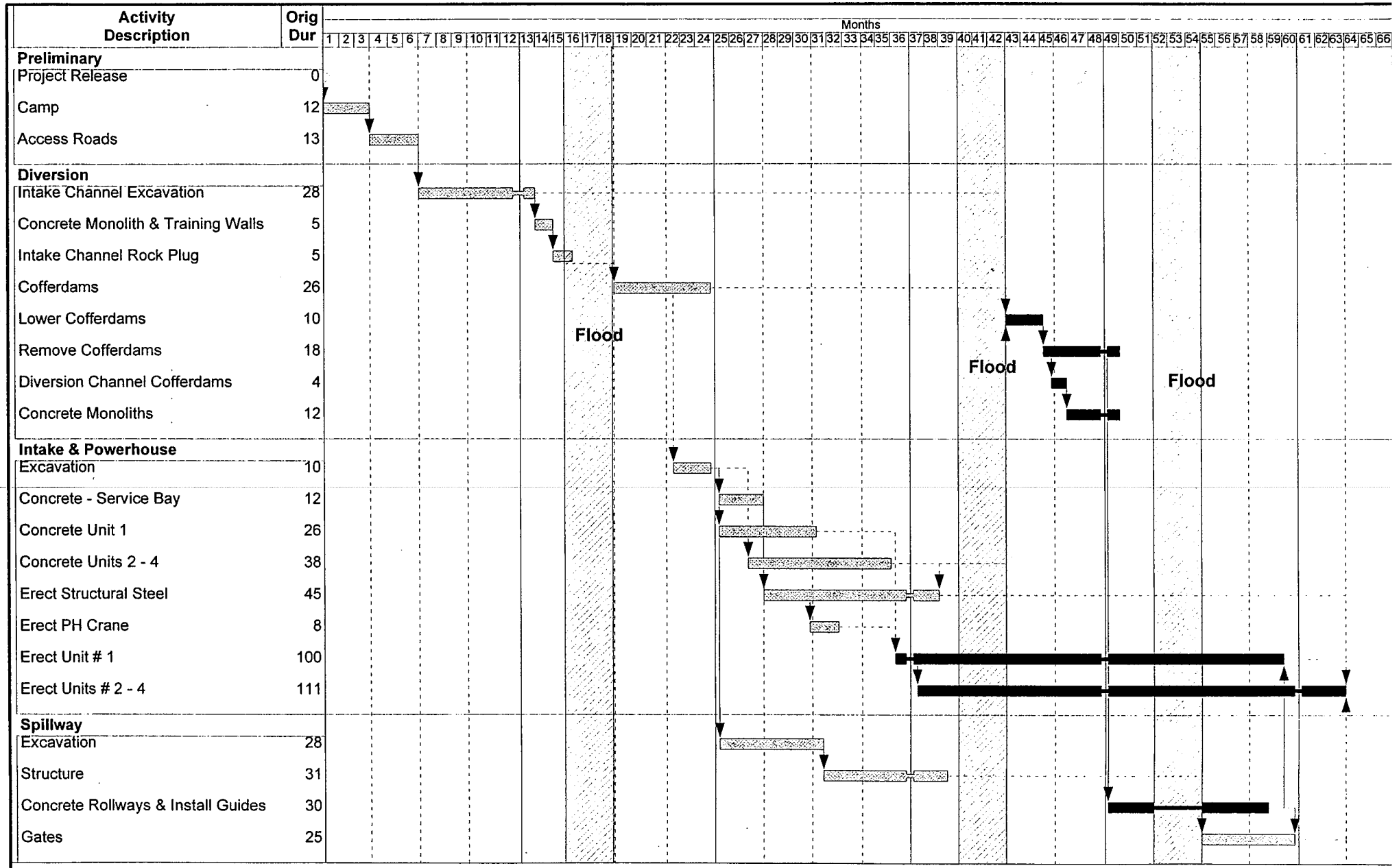

Activity Description
Orig Dur
Months
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61
Preliminary
Project Release 0
Camp 12
Access Roads 13
Diversion
Intake Channel Excavation 5
Concrete Monolith & Training Walls 5
Intake Channel Rock Plug 5
Cofferdams 12
Lower Cofferdams 10
Remove Cofferdams 18
Diversion Channel Cofferdams 4
Concrete Monoliths 12
Intake & Powerhouse
Excavation 10
Concrete - Service Bay 12
Concrete Unit 1 26
Concrete Units 2 - 4 38
Erect Structural Steel 45
Erect PH Crane 8
Erect Unit # 1 100
Erect Units # 2 - 4 111
Spillway
Excavation 28
Structure 31
Concrete Rollways & Install Guides 30
Gates 26
Flood
Run Date 29JAN99
Early Bar
Progress Bar
Critical Activity
Sheet 1 of 1
Muskrat Falls - Variant 11
Master Schedule

Figure 6.4



Run Date 29JAN99

Early Bar
Progress Bar
Critical Activity

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Sheet 1 of 1

Muskrat Falls - Variant 11a
Master Schedule - January Start

Figure 6.4a

(a)Variant 5

Project release for Variant 5 was assumed to be in January. In this Variant four tunnels would be excavated through the rock knoll on the north side of the river. The two most northerly tunnels would initially be used as diversion tunnels.

As soon as access is available to the upstream portal area, work would start. The permanent intake structure would be used as the diversion tunnel inlet. To minimize the duration of the work, an adit would be excavated from the south side of the knoll to provide access to the tunnels. Once minimum tunnel excavation has been completed from the upstream portals, the underground excavation would be completed from the adit. The intake would then be concreted in parallel with the tunnel work. The final pre-diversion activity would be the installation of the intake gates.

In parallel with the diversion tunnel work the remaining two power tunnels would be excavated from the adit. The powerhouse would also be constructed during this time period.

Once the intake has been completed, the river would be diverted into the tunnels with a cofferdam constructed upstream of the upper falls. This would allow the construction of the spillway in the dry riverbed.

Once the spillway has been completed, the cofferdam would be removed and the two diversion tunnels closed. This would allow the rock plugs in the two power tunnels to be excavated and the concrete plugs constructed in the diversion outlet tunnels. Access to this work would be through the powerhouse and diversion tunnel outlet portals.

Once the powerhouse structure has been completed and the draft tube stoplogs installed, the powerhouse rock plug would be removed.

The critical path for this Variant would be through the cofferdamming of the river and the construction of the spillway. Total duration to first power would be 4 years and 10 months (58 months).

(b)Variant 7

Project release for Variant 7 was assumed to be in January. Work would start immediately in the two diversion tunnels through the rock knoll on the north bank. Once completed, the main upstream cofferdam would be constructed above the upper falls. The main river channel would then be completely dewatered.

Work would then commence on both the intake/powerhouse and spillway. The spillway concrete would be completed and the gates installed in the dry. The powerhouse and intake concrete, superstructure and intake gates would all be completed in the dry. Stoplogs would then be installed for all the draft tubes. The cofferdams would then be removed and the reservoir impounded. After the cofferdams have been breached, and the river is flowing through

the spillway, the concrete plugs would be constructed in the two diversion tunnels. Turbine/generator erection would continue in parallel with the tunnel work.

The critical path would be through the diversion works, powerhouse excavation and concrete, turbine/generator erection and commissioning. Total duration to first power would be 4 years and 8 months (56 months).

(c)Variant 10

Project release for this Variant was assumed to be in January. The first season of work would be required to construct the access to the south shore of the river. Once this has been completed, work would start on the excavation for the powerhouse and spillway, in the dry on the south bank. The main thrust would be to complete the spillway structure and the approach and discharge channels, as quickly as possible.

Once the spillway structure has been completed and the gates installed, the river would be diverted through the spillway. Work on the powerhouse would continue behind rock plugs.

Once the diversion has been achieved, the dam area would be dewatered and the foundation prepared. At the start of the next summer season, fill placement in the main dam would start. The dam would be constructed in a single construction season.

With the dam complete, the spillway rollways would be completed and the reservoir impounded. This would then allow the units to be commissioned. Total duration to first power would be 4 years and 11 months (59 months).

(d)Variant 11 and 11a

Project release for Variant 11 was assumed to be in June. Work would start with the excavation of a diversion channel on the north bank of the river, behind a small upstream cofferdam. The lower levels of the gravity dam monoliths and training walls would be concreted.

Once the spring flood has past, the cofferdam would be removed and the first stage of the main upstream cofferdam would be constructed to close the river. In parallel with the second stage of the cofferdam construction, a rock spur would be constructed at the upper falls to partially control the upstream water level and the generation of frazil ice.

With the river flowing through the diversion channel, and upon completion of the downstream cofferdam, work would start on the powerhouse and spillway. The structures would be completed and the gates installed in the powerhouse and spillway. Stoplogs would be installed in the draft tubes.

In order to remove the upstream cofferdam the rollways of the spillway would be left down to enable passage of the river flow through the spillway.

After the spring flood has past, the upstream cofferdam would be lowered and as much as possible of the fill would be removed in the dry. Once this has been completed, the remaining cofferdam would be breached and a closure cofferdam would be constructed in the diversion channel.

Once the cofferdams are removed and the gravity dam constructed in the diversion channel, work would start on the spillway rollways. Each rollway would be dewatered by installing temporary upstream and downstream stoplogs. The river would continue to flow through the remaining spillway openings and slowly rise as the number of openings is reduced. When the final opening is closed for concreting the river will impound to a high enough level to allow the first unit to be wet tested.

Total duration to first power would be 4 years and 7 months (55 months).

Variant 11a is similar to Variant 11 in sequence. The principle difference would be with a project release in January. This would cause an unfavourable interaction with the flood seasons and would increase the total duration to first power to 4 years and 11 months (59 months).

The critical path of the construction would be through the turbine/generators. If the contract for the turbine generators could be awarded prior to project release, with an early termination clause, then the opportunity would exist to further advance the date of first power.

As previously noted, the schedules for Variants 5, 7, 10 and 11a are based on a project release in January. Should a different project release be used, the schedule durations may change.

6.5.2 Preliminary Layout Selection

Based on the 1979-1980 SNC-Lavalin Study, for a three-unit (618 MW) development of Muskrat Falls, Variant 5 was selected as the preferred development scheme, after analysis of six basic schemes and numerous related variations. The following summarizes the reasons for the selection or rejection of each variant.

Variant	Selected/Rejected	Reason
1	Rejected	Construction difficulties, together with cost and schedule risks associated with cellular cofferdams.
2	Rejected	Construction difficulties, together with cost and schedule risks associated with cellular cofferdams.
3	Rejected	Similar cost but longer schedule than Variant 5, due to higher concrete quantities.
4	Rejected	Construction difficulties, together with cost and schedule risks associated with cellular cofferdams.
5	Selected	Accepted by Client and Board of Consultants as

Variant	Selected/Rejected	Reason
		feasible, with lowest cost and shortest schedule.
6	Rejected	Underground powerhouse not cost effective for a low head development. Higher cost than Variant 5.

The 1989 SNC Study, which considered development schemes suitable for a higher plant capacity, i.e. four units of 206 MW, did not include Variant 5 of the 1979-1980 Study. Based on the 1979 geotechnical information, because of the fault zone through the southeast corner of the rock knoll, a four-unit powerhouse option for Variant 5 was considered too risky and costly. Variant 3, which in 1980 was considered as the second choice for development, and was suitable for a four-unit development of Muskrat Falls (824 MW), was revisited in the 1989 Report, and called Variant 7. Three other Variants, 8, 9 and 10, as previously described in Sections 6.2.5, 6.2.6 and 6.4.1, respectively, were included in the 1989 Report.

The 1989 Report did not recommend either of the four Variants as a preferred scheme of development. The report concluded that the costs of the four schemes were similar (within about 2%).

In the current (1998) study, the ten previous Variants were reviewed, together with Variant 11, suggested by Hydro Quebec.

Based on the qualitative comparison of the eleven Variants, four of the Variants were selected for a quantitative comparison. The following summarizes the reasons for selection of Variants 5, 7, 10 and 11 for the quantitative comparison.

Variant	Reason
5	Based on the 1998 geotechnical investigations and review of the fault zone at the southeast corner of the rock knoll, it was deemed to be feasible to construct a four-unit powerhouse in this area and that any additional costs as a result of the fault would be of relatively minor consequence. Variant 5 was accepted by the Board of Consultants and was proposed to, and accepted by, the Client, in 1980, as the preferred scheme of development.
7	Previously referred to as Variant 3 (3-units) this was the second Variant of choice in 1980, with a similar cost as Variant 5 but with a slightly longer construction schedule.

Variant	Reason
	It offers several advantages, and few disadvantages, as noted in Table 6.1, including: • total frazil ice control; • simple diversion arrangement; • less hydraulic headloss than Variant 5; • simpler sequence of construction than Variant 5; • fewer unwatering closures; • complete unwatering of the power development works throughout the construction period; and • possible improvement in the construction schedule (Variant 8) due to earlier access to the powerhouse.
10	It offers the significant advantages of: • excavation for all the major structures in the dry; • less hydraulic headloss than Variant 5; • only main dam construction in the river; and • a construction schedule similar to Variant 5.
11	It offers the advantages of: • less hydraulic headloss than Variant 5; and • construction schedule shorter than Variant 5 and similar to Variant 7.

Variants 1, 2, 3, 4 and 6 of the 1979-80 study were not included in the quantitative comparison for the reasons stated above. Variant 8 of the 1989 study was not included since it is similar to Variant 7 but more costly due to the reverse locations of the intake/powerhouse and spillway, and due to the more adverse bedrock profile at their foundations. Variant 9 of the 1989 study was not included due to the disadvantages and higher risk as stated in Table 6.1.

6.5.3 Quantitative Comparison

A quantitative comparison was made of the selected Variants 5, 7, 10 and 11. The comparison is expressed as relative costs which are summarized in Table 6.3. This comparison was based on the previous capital cost estimates, escalated to June 1998 Canadian dollars, as follows:

- Variant 5 was escalated from 1980, and
- Variants 7 and 10 were escalated from 1989.
- Variant 11 was based on prices escalated from 1989.

The escalated cost estimates were adjusted to account for any changes from the original estimates. For example:

- A fourth unit at 206 MW, and associated intake, tunnel and powerhouse costs were added to Variant 5.
- Additional quantities were added to Variant 5 for change in design of the surge shafts since 1980.
- Costs were added for removal of the upstream cofferdam in Variant 7. These costs were not previously included.

In addition, relative costs as a result of differences between the Variants were taken into account. For example:

- A 15% reduction in the cost of turbines in Variants 7 and 10 was made, relative to Variant 5, for substitution of the steel spiral case with concrete semi-spirals in Variants 7 and 10.
- The differences in construction schedule were accounted for by the inclusion of estimated interest during construction (IDC), based on an interest rate of 8%.
- The value of lost generation (sales), resulting from an increase in construction schedule, based on a value of $0.04/kWh, was added to Variants with longer schedules.
- The value of increased headloss in Variant 5, versus Variants 7, 10 and 11 was added as a cost to Variant 5, based on a value of $0.04/kWh.
- The value of the estimated increase in turbine efficiency of 0.3% for a concrete semi-spiral versus a steel spiral was added to Variants 7, 10 and 11, based on a value of $0.04/kWh.

The total relative projects costs, including all of the above relative costs for schedule and efficiency differences, are:

Variant	Total Cost (June, 1998$)
5	$1,744,310,000
7	$1,554,040,000
10	$1,659,730,000
11	$1,695,200,000

6.6 CONCLUSIONS

By means of a series of qualitative and quantitative comparisons the list of eleven alternatives (or variants) was reduced to two, Variant 7 and Variant 11. These two remaining Variants which were of similar costs, were then examined in greater detail. The following points emerged from this comparison:

- The key feature of Variant 11 is the rockfill spur that would be constructed partway across the river to raise the water level upstream, to permit formation of an ice cover and limit the formation of frazil ice. The basic design requirement, as recommended by LCG (Reference 53), is to maintain the upstream water level to El. 24.0 m. The spur should also be able to resist the 1 in 40 year flood. The viability of Variant 11 depends on the reliable functioning of the spur. SNC-AGRA's preliminary calculations estimated the flow velocity through the gap (between spur and opposite shore) to be about 9.0 m/s increasing to about 13.0 m/s further downstream. These flow velocities would required armour stones weighing over 8 tonnes. Such large armour stone cannot be produced from structure excavations and will have to be obtained from a special off site quarry or alternatively by concrete tetrapods which would have to be manufactured for the critical end sections of the spur - an expensive requirement. The unit flow, flow velocity and head drop through the gap between the spurs are without precedent according to SNC-AGRA inquiries, as shown in Table 6.4 which compares the hydraulic parameters of the Variant 11 Rock Spur with the Manasan Ice Control Structure in Manitoba. The Manasan Structure is believed to represent the upper limit of experience with this design. In these circumstances Variant 11 must be considered as very risky and should only be considered if it provides substantial cost saving. It will require careful study, including model testing.

Table 6.4
Variant 11 Rock Spur - Summary of Hydraulic Calculations
For Q_{40} = 5300 m^3/s

Option	W.L. for Ice Control	Gap Between Spurs (m) (1)	Unit Flow (m^3/s per m) (2)	H.W.L. (m)	ΔH (m) (3)	Velocity in gap (m/s)	Max. Velocity Downstream (m/s) (4)	Weight of Armour (t) (5)
1	24.0	41.0	82.1	29.7	8.7	9.0	13.3	8.2
2	23.0	50.0	74.2	28.3	7.3	7.3	12.0	6.8
			For design flow = 1100 m^3/s					
Manasan Ref. 56 & 57	N/A	~23	31	179.9	4.2	4.2	9.0	2 to 4

Notes:

1. Gap between spurs at river bottom
2. Unit flow = Q_{40} / width at mid depth
3. ΔH = difference between H.W.L. and 21.0 m, water level under natural conditions

4. Max. velocity = $\sqrt{2g.\ \Delta H}$

5. Weight of armour stone per Isbash curve (reference 6) based on velocity in gap.

- The main advantage of Variant 7 is the simplicity of design concept, reliability of ice control and low risk;

- Variant 11 may only provide partial upstream water level control and frazil ice control, necessitating a higher downstream cofferdam than Variants 5, 7 and 10;

- Variant 11, with a rockfill spur for upstream water level control, carries the risk of fluctuating upstream water levels, breakup of the ice cover and consequential risk of ice-jams at the spur or in the diversion channel, possibly resulting in flooding of the powerhouse and spillway construction area;

- Variants 7 and 11 provide similar construction schedules, of shorter duration than Variants 5, 10 and 11a;

- Variant 7 offers the least relative risk and Variant 11 would have the highest relative risk; and

- Variant 7 offers the least total direct cost and total relative project cost.

In conclusion, Variant 7 was selected as the optimal scheme of development for Muskrat Falls as it was found to be the least expensive, although by a small margin over Variant 11. Variant 7 was also judged to be much less risky than Variant 11.

6.7 ALTERNATIVE SPILLWAY LAYOUTS

6.7.1 Background

The spillway, being an expensive component of the Muskrat Falls Development, was targeted for concept optimization. In this respect it is worth noting the features of the Muskrat Falls site that offer potential for alternative design of spill facilities.

- river diversion facilities can be upgraded to permanent spill facilities at relatively low cost;

- the north and south dams are relatively low and can be designed as overflow spillways at relatively low cost in either conventional or roller compacted concrete (RCC); and

- rubber dams, mounted on the crest of concrete monoliths, offer reliable and economical flood control.

Preliminary cost comparisons indicated there could be substantial cost savings, if some of the above features were incorporated into project design so as to reduce the number of spillway gates.

On the basis of these observations, six spillway alternatives were identified. Preliminary designs and comparative cost estimates were prepared.

6.7.2 Design Criteria

Preliminary design for the six spillway alternatives was done in accordance with the following criteria, from the Canadian Dam Safety Association Guidelines:

- The main spill facility, gated spillway and north dam, should be capable of handling the 1 in 1,000 year flood (10,100 m^3/s) with a maximum flood level of 40.0 m. This corresponds to a 5% lifetime risk of occurrence, based on a 50 year service life.

- The main and auxiliary spill facilities should be capable of handling the PMF (22,100 m^3/s).

  (The governing concern during the PMF is that the structural integrity of the dams containing the reservoir should be assured although some damage would be acceptable.)

- The spill facilities should be sized to handle the 1 in 1,000 year and PMF floods assuming the power plant is out-of-service.

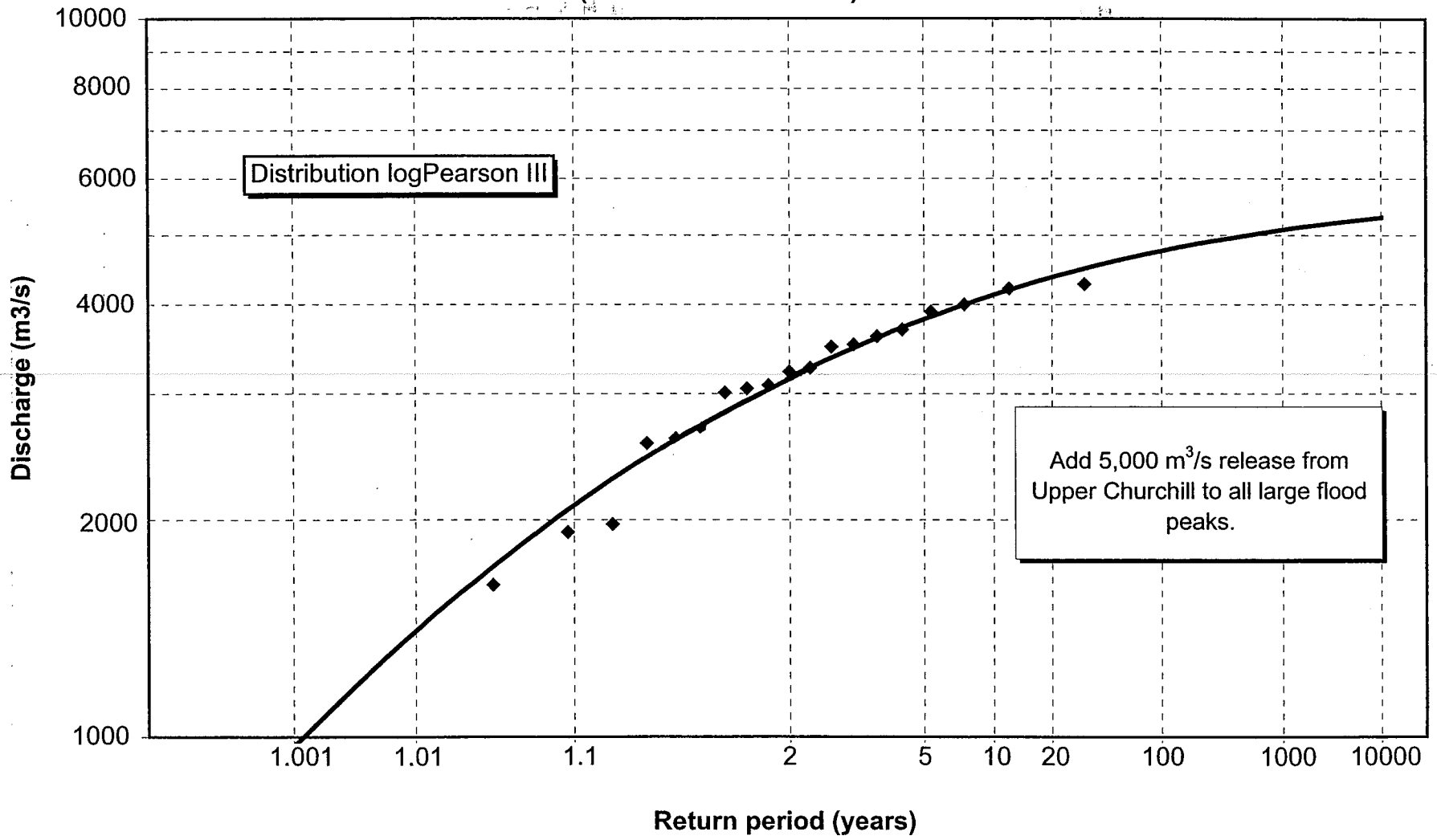
These criteria are consistent with international practice, notably, in Norway, "The Norwegian design criteria are laid down in legislation. They contain a double flood criterion; the spillway system should handle the 1,000 year return period design flood (Q_{1000}) in normal operation and without damage to the dam and spillway, and the dam should withstand the probable maximum flood (PMF) without failure. Originally the design criteria were not based on dam classification. They are now modified, and contain a classification according to consequences of failure, with three classes. In the low hazard class - no permanent settlements endangered - dams are only evaluated against Q_{1000}. In the high hazard class, dams are always evaluated against the PMF."(Reference 45).

A further criterion, where economically feasible, is to be able to discharge the maximum plant flow (Q_p = 2,667 m^3/s) over closed spillway gates plus auxiliary overflow spillway(s) without excessive (dangerous) rise in reservoir water level in an emergency situation. Excess plant flow would normally be handled by opening a spillway gate, in the event of a transmission line outage or similar situation.

The 1 in 1,000 year flood (10,100 m^3/s) was determined from frequency analysis of flows from the Lower Churchill River (downstream of Churchill Falls) plus a flow release of 5,000 m^3/s from Churchill Falls similar to the assumption applied in determining the PMF (see Figure 6.5).

The proposed spillway alternatives comprise a number of component flow handling structures, as below:

Figure 6.5
Muskrat Falls Peak Discharge
(from Lower Churchill)



a) **Main Facilities** (used during normal operation)

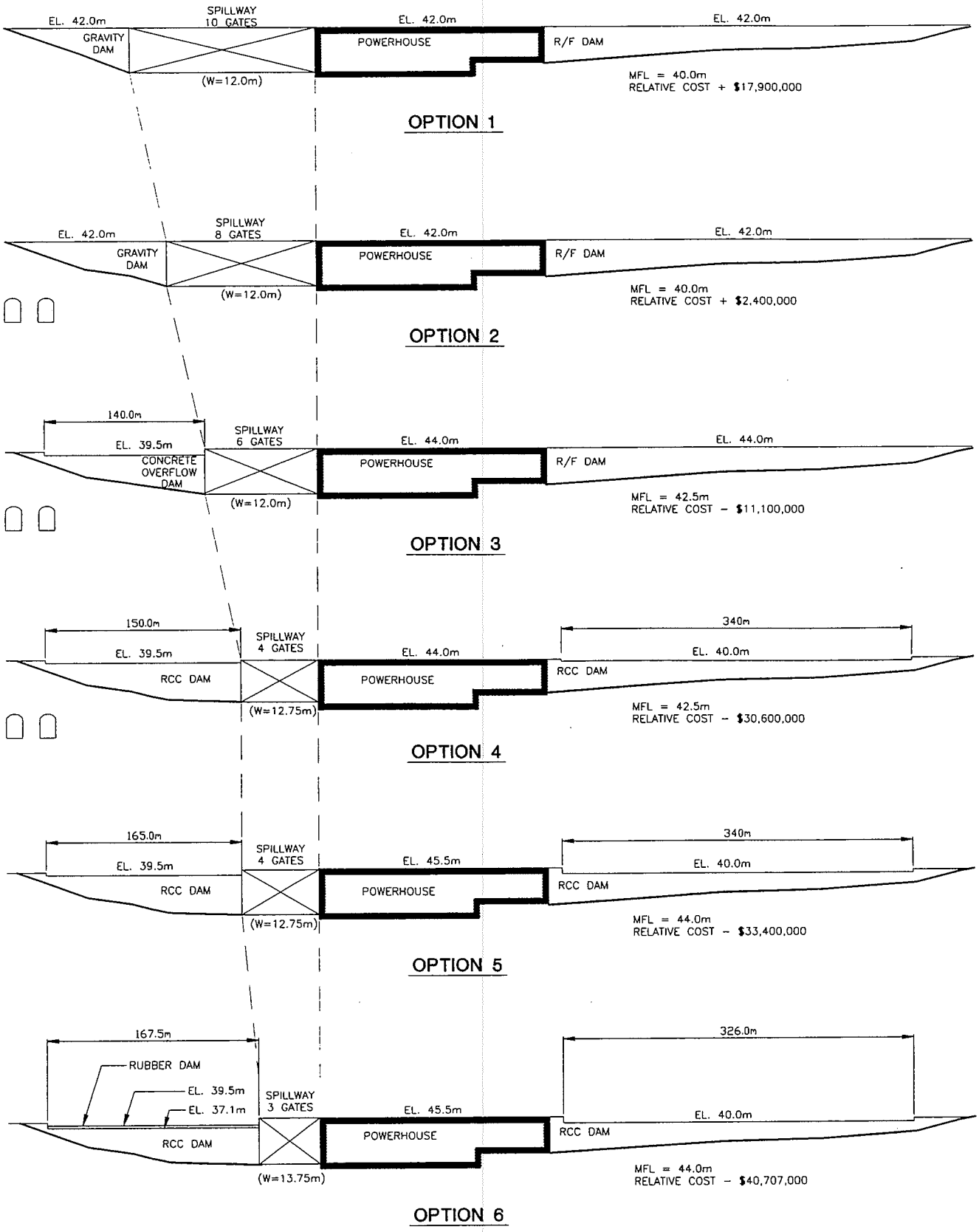
- Gated spillway, number and size of gates vary;
- North abutment overflow dam, of conventional concrete or RCC construction;
- North abutment (RCC) dam, with rubber crest dam.

b) **Emergency Facilities** (only used for floods greater than Q_{1000})

- South abutment (RCC) dam;
- Diversion tunnels.

(Some authorities advise against the use of diversion gates (or similar gated structures) for rare emergency flood handling, pointing out that there may be little incentive to exercise or maintain such facilities. Accordingly, the reliability of such facilities could be called into question. The diversion tunnel option would only be entertained if very significant savings were to be obtained. An inherently reliable structure such as an overflow dam is generally preferred for rare emergency service.)

The main features of the six spillway alternatives are summarized in Table 6.5; and shown schematically in Figure 6.6. Table 6.6 gives pertinent data on flood handling characteristics.



COMPARISON OF SPILLWAY ALTERNATIVES
(BASE CASE IS VARIANT 7)

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FIGURE 6.6

Table 6.5
Description of Spillway Alternatives

Option	Gates	Diversion Tunnels	North Dam	South Dam	Dam Crest	Remarks
Base Case	8, 12.0 x 20.2 m				42.0 m	• Variant 7
1	10, 12.0 x 20.2 m				42.0 m	• Similar to Variant 7
2	8, 12.0 x 18.7 m	Two tunnels 14 x 17.0 m			42.0 m	• Diversion intake and gates upgraded for permanent service
3	6, 12.0 x 19.0 m	Two tunnels 14.0 x 17.0 m	Crest at 39.5 m Net Length = 140 m Conventional Concrete		44.0 m	• Rockfill dam crest raised to 44.0 m
4	4, 12.75 x 20.2 m	Two tunnels 14.0 x 17.0 m	Crest at 39.5 m Net Length = 150 m RCC	Crest at 40.0 m Net Length = 340 m RCC	44.0 m	• road and powerhouse • no rockfill section • erosion below south dam, temporary loss of access to powerhouse with PMF
5	4, 12.75 x 20.2		Crest at 39.5 m Net Length = 165 m RCC	Crest at 40.0 m Net Length = 340 m RCC	45.5 m	• road and powerhouse • erosion below south dam with PMF
6	3, 13.75 x 20.2 m		Rubber dam Crest: Inflated = 39.5 m Deflated = 37.1 m Net Length - 167.5 m RCC	Crest at 40.0 m Net Length = 326 m RCC	45.5 m	• road and powerhouse • erosion below south dam with PMF

Notes:

- RCC - Roller Compacted Concrete.
- For hydraulic and structural design criteria, see Appendix B.

Table 6.6
Flood Handling Characteristics

Option	Case	Flood Discharge Through or Over (m^3/s)				Flood Level (m)
		Gates	Diversion Tunnels	North Dam	South Dam	
Base Case	PMF	18,700	-	-	-	40.0
	Q_{1000}	10,100	-	-	-	39.0
	Q_p	2,667[1]	-	-	-	45.3
1	PMF	22,100	-	-	-	40.0
	Q_{1000}	10,100	-	-	-	39.0
	Q_p	2,667[1]	-	-	-	44.5
2	PMF	16,400	5,700	-	-	40.0
	Q_{1000}	10,100	nil	-	-	39.0
	Q_p	2,667[1]	nil	-	-	45.3
3	PMF	14,800	5,700	1,600	-	42.5
	Q_{1000}	10,100	nil	nil	-	39.0
	Q_p	850[1]	nil	1,817	-	42.8
4	PMF	11,800	5,700	1,700	2,900	42.5
	Q_{1000}	9,980	nil	120	nil	40.0
	Q_p	290[1]	nil	953	1,424	41.6
5	PMF	12,800	-	3,400	5,900	44.0
	Q_{1000}	9,980	-	120	nil	40.0
	Q_p	290[1]	-	953	1,424	41.6
6	PMF	10,800	-	5,800	5,500	44.0
	Q_{1000}	8,450	-	1,650	-	40.0
	Q_p	2,667[2]	-	-	-	39.2
	Q_p	98[1]	-	2,214[3]	355	40.6

Note:

- PMF =22,100 m^3/s
- Q_{1000} =10,100 m^3/s
- Q_p =2,667 m^3/s
- The main flood handling facilities are the gated spillway and the north overflow dam (in Options 3 to 6). The diversion tunnels and south overflow dam are considered as emergency spillways that would only be required for floods greater than Q_{1000} up to the PMF.

(1.) Flow over top of closed gates, with powerplant out of service
(2.) Flow through one gate
(3.) All five sections of rubber dam deflated

It is worth noting that the maximum plant flow can be safely handled over closed spillway gates, without exceeding the PMF flood level in options 4, 5 and 6, but not in the other options.

6.7.3 Comparison of Alternatives

The approach taken was to evaluate the cost differences between the various alternatives with reference to Variant 7. Costs common to all options were omitted since they would not influence the outcome of the comparisons. Cost increases are shown as positive while benefits (or cost reductions) are shown as negative, with respect to Variant 7.

Operating and Maintenance costs (O&M) were not explicitly considered since O&M cost differences between alternatives are relatively small compared to capital costs. The only exception was for the rubber dam which included an allowance to account for complete replacement of the dam after 25 years in service (a conservative assumption).

The resulting comparison of costs was, as follows:

Table 6.7
Comparison of Cost Differences - Spillway

Option	Difference from Variant 7
1	+$17,900,000
2	+$2,400,000
3	-$11,100,000
4	-$30,600,000
5	-$33,400,000
6	-$40,707,000

6.7.4 Recommendations

Table 6.7 shows progressive cost improvements from Option 1 to Option 6.

This comparison shows that there is a clear economic advantage for Option 6, which meets all operational requirements, over the next best option. Operationally option 6 is inherently more reliable than options 2, 3 and 4 as noted in Section 6.7.2, due to the latter's dependence upon seldom used, submerged gates. Also, option 6 is operationally superior to option 5 in that the flood discharge over the south dam would be less.

Option 6 was therefore selected for final project layout.

SECTION 7.0

RECOMMENDED SCHEME OF DEVELOPMENT

SECTION 7.0

RECOMMENDED SCHEME OF DEVELOPMENT

7.0 RECOMMENDED SCHEME OF DEVELOPMENT

7.1 GENERAL

Development of the Muskrat Falls site will include stabilization of the natural dam, called the north spur, which connects the rock knoll to the north bank of the original river valley, and power development facilities in the river channel, including diversion, reservoir control and power facilities. The stabilization of the spur is a common element to any alternative power development schemes, of which several were considered.

Based on the qualitative and quantitative comparisons made in Sections 6.5.1 and 6.5.3, Variant 7 is recommended as the scheme of development for the power development facilities for Muskrat Falls. This scheme offers the alternative with the least total project cost, least risk, as well as the simplest diversion and construction sequence.

The recommended scheme benefits from the natural geography at the site. The rock knoll on the north bank will contain two tunnels that will be used for diversion of the River discharges around the construction area. A diversion cofferdam will be located in the river channel, upstream of the upper falls. An integrated intake/powerhouse, main spillway and flanking overflow dams will be located in the bedrock river channel at the lower falls. The overall general arrangement is shown on Plate 10.

7.2 PRINCIPAL ELEMENTS OF THE PROJECT

The principal elements of the project are described in detail in Sections 8 to 11. The principal elements comprise:

- stabilization works for the north spur;
- diversion facilities;
- reservoir control facilities;
- dams; and
- power facilities.

A brief description of these elements is given below.

7.2.1 Stabilization Works for the North Spur

The spur of land connecting the rock knoll with the north shore forms a natural dam with steep upstream and downstream slopes, close to the limit of their natural stability. The works required to stabilize these slopes for post-project conditions will include a rockfill berm on the upstream slope, a rockfill berm and slope trimming on the downstream side, unloading of the downstream slope adjacent to the rock knoll, additional pumpwells in the existing pumpwell line, and provision of additonal drainage ditches to lower the groundwater table in the spur. The overburden in this spur ranges from impervious to semi-pervious, and seepage quantities will be minor.

7.2.2 Diversion Facilities

For most of the construction period, two unlined diversion tunnels will carry the river discharge through the rock knoll and around the construction area. During the diversion period, gated concrete inlet structures will regulate water levels in winter to create a stable ice cover upstream of Muskrat Falls. This will prevent the formation of the ice dam that frequently occurs downstream of the lower falls. Downstream water levels will remain constant in winter, enabling work to proceed on the power development structures all year round. A plan view and details of the diversion facilities with a cofferdam upstream of the upper falls can be seen on Plates 12 and 13. The diversion tunnel inlet connects to the reservoir via a rock channel 67 m wide. Separate inlet structures are provided to each of the two tunnels. Each tunnel inlet will be provided with twin gates 6.6 m x 17.0 m, designed for heated winter operation. Concrete plugs will be provided in the diversion tunnels after reservoir impounding.

7.2.3 Reservoir Control Facilities

The reservoir control structures at Muskrat Falls comprise a three-bay gated spillway and north and south RCC overflow dams. The north dam is fitted with five sections of rubber dam crest. The dam will raise the water level from its current elevation of about 19 m to a maximum operating level of 39 m. The narrow reservoir thus created will remain within the valley and will extend upstream 60 km to the foot of Grizzle Rapids, the site of the proposed Gull Island hydroelectric project. The spillway, with its three gates of 13.75 m x 20.2 m, is capable of discharging 49% (10,800 m^3/s) of the PMF (22,100 m^3/s) with a 5.0 m surcharge in reservoir level. Flow from each bay is contained between piers and is discharged out over a 20° flip on a concrete apron to some distance from the concrete structure, preventing erosion of the bedrock at the toe of the apron. The north overflow dam with rubber dam crest and the south RCC overflow dam are capable of discharging 26% and 25% of the PMF (5,800 m^3/s and 5,500 m^3/s), respectively.

The north RCC dam closes the valley on the north side of the spillway. The south RCC dam closes the valley on the south side of the intake/powerhouse block. These dams, with a maximum height of 34.5 m, are roller compacted concrete gravity structures with conventional concrete crest and facings. They are constructed on prepared bedrock surfaces and abut concrete training walls at the spillway and intake/powerhouse structures.

Both dams are designed to spill water by overtopping. The north dam and the gated spillway are the main flood handling structures and these will spill the plant flow during unit shutdown and during floods up to the 1 in 1,000 year occurrences. The spill capacity of the north dam is increased by the incorporation of a rubber dam crest on the RCC structure. The rubber dam is 2.4 m high when inflated to a crest elevation of 39.5 m. It consists of five sections which can be deflated together, or in succession, as required, to control the reservoir level.

The sequence of operation of the flood discharge facilities would be:

- 1st: main spillway;
- 2nd: north RCC overflow dam; and
- 3rd: south RCC overflow dam.

In the event of one or more units shutting down, the flow would normally be discharged through one of the main spillway gates. In the rare event that all units shutdown and the gates fail to open, the flow could be discharged over the top of the gates and over the north and south dams. The flow over the south dam would be minimized by deflating all five sections of the rubber dam. All floods up to the 1 in 1,000 year flood would be discharged through the three gates of the main spillway and over the rubber dam on the north RCC dam.

For floods in excess of the 1 in 1,000 year flood, all of the reservoir control structures would be involved and water would start to spill over the south dam. The south dam would be the last to spill, to protect the permanent access to the powerhouse, located downstream of the dam.

7.2.4 Power Facilities

The intake and powerhouse are combined in an integrated block, located adjacent to the spillway in the river, just upstream of the lower falls. The powerhouse contains four generating units, including 3-206 MW Propeller turbines, 1-206 MW Kaplan turbine and four identical generators. Access to the site is provided by a permanent access road from the north, along the south slope of the rock knoll. The road crosses over the dams, spillway and intake structures to the south side of the River and thence is located along the downstream side of the south RCC dam, connecting to the service bay at the south end of the powerhouse. Bridges erected on concrete piers, built integrally with the dams and spillway structure, carry the road across the structures and facilitate concurrent reservoir spilling.

Water enters the turbines via rectangular intake passages into the concrete semi-spiral case and is discharged via an elbow draft tube directly to the tailrace below the lower falls.

The four generating units are connected to two power transformers located on the downstream deck of the powerhouse. From there, lines are connected to a conventional switchyard with an open ring bus arrangement located on the roof of the powerhouse. Two 230 kV transmission lines are expected to proceed along the south shore of the reservoir to the area of the upper falls, thence across the river to the north shore and along the north shore to the HVDC converter station near Gull Island.

The reservoir for the Muskrat Falls site is maintained at a relatively constant operating level of El. 39.0 m with the unit operating essentially as a "base load" generator. Conditions in the upstream river channel and the estuary downstream will have minimal disturbance resulting from the proposed project. The dam will have a positive effect in that the ice buildup as presently experienced, will be eliminated and erosion of the riverbanks would be less.

7.3 SALIENT DATA

Salient data on components of the project are as follows:

(Note: All elevations are based on geodetic datum)

7.3.1 Drainage Basin

• Total drainage basin area (including St-Jean and Romaine River diversions)	101,938	km^2
• Regulated drainage basin area (Churchill Falls)	78,850	km^2
• Long term average flow (including St-Jean and Romaine River diversions)	2,026	m^3/s
• Maximum plant flow capacity	2,667	m^3/s
• Diversion design flood (1:40)	5,300	m^3/s
• Design flood (PMF)	22,100	m^3/s

7.3.2 Reservoir

• Maximum flood level at PMF	44.0	m
• Maximum operating level	39.0	m
• Minimum operating level	38.5	m
• Maximum water level at diversion design flow (1:40)	30.5	m
• Flooded area at maximum operating level	106.8	km^2
• Reservoir length	60.0	km
• Total storage at elevation 39.0 m	1,520	x 10^6 m^3

7.3.3 Diversion Facilities

7.3.3.1 *Upstream Cofferdam*

• Freeboard (on MFL at Q_{40})	1.0	m
• Crest elevation	31.5	m
• Maximum height	23.5	m
• Crest length	320.0	m
• Volume fill	306,800	m^3

7.3.3.2 *Approach Channel*

• Sill invert elevation	12.0	m
• Width at invert	67.0	m

7.3.3.3 *Inlet Concrete Structures (2)*

• Number of water passages/structure	2	
• Steel deck elevation	40.0	m
• Top of concrete	31.0	m
• Invert elevation	4.0	m
• Width (each)	20.0	m

7.3.3.4 *Inlet Gates*

• Number/structure	2	
• Total Number	4	
• Width	6.6	m
• Height	17.0	m

7.3.3.5 Tunnels

• Number	2	
• Type	Inverted U	
• Length (portal to portal)	600	m
• Cross Section (W x H)	14.0 x 17.0	m

7.3.3.6 Outlet Portal

• Number of tunnel outlets	2	
• Invert elevation	-12.0	m
• Width at invert (each)	14.0	m

7.3.3.7 Outlet Channel

• Invert elevation	-4.0	m
• Width at invert (each)	28.0	m

7.3.4 Intake Facilities

7.3.4.1 *Approach Channel*

• Invert elevation	1.0	m
• Width at invert	138	m

7.3.4.2 *Intake Structure*

• Close coupled to powerhouse		
• Deck elevation	45.5	m
• Sill elevation	3.0	
• Width	144.5	m

7.3.4.3 *Intake Head Gates*

• Number	12	
• Width	7.33	m
• Height	16.3	m

7.3.4.4 *Bulkhead Gates*

• Number	3	
• Width	7.33	m
• Height	17.8	m

7.3.4.5 *Trashracks*

• Number	12	
• Width	7.33	m
• Height	22.0	m
• Clear spacing between bars	100	mm

7.3.5 Power Facilities

7.3.5.1 *Power Installation*

• Gross head	37.0	m
• Rated net head	35.0	m
• Power generation		
• Number of units	4	
• Rated plant output	824	MW
• Speed	81.82	rpm

7.3.5.2 *Powerhouse*

• Length	187.5	m
• Total width	68.8	m
• Width of main generator floor	28.0	m
• Roof elevation	37.5	m
• Draft tube invert elevation	-30.4	m
• Unit spacing	36.0	m
• Concrete semi-spiral inlet dimensions (W x H)	28.0 x 14.0	m

7.3.5.3 *Turbines*

• Type	3-Propeller, 1-Kaplan	
• Rated discharge, per unit (at efficiency 93% (P), 92% (K))	665	m^3/s
• Runner diameters	9.0	m

• Runner mass	- Propeller	140	tonnes
	- Kaplan	230	tonnes
• Total turbine mass	- Propeller	1,600	tonnes
	- Kaplan	1,800	tonnes

7.3.5.4 *Generators*

• Rated output, each	208	MW
• Rated voltage	15.0	kV
• Power factor	0.95	
• Rotor diameter	14	m
• Rotor mass	740	tonnes
• Total generator mass	1,250	tonnes

7.3.5.5 *Powerhouse Cranes*

• Number	2	
• Main hook capacity, each	400	tonnes
• Auxiliary hook capacity, each	25	tonnes
• Equalizing beam mass	50	tonnes

7.3.5.6 *Power Transformers*

• Number	2	
• Rating	165/165/330	MVA ONAN
	220/220/440	MVA ONAF
• Voltage	15/15 - 230	kV
• Shipping mass, each	250	tonnes

7.3.6 Flood Handling Facilities

7.3.6.1 *Main Spillway*

• Width out to out of end piers	57.25	m
• Length to end of apron	72.0	m
• Crest elevation of ogee	18.8	m
• Number of fixed wheel gates	3	
• Width of gate	13.75	m
• Height of gate	20.2	m
• Stoplogs	1	set
• Maximum flow capacity (MFL = 44.0 m)	10,800	m^3/s

7.3.6.2 *North RCC Overflow Dam with Rubber Dam Crest (see 7.3.7.3)*

• Maximum overflow capacity (MFL at PMF = 44.0 m)	5,800	m^3/s

7.3.6.3 *South RCC Overflow Dam (see 7.3.7.4)*

• Maximum overflow capacity (MFL at PMF = 44.0 m)	5,500	m^3/s

7.3.7 Dams

7.3.7.1 *Upstream Cofferdam*

- See item 7.3.3.1, above

7.3.7.2 *Downstream Cofferdam*

• Maximum tailwater level at 1:40 flood	6.0	m
• Freeboard	1.0	m
• Crest elevation	7.0	m
• Maximum height	3.5	m
• Crest length	180.0	m
• Total volume	6,200	m^3

7.3.7.3 *North RCC Overflow Dam (North Abutment)*

• Freeboard	0.5	m
• Rubber Dam - crest elevation - inflated	39.5	m
- deflated	37.1	m
• Rubber Dam crest length (net)	167.5	m
• Concrete - top elevation	37.0	m
- length	178.5	m
- total volume	72,000	m^3
• Maximum height	32.0	m
• Bridges: 5 spans @ 36 m	180	m

7.3.7.4 *South RCC Overflow Dam (South Abutment)*

• Freeboard	1.0	m
• Crest elevation	40.0	m
• Maximum height	29.0	m
• Length	370.0	m
• Overflow crest length (net)	326.0	m
• Total volume	69,000	m^3
• Bridges: 10 spans @ 34 m	340	m

SECTION 8.0

RIVER DIVERSION

SECTION 8.0

RIVER DIVERSION

8.0 RIVER DIVERSION

8.1 GENERAL

During construction of the power complex structures the river flow will be diverted by an upstream cofferdam, located just above the upper falls, into two diversion tunnels through the rock knoll, as shown on Plate 12. The flow will be returned to the river, downstream of the lower falls, from the outlet portals of the tunnels. The inlet structure of the diversion tunnels will be gated to permit regulation of flow through the tunnels in order to raise the upstream water level for the purpose of winter ice control. A downstream cofferdam at the crest of the lower falls will prevent flooding of the main construction area from downstream.

The diversion facilities, cofferdam and diversion tunnels were designed to handle a 1 in 40 year flood of 5,300 m^3/s, assuming flow control would be provided by Churchill Falls Generating Station. The dimensions of the diversion works, the tunnel size and the crest elevation of the cofferdam were determined on the basis of an economic optimization - giving a tunnel size of 14.0 m wide x 17.0 m high and a maximum diversion flood level of 30.5 m (cofferdam crest = 31.5 m).

In the event flood control would not be provided at Churchill Falls, these facilities would have to be sized to handle a flood of 8,800 m^3/s. It was estimated this change would result in an increase of about $16 M in the cost of the diversion facilities.

Each inlet structure will have two gates 6.6 m wide x 17.0 m high, which are the same size as used at Gull Island. This provides the opportunity to utilize the Gull Island diversion gates, if it is feasible to retrieve them after closure of the Gull Island Diversion Tunnels.

The following sections of this chapter present general information on the site conditions for the diversion structures, ie. flood levels and ice control required during the construction, as well as descriptions of the individual structures.

8.2 SITE INFORMATION

8.2.1 Field Explorations

Details of the subsurface and foundation conditions in the upstream cofferdam area are based on five boreholes V1, V2 and V9 on the south bank, and V3 and V4 on the north bank and seismic line SL-12 completed during the 1979 field investigation as shown on Plate 7.

The proposed location for the diversion inlet structure, on the west side of the rock knoll, was investigated during the 1979 and 1998 field investigations. Boreholes V3, V4 completed in 1979, and M1 in 1998, were located close to the structure. This area was also covered by five seismic refraction lines, SL-12-79, SL-16-79, SL-17-79, SL-18-79 and SL-1-98.

Boreholes M2 and M3, and seismic lines SL-2 and SL-3 from the 1998 investigation are located along or near the axis of the proposed diversion tunnels. Part of seismic line SL-1 (1998

investigation) and the entire line SL-16 from the 1979 investigation were located to provide detail in the area of the proposed inlet structures.

The original location for the diversion outlet was extensively investigated during the 1979 field investigations. Boreholes V5, V6, V7, and V8, and seismic lines SL-15, 19 and 20 cover more than 10,000 m^2 of the area at the southeast extremity of the rock knoll. Borehole M4 and seismic line SL-4 of the 1998 investigation are located at the northeast and the east side of the knoll. This new investigation zone was investigated to find the most appropriate and economical site for excavation of the outlet.

The subsurface conditions in the area of the main project features near the lower falls have been investigated by two boreholes (C5 and C6) and two seismic line; SL-14 completed during the 1979 investigation and SL-5 completed in 1998.

Geological mapping and air photo interpretation were performed for the entire project area.

8.2.2 Geology

The rock knoll has a thin blanket of overburden along the proposed tunnel alignments. Overburden generally consists of 1 to 4 m of sand and gravel with varying amounts of fines, cobbles, and boulders. Some large talus blocks occur at the outlet portal area and on the south side of the knoll in the vicinity of the proposed access road.

Bedrock is exposed at the crest of the hill over the mid-portion of the tunnel alignments and sporadically near the tunnel inlet. Bedrock outcrop is nearly continuous at the base of the hill at the water's edge. Bedrock consists mainly of Precambrian granitoid gneiss with amphibolite gneiss and pegmatite present locally.

8.2.2.1 *Upstream Cofferdam*

Results from these investigations show that the existing foundation conditions vary on both river banks and over the length of the proposed dam. At the north end of the cofferdam (left bank) good quality granite gneiss, overlain by 0 to 2 m of glaciofluvial soil, was encountered. At the shoreline (left bank) a mass of amphibolite was found. Proceeding south from the falls along the right bank, the ground surface rises in four separate fluvial terraces with each terrace step a few metres higher than the other as shown on Plate 8. The surface of these terraces comprises washed sand and gravel. At borehole V1, 2.8 m of stratified sand and silt overlays 5 m of silty sand and gravel with some cobbles and boulders (till like material). At borehole V9, 2 m of fine silty sand overlays 7.2 m of fine to coarse sand with traces of silt and some gravel and cobbles (till like material). Both boreholes recovered poor to excellent quality granite gneiss. In outcrop, near the shoreline, the bedrock is well foliated granite gneiss striking east-west and dipping in a southerly direction 25° to 55°.

In the riverbed, due to the strong velocity of water in this area, overburden cannot be accumulated but the presence of large boulders is possible. Bedrock velocities vary from 4,700 to 5,700 m/sec, with a narrow, low velocity zone of 2,700 m/sec, near the south-centre area of

the river. This anomaly in the bedrock was interpreted to be a weak rock zone or sediment filled gulley. Borehole V2 shows the presence of numerous small shear zones which could indicate a possible wide fault running in the mid-river in a southeast/northwest direction. The rock from this borehole is a well foliated granite gneiss, approaching amphibolite mineralogy, with alternating bands of amphibolite and pegmatite. The foliation is inclined at approximately 60° to the core axis with variations between 40° and 70°. The joint spacing is moderately close and their surfaces are generally weathered. The hydraulic conductivity is generally low (under 5×10^{-5} cm/sec); hydraulic conductivities of about 1×10^{-4} cm/sec were recorded at depth between 25.8 m to 28.9 m. These higher permeability's generally occur in the vicinity of shears. The shears are, in general, brecciated, well-sealed and moderately weathered.

Before final closure of the diversion tunnels the crest of the cofferdam will be lowered to elevation 26.0 m to reduce local flow velocities at the river section to less than 0.6 m/s.

8.2.2.2 *Diversion Inlet Structure*

At the proposed inlet channel location the overburden thicknesses, based on seismic surveys, varies from 0 to 8 m but is about 1.0 m at the portal location. The overburden is composed of glaciofluvial soil comprising cobbles and boulders with a sand and gravel matrix overlain by organics. The rock has a seismic velocity near the surface between 4,000 to 5,800 m/s. A small zone of low seismic velocity (3,400 and 3,500 m/s) was found, during the seismic refraction survey in 1979, at the intersection of lines 12, 16 and 18. Boreholes V3 and V4 were drilled to intercept this anomalous zone and one located close to the inlet structure locations.

At borehole V3, the rock is exposed at the ground surface. The overburden thickness at V4 is 1.37 m and at M-1-98 it is 0.61 m. Bedrock in this area was generally encountered near the surface, except to the north, near the spur, where it dips steeply. Rock at this location is of fair to excellent quality, well foliated, granite gneiss with some pegmatite and amphibolite. The RQD is generally over 75% except for occasional thin layers of low RQD encountered in the rock mass. Borehole V3 was drilled to intercept the seismic anomaly noted previously. The RQD in the first 30 m of the borehole is under 50%. This zone is affected by the presence of an amphibolite band between 20 to 26 m depth and associated with brecciated shear zones. Joint spacing varies in this rock from very wide to close with occasional narrow (< 0.1 m) shattered zones.

8.2.2.3 *Diversion Tunnels*

Based on the 1979 and 1998 geophysical survey, 1965 to 1998 geological mapping and the 1998 boreholes, the rock knoll has a thin blanket of overburden along the proposed tunnel alignments. Overburden generally consists of 1 to 3 m of boulders and cobbles with sand, gravel and fines matrix overlain by organic materials. Bedrock is exposed at the hill crest, over the mid-portion of the tunnel alignments, sporadically along seismic line SL-1 and near the tunnel inlet and outlet areas. The bedrock consists mainly of granite gneiss with amphibolite gneiss and minor pegmatite present locally. Bedrock along the tunnel alignments appears to be generally of good to excellent quality although some zones of fair to poor quality reflect narrow shear zones.

At boreholes M2 and M3, 1.50 m of organic, black topsoil, cobbles and boulders with sand and gravel matrix overlays 1.00 m to 1.50 m of glacial till containing sand and gravel with sub-angular to angular boulders, densely packed, are found. Bedrock was encountered at a vertical depth at 2.81 m.

Rock at this location is fair to excellent quality with some thin zones (< 1 m) with poor RQD. This rock is well foliated (45° to 70°), granite gneiss alternating with amphibolite gneiss and minor pegmatite. Joint spacing parallel to the foliation at borehole M2 is moderately close to extremely wide. The joint spacing parallel to the foliation at borehole M3 is generally close to very wide with occasional narrow shattered zones. Joint spacing perpendicular to the foliation is very close to wide.

8.2.2.4 *Diversion Outlet Portal*

At the Diversion Outlet Portal bedrock outcrop is generally observed between the waters edge to about elevation 15 metres. Overburden exists above this to the base of the cliffs. Overburden is variable. Near the cliff at the east side of the knoll, local accumulation of talus is present. The thickness of this accumulation could be greater than the 2 to 3 m indicated on seismic line SL-4 presented in the Refraction Seismic Report (Appendix C of Volume 2). The size of blocks can vary from 0.5 m^3 to more than 5 m^3. The maximum overburden thickness was observed at the southeast flank of the rock knoll in borehole M4. 3.06 m of overburden, comprising a thin surficial layer of moss, resting on fluvial soil consisting of cobbles and small angular boulders with a sand and gravel matrix overlays the bedrock. At this location bedrock consists of amphibolite gneiss with granite gneiss and rare pegmatite, generally of fair to excellent quality. The joint spacing is close when parallel to the foliation and close to wide when perpendicular to the foliation.

The rock intercepted at borehole V5, V6 and V7 (southeast side of the knoll) consists mainly of granite gneiss with occasional amphibolite gneiss and pegmatite bands. The gneiss is well foliated and generally fair to excellent quality except the fault zone intercepted in boreholes V5 and V7. At borehole V7 granite gneiss is present in the top half of the borehole with amphibolite gneiss rock in the bottom half. The foliation is inclined at approximately 50° to the core axis in the gneiss and 30° to 40° in the amphibolite. Joint spacing is moderately close throughout the rock mass with an occasional zone where it is very close to extremely close. The overall hydraulic conductivity of the rock varies, ranging from 1×10^{-3} to 1×10^{-5} cm/sec.

8.2.2.5 *Downstream Cofferdam*

The river channel is underlain by generally good to high quality bedrock. Occasional faults cut the area, generally with an east-west strike and near vertical dip. Most faults are in hard rock and are healed or reconsolidated with good hard rock contacts.

8.2.3 Geological and Geotechnical Design Considerations

Bedrock is generally fresh Precambrian granite gneiss with amphibolite gneiss bands and pegmatite stringers. Foliation is well developed, striking east-west and dipping at 30° to 60° to the south. Rock quality is generally good to excellent but zones of poor quality, fractured rock, occur locally due to the shear zones and minor faults which, when intersected in the tunnels and surface structural excavations, will necessitate treatment with rock support. Three joint sets exist which in conjunction with shear zones and faults, will form wedges requiring support. The geological mapping shows some minor faults, and one major fault associated with an aplite dyke trending dominantly east-west parallel with the river and dipping either sub-vertically or parallel with the foliation. These parameters need to be considered for the geotechnical design. Other considerations and criteria to be taken into account are listed below:

- the criteria adopted for locating the inlet and outlet portals are that the bedrock in the portal area must be sound and the thickness of the rock above the tunnel crown must be a minimum of 1.25 x tunnel width at upstream portal (with extra rock support included in the capital cost estimate) and 1.5 x tunnel width at downstream portal;

- geotechnical considerations are the main basis for the design shape of the tunnels. The tunnel shape was selected:

  a) to minimize the tensile stresses in the tunnel walls and roof,
  b) to minimize rock reinforcement by using the inherent strength characteristics of the rock mass,
  c) to minimize detrimental influence of the stress field in the central pillar, and
  d) to accommodate construction;

- the minimum required width of the central pillar to attain a stable tunnel configuration was determined by assuming possible rock wedge failures on either side of a central pillar along the rock structural discontinuities and then analyzing the stability of the remaining central pillar;

- rock quality along the diversion tunnels was evaluated from drilling and geological mapping data and rock quality established using the NGI Q system (Barton and al, 1974, reference 9) and the geomechanics (RMR) system (Bienawski, 1974, reference 10) of rock classification. Rock support requirements will be designed based on rock quality class, with provision for modification as necessary during the tunneling. Due to the large span of the tunnels, crown pattern bolting and shotcrete support will be necessary. Side wall support will range from spot bolts to pattern bolting depending on rock quality. Potentially erodable gouge and shear zones will be treated with shotcrete (EPRI, 1987, Reference 32);

- on the basis of the known rock structure, rock cuts undercutting the foliation will require support with pattern, tensioned rockbolts, shotcrete and drains, or relatively flat benched slopes;

- blasting will be required to remove the rock to the required dimensions. Excavations in rock can be carried out using controlled blasting techniques. Pre-shearing, cushion blasting, line drilling and other techniques should be used to eliminate damage to the rock walls and the crown of the excavations;

- joint spacing in the rock outcrops is close to moderately close with most joints striking east-west and dipping towards the south. Most joints are rough with hard rock contacts. There are occasional open joints. Major rock slope instabilities are not expected in this area. However, the north walls of east-west trending excavations will expose the major joint system, i.e. they will daylight into the excavation. The blasting procedures will most likely open up these joints, especially near the top of excavations;

- narrow, east-west trending faults and sheared rock exist in the area. Several were identified on the shoreline of the river and in previous drilling. It is possible that one or more of these features will be encountered. These items should be examined at that time and, if necessary, decisions for excavating and rock support or enhancing with other structural means will be made;

- for the upstream cofferdam, remove all organic materials from within the footprint of the cofferdam on both banks and dispose of this material in disposal areas as shown on Plate 24;

- armour the right bank with rockfill prior to closure to minimize overburden erosion during closure by construction of at least 10 m of the closure groins from the right bank; and

- the south abutment of the cofferdam will be constructed on soil identified as Terrace 3. This flat topped erosional morphological feature suggests that it has a soil core that was easily erodable by the Churchill River during periods of higher river levels. Soil conditions expected are thin fluvial sands overlying a till-like soil consisting of sand and gravel with some fines, some cobbles and small boulders. The depth of these soils is not known at this location. The nearby seismic line SL-12 shows the bedrock profile rising to about El. 25 m at station 4 + 250 (30 m from water's edge), but dipping to about 21 m at station 4 + 270 (50 m from the water's edge), and not rising to El. 32 m until about station 4 + 400 (180 m from water's edge). Details of the abutment contact will require further investigations for design.

8.3 DIVERSION HYDRAULICS AND ICE CONTROL

8.3.1 Planned Diversion Sequences

The project construction schedule calls for river closure in late July of Year 2. Mean river flow in July/August is about 2,000 m^3/s according to the Energy Study (Reference 54), but for the purposes of this study a closure flow of 2,000 m^3/s is recommended for design. Notwithstanding this design approach, the actual closure can be substantially simplified if power plant discharges from upstream can be reduced during the critical four-to-five days required for

closure of the cofferdam. Construction for the Diversion Facilities are on the critical path. For details on the construction schedule of these facilities, refer to Section 15.2.2.

8.3.2. Flows and Water Levels

Figure 8.1 shows preliminary flow rating curves for both the river and diversion tunnels. These curves illustrate how head-flow relationships vary as the cofferdam is constructed (left panel) and also show head-flow relationships for the diversion tunnel (right panel).

For closure against a flow of 2,000 m^3/s the head difference across the gap would be in the order of 4 m with regulation and closure against a flow of 1,000 m^3/s, the head drop across the groins would reduce to 3 m and rockfill erosion losses would reduce.

A hydraulic model study is necessary to reliably determine river water level flow relationships for final hydraulic design of cofferdam and diversion facilities.

8.3.3 Ice Control

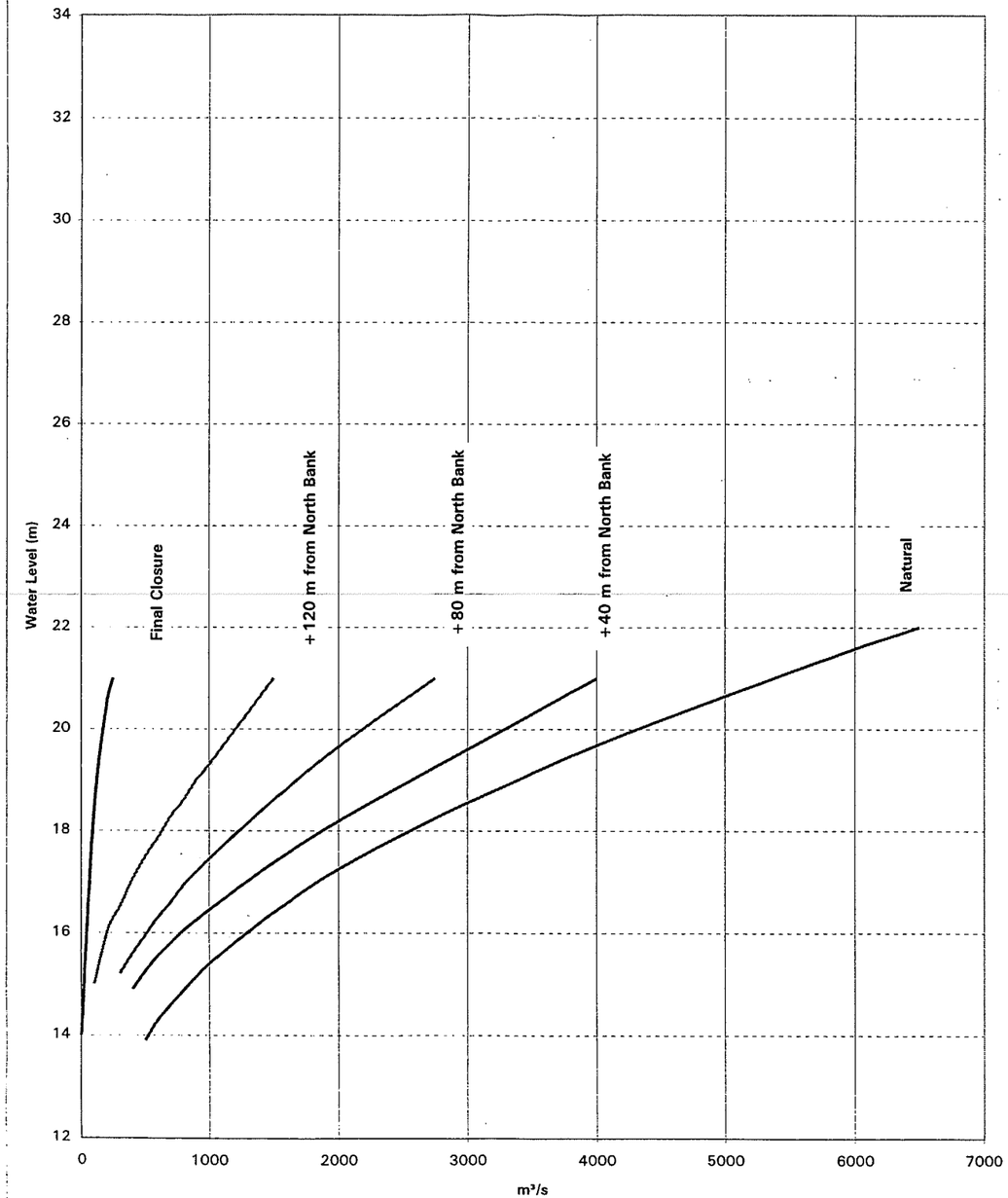
During the construction of the Muskrat Falls Development, the formation and the maintenance of an ice cover upstream of the site must be ensured throughout the two winter seasons of the construction period, without creating any operating constraint on the Churchill Falls Powerplant. In order to guarantee the formation of a stable ice cover, the water level upstream of the diversion has to be maintained at or above El. 24.0 m, by regulation of the discharge through the diversion tunnels, as recommended by LaSalle Consulting Group Inc. (Appendix F, Reference 53).

It is to be noted that a complete shutdown of the Churchill Falls Powerplant is also possible during the period of diversion in which case closure of the diversion may be required.

Because of the above requirement, the use of gates at the upstream portal of the diversion tunnels will be necessary. For the normal expected river flow range (1,500 to 2,500 m^3/s) during the winter periods, one tunnel with two gates fully open will be required at all times to maintain the water level at El. 24.0 m.

For flows in excess of 2,000 m^3/s, a third gate in the other tunnel will have to be opened which will permit water level control for flows up to 2,500 m^3/s, the maximum winter flow expected. Where there is an unbalance between river flow and gate discharge capacity, the correct level will be maintained by fully opening or closing one gate.

It is expected that two cycles a day (two openings and two closings) will be sufficient to control the water level within a range 24.0 m $\pm$ 0.15 m. This will require that the water level during winter operation be carefully monitored so the operation of the gates can be performed when required (installation of a simple on/off water level control system should be investigated in final design). This mode of operation is recommended to avoid the risks of creating flow induced vibrations which can occur during operation with partially open gates.


Muskrat Falls
Rating Curves
Flow in River Channel
Water Level (m)
34
32
30
28
26
24
22
20
18
16
14
12
0
1000
2000
3000
4000
5000
6000
7000
m³/s
Final Closure
+120 m from North Bank
+80 m from North Bank
+40 m from North Bank
Natural

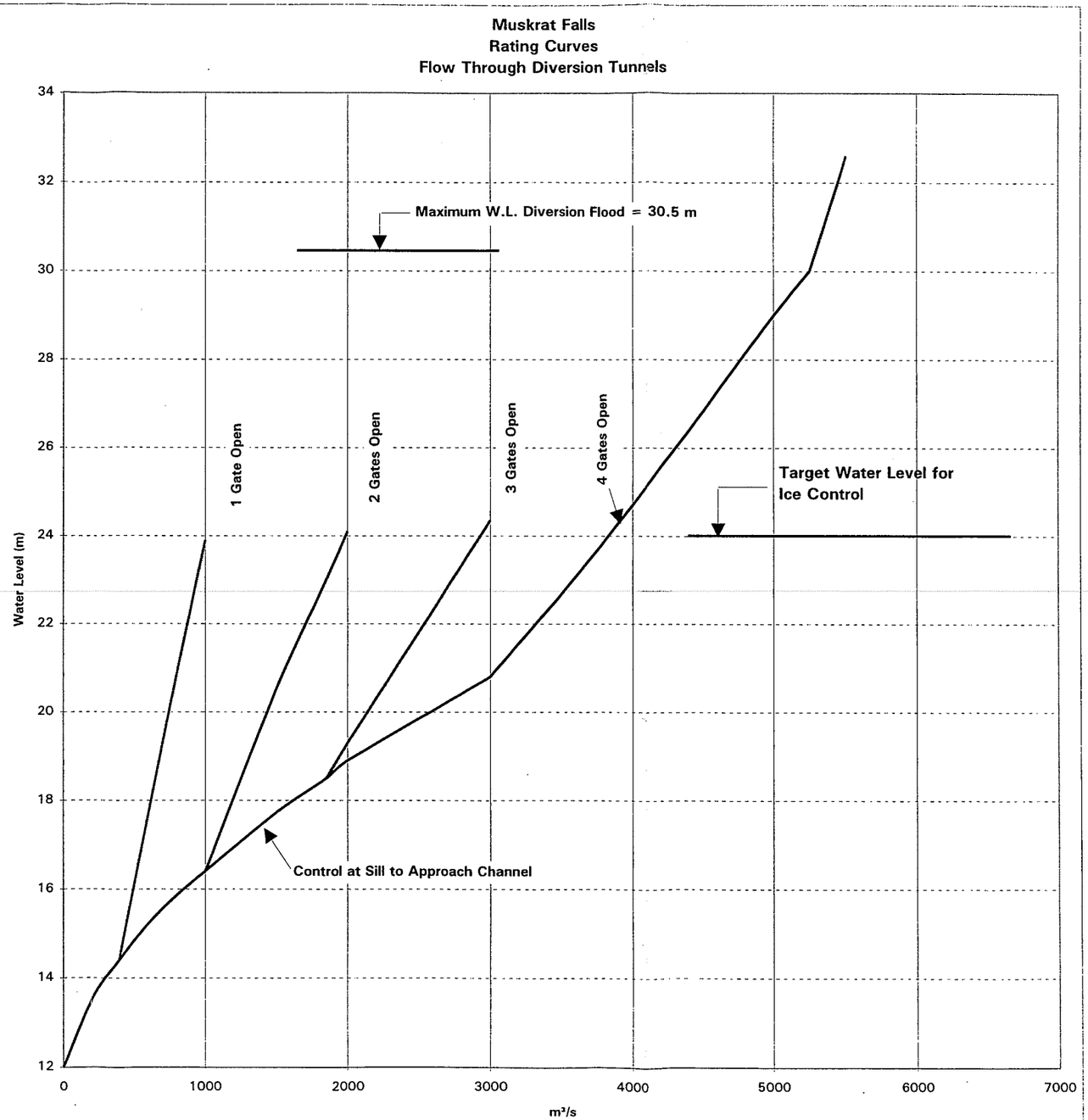

Muskrat Falls
Rating Curves
Flow Through Diversion Tunnels
Water Level (m)
34
32
30
28
26
24
22
20
18
16
14
12
0
1000
2000
3000
4000
5000
6000
7000
m³/s
Maximum W.L. Diversion Flood = 30.5 m
1 Gate Open
2 Gates Open
3 Gates Open
4 Gates Open
Target Water Level for
Ice Control
Control at Sill to Approach Channel

Figure 8.1

8.4 HYDRAULIC DESIGN CONSIDERATIONS

The following are the significant design considerations for the diversion structures. Salient data for the diversion structures is presented in Section 7.3 and design criteria are provided in Appendix B.

- Design flood will be the 1 in 40 year flood flow (5,300 m^3/s).
- Tunnel design is based on pressurized flow.
- Four service gates (two per tunnel) are provided with no bulkhead gates or stoplogs.
- Gates will be of vertical lift, fixed wheel design.
- Gates will be heated and will be operated fully open or fully closed (no throttling of flow).

8.5 DIVERSION STRUCTURES

As previously noted, sizing of the cofferdam and tunnel dimensions were determined by economic optimization. The diversion structures include an upstream cofferdam, approach channel, inlet structures, diversion tunnels, outlet portals and a downstream cofferdam. These structures are described in the following Sections 8.5.1 to 8.5.6.

8.5.1 Upstream Cofferdam

The upstream cofferdam is located immediately upstream of the upper falls as shown on Plate 12. The first stage cofferdam consists of two rockfill closure groins with crest elevations 20 m upstream and 19 m downstream. Rockfill groins will be constructed from the south bank prior to closure to armour the south bank overburden against erosion during closure. Following closure, granular transition and impervious till will be dumped in the wet between the closure groins to seal the cofferdam. Closure by two rockfill groins will limit the differential head across each groin to about 2 m and minimize rockfill erosion losses.

Following diversion and sealing of the Stage 1 cofferdam, the cofferdam will be raised to the Stage 2 crest level (El. 31.5 m) as required to pass the spring flood, with compacted rockfill, granular transition, and till upstream, with the upstream shell sloped at 2.5H:1V for drawdown stability. Erosion protection for the upstream face is provided by 1 m of dumped rockfill in view of an anticipated water velocity along the face of the cofferdam during passage of the maximum diversion flood, in the order of 3 m/s.

The cofferdam is designed for river closure in the summer and will be topped-up to Stage 2 crest level (El. 31.5 m) by mid September of Year 2.

On the abutments, organic surface material will be stripped from the cofferdam footprint. Class 2 granular fill will be used in the downstream shell of the Stage 2 cofferdam in the south abutment to provide a filtered exit for seepage through any pervious strata in the overburden forming the abutment.

Before final closure of the diversion tunnels, the upstream cofferdam will be lowered to El. 26.0 m, so as to limit local flow velocities in this area to less than 0.6 m/s during plant operation.

8.5.2 Approach Channel

The configuration of the approach channel has been based on the required hydraulic performance. Near the river the invert level of the approach channel is at El.12.0 m, then it slopes downward to El. 3.5 m just upstream of the inlet structure as shown on Plate 13.

8.5.3 Inlet Structures

A separate inlet structures is provided for each tunnel. Each inlet has twin gates and a centre pier. The general dimensions of the structure are illustrated on Plate 13 and the proposed mechanical and electrical equipment are described in Sections 8.5.7 and 8.5.8.

Completion of the structure with gates and hoists is necessary prior to diversion. The gates of both tunnels will be operated in winter to maintain the upstream water levels at El. 24 m as required for ice control.

8.5.4 Diversion Tunnels

The two proposed unlined tunnels, 17.0 m high x 14.0 m wide inverted U shape will have a length of approximately 600 m and will pass the diversion flood peak of 5,300 m^3/s at a maximum velocity of 12.0 m/s. The tunnel location and alignment were selected on the basis of hydraulic, geological and geotechnical criteria. The geological and geotechnical design considerations have been discussed in Sections 8.2.3.

Concrete lining is provided for a length of about 15 m downstream of the inlet for rock support and as protection against high velocity flows.

A construction adit, 10.0 m high and 11.0 m wide inverted U shape, will have a length of approximately 390 m. The adit will be driven from the south side of the rock knoll to provide access for excavation of the two diversion tunnels in two directions.

Upon completion of the diversion tunnels excavation, concrete plugs will be installed in the adit at each side of the diversion tunnels to reduce hydraulic turbulence in the tunnels during diversion.

For final closure of the diversion tunnels, a concrete plug will be installed in each tunnel. The plugs will be installed immediately downstream of the inlet gates, as shown on Plate 13, where the tunnel invert elevation is above -3 m. Access for installation of the plugs will be via the inlet structure vent shafts. The length of the plugs will be confirmed during final design but, for this report, is based on one half of the tunnel width.

A plan and profile of the tunnels are shown on Plate 12. The final location of the adit and the tunnel plugs will be optimized during the final design.

8.5.5 Outlet Portals

The diversion outlet is located on the east side of the rock knoll as shown on Plates 12 and 13.

The outlet is an open excavation comprising two tunnel outlet portals and a downstream channel. The outlet portals are separated by a rock pillar 32 m wide. The portals are each 17.0 m high x 14.0 m wide.

A rock plug is provided in the downstream channel to allow construction of the outlet in the dry.

8.5.6 Downstream Cofferdam

The downstream cofferdam is located along the top of the lower falls as shown on Plate 12. It is a low compacted rockfill/till berm which will be used for access along the downstream side of the main construction area. With a crest elevation of 7.0 m, it will prevent the main construction area from flooding, from downstream. The cross-section of the cofferdam, with a crest length of 180 m, at its crest elevation of 7.0 m, is shown on Plate 12.

8.5.7 Mechanical Equipment

Refer to Sections 2.0 and 6.0 of the Design Criteria presented in Appendix B for the dimensional and selection parameters of the mechanical equipment for the diversion structures.

Each of the two diversion tunnels has an inlet portal with two openings equipped with fixed wheel vertical lift gates. The gates have downstream seals. These gates are used for water level control during winter operation of the diversion scheme and also for final closure of the diversion tunnel after which they will not be removed. The gates are fabricated in sections and assembled at site.

Because of the winter operation of the gates, they are all heated and provided with heated guides. Also, in order to guarantee that all gates can be operated individually at any time, all four gates are equipped with individual cable hoists installed in temporary enclosures. The gates are designed to discharge the construction flood of 5,300 m^3/s and to operate under the design flood level of 30.5 m.

The flow anticipated at the time of closure ranges between 1,500 and 2,500 m^3/s. The embedded parts of the diversion gates are fitted with a compressed air piping system for flushing any obstruction from the gain and sill areas before lowering the gates. No other mechanical auxiliaries are required.

Since the diversion gates will not be used for flow throttling, hydraulic model testing of the gates will not be necessary.

After final closure, the upstream reservoir will fill to the maximum operating level within one week and, for this reason, the gates are designed for the full reservoir static head.

The approximate mass of each gate and each set of embedded parts is 227 t and 31 t respectively. Each hoist has a mass of 18,000 kg.

8.5.8 Electrical Power Requirements

Power is required at the diversion inlet structures during construction and throughout the diversion period of approximately two years. Power is supplied from the nearby 25 kV distribution line with suitable transformation to 600/347 V. Distribution is made at 600 V to an MCC panel for the hoist motors and for heating of the gates and gate guides during winter. A 120/208 V supply is also provided for lighting and general services.

SECTION 9.0

DAMS AND SPILLWAY

SECTION 9.0

DAMS AND SPILLWAY

9.0 DAMS AND SPILLWAY

9.1 GENERAL

The dams and spillway, together with the integrated intake and powerhouse, are located in the river channel, upstream of the lower falls, to create a full closure of the river valley and the reservoir for the Development. The spillway and the intake/powerhouse are adjacent structures and are located in the river channel. The north dam closes to the north side of the valley and the south dam closes to the south side of the valley.

As previously noted (Section 7.2), the north and south dams are designed for overtopping at high water levels. These structures, combined with the gated spillway, provide the flood handling facilities of the Project. The north dam includes a rubber dam crest which, when deflated, increases its flood discharge capacity. The south dam includes a short, freeboard dyke with crest at El. 45.5 m at its southern extremity.

Dambreak analysis, inundation mapping and flood contingency plans must be prepared during final design to meet requirements of the Department of Environmental and Labour.

9.2 FOUNDATION CONDITIONS

9.2.1 Field Exploration

Boreholes V5, V6, V7, V8, and C6 from the 1979 investigations are located approximately 40 to 100 m east of the north dam. Seismic line SL-15 passes just north of the dam. SL-19 passes just west of the structure. The north end of seismic line SL-13 is located just west of the dam. The south part of Line SL-20 passes through the central part of the dam. Seismic line SL-5 from the 1998 investigations passes through the dam. Air photo interpretation was performed in the area.

Borehole M-5 from the 1998 investigation is located 25 m from the downstream toe of the south dam at its mid-point. Parts of seismic lines SL-5 and SL-8 are located within the area of the proposed dam. Borehole C7 and seismic line SL-6 from 1979 are located approximately 100 m downstream.

The subsurface conditions in the spillway area are based on two boreholes (C5 and C6) and one seismic line (SL-14) completed during the 1979 investigation and a seismic line (SL-5) completed in 1998.

To obtain bathymetric and additional subgrade details in the riverbed areas where high water velocities had restricted earlier investigations, a ground probing radar (GPR) survey was carried out with coverage in both directions at approximately 10 m intervals over the area from upstream of the upper falls to downstream of the lower falls.

9.2.2 Geology

9.2.2.1 *North RCC Dam*

The site investigations show that bedrock in this area is amphibolite gneiss and granite gneiss with relatively thin soil cover. The exposed shoreline consists of 50% bare rock and 50% small boulder pavement with a sand and gravel matrix. Continuing north into the hillside, marine sediments, comprised of silt and clay, cover the rock surface to depths of approximately 6 to 16 m in a local area at the southeast side of the knoll. Silty sand and gravel till with cobbles and boulders may comprise part of the deeper overburden. Depth of overburden along the dam axis is expected to range from 0 at the south end, to 10 m at midpoint, to 6 m at the north abutment.

9.2.2.2 *South RCC Dam*

Investigations in this area show that the existing foundation materials are varied over the length of the structure. At the north end of the dam generally good quality granite gneiss is exposed at the river shoreline. Proceeding south, the ground surface rises in five separate fluvial terraces along the axis of the dam with each terrace step 8 to 12 m high. The surface of these terraces is composed of washed sand and gravel, however, dense, gray glacial till or stratified marine sediments will be encountered within a few metres depth. Thin bog deposits have developed in flat and low lying areas especially at the bases of the stepped terraces and areas where the glacial till and/or bedrock rises close to the ground surface. Depth of overburden is expected to average 4 m along the axis of the dam, varying from 0 to 10 m.

At borehole M-5, 0.73 m of bog overlays 0.78 m of glaciofluvial sand and gravel with cobbles and small boulders which in turn overlays sand and gravel glacial till with some silt, cobbles and small boulders. Bedrock was encountered at a vertical depth of 5.43 m. Rock at this location comprises excellent quality, well-foliated, amphibolite gneiss. Joint spacing is close to moderately close throughout the rock with occasional narrow (< 0.2 m) shattered zones.

9.2.2.3 *Spillway*

The river channel in the spillway area is underlain by generally good to excellent quality granite gneiss and amphibolite gneiss bedrock. Little or no overburden is anticipated. Occasional faults cut the area, generally with an east-west strike and near vertical dip. Most faults are in hard rock and are healed or reconsolidated with, hard rock-to-rock contacts.

9.2.3 Geological and Geotechnical Design Considerations

9.2.3.1 *North and South RCC Dams*

The exact quantity and distribution of the various classes of natural soil and bedrock were not determined during the investigations, however, based on observations of these materials elsewhere on the site, the following procedures are recommended:

- remove all soil and loose materials overlying the bedrock to approximately 3 m outside the footprint of the dams;

- design the foundations, resting on bedrock, for an allowable bearing capacity of 3,000 kPa. The near surface weathered rock usually has a higher hydraulic conductivity and more open fractures than deeper, sound rock. A grout curtain is necessary to reduce and control foundation seepage in this area; and

- narrow, east-west trending faults and sheared rock conditions exist in the area. Several were identified along the shorelines of the river and in previous drilling. It is possible that one or more of these features will be encountered. These items will be examined during foundation cleanup and geologic mapping and, if necessary, decisions for grouting, excavating, backfilling with dental concrete or treatment with other means will be made at that time, as per normal foundation preparation procedures.

9.2.3.2 *Spillway*

The following recommendations are proposed:

- narrow, east-west trending faults and sheared rock exist in the area. There exists the likelihood that one or more of these features will be encountered in the spillway foundation area. These features should be examined during foundation cleanup and geologic mapping and, if necessary, decisions for grouting, excavating, backfilling with dental concrete or treatment with other means will be made at that time, as per normal foundation preparation procedures; and

- excavations in rock can be carried out using controlled blasting techniques. Pre-shearing, cushion blasting, line drilling and other techniques should be used to eliminate damage to the rock walls of the excavations.

9.3 NORTH RCC DAM

The north RCC dam is a concrete gravity overflow structure founded on rock which extends from the spillway to the north abutment. This dam will provide additional spill capacity to pass floods greater than the gated service spillway capacity. Maximum height of the dam is about 32 m above foundation level. The crest length of the overflow section is 178.5 m and there is a 20 m long non-overflow section at the north abutment as shown on Plate 14.

A steel truss bridge 6.3 m wide, with deck level El. 45.5 m and piers at 36.0 m centre-to-centre spacing provides access across the dam to the spillway. Five 2.4 m high rubber dam sections are provided between the bridge piers. These will normally be inflated to El. 39.5 m and can be deflated to El. 37.1 m to increase spill capacity to pass a major flood.

The dam is a composite concrete section, with a roller compacted concrete (RCC) interior of 15 MPa concrete, with upstream and downstream facings of 0.5 and 1.5 m thick respectively, of conventional 25 MPa air-entrained concrete for better freeze-thaw durability. The upstream

facing is reinforced and has vertical, waterstopped, contraction joints at 12 m spacing which are drained by half-round drains to the drainage gallery. Although the interior RCC dam is of low permeability, this upstream face provides the primary seepage control barrier. A 2.5 m x 2.2 m grouting and drainage gallery is provided which extends about 2/3 of the length of the structure to allow foundation treatment (grouting and pressure relief) and monitoring of long-term performance.

Foundation treatment follows conventional practice for a concrete gravity dam with a single-line grout curtain to a depth in rock equivalent to H/2 (8 m minimum), where H is the hydrostatic head, installed from the grouting gallery along with a curtain of drainholes. The downstream slope of the dam is sloped at a relatively conservative 0.75 H:1V to provide adequate stability with the reservoir surcharged to pass the PMF. Overburden downstream of the dam is stripped to the bedrock surface to minimize erosion when spillage occurs over this dam. Unsuitable overburden material will be disposed of in a disposal area on the north bank, as shown on Plate 24.

9.4 SOUTH RCC DAM

The south RCC dam is a concrete gravity overflow structure founded on rock. The dam extends from the service bay of the powerhouse to the south abutment and is designed to pass part of the PMF. Maximum height of the dam is about 29 m above foundation level. The dam has a length of 370 m with a net overflow section length of 326 m and crest at El. 40.0 m. A short non-overflow free-board dyke section described below, connects the south RCC dam to the south abutment as shown in Plate 14.

The south dam has a conventional concrete ogee crest at El. 40.0 m and a composite concrete section with a RCC interior of 15 MPa concrete, and upstream and downstream facings of conventional 25 MPa air-entrained concrete, as described for the north RCC dam in Section 9.3. Foundation treatment is also similar to the North RCC Dam. As the south dam would only be overtopped by floods in excess of the 1,000 year event, overburden downstream from the dam is not removed. A drainage ditch with riprap erosion protection is provided to drain wave-splash to tailwater downstream of the powerhouse. A 50 m long RCC training wall extends from the start of the overflow section to the end of the powerhouse parking lot to protect the powerhouse against flood discharge, in the event that the dam is overtopped. Even with the 1 in 10,000 year flood event, a relatively small volume of water (30 m^3/s) will discharge over the dam. This will be handled by the drainage ditch with minimal erosion of the overburden downstream of the dam.

Unsuitable overburden material will be disposed of in a disposal area on the south bank, as shown on Plate 24.

9.4.1 Design of RCC Dams

A preliminary stability analysis was carried out for the normal and extreme loading cases, following the conventional criteria for gravity structures as summarized in Appendix B. The normal case considers uplift pressure equal to 1/3 of the hydrostatic head under maximum operating level (El. 39.0 m) and ice thrust acting upon the dams. The extreme case considers

uplift pressure to the full hydrostatic head (El. 44.0 m with the reservoir surcharged by 5.0 m), and drains ineffective.

In both cases, the safety factors calculated were somewhat higher than the required criteria when using a shear friction coefficient of 1.0 and a unit shear resistance of 700 kpa. The loading due to the maximum flood level from the PMF, with drains ineffective, was found to be the governing case.

Appendix G contains a more general description of RCC dams and a table of dams constructed world wide between 1980 and 1989.

9.5 FREEBOARD DYKE

The south RCC dam is founded on bedrock to a point near the south abutment where the bedrock surface rises to El. 39 m. Beyond that point a short (30 m long) freeboard dyke section is constructed on overburden to provide a transition between the concrete overflow dam section and the right abutment. The freeboard dyke will have a head of water against it for only short periods when the reservoir is surcharged to pass major floods. It is designed as a conventional embankment dam.

9.5.1 General Description

The dyke cross-section consists of a central core of compacted impervious till, full width of the concrete overflow dam section at its south end, and granular shells upstream and downstream. The dyke will have a crest width of 9.0 m at bridge deck level (El. 45.5 m) for a length of about 30 m to its junction with the powerhouse access road. The shells are sloped at 2.5H:1V upstream and downstream with dumped rockfill erosion protection, as wave action, even during periods of reservoir surcharge, is expected to be relatively minor. The dyke is founded on sound overburden consisting of till, silt and sand. No further cutoff is required due to the limited head that would occur only during periods of reservoir surcharge.

Detailed geotechnical investigations are required in the transition area between the dyke and RCC dams for final layout and design.

9.5.2 Dyke Design

In view of the very limited head and the infrequent loading that would be imposed on this structure, normal design considerations, i.e. stability analysis, seepage control and erosion protection, will be checked relative to the particular foundation conditions which may be presented at the time of construction.

9.6 SPILLWAY

9.6.1 General

The gated spillway structure is located in the river channel above the lower falls, abutting the north RCC overflow dam on the north side and a 10 m thick concrete bulkhead closure to the

intake structure on the south side. The spillway consisting of three gates and a downstream flip bucket is founded on rock. Plate 15 shows the general arrangement and details of the structure.

Releases at the gated spillway will usually occur during spring flood run-off when it will be used in conjunction with releases at other structures. However, diversion of power flows may also be necessary following unit outages to prevent rise in the reservoir level due to lack of storage. For the latter event, the full plant flow of 2,667 m^3/s can be passed through one gate fully opened. In the rare event of all gates failing to open with a plant outage, the full plant flow can be passed over the top of the gates, combined with overtopping of the north and south dams at water level 40.6 m.. The spillway is equipped with three gates with provision for year-round remote operation.

9.6.2 Design Flood Flows

The handling of floods involves three structures:

- the gated spillway;
- the north dam; and
- the south dam.

The design flood criteria, follow the Canadian Dam Safety Association Guidelines, as further explained in Section 6.7.2.

The design flood flows are:

- Normal Operation, 1 in 1,000 year flood

– Q_{1000}	10,100 m^3/s
– Flood Level	40.0 m
– Flow Distribution	
– Gated Spillway	8,450 m^3/s
– North Dam	1,650 m^3/s
– South Dam	0 m^3/s
Total	10,100 m^3/s

- Extreme Condition

– PMF	22,100 m^3/s
– Flood Level	44.0 m
– Flow Distribution	
– Gated Spillway	10,800 m^3/s
– North Dam	5,800 m^3/s
– South Dam	5,500 m^3/s
Total	22,100 m^3/s

The bottom chord of the bridges are set about 1.0 m above the profile of the discharging water (PMF = 22,100 m^3/s) to permit safe passage of floating debris without affecting the safety of the bridges.

9.6.3 Description of the Gated Spillway

The gated spillway comprises three bays, each 13.75 m wide. The spillway crest at El. 18.8 m is a standard ogee shape with a 1H:3V upstream face and with downstream slope terminating in a concrete apron with a curved flip of 20° at its downstream end, as shown on Plate 15. Adjacent bays are separated by a 4 m thick pier. The approximate dimensions of the structure are 57.25 m wide, 72 m long (upstream - downstream) and the overall height, excluding the steel gate support superstructure, is 40.5 m. The required spillway crest length is 41.25 m for a gate height of 20.2 m, and a weir coefficient of 2.10.

A concrete road bridge is located on the upstream side of the gates at El. 45.5 m and, together with bridges over the adjacent structures, will provide a permanent crossing of the Churchill River. A 2.2 m wide by 2.5 m high inspection gallery is provided beneath the rollway for the full width of the structure and will be used for grouting and pressure relief drains.

9.6.4 Stability Analysis

A preliminary stability analysis of the structure has been made for the normal and for the extreme loading cases. The normal case considers uplift pressure equal to 1/3 of the hydrostatic head under maximum operating level (El. 39.0 m) with the gates closed and ice thrust acting upon it. For that case the minimum safety factors required against overturning and sliding are 1.5 (with the resultant within the middle-third) and 3.0, respectively.

The extreme case considers uplift pressure equal to the full hydrostatic head (El. 44.0 m with the reservoir surcharged by 5.0 m and the spillway gates closed). For this extreme case, the minimum safety factor required against overturning and sliding are 1.1 (with the resultant within the middle-third) and 1.0, respectively.

In both cases, the safety factors calculated were somewhat higher than the criteria mentioned above when using a shear friction coefficient of 1.0 and a unit shear resistance of 700 kPa.

Refer to Section 5.0 of the Design Criteria presented in Appendix B for dimensional and loading criteria.

9.6.5 Spillway Gates and Stoplogs

Refer to Sections 2.0 and 6.0 of the Design Criteria presented in Appendix B for the dimensional and selection parameters of the mechanical equipment.

The spillway gates are of the vertical roller type and are operated by individual cable hoists. The dimensions of the gates are 13.75 m wide by 20.2 m high. All three gates, and their guides, are equipped with electric heaters for winter operation and all gates are designed for ice load.

The gates are designed to operate at maximum forebay level under a design head of 25.2 m with allowance in hoist capacity for maximum probable flood level of El. 44.0 m.

Each gate is provided with upstream staunching rod seals and downstream elastomer wind seals.

The embedded guides have stainless steel surfaces for the gate seal and roller paths. Dogging devices are provided for securing the gates in their full open position.

Each gate is provided with a two point lift wire rope hoist. Clutches allow the operation of one side only to facilitate maneuvering in case of gate jamming. The hoists are installed on a common covered hoist bridge mounted on steel towers supported on the concrete piers. The hoists are equipped with local and remote push-button controls, limit switches, local and remote gate position indicators and brakes suitable for positioning the gates over the full range of gate openings.

In emergency, the gates can be operated using a diesel generation power supply.

To allow for inspection and maintenance of the spillway gates and their guides, one set of stoplogs is provided. The stoplogs include synthetic seals and leaf springs to assist the sealing.

The stoplogs are fabricated from steel in sections with a mass of no more than 21,000 kg and are designed for a static head of 20.2 m. Each stoplog section is complete with dogging and lifting lugs for handling by a mobile crane and are designed to be installed under balanced conditions. The top section is equipped with a filling valve. It has been assumed that the mobile crane required for handling the stoplogs can be rented from a supplier in Happy Valley-Goose Bay.

The approximate mass of the equipment is as follows:

Equipment	Number	Mass (each)
Spillway gate	3	193,000 kg
Spillway guides	3 sets	41,000 kg
Stoplogs	1 set (8 logs)	164,000 kg
Stoplog guides	3 sets	23,000 kg
Spillway gate hoists	3	25,000 kg

9.7 RUBBER DAM

The rubber dam comprises five sections, each 33.5 m long, secured to the top of the concrete gravity section of the north RCC dam, between the bridge piers which are 1.5 m thick. The rubber dam is 2.4 m high with a crest elevation of 39.5 m, when inflated. Deflated, the dam will provide a weir crest at El. 37.1 m for discharge of water over the north dam.

The rubber dam may be deflated section by section as required to match rising flood discharge. It can also be used to flush floating debris from the forebay. Control of inflation and deflation is

automatic with manual override. The control system is connected to the plant PLC system for local or remote operation.

The control system will be connected to the emergency diesel backup power supply. In the rare event of a total power failure, including the emergency power supply, it will take many days for the rubber dam to deflate, since the dam has a check valve type closure system.

The design life of the rubber dam is approximately 25 years. The replacement cost, after 25 years, is included in the operation and maintenance (O&M) costs.

The spill capacity of the rubber dam, fully deflated, and with the reservoir at El. 40.0 m, is 1,650 m^3/s. With the reservoir at El. 44.0 m, the capacity is 5,800 m^3/s.

The rubber dam is constructed of heavy duty, reinforced and UV and ozone resistant Ethylene Propylene Diene Monomer EPDM rubber, with a minimum thickness of 12.4 mm. It is secured to the concrete dam along its upstream side with a continuous galvanized clamp and anchor bolt system designed with a factor of safety against pull-out ≥ 3.0.

Inflation and deflation of the dam is controlled automatically. Inflation is provided by two electrically powered air blowers (one on stand-by), each designed for 100% of the required capacity. Deflation is controlled by a bucket-operated air exhaust valve, designed for a multiple span installation. The mechanical equipment is located in an enclosure near the south end of the dam and is connected to each section of the dam with stainless steel piping embedded in the dam.

Deflation of the dam will be automatically controlled by the upstream water level at preset trip levels. The deflation will progress from the north (span 1) to the south (span 5) as follows:

Span	Water Level
1	40.0 m
2	40.5 m
3	41.0 m
4	41.5 m
5	42.0 m

Inflation of the dam will progress in the same direction, north to south.

For inspection and maintenance of the rubber dam, a set of stoplogs has been provided, for installation upstream of a rubber dam section.

Appendix G contains technical literature and experience lists for rubber dams.

SECTION 10.0

RESERVOIR RIM AND SPUR STABILIZATION

SECTION 10.0

RESERVOIR RIM AND SPUR STABILIZATION

10.0 RESERVOIR RIM AND SPUR STABILIZATION

10.1 INTRODUCTION

The previous engineering studies on the Muskrat Falls site identified landslide activity along the river and spur as a significant problem in development of the site. The spur forms a natural earthfill dam which infills the preglacial valley of the Churchill River, and currently has a head of 16 m across it from river levels of about 19 m upstream to 3 m downstream. After reservoir impounding, the head across the spur will increase to about 36 m.

The spur extends about one km from the rock knoll to the north bank of the river with a crest at about El. 60 m. The width narrows from about 1,000 m at the junction with the north bank to about 70 m at its south end, where it has been narrowed by landslide activity.

Feasibility investigations in 1979-80 showed the Muskrat Falls site is a viable site for a hydroelectric development, although stabilization measures would be necessary to prevent continued landsliding from breaching the spur under existing conditions. In 1982, an interim system of 22 pumpwells was installed on the spur to lower the groundwater table and prevent continued regression of the slopes due to landslide activity. The interim pumpwell system has performed well and no landslides have occurred in the last 17 years.

Details of previous investigations, soil stratigraphy and properties, groundwater conditions, and hydrogeology are outlined in Appendix C along with stability analyses and the basis of design of the stabilization measures.

10.2 SITE CONDITIONS

10.2.1 Topography

The Muskrat Falls site is dominated by a rock knoll, immediately north of the rapids and falls. A narrow spur of postglacial sediments extends about 1 km north to connect the rock knoll with the terrace at forming the north bank of the river. The spur is narrow near its south end where the crest at El. 60 m narrows to about 70 m. It gradually widens to the north where it has a width of about 1 km at its junction with the north bank of the river as shown in Figure 10.1.

The ground surface is relatively flat, except for a ravine at the north edge of the spur which drains a series of three small kettle lakes into the Churchill River at the downstream side of the spur.

10.2.2 Stratigraphy and Geology

The soils in the spur are postglacial sediments consisting of four units:

(a) Upper Sand (El. 60 m to 45 m) consisting of fine to medium sand approximately 10 to 15 m thick.

(b) Stratified Drift (El. 45 to -10 m) consisting of a sensitive marine upper clay unit underlain by varying thickness of sand and silty sand, interbedded with clay and silt strata. The intermediate sand components are relatively thick in the southern part of the spur. The thickness of the upper clay increases to the north. Landslide activity on the downstream side of the spur has occurred in this unit. The soil properties and groundwater conditions in this unit are key to stabilization of the spur.

(c) Lower Clay (El. -10 to -70 m) is a relatively uniform silty marine clay grading to silt and silty sand with depth.

(d) Lower Aquifer (El. -70 to - 210 m) consisting of glacial sand, gravel and boulder infill of the preglacial valley of the Churchill River to bedrock.

The stratigraphic profile results from the geologic history of the site. Preglacially, the Churchill River flowed easterly in a wide U shaped valley north of the rock knoll which is now covered by the spur. Following deglaciation sea level rose and submerged the Churchill River valley as far upstream as Gull Island. Sediments carried into this estuary resulted in the deposition of thick marine clay and estuarine silty-sand deposits at the Muskrat Falls site. Isostatic rebound following deglaciation caused gradual recession of the sea and resulted in deposition of fine sand deposits over the underlying marine clay deposits.

10.2.3 Soil Properties and Groundwater Conditions

The upper sand grades from silty to fine-medium sand with an average of 9% passing the 0.074 mm sieve and density ranging from compact to very dense. This unit is almost fully drained although a perched watertable occurs locally, recharged from precipitation.

Within the Stratified Drift, the upper clay is a silty marine clay with sensitivity averaging 11 (range 2 to 28) from field vane tests with firm to very stiff consistency. The clay is normally consolidated with effective strength parameters of $\varnothing' = 32^\circ$ $c' = 0$ from triaxial tests at large strains. The intermediate sand consists of fine to silty sand with 5 to 50% passing the 0.074 mm sieve, and is generally very dense. Anticipated permeability ranges widely from 1×10^{-2} to 1×10^{-5} cm/sec. Piezometric levels over most of the spur during 1979 feasibility investigations averaged El. 30 m, iwith predicted recharge from the northwest.

The Lower Clay, although a marine clay, is only slightly sensitive with sensitivities ranging from 2 to 6 and averaging 4, with stiff to very stiff consistency.

A pump test in the Lower Aquifer showed the granular infill of the preglacial valley to be pervious with an average permeability (k) of 1.4×10^{-2} cm/s. Groundwater levels in the Lower

Aquifer range from El. 7.0 to 5.0 across the spur from upstream to downstream indicating that the marine clay deposits separating it from the river to form an aquiclude. A pump test (Appendix C) confirmed that no connection exists between the river and the Lower Aquifer.

10.3 LANDSLIDE ACTIVITY AND GROUNDWATER CONTROL

The riverbanks between Gull Island and Goose Bay are formed by downcutting of the river through surficial sands and stratified sands and marine clays with riverbanks at relatively steep angles of 37° to 43°, indicating that these sediments have relatively high strengths. However, numerous landslides have occurred along the river. Three types of slides have been identified.

- Shallow slab failures with the failure surface parallel to the slope in areas where active river toe erosion occurs.

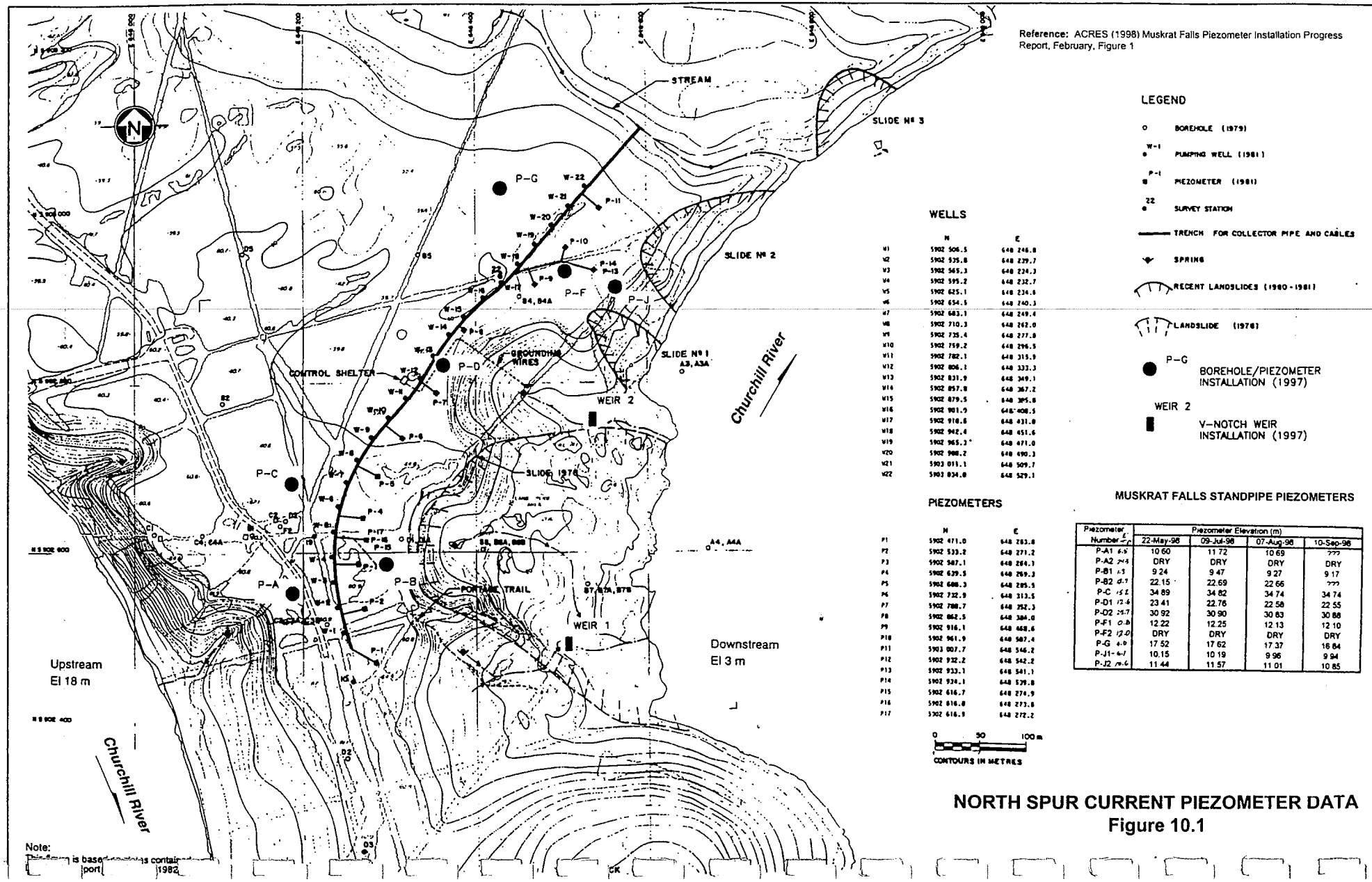
- Block movements which are predominantly translational with the backscarp of the slide 30 to 100 m away from the original face of the slope, which appear to have been triggered by toe erosion and high piezometric levels in sand strata between El. 10 to 20 m.

- Flowslides which can be up to 1 km^2 in size and 2.5 million m^3 in volume which have commonly very little debris in the slide bowls which are typically at El. 20 to 25 m, showing that the slides were based in sensitive clays which liquify after failure, resulting in large retrogressive landslides.

Along the spur, the scars of four major slides are visible, three on the downstream side and one upstream. The most recent slide in 1978 involved about 1 million m^3 of soil. High piezometric levels in the Intermediate Sand, toe erosion, and river level fluctuations due to winter ice staging are the major features contributing to instability along the spur.

Stability analyses in the 1979-80 feasibility study (Appendix C) showed control of groundwater levels in the spur by a line of pumpwells would be the most economical method to stabilize the spur. In order to protect the spur from further instability, an interim pumpwell system was installed in 1982 consisting of 22 pumped wells spaced at 30 m with an average depth of 63 m. The wells consist of 30 cm diameter PVC screens and riser pipe with submersible pumps at El. 0 m, with a common buried header pipe. The pumpwells are intended to lower the groundwater level in the spur from El. 30 m to at least El. 15 m. The system layout is shown on Plate 18.

The pumpwell system was commissioned in September 1982 and has operated continuously to the present. A lightning strike in 1983 damaged the vibrating wire monitoring piezometers and piezometric data was not available until replacement standpipe piezometers were installed in 1997.


Reference: ACRES (1998) Muskrat Falls Piezometer Installation Progress Report, February, Figure 1
LEGEND
BOREHOLE (1979)
PUMPING WELL (1981)
PIEZOMETER (1981)
SURVEY STATION
TRENCH FOR COLLECTOR PIPE AND CABLES
SPRING
RECENT LANDSLIDES (1980 - 1981)
LANDSLIDE (1976)
P-G BOREHOLE/PIEZOMETER INSTALLATION (1997)
WEIR 2 V-NOTCH WEIR INSTALLATION (1997)
WELLS
N E
W1 5902 506.5 648 246.8
W2 5902 535.8 648 239.7
W3 5902 565.3 648 234.3
W4 5902 595.2 648 232.7
W5 5902 625.1 648 234.6
W6 5902 654.5 648 240.3
W7 5902 683.1 648 249.4
W8 5902 710.3 648 262.0
W9 5902 735.4 648 277.8
W10 5902 759.2 648 296.5
W11 5902 782.1 648 315.9
W12 5902 806.1 648 333.3
W13 5902 831.9 648 349.1
W14 5902 857.8 648 367.2
W15 5902 879.5 648 385.8
W16 5902 901.9 648 408.5
W17 5902 918.8 648 431.8
W18 5902 942.4 648 451.6
W19 5902 965.3 648 471.0
W20 5902 988.2 648 490.3
W21 5903 011.1 648 509.7
W22 5903 034.0 648 529.1
PIEZOMETERS
N E
P1 5902 471.0 648 283.8
P2 5902 533.2 648 271.2
P3 5902 587.1 648 264.1
P4 5902 639.5 648 269.3
P5 5902 688.3 648 285.5
P6 5902 732.9 648 313.5
P7 5902 788.7 648 352.3
P8 5902 862.5 648 384.0
P9 5902 916.1 648 468.6
P10 5902 961.9 648 507.4
P11 5903 007.7 648 546.2
P12 5902 932.2 648 542.2
P13 5902 933.1 648 541.1
P14 5902 934.1 648 539.8
P15 5902 616.7 648 274.9
P16 5902 616.8 648 273.8
P17 5902 616.9 648 272.2
0 50 100 m
CONTOURS IN METRES
MUSKRAT FALLS STANDPIPE PIEZOMETERS
Piezometer Number / Elevation (m)
Piezometer Elevation (m)
22-May-98 09-Jul-98 07-Aug-98 10-Sep-98
P-A1 6.5 10 60 11 72 10 69 ???
P-A2 7.5 DRY DRY DRY DRY
P-B1 1.5 9 24 9 47 9 27 9 17
P-B2 6.1 22.15 22.69 22 66 ???
P-C 15.2 34 89 34 82 34 74 34 74
P-D1 12.6 23 41 22.76 22 58 22 55
P-D2 25.7 30 92 30 90 30 83 30 88
P-F1 0.8 12.22 12.25 12.13 12 10
P-F2 12.0 DRY DRY DRY DRY
P-G 4.9 17 52 17 62 17 37 16 84
P-J1 4.7 10.15 10 19 9 96 9 94
P-J2 7.6 11 44 11 57 11 01 10 85
NORTH SPUR CURRENT PIEZOMETER DATA
Figure 10.1
Note:
is base port is contain 1982
Upstream El 18 m
Downstream El 3 m
Churchill River
Churchill River
STREAM
SLIDE Nº 3
SLIDE Nº 2
SLIDE Nº 1
SLIDE 1976
CONTROL SHELTER
GROUNDING WIRES
WEIR 1
WEIR 2
PORTAGE TRAIL
P-A
P-B
P-C
P-D
P-F
P-G
P-J

The possibility of 10 or 15 m rapid drawdown of the reservoir is remote but would result in slope failures. Along the reservoir no protection is feasible, or necessary, as surge waves from any slides triggered by rapid drawdown would not pose a serious threat to the project since they would occur in a lowered reservoir when extra freeboard is available. However, a stabilizing rockfill berm is required along the upstream face of the spur to protect the upstream slope from sliding, if the reservoir ever has to be lowered rapidly in the future.

10.5 STABILIZATION MEASURES

The spur is a natural earth dam of considerable size with a minimum width at the reservoir level (El. 39 m) of approximately 180 m, and a minimum base width at tailwater level of about 450 m. Investigation data and interim pumpwell performance since 1982 shows that the stratigraphy, soil properties and groundwater conditions are complex and vary rapidly along the spur. Hence, to increase the stability of the spur sufficiently to provide adequate stability for long-term operating conditions after reservoir impounding, a number of stabilization measures will be necessary as shown in Plates 18 and 19. These measures include:

- lowering the groundwater table in the downstream part of the spur by the installation of additional pumpwells in the line of pumpwells installed in 1982, with the final pumpwell spacing selected from performance of the interim pumpwell system;

- provision of downstream erosion protection and downstream stabilizing fill in selected areas to improve local toe stability, and eliminate the potential for retrogressive failures due to the presence of sensitive marine clays in the Upper Clay unit;

- local topcutting of the spur and unloading of the top of local high, steep slopes to improve local stability; and

- provision of erosion protection and a stabilizing upstream berm to prevent raveling of the upstream slopes from reservoir wave action and eliminate the potential for sliding in the unlikely (but possible) event of rapid drawdown during future project operation.

Previous engineering studies have considered a number of different methods of stabilization of the spur as discussed in Appendix C. The information gained from installation and operation of the interim pumpwell system since 1982 shows that control of the groundwater in the spur is the most cost-effective and reliable approach. However, a combination of stabilization measures is necessary for the most effective and economical approach.

Engineered earth dams are designed for a minimum factor of safety (FS) of 1.50 for the downstream slope based on a long-term steady-state flownet and limit equilibrium analysis. The complex stratigraphy and groundwater conditions and the presence of sensitive clays, which will liquify when overstressed, limit the application of this approach to the spur. The 1979-80 feasibility study adopted an approach where the stabilization measures adopted increased the factor of safety for sliding by 50% over existing conditions on the downstream side of the spur (i.e. to FS=1.50) for groundwater conditions in the spur with the reservoir impounded. Similarly, stabilization measures were designed to increase the stability of the

Performance of the pumpwells, discussed in detail in Appendix C, is summarized below:

- The south portion of the pumpwell line (W1 to W7) is installed in the most pervious section of the spur where about 55% of the Stratified Drift Unit to El. 10 m consists of pervious sands and silty sands, with an average permeability of 1 x 10^{-3} cm/s. Groundwater levels were lowered rapidly by about 15 m in 1982 when pumping commenced and the area is well drained to below El. 15 m. One pumpwell (W4) operates about 19 hours daily to drain this area.

- The central portion of the pumpwell line (W8 to W17) is installed in lower permeability soils, with about 20% of the Stratified Drift Unit to El. 10 m composed of pervious sands and silty sands. Drawdown during the first year of pumpwell operation was generally moderate (<5 m), although drainage of low permeability soils is slow and long-term drainage can be expected to be higher. Wells in this section of the pumpwell line operate only for short periods, generally less than 1 hour daily.

- The northern portion of the pumpwell line (W18 to W22) is installed in low permeability soils, with about 5% of the Stratified Drift Unit to El. 10 m composed of pervious sands and silty sands. Drawdown during the first year of pumpwell operation was minor (<0.5 m). However, drawdown measured when the 1997 replacement piezometers were installed indicates drawdown in the order of 15 m has developed since 1983. The quantity of water pumped is low, with the five pumps in this section of the pumpline operating a total of about 1 hour daily.

The data obtained during pumpwell installation and operation provides detailed information on the stratigraphy and groundwater conditions in the spur, and provides sufficient data for optimization of the design of stabilization measures for the spur.

10.4 EFFECTS OF RESERVOIR IMPOUNDING ON SLOPE STABILITY

Impounding the Muskrat Falls reservoir will create a reservoir at El. 39.0 m, 20 m above the present river level upstream of the spur. The new reservoir will create a new groundwater region in the upper part of the spur, with horizontal flow occurring in the sand strata across the spur of land. The magnitude of flow will be relatively low as shown by the relatively low quantities of water being pumped by the interim pumpwell line where the present head on the pumpwells (9 m) could be handled by about 24 hours of pump operation daily with a single pump. (i.e. 5% of the total installed pump capacity). Reservoir impounding will increase pumpwell operation to the equivalent of 3 pumpwells in continuous operation. Without the pumpwell line, groundwater levels would increase due to the combined effects of the reservoir and recharge from rainfall, and consequently endanger the spur stability.

The impounding of the reservoir will generally improve the stability of slopes upstream of the dam. Water level fluctuations during normal reservoir operation will not exceed 3 m in magnitude or 1 m per day, a condition which will not cause major instability along the reservoir banks. However, erosion by wave action will cause local bank regression for a few years, until a stable beach slope develops.

upstream side of the spur by 30% over existing conditions (ie. FS=1.30) with the reservoir impounded to El. 39 m, and by 10% (i.e. FS=1.10) for rapid drawdown to El. 25 m. Details of stability analyses conducted in the 1979-80 feasibility and this study are included in Appendix C.

10.5.1 Control of Groundwater

The groundwater levels within the intermediate sand strata will be raised following reservoir impounding which would reduce the stability of the downstream slope and could result in sliding which could breach the spur. In the 1979-80 feasibility study it was determined that the most effective method of stabilization was the installation of a line of pumpwells to lower the groundwater level from the El. 30 m then existing, to at least El. 15 m. Available data at that time indicated a relatively close spacing of 6 m would be required. Each well would be equipped with a submersible pump set at El. 0 m and discharge into a buried header pipe. The 1982 interim pumpwell system was installed with pumpwells at 30 m spacing. Details of the existing interim pumpwell system and its performance since 1982 are give in Appendix C.

The performance of the interim pumpwells spaced at 30 m allows selection of final pumpwell spacing. As shown in Plate 18, pumpwell spacing in the south part of the pumpwell line will be 15 m, 10 m in the middle part of the pumpwell line, and 6 m in the north part of the pumpwell line. A total of 44 additional pumpwells will be installed between the existing pumpwells. The additional pumpwells will discharge into the existing header pipe which is provided with manhole access for flow monitoring. A backup diesel power supply exists to ensure continued pump operation in the event of outage of the primary power supply. The additional pumpwells will be incorporated into the monitoring/control system which allows for remote control and monitoring of operation. Although the existing 22 pumpwells are in good condition, all pumps, risers and controls will be replaced.

The pumpwell system has been proved effective and reliable since 1982. The quantity of water which will be pumped will be relatively small due to the generally low permeability of the soils constituting the spur. Pumpage will increase after reservoir impounding from the present 50 L/s to about 150 L/s, which can be pumped by 3 of the 66 pumps which will be installed.

Two additional systems are included for groundwater control. Most (90%) of the present pumpage from the pumpwell line occurs in the southern 200 m of the line (W2 to W7) in the only section of the spur where relatively thick pervious sandy soils occur. On the upstream face of the spur, a 4.0 m wide blanket of impervious fill will be placed against the bank inside the stabilizing rockfill berm (Plate 19, Section D-D). The blanket provides a partial cutoff to reduce flow through this section of the spur after impounding, and reduce the pumpage from the wells in the south portion of the pumpwell line. Some seepage will still flow under this blanket (as the Intermediate Sand extends from riverbed down to about El. 0 m) but, due to the stratified nature of the soils, a moderate reduction in pumpage will result. Pumpwell spacing of 15 m in this section of the pumpwell line provides three times the number of pumps necessary to handle anticipated seepage after reservoir impounding, even without the impervious blanket.

The second system to control groundwater levels in the spur after reservoir impounding consists of a line of relief drains at the downstream toe as shown on Plates 18 and 19 (Sections

D-D and H-H). The relief drains are 50 m deep with a 50 mm diameter slotted PVC pipe (with geotextile wrap) which will depressurize any pervious beds under the downstream slope of the spur not effectively drained by the pumpwell system. The relief drains also provide a backup drainage system to control groundwater levels in the spur in the event of local reduction in pumpwell efficiency due to screen clogging, or long-term pump outage.

Flow from these drains will be by gravity and efficiency will vary depending on collar elevation. Spacing of the relief drains will vary from 15 m at the south end of the pumpwell line to 6 m at the north as shown in Plate 18, reflecting the decrease in mass permeability of the soils in the spur from south to north. Capacity of the relief drains is sufficient to efficiently handle the relatively low volume of seepage which would occur with the pumpwells out of service. The relief drains would limit groundwater levels in the downstream part of the spur to levels below those existing prior to 1982 installation of the interim pumpwell line.

Three of the relief drains will be extended to 80 m depth to penetrate the top of the Lower Aquifer to:

- serve as standpipe piezometers to monitor groundwater levels in this unit after reservoir impounding; and

- provide additional pressure relief should, on impounding, a connection develops between the reservoir and the Lower Aquifer.

As discussed in Appendix C, the 1979 pumptest of the Lower Aquifer showed the Lower Clay forms an effective aquiclude and no connection exists between the river and the Lower Aquifer within the 1 km radius of drawdown of the pumpwell. Connection of the Lower Aquifer with reservoir further upstream is very unlikely but, if it occurs, effects would be limited by pumping from the camp water supply well which is located on the spur. However, the use of a limited number of the relief drains to monitor performance of the Lower Aquifer and provide supplementary pressure relief, if necessary, will provide additional security at a minimal additional cost.

10.5.2 Downstream Slope Treatment

In addition to lowering the groundwater levels in the spur by the pumpwells and toe relief drains, the following additional stabilization measures are provided as shown on Plates 18 and 19 for the downstream slopes of the spur:

- Placing a rockfill dyke along the existing shoreline to prevent continued shoreline erosion in the area of past landslides, near the south end of the spur, and to protect the toe of intact slopes further north from river erosion, and to buttress the toe of these slopes.

- Excavation of 20 m wide berms at El. 45 m above sections of the downstream slope where river erosion or previous landslides have produced high, steep slopes where stability may still be marginal, despite lowering of groundwater levels in the spur with the interim pumpwells.

- Placing of a stabilizing blanket of compacted free-draining granular fill at the downstream toe of the spur, between the rockfill toe dyke and the El. 20 m contour immediately north of the rock knoll, to buttress the bottom of the slope and improve local stability against retrogressive failure.

- Topcutting 200 m of the spur between the south end of the pumpwell line (W1) and the rock knoll to El. 45 m, excavation of a drainage trench to El. 17 m, and infill with compacted free-draining granular fill. As discussed in Appendix C, stratigraphy in the south 200 m of the spur consists of the Upper Clay. Pumpwells would have low efficiency in this low permeability soil. A combination of gravity drainage with a large drainage trench, and granular fill downstream to buttress the lower part of the slope, is more effective to improve stability of this local area than closely spaced pumpwells. Details of the layout and design of stabilization of this section of the spur are given in Appendix C.

10.5.3 Upstream Slope Treatment

From the rock knoll to a point about 300 m to the north, the upstream slope of the spur is relatively flat below El. 42 m at an average inclination of 4H:1V. Further north to slope steepens to about 1.5H:1V and several small old landslide scars exist. The southern portion of the upstream slope of the spur is stable and appears to have been subjected to occasional drawdown effects from winter ice staging. No stabilizing berm is required in this area and only erosion protection against wave action during diversion and after reservoir impounding is necessary. As shown on Plates 18 and 19, Section C-C, a 1.0 m thick blanket of rockfill underlain by a 0.5 m thick transition of tunnel muck provides erosion protection.

The northern section of the upstream slope sloping at 1.5H:1V below El. 42 m is presently close to limiting equilibrium and would become unstable if subjected to rapid drawdown of more than a few metres. A 20 m wide stabilizing rockfill berm to El. 42 m will be provided as shown in Plates 18 and 19, Section D-D, for the next 200 m north with a 4 m wide blanket of compacted till against the bank to limit seepage to the south part of the pumpwell line. A 4 m wide zone of granular filter from El. 42 to 35 m is provided in the zone of reservoir wave action and normal operating fluctuation. Below El. 35 m, a 4 m wide zone of tunnel muck forms a transition between the till blanket and the rockfill. Further north, the 20 m wide berm will consist of 16 m of rockfill and 4 m of tunnel muck which forms a transition between the bank and the rockfill.

During construction of the upstream berm, larger rock sizes will be placed at the upstream face to protect the berm against wave action. Placement of select, and costly, riprap along the upstream face of the berm is not necessary in view of the width of the rockfill berm and its crest level (El. 42 m - 3 m above reservoir maximum operating level) which will safely allow minor beaching on the face of the berm without need for remedial work.

10.5.4 North End of the Spur

The north end of the spur occurs at the ravine which drains a chain of three small kettle lakes immediately northwest of the spur, at El. 31 m, 27.5 m and 25 m respectively. The creek draining the lakes is about 400 m long with an average gradient of 5.5% to its discharge into the Churchill River downstream of the Spur. The creek bed is at about El. 22 m at the projection of the pumpwell line.

The ravine and chain of kettle lakes forms a large, natural drainage sump which partially drains the north end of the spur to the creekbed elevation. The radius of influence of this large natural drain is much larger than that of a small diameter (0.3 m) pumpwell. The ravine will be developed to improve the efficiency of this large natural drain by the following measures as shown on Plate 18:

- Excavation of a 2 m wide (backhoe) drainage trench to El. 15 m in the creekbed to the projection of the pumpwell line and backfill with free-draining granular fill.

- Excavation of a similar 4 m deep (backhoe) drainage trench down the side of the ravine along the projection of the pumpwell line and backfill with free-draining granular fill.

- Stripping of the lower sides of the ravine downstream of the lower kettle lake, installation of a 1.7 m diameter corrugated steel pipe (CSP) and backfill of the lower part of the ravine with free-draining granular fill to improve the efficiency of the natural drainage along the ravine.

The resulting large granular drain will effectively drain the north end of the spur north of the end of the pumpwell line to about El. 15-20 m, more economically than extension of the pumpwells at 6 m spacing to the bottom of the ravine.

10.5.5 South End of the Spur

As discussed above, the south 200 m of the spur between the south end of the existing pumpwell line (W1) and the rock knoll is composed of Upper Clay with a thin capping of sand. In this low permeability material, effective drainage with pumpwells is not possible. Stabilization measures will consist of the following measures as shown on Plates 18 and 19:

- Topcutting the spur full width to El. 45 m to improve overall stability;

- Excavation of a 10 m wide drainage trench to El. 20 m downstream of the projection of the pumpwell line and a 2.5 m wide drainage trench by backhoe to El. 17.0 m, which is backfilled with free-draining granular fill; and

- Placement of a downstream stabilizing blanket of compacted granular fill between the rockfill toe dyke and the downstream slope of the spur at El. 20 m to improve toe stability of the slope.

Details of soil properties and the design of the stabilization measures in this area are given in Appendix C.

10.5.6 Instrumentation

To assess the effectiveness of the drainage and slope stabilization measures in the complex soil and groundwater conditions existing in the spur, the area must be instrumented and monitored during construction, reservoir impounding and the operation of the project. The instrumentation program will consist of:

- installation and monitoring of piezometers to confirm the efficiency of the pumpwell system;
- monitoring of pumpwell performance (water level in each well, quantity of water pumped, pumping time) and discharge from the large granular drains at the north and south ends of the pumpwell line;
- monitoring of groundwater levels and discharge from the line of toe relief drains;
- periodic surveys on surface monuments at select locations; and
- monitoring piezometric level of the Lower Aquifer during reservoir impounding to confirm that no connection exists upstream with the reservoir.

10.6 RESERVOIR SLOPES

The operating reservoir level is at El. 39 m. At this elevation the reservoir will be mainly confined to the present river course, submerging a portion of the banks. The behavior of the reservoir slopes after impounding will be controlled by the influence of flooding on factors which have been previously identified as the causes of instability, namely, toe erosion high groundwater tables, and fluctuation in river levels.

Reservoir impounding will generally increase the stability of the reservoir banks. However, minor (1-2 m) variations in the reservoir level and wave action can be expected to cause beaching and small local slides based at normal operating range (i.e. El. 37-39 m). Major slope regression is unlikely and reservoir slopes are expected to stabilize after a few years of operation.

The major detrimental effect to the stability of the reservoir slopes would come from rapid drawdown should this unlikely event ever occur. This could cause slope failures similar to the large slope failures experienced in the past in local areas of the reservoir where high steep terrace slopes exist containing sensitive marine clay in the zone of reservoir drawdown (El. 39-25 m). However, little potential exists for large drawdown - induced slope failures in the upper reaches of the reservoir, where the amount of drawdown would be minor and less than natural river fluctuations due to flood and winter ice-staging. For about 10 km upstream of the dam, the right bank is bedrock controlled and relatively flat with no potential for large drawdown - induced slides. Elsewhere, drawdown - induced sliding could occur but size of the slides would generally be smaller than previous landslides, due to the reduced height and inclination of the slopes from slope retrogression and beaching which will occur under normal operating conditions.

The occurrence of landslides within the reservoir will have to be accepted as unavoidable, although frequency and size will be less than under existing conditions. It is recommended that the construction of any services, roads or structures in the vicinity of the reservoir shorelines be

restricted only to areas where large-scale sliding is unlikely in the event of drawdown [i.e. areas with flat banks, low groundwater tables, and where sensitive marine clays do not occur in the slope in the range of possible emergency drawdown (i.e. El. 39-25 m)].

10.7 INDUCED SEISMICITY

The impounding of the reservoir can, in certain cases, induce some seismic activity. The present knowledge indicates that this depends on two factors: the total depth of water and the total area of the reservoir.

A chart of induced seismically has been produced and is shown in Figure 10.2. The volume of the Muskrat Falls reservoir will be of the order of 1,520 x 10^6 m^3, with an average depth of 20 m. The potential that the relatively small Muskrat Falls reservoir will initiate any significant seismic activity is minor.

10.8 SILTATION OF THE RESERVOIR

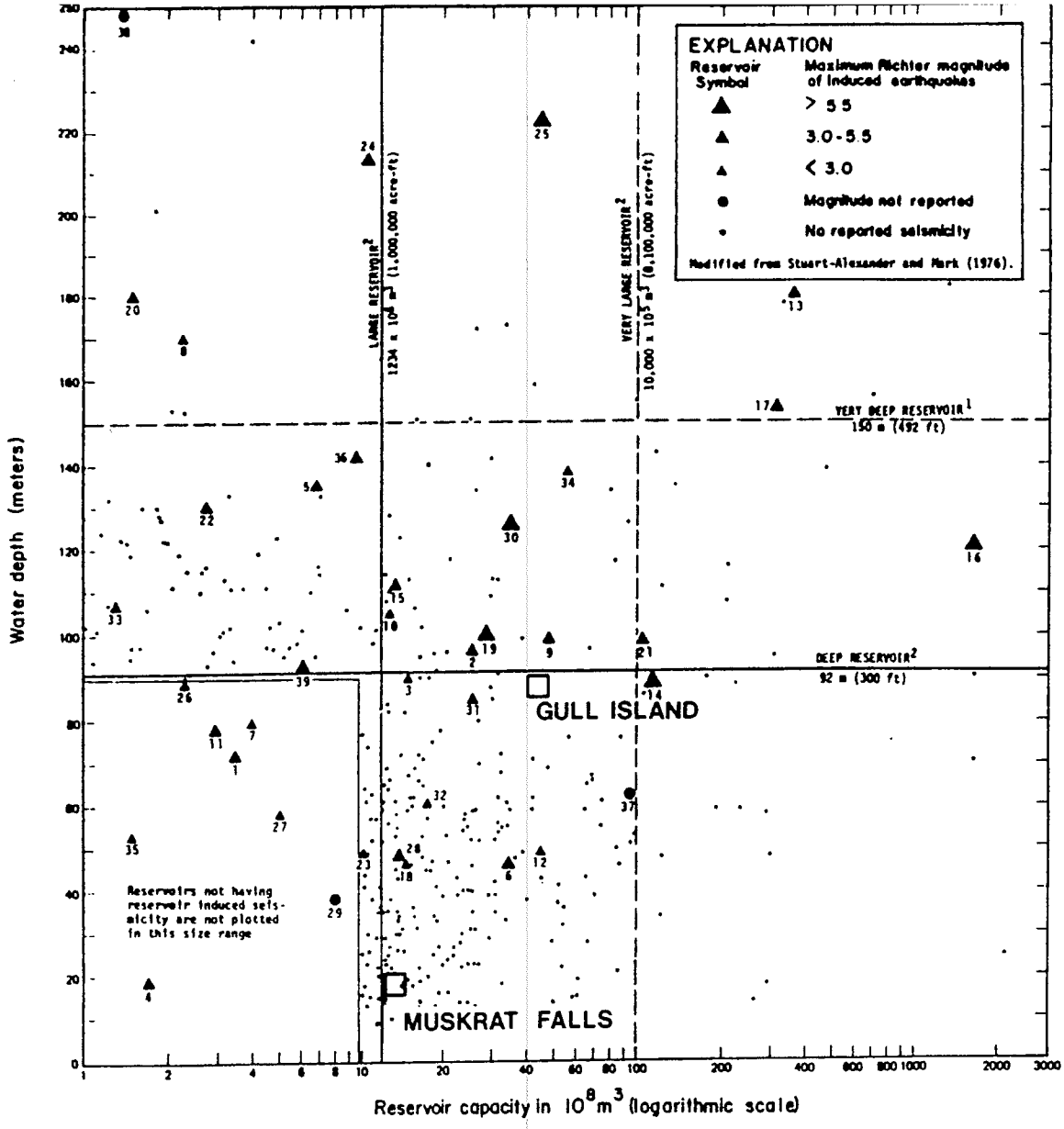
The Churchill River now flows through a channel filled with marine, glacial and glaciofluvial sediments. The river carries fine sediment in suspension and moves larger solids along the channel bed. The construction of the Muskrat Falls development will raise the reservoir to El. 39 m and extend it to Gull Lake.

With the impoundment of the reservoir the bed load will be deposited at the head of the reservoir, in the vicinity of Gull Lake. The amount of sediments likely to be transported into the future Muskrat Falls reservoir will be reduced significantly if the Gull Island development is undertaken, as the suspended load will be deposited in the Gull Island reservoir. In the absence of the Gull Island development, the suspended loads will be carried into the reservoir through a distance directly proportional to the velocity and turbulence and inversely proportional to the grain size of the sediments. During normal operation of the Muskrat Falls powerhouse, suspended sediments in the vicinity of the intake will be carried through the units and reach the tailrace. The Muskrat Falls development is a run-of-the-river power plant. The reservoir is merely a flooded channel. The siltation of the reservoir will be limited by the upstream development of Gull Island and is unlikely to result in significant loss of storage or other problems.

10.9 DOWNSTREAM DEGRADATION

Construction of Muskrat Falls will not have a significant effect on the existing equilibrium for sedimentation and sediment transport in the river channel downstream of the lower falls.

The river between Muskrat Falls and Goose Bay has a difference in elevation of 2.0 m. Levels at Goose Bay are controlled by sea level and will not change. A positive gradient must always exist. It is therefore evident that potential for degradation and river scour between Muskrat Falls and Goose Bay is negligible.



Key to dams and reservoirs having accepted and questionable reservoir induced seismicity.

Dam names, where different from reservoir names, are in parentheses.

1. Bajina Bašta, Yugoslavia
2. Lake Benmore, New Zealand
3. Blowering, Australia
4. Cajuru, Brazil
5. Canelles, Spain
6. Clark Hill, USA
7. El Grado, Spain
8. Emosson, Switzerland
9. Eucumbene, Australia
10. Ilijeća (Gračanica), Yugoslavia
11. Grandval, France
12. Hendrik Verwoerd, South Africa
13. Lake Mead (Hoover), USA
14. Hsinfengkiang, China
15. Jocasse, USA
16. Lake Kariba, Zambia/Rhodesia
17. Keban, Turkey
18. Flathead Lake (Kerr), USA
19. Shivaji Sagar Lake (Koyna), India
20. Kurobe, Japan
21. Manicouagan 3, Canada
22. Lake Monteynard, France
23. Mula, India
24. Nurek, USSR
25. Oroville, USA
26. Oued Fodda, Algeria
27. Pyrambikulam, India
28. Pôrto Colombia, Brazil
29. Lake Entall (Rocky Reach), USA
30. Kremasta (Rel Paul), Greece
31. San Luis, USA
32. Lake Meredith (Sanford), USA
33. Schlegeis, Austria
34. Shasta, USA
35. Sholayar, India
36. Talbingo, Australia
37. Ukai, India
38. Vajont, Italy
39. Vouglans, France

NOTES

1. Woodward-Clyde Consultants' definition for the Auburn Dam study.
2. National Academy of Sciences-National Academy of Engineering (1972) definition.
3. There are two cases of questionable reservoir induced seismicity for which water depth and reservoir volume values were not obtained.
4. No reservoirs having volumes less than $10^8 m^3$ have been plotted on this graph because of their extremely large number.
5. There are ten cases of accepted and questionable reservoir induced seismicity that have occurred at reservoirs having volumes less than $10^8 m^3$.
6. Water depth and reservoir volume for Nurek Reservoir are the maximum as of January 1977, rather than the maximum potential for the reservoir.

INDUCED SEISMICITY VS DEPTH AND VOLUME OF RESERVOIR

Figure 10.2

It is recommended, however, that a review of siltation and degradation be carried out during final design to confirm the above conclusions.

SECTION 11.0

POWER FACILITIES

SECTION 11.0

POWER FACILITIES

11.0 POWER FACILITIES

11.1 GENERAL

The power facilities comprise the integrated intake/powerhouse structure and associated equipment. The intake/powerhouse structure is located approximately in the centre of the river channel, upstream of the lower falls, abutting the spillway closure bulkhead to the north, and the south RCC dam transition to the south.

11.2 SITE INFORMATION

11.2.1 Field Explorations

The subsurface conditions in the power facilities area are based on two boreholes (C5 and C6) and one seismic line (SL-14), completed during the 1979 investigations, and a seismic line (SL-5) completed in 1998. A ground probing radar (GPR) survey was completed over the area to provide contours of the riverbed. Bedrock structural mapping was performed in 1965, 1978, 1979 and 1998.

11.2.2 Geology

Results of these investigations show that the whole of the existing riverbed channel is underlain by good to high quality bedrock. Little or no overburden is anticipated. Occasional faults cut the area, generally with an east-west strike and near vertical dip. Most faults are in hard rock and are healed or reconsolidated with good hard rock contacts.

In Borehole C5 (south bank), from the surface to a depth of 88.8 m the coefficient of permeability is in the order of 1×10^{-4} cm/s. Notable weak and altered zones appear below a depth of 108.5 m (elevation - 42 m). Borehole C6 (north bank) was drilled nearly parallel to the foliation. From 76.0 m to 115.52 m (elevations - 24 to - 43 m), the core is fragmented and shows alterations.

11.2.3 Geological and Geotechnical Design Considerations

The following recommendations are proposed to accommodate particular geological conditions which may be encountered.

- In areas where structures will be founded on fresh undisturbed bedrock, structures shall be designed for an allowable bearing capacity of 3,000 kPa. The near surface weathered rock usually has a higher hydraulic conductivity and more open fractures than deeper, fresh rock. A grout curtain is recommended to reduce and control any seepage under structures in these areas.

- Narrow, east-west trending faults and sheared rock seams exist in the area. Several of these areas were identified on the shoreline of the river and in previous drilling. It is possible that one or more of these features will be encountered. These features should

be examined when exposed during foundation clean up and geologic mapping, and if necessary, decisions for grouting, excavating, backfilling with dental concrete or treatment by other means, will be made at that time as per normal foundation preparation procedures.

- Blasting will be required to remove the natural sill at the lower falls down to an elevation of -10 m for the tailrace and to -32.4 m for the powerhouse. Excavations in rock can be carried out using conventional drill/blast techniques. Pre-shearing, cushion blasting, line drilling and other techniques should be used to minimize damage to the rock walls of the excavations. After the cofferdam is removed and the lower powerhouse is flooded, an air bubble curtain wall or equivalent should be used to prevent any damage to this structure during the subsequent blasting operations in the tailrace area.

- Joint spacing in the drill core and rock outcrops was close to moderately close with most joints striking east-west and dipping towards the south. Most joints were rough with hard rock-to-rock contacts. There were occasional open joints. Major rock slope instabilities are not expected in this area. The north walls of east-west trending excavations will, however, be exposed to the major joint system, i.e. they will daylight into the excavation and support with rock bolts, shotcrete or mesh will be necessary.

11.3 GENERAL ARRANGEMENT

The general arrangement of the power facilities is shown on Plates 10 and 11. The powerhouse is located in the river channel, just upstream of the lower falls. Access is provided via a permanent road constructed along the south flank of the rock knoll and across the permanent structures to the south bank of the river. The road is then routed along the south bank, downstream of the south dam, approaching the south end of the powerhouse at the generator floor elevation.

The facilities include four generating units, three of which have Propeller turbines and one has a Kaplan turbine. The flow to each unit is provided through an intake located immediately upstream of the unit. Three large water passages carry water from the forebay, through the trashracks, to the entrance of the spiral case. The concrete semi-spiral case distributes the water around the unit.

Two power transformers (one for each pair of units) are located on the downstream deck of the powerhouse. These transformers will be connected to a single load bus system switchyard located on the roof of the powerhouse. Two 230 kV outgoing lines will tie the Muskrat Falls powerhouse to the HVDC Converter Station. Minimum provision for connection of a third line to Happy Valley-Goose Bay is included.

The proposed power facility with four units will provide a total plant output capacity of 824 MW at Muskrat Falls.

11.4 INTAKE AND POWERHOUSE STRUCTURE

11.4.1 Intake and Powerhouse

Refer to Sections 2.0 and 5.0 of the Design Criteria presented in Appendix B for dimensions and structural criteria.

The intake and powerhouse arrangement provides a close-coupling of the intake water passages to the powerhouse spiral cases. The two structures at each unit are constructed as an integrated block of reinforced concrete from the lowest foundation level of El. –32.5 m to the intake deck at El. 45.5 m. The powerhouse is 187.5 m long x 68.8 m wide (upstream-downstream) and is divided lengthwise into an erection/repair bay 43 m long, and four unit blocks, each 36 m long. The five blocks are separated by contraction joints. The north end unit has a 0.5 m extension to support the superstructure frame for the powerhouse end wall.

The intake section includes four intakes, each divided into three water passages by two piers 3 m thick which support the gates and trashrack guide loads. The center-to-center spacing of the intakes is 36 m, to match the unit spacing. Each intake is equipped with guides for three intake gates, three sets of bulkhead gates and trashracks.

Along the centreline of the water passages the intake portion of the structure has an overall upstream/downstream length of approximately 25 m, between the trashracks and the entrance to the concrete semi-spiral case of each unit. The powerhouse portion has an upstream/downstream dimension of 42 m from the entrance to the concrete semi-spiral case to the downstream end of the draft tube. Each draft tube is divided by a central pier, 3 m thick, and two sets of two gates are provided for complete dewatering of two units as required.

The four generating units in the powerhouse are spaced at 36 m centers. The erection/repair bay is located at the south end of the powerhouse, adjacent to the main access road. A steel superstructure supports the powerhouse roof at El. 37.5 m. Plans and cross sections of the intake/powerhouse are presented on Plates 16 and 17.

The main powerhouse equipment and facilities are located on the turbine floor at El. 8.0 m and the generator floor at El. 14.0 m. The repair bay area at generator floor level is completely open for erection and repair of the turbine and the generator and is equipped with a large door at its south end. At El. 8.0 m, this area is used for mechanical and electrical equipment and services. Telecommunications, terminal room and control room are located above the generator room floor level at the upstream side of the repair bay area, as shown on Plates 16 and 17.

After erection of the turbines and generators, the south end of the repair bay will be available for provision of multilevel shops and offices.

A low level gallery at El. -17.6 m provides access to the draft tubes and the powerhouse drainage and dewatering system. A central unwatering and drainage sump with vertical turbine pumps is located at the south end of the powerhouse, below the repair bay, with the pump

motors located at El. 8.0 m.

The two main power transformers are located on the transformer deck at El. 18 m on the downstream side of the powerhouse. Vehicle access is provided from a parking area, adjacent to the repair bay entrance, onto the transformer transfer deck at El. 16.5 m, adjacent to the transformer deck.

A crane to handle the draft tube gates is located in an enclosed gallery below the transfer deck at El. 16.5 m. A cable gallery, situated below the transformer deck, extends across the full length of the powerhouse.

11.4.2 Approach Channel

The approach channel, is excavated in rock to an invert elevation of 1.0 m for a distance of 10 m upstream of the intake, then rising with a slope of 3H:1V to the riverbed, at approximately El. 5 m. The channel has an average length of 13 m (upstream - downstream) and a minimum width of 138 m, at the intake.

The approach channel, with an average depth of approximately 36 m, will provide a maximum approach velocity of less than 0.6 m/sec., which will ensure a stable ice cover forms upstream.

11.4.3 Tailrace Channel

The tailrace channel is 138 m wide with an average length of 100 m.

The bottom of the channel at the draft tube exit is at El. -28.5 m for 10.0 m, then rises with a slope of 3H:1V to El. -10.0 m. This elevation has been selected to keep the velocity of the water less than 1.5 m/sec.

11.4.4 Stability Analysis

A preliminary study has been made to check the overall stability of the structure for the following loading conditions.

Normal

- Upstream water level 39.0 m
- Tailwater level 3.0 m
- 2/3 uplift

Extreme

- Upstream water level: 44.0 m
- Tailwater level 11.0 m
- Full uplift
- Earthquake

The minimum factors of safety for normal loading conditions are 1.5 for overturning and 3.0 for sliding, taking into account the shear resistance between concrete and rock with the shear friction coefficient as 1.0 and the unit shear resistance as 700 kPa. The calculated factors are 1.62 for overturning and 4.40 for sliding. The overturning resultant falls within the middle third of the base, therefore, no contact area between concrete and rock will be in tension.

For extreme conditions, the required minimum factors of safety are 1.1 for overturning and 2.0 for sliding. The calculated factors are 1.56 for overturning and 4.20 for sliding.

The lowest factor of safety was found to be that against overturning which is primarily due to the shape and downward slope of the powerhouse.

11.5 TURBINES AND GENERATORS

11.5.1 General

Each of the 4 turbines provides a rated power output of 212 MW (resulting in a plant output of 206 MW) under a rated net head of 35.0 m. The selection of the number of units is treated in Section 5.4 of this report.

Refer to Sections 2.0 and 6.0 of the Design Criteria presented in Appendix B for the main design parameters of the mechanical equipment.

Each turbine is equipped with an electro-hydraulic governor, which regulates the unit speed through the operation of the wicket gate servomotors using a 6.3 MPa rated pressure oil system.

The generators are of the vertical shaft type and directly coupled to the turbine shaft. The generator inertia constant H will be approximately 4 unless system studies dictate otherwise. The generators are provided with the accessories required to enable them to be used for powerhouse heating.

A conventional unit thrust bearing arrangement is proposed. The turbine guide bearing is located inside the head cover and the generator guide bearing is supported by the upper bracket above the generator. The thrust bearing located below the generator rotor is supported by a conical structural pedestal which transmits the vertical loads to the turbine head cover.

Table 11.1 summarizes the main unit characteristics.

Table 11.1
Muskrat Falls Turbine Characteristics

CHARACTERISTICS	Units	Propeller	Kaplan
Synchronous speed	rpm	81.8	81.8
Plant Turbine Installed Capacity	MW	636	212
Number of Units	Nb	3	1
Turbine Rated Capacity	MW	212	212
Rated Net Head	m	35	35
Specific Speed at rated capacity	kW-m	443	443
Rated Flow per Unit	m^3/sec	665	665
Distributor centerline	m	-4.5	-4.5
Approximate Runner Outlet Diameter	m	9.0	9.0
Approximate Runner Mass	t	140	230
Approximate Semi-Spiral Inlet Area	m^2	392	392
Approximate Draft Tube Outlet Net Area	m^2	231	231
Approximate Draft Tube Height	m	26.0	26
Unit Spacing Selected	m	36.0	36.0
Approximate Generator Rotor Mass	t	740	740
Expected Turbine Efficiency at Rated Load	%	92.0	94.2
Expected Peak Turbine / Generator Efficiency	%	94.0	95.0
Expected Generator Efficiency at Rated Load	%	98.6	98.6

11.5.2 Turbines

Based on a general review of the equipment available for very large units and the arrangement of structures developed by the scheme layout study, a conventional vertical axis turbine arrangement using a concrete semi-spiral case was ultimately selected.

Both the Propeller and Kaplan type turbines are suitable for this head range. As mentioned in Section 5.4, the initial selection of the optimum number of units was based on the use of Propeller turbines only. However, for further analysis, it was conservatively estimated that the use of one Kaplan type turbine in place of one of the four Propeller turbines would provide approximately the same energy gain as would be provided by six Propeller turbines. For the purpose of this report, the above consideration, combined with a favorable cost advantage, became the basis for the selection of three Propeller and one Kaplan type turbines for the powerplant.

A specific speed in the neighborhood of 440 (in kW-m units) represents an adequate modern design selection for a relatively high speed turbine. This feature will lower the equipment cost while keeping the civil costs at an acceptable level for a surface powerplant due to reasonable setting requirements. Figure 11.1 and Table 11.2 illustrate the world experience for the

Table 11.2
Axial Flow Turbine Statistics (>100 MW)

No.	Power Plant Name	Supplier	Date	Type	Capacity (MW)	Head (m)	Speed (rpm)	Sp. Speed Ns (m-kW)	Const. k (m-kW)	No. of units	Runner Dia. (m)
1	Porto Primavera	NEYRPIC	1980	K	103.0	18.3	75.0	636	2720	18	8.600
2	Gezouba (A)	DEMW	1981	K	176.0	18.6	54.5	592	2555		11.300
3	Gezouba (B)	HEMW	1981	K	129.0	18.6	62.5	581	2506	10	
4	Solbergfoss	KMW	1981	K	106.7	20.0	79.0	610	2727		8.300
5	Yacyreta	VOITH	1994	K	138.0	21.3	71.4	580	2675	4	9.500
6	Moxoto	DEW	1973	K	110.5	22.0	80.0	558	2617	4	8.382
7	Jupia	SEW	1969	K	107.0	23.0	78.3	509	2439		8.397
8	Long Spruce	DEW	1973	P	100.7	24.4	81.8		2366	10	7.938
9	Salto Grande	KHTW	1978	K	138.0	25.3	75.0	491	2470		8.500
10	Volta Grande	DEW	1972	K	103.0	26.2	85.7	464	2376	4	7.620
11	Rocky Reach	DEW	1967	P	142.0	26.4	85.7		2775		7.899
12	Ice Harbor	ALLIS	1971	K	129.8	27.1	85.7	499	2597		
13	Jerdap-Iron Gates	LMW	1969	K	178.0	27.2	71.5	487	2536		9.500
14	Palmar	NEYRPIC	1977	K	113.4	27.3	88.2	476	2487		7.600
15	Laforge 2	GE Hydro	1996	K	146.8	27.4	85.7	524	2742	2	8.450
16	LG1	DEW	1994	P	116.3	27.5	85.7		2434	8	8.000
17	Limestone	DEW	1990	P	125.4	27.6	90.0		2647	10	7.938
18	Vamma	NOHAB	1968	K	110.0	28.0	100.0	515	2725	7	
19	Little Goose	ALLIS	74/78	K	156.1	28.3	90.0	545	2898		8.130
20	John Day	BLH/Voest	1961	K	158.0	28.7	90.0	539	2885		7.925
21	Nova Avanhandava	HYDRO-ART	1967	K	103.0	30.0	94.7	433	2372		7.400
22	Kettle	DEW	1984	P	104.4	30.0	90.0		2267	2	7.366
23	Cachoeira Dou. Iv	NEYRPIC	1988	K	115.0	32.0	102.9	458	2594		6.750
24	Porsi 3	KMW	1970	K	100.0	32.1	115.4	478	2709		6.800
25	Mactaquac	DEW		K	104.4	33.5	112.5	451	2609	1	6.655
26	Muskrat Project		1996	P	212.0	35.0	81.8		2618	4	9.120
27	Brisay	DEW	93/03	K	193.0	37.5	94.7	449	2747		8.600
28	Caruachi	Kvaemer	1995	K	188.0	38.0	97.7	449	2769	12	7.790
29	Grand Rapids	GEC Alstom	1976	K	124.0	38.0	112.5	420	2588	2	6.450
30	Kainji #11, 12	VOEST		K	108.2	38.1	115.4	401	2475		6.500
31	Castro	NOHAB	1982	K	120.2	38.4	107.1	388	2407		
32	Ligga 3	NOHAB	1969	K	181.7	39.0	107.1	468	2925	1	7.500
33	Akkats	KMW	1994	K	155.0	42.8	115.4	415	2715		6.800
34	Shuikod	Hitachi	1983	K	204.0	47.0	94.7	348	2383		8.000
35	Shulbinskaya	Kharkov	1983	K	230.0	48.3	93.3	351	2442	6	8.500
36	Macagua II	Hitachi	1993	K	216.0	55.0	82.0	254	1887	12	7.900
37	Shamkor	Kharkov	1981	K	195.0	55.0	107.1	316	2342	2	7.350

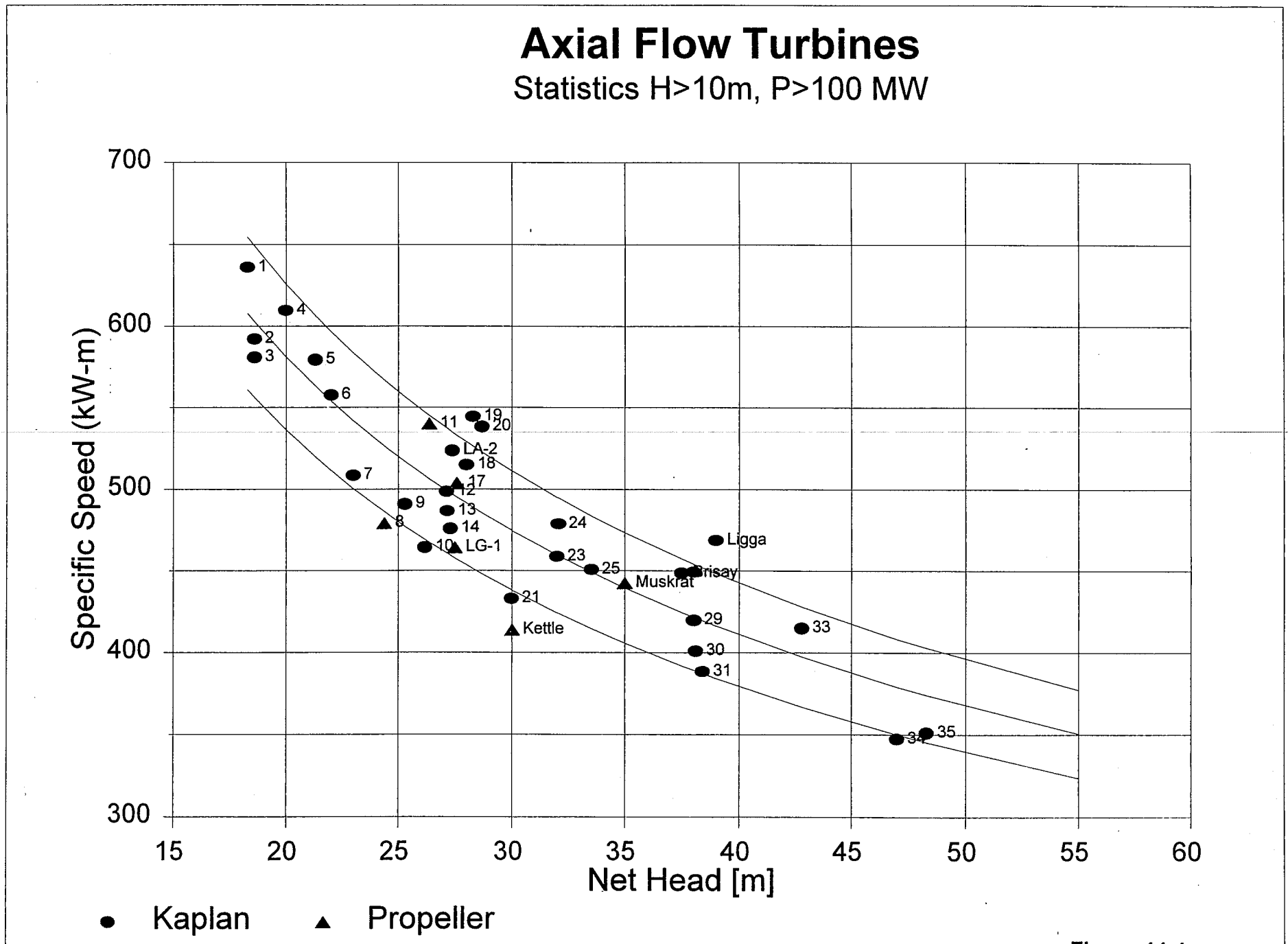

Axial Flow Turbines
Statistics H>10m, P>100 MW
Specific Speed (kW-m)
700
600
500
400
300
15
20
25
30
35
40
45
50
55
60
Net Head [m]
1
4
2
3
5
6
11
19
20
LA-2
18
17
12
7
9
13
8
14
10
LG-1
24
23
25
21
Kettle
Muskrat
Ligga
Brisay
29
33
30
31
34
35
Kaplan
Propeller

Figure 11.1

applicable head range and confirm the Muskrat specific speed selection. The constant "k" is associated with a measure of the trend to the selection of higher specific speeds. The selected synchronous speed of 81.8 rpm (88 poles) has been effectively confirmed by the two major Canadian turbine manufacturers consulted, one of which has, in fact, recommended using yet a higher speed of 90 rpm.

The best efficiency of a Propeller turbine has a narrow range normally positioned at approximately 95% of the unit rated power, whereas best efficiency of the Kaplan turbine is normally selected at about 75% load and has a broad high level range which drops gradually as it approaches rated power.

Turbine model tests are strongly recommended for this design to ensure that, at an early stage of conceptual design, the hydraulic behavior and performance are adequate. Field efficiency acceptance tests will be carried out in accordance with IEC prescriptions. It is recommended that the turbine flow be measured using the current meter method.

The four turbines require runners with an approximate diameter of 9.0 m. The expected runner mass is approximately 140 t for the Propeller type and 230 t for the Kaplan.

The runners blades will be fabricated using stainless steel in accordance with ASTM A-743, grade CA-6NM and they will be provided with anti cavitation lips. The runner hubs may be fabricated either from stainless steel or mild steel.

The turbine shafts connecting the runner to the generator rotor is made in two pieces so that the runner and shaft assembly can be installed and removed as a whole. The shaft material is ASTM A-668 class D forged steel or equivalent and the maximum torsional shear stress should not exceed 41 MPa.

The shaft seal is of the carbon ring type, lubricated by filtered water.

The stay-ring is made in several pieces to be welded and machined at site. The completed head cover is expected to have a mass of approximately 200 t. The outer head cover ring containing the wicket gate pots and bushings will be fabricated and shipped in several pieces to be assembled at site.

11.5.3 Governors

The type of governor considered for this plant is electro-hydraulic PID. This type is preferred as it permits improved system regulation and the best possible response to a wide variety of system control parameters using remote system control equipment. A wicket gate closing time of approximately 8 seconds will initiate a transient pressure rise of approximately 10% and a speed rise of around 30%.

11.5.4 Generators

The four generators are each rated 220 MVA, 15 kV, 60 Hz, 81.8 rpm, and 0.95 power factor. The generators are air-cooled, of the vertical axis design, salient poles and directly coupled to the turbine shaft. The rotor diameter is expected to be approximately 14 m and its mass 740,000 kg.

Refer to Section 7.0 of the Design Criteria in Appendix B for additional generators parameters.

11.6 PLANT ELECTRICAL

11.6.1 General

The Muskrat Falls hydroelectric power development consists of four units with a total plant output capacity of 880 MVA.

The power is produced by generators at 15 kV, and is transformed by step-up transformers to 230 kV.

The powerhouse is connected to the HVDC Converter Station by two 230 kV AC transmission lines. It is assumed that each line can carry the total powerhouse output. This assumption allows for a simplified HV single line diagram using only three circuit breakers in an H configuration.

Each two units are connected via a generator breaker to a three winding transformer. Each LV (15 kV) transformer winding has a capacity of 220 MVA with the HV (230 kV) winding having a total capacity of 440 MVA.

The auxiliary services are supplied by four auxiliary transformers directly tapped to each generator. The scheme eliminates the medium voltage auxiliary switchgear as there is no requirement to supply remote loads.

Refer to Sections 2.0 and 7.0 of the Design Criteria presented in Appendix B for governing parameters of the electrical equipment.

11.6.2 Single Line Diagram

The main power single line diagram is shown on Plate No. 20.

A scheme comprising two generators - one transformer - and medium voltage generator breakers is considered as being a simple, reliable and most economical configuration. Each two units are connected to the 230 kV switchyard through a 3-phase, three winding, step-up transformer.

Each unit is connected to the low voltage winding of the step-up transformer via a 15 kV circuit breaker. This configuration is more economical than the individual unit scheme (four generators and four step-up transformers).

This scheme also minimizes the equipment required on the roof of the powerhouse.

A rated voltage of 15 kV is quoted herein for the generators as being the best level to accommodate the interchange of spare parts for all Lower Churchill Hydroelectric developments units.

11.6.3 Generators and Related Equipment

11.6.3.1 *Generator Characteristics*

The four generators each rated at 220 MVA, 3 phase, 60 Hz, 0.95 power factor are of the vertical axis type, air cooled and directly coupled to the turbine shaft. The power factor of the generator was established by studies carried out by the Client and provided to SNC-AGRA as one of the parameters to establish the rating of the unit. The generator has class F windings in the stator and rotor however the temperature rise on the unit when operated at base rating and 0.95 power factor shall not exceed 80^0C. A Machine Condition Monitoring System (MCM) is provided. The cooling water temperature for design purposes has yet to be established but is tentatively set at 15^0C in the summer.

The main generator characteristics are given in Section 7.3.5.4.

Factory and field testing in accordance with IEEE and ANSI standards is recommended. Acceptance tests are recommended to check design, verify construction performance and to calculate generator constants.

11.6.3.2 *Excitation*

The recommended excitation systems are digitally controlled static excitation systems of the latest design. The excitation control system is to be compatible with the overall station control, which includes automatic, local or remote manual control and full monitoring of the AVR and excitation system via the DCS. The system will also be able to take set point control of generator voltage and/or scheduled VAr's from the control centre via the DCS station group control if this feature is required for the AGC system.

The excitation system consists of a dry type excitation transformer, the main SCR or equivalent power electronics panels, field flashing inputs to bootstrap (self start) the system and the digital AVR and local control and interface panel. The general configuration is for the AVR and power electronics panels to be located next to the local control and protection panels in the machine hall.

The rectifier includes an extra redundant SCR bridge as well as redundancy in the cooling unit fans.

The exciter has complete internal protection, control and monitoring functions, and redundant sensing of machine parameters, together with input voltage fail monitoring.

The voltage regulator response, gain, range of adjustment, ceiling voltage and transfer functions can be programmed into the digital memory following system studies when final generator characteristics and constants are known. Fine-tuning of these characteristics will similarly be possible as the design develops.

11.6.3.3 *Generator Connections*

The power from each generator is carried to the 15/15-230 kV main power transformer via the isolated phase bus rated at 10 kA.

As the main transformers are located outdoors, the isolated phase has to be suitable for indoor and outdoor service. For the short lengths involved, about 15 m, a self-cooled design is more economical. Conductors and enclosures are made of aluminium. The design is of the short-circuited type (bonded enclosures) to minimize external magnetic fields and heating effects in the vicinity of the bus.

A main 10 kA iso-phase bus type generator circuit breaker is provided in the main run between the generator and transformer for switching and synchronizing the units. The presence of the circuit breaker allows the provision of auxiliary power from the HV system when the units are not operating or when the generator is being switched. An integral disconnect is also provided to allow the generator to be locked out for maintenance while keeping the station service feed operational.

Bus taps are provided for connections of the excitation transformer, potential transformers, and surge protection. Current transformers are installed inside the isolated phase bus. The direct connection of the station service transformers to the iso-phase bus via 15 kV fuses avoids the need for medium voltage switchgear and associated reactors and the complexity and increased fault risk these imply. Fuses are provided in order to achieve selective tripping and fault isolation in case of auxiliary transformer faults.

Three sets of three single phase potential transformers wye connected are required for each unit for relaying, metering, synchronizing and voltage regulation; two on the generator side and one on the transformer side of the generator circuit breaker. The potential transformers are of the drawout type, mounted in free standing phase isolated enclosures. Each drawout compartment is provided with primary and secondary disconnect devices and fuses.

The surge protection equipment consists of three generator protection lightning arresters to reduce the amplitude of incoming surges to a safe level. It is normal practice to include surge capacitors to reduce the rate of rise of the incoming surge. Further studies at the detailed design stage will determine their characteristics or if they can be omitted. Surge arresters and capacitors are part of the isolated phase bus cubicle system.

The generator neutral is connected to ground through a single-phase disconnectable link and a single phase distribution transformer. A loading resistor and a relay is connected across the secondary of the transformer to provide sensitive low current grounding. This design is optimal as separate transformer windings are provided for each generator. If a single winding shared by two generators were provided, the ground protection would be compromised (de-sensitized) as the excess capacitance of these connections would have to be allowed for, also a fault on either generator, would trip both units, and the short circuit level on the 15 kV common connections would be much higher.

With a direct station service transformer connection, current limiting reactors are not required.

11.6.4 Major Electrical Equipment

11.6.4.1 *Main Power Transformers*

The main power transformers are located outdoors on a concrete deck at El. 18.0 m on the downstream side of the powerhouse as shown in Plate No. 17. An adjacent transfer deck has been provided at a slightly lower elevation to facilitate installation and removal of the 250 t transformers. The main characteristics of the transformers are given in Section 7.3.5.6.

Three current transformers are provided on each HV and LV bushing, for protective relaying and indication. Current transformers are also installed on the high voltage neutral for ground fault differential and backup protection. Hot spot winding temperature detector systems are provided for temperature indication, protection trip and alarm. A "hydrogen and other dissolved gases in oil" detector system will be installed as required.

Lightning arresters are provided to protect the HV bushings and windings of the transformers.

Fire protection is by a permanently installed automatic spray system having automatic deluge valve, water spray nozzles and temperature detectors. These are linked to both the central fire alarm and central control systems. Fire walls are provided on both sides of the Transformer.

A containment dyke and drainage piping to an oil interceptor sump is provided to contain the oil in case of a transformer tank rupture.

11.6.4.2 *Spare Power Transformer*

The possibility of using the same transformer at Muskrat Falls and Gull Island was reviewed during the Study. The three winding transformer at Muskrat Falls with two units per transformer was considered as the best technical solution for Muskrat Falls. The cost of a spare transformer is included in the capital cost estimate.

11.6.5 Station Service Auxiliaries

11.6.5.1 *AC Distribution*

The station service single line diagram is shown on Plate No. 21.

All units are provided with an auxiliary transformer (dry type, if indoors) to supply the auxiliary loads of their respective unit. The unit auxiliary transformers on units 1 and unit 4 also contribute to the main station service supply as a whole. The transformer rating is presently estimated at 1,000 kVA for units 2 and 3 and 2,500 kVA for units 1 and 4. The transformers are equipped with on-load tap changers and have relatively high impedance to limit the 600 V short circuit level.

Each unit supplies auxiliary power via a tap-off at a point between the main generator circuit breaker and main power transformer. For units 1 and 4 the unit auxiliary transformers also feed their respective 600 V station service bus. The tie circuit breaker between the two buses allows transfers to be effected from one bus to the other in case of failure of a unit transformer.

The total station service power requirements for the generating station will be estimated at a later time when pertinent data on auxiliary loads is known. Power for the intake structures, the spillway, and other structures, located outside the powerhouse is supplied from the plant 600 V station service buses as required.

11.6.5.2 *DC Power Supply and Distribution*

For control and protection, the plant is provided with two 125 V DC, 60 cell, lead acid batteries sized for the required loads at an eight hour discharge rate. One extra 125 V DC, 60 cell battery, is provided for field flashing the four units. This unit will be sized for high power and short time duty. Alternatively, field flashing could be by AC from station service, this should be reviewed at the design stage. The batteries are rack mounted and the room in which they are housed is properly vented although the provision of high power sealed units will be reviewed as this will reduce the criticality of any vent fans.

The design of the battery room, including requirement for containment of spills, etc. will be as per applicable codes and regulations.

DC power for battery charging is supplied by two chargers per battery. Each charger is of sufficient size to carry the normal continuous DC load required and to recharge the battery from the discharged (1.75 V per cell) level in ten hours.

Separate batteries and chargers are provided at any other location outside the powerhouse where dc power is required.

11.6.5.3 *Uninterruptible Power Supply (UPS)*

The plant will have two 120 V, single phase, 60 Hz UPS systems to provide power for operation of the computer systems and other vital plant instrumentation that require AC power. Each UPS is equipped with a static transfer switch for transfer to the other UPS feed when required. A manual bypass to bridge and isolate each UPS is provided for emergency or maintenance operation only as unconditioned power could damage the computer systems.

A 347 V, three phase, 60 Hz, UPS system is provided for emergency lighting.

11.6.5.4 *Grounding*

A system of interconnected copper electrical conductors provides safe touch and step potentials, under maximum ground fault current conditions. The powerhouse, the switchyard and intake structure grounding systems are connected together to constitute one low resistance grounding system.

11.6.5.5 *Lighting*

Lighting systems will generally be 347 V and supplied from isolation transformers, solidly grounded on the secondary. The generator floor will be illuminated by metal halide (MH) fixtures; fluorescent fixtures will be used for control room, offices and shop areas. Ten percent of the lighting fixtures shall be connected to an uninterruptible power supply to provide enough emergency lighting during a power failure. The areas covered include the generator floor, service areas, control room, and other critical areas.

Emergency and exit lighting, as required for egress, shall provide illumination for 2 hours in the event of a power failure of the normal supply.

Exterior lighting fixtures shall be high pressure sodium (HPS) and shall be controlled through a lighting contactor and hands-off-auto control circuit with a photocell in the auto circuit.

Illumination levels and uniformity will be in accordance with IESNA. Energy saving equipment and design will be used throughout the plant.

11.6.5.6 *Emergency Generator Units*

A 600V, three phase, emergency diesel generator is installed and connected to one of the two Essential Services Boards. Automatic transfer switch will be provided to transfer the diesel generator from one essential service board to the other. The unit will be provided with automatic start and load testing facilities. It's power rating is estimated at 500 kW. This rating shall be confirmed during detail design.

11.6.6 Plant Control and Protection

11.6.6.1 *Control Systems*

It is recommended that the generators and the station be controlled by a Distributed Digital Control System (DCS) with control intelligence located as close as possible to each element of the Plant. In this respect the start-stop sequence logic, the local control and the protection panels would be located on the generator floor near each generator. The main display and data storage part of the control system will be located in the control room providing control via VDU, key pad, mouse and keyboard control. The system can also provide, either on a routine or on request basis, printed station logs, graphs and similar station recording facilities. Data is also consolidated into monthly records and stored on bulk storage (diskettes or tapes) and/or transmitted to a remote record keeping location.

It is also recommended to provide a mosaic mimic diagram in the control room to give an overall display of the station operation to be used in conjunction with the single line diagram displays on the display screens. This mimic is driven by the control computer systems and functions through them. However, it is possible to do some basic manual switching, including starting and stopping the generator from either the mimic or the computers. In addition, a directly wired emergency stop is provided on the mimic to shut down any of the generators via the protective systems, independently of the control computers.

The main connection between the central control computer and the PLC/DCS interface unit, and Data Acquisition Units (DAU's) is via a Local Area Network (LAN) type communication using fibre optic, co-axiac or twisted pair cables to suit the system selected. An open system is recommended although a proprietary LAN system may still be more economical to purchase, particularly if proven operating experience and speed of delivery is paramount. The display computers will, however, be either standard Pentium (Windows NT) or Sparc's (Unix/X-Windows) standard computers and display units which can be replaced and reprogrammed with commercially available displays and will not be proprietary systems.

The system considered should interface directly with the generator governor and excitation systems and read the alarms generated by the protection systems directly. Similarly, direct digital data input from digital instrumentation is preferred although direct CT and VT inputs to the DCS data acquisition units (DAU's) units is also possible. It is not recommended to use the protection relay as the sole source of metering and display data, as a single failure would result in the simultaneous loss of protection and indication. However, connection into the system for use as backup or for data cross-checking is recommended.

In addition to the DCS, the relays provide a separate data exchange for fault analysis, fault recording (analogue currents, voltages and alarms) and setting check and/or change. Such facilities are usually separately password protected and may best be provided via a separate LAN connection. This could then be accessed remotely via a dial up communication link or via one of the control computers switched to maintenance mode. All the systems are accessible in maintenance mode via a suitably programmed lap top computer at the relay or digital meter.

The recommended system has the ability to interface directly, at digital level, with the RTU terminals of the Regional Control Centre at Happy Valley-Goose Bay. This avoids numerous coil, contact and transducer interfaces. However, it is necessary to establish an appropriate protocol to exchange this data and to identify the requirements of both systems.

The data transmitted to the Regional Control Centre will be limited to the normal switchgear status and control and the power flow data required by the systems operator. The option to have generator remote start-stop facilities would also be available using the conventional RTU interfaces. The operating data presented to the dispatch centre operators would be limited to that required for overall power system operation and does not include the internal operation of the station. A second series of displays will provide full station remote control data and control access on a permissive basis. This system, to be used for full control of each station, could be located in a separate area at the Regional Control Centre, providing full control of all aspects of station operation.

11.6.6.2 *Local Control and Instrumentation*

The local control panels will be adjacent to each generator and will allow some level of direct hard wired control. This will allow a generator that is on line to continue to be operated locally without the control system and to be shut down, if necessary, independently of the DCS system. However automatic start would only be available via the local PLC unit. Basic direct indication and alarms would be provided through display units which also provide the interface to the DCS control system via the individual generator data acquisition unit (DAU). The conventional electrical measuring, bearing, cooling system, and the turbine and generator temperature monitoring systems can be read locally and are also connect into the overall station DCS control system via the DAU. Two additional systems are also provided, vibration monitoring and generator winding PD monitoring. These are digital systems that monitor, respectively, the mechanical performance and the electrical state of the generator. Both these systems provide direct alarm status and long term storage of their measurements to display trend data of the machine vibration and electrical noise to detect deterioration. This trend data is interfaced with the overall control system so that the data can be stored and analyzed locally and remotely at the control and maintenance centre. The electrical parameters of the generator such as voltage, current, active and reactive power, energy, power factor and frequency are monitored to insure proper operations.

11.6.6.3 *Relay Protection*

The protection system is recommended to be by dual multifunction digital generator packages together with other digital protection providing backup. All primary functions are backed up.

The listed protections will be split over a minimum of two digital packages with critical functions duplicated between the two systems. Independent dc or dual dc powered relays tripping into dual trip logic will be used powered by the group A and group B battery systems. The generator and overall differential protections will be in separate relay systems. The protection system will consist of the following:

- 100% stator ground fault;
- 90% stator ground fault (60Hz Voltage) as backup;
- loss of excitation;
- negative phase sequence current (Alarm & Trip stages);
- reverse power;
- over and under frequency;
- split phase if dual windings used;
- over /under voltage;
- voltage controlled or voltage restraint overcurrent and/or distance backup;
- out of Step;
- stator / rotor over temperature (RTD direct and trip logic);
- generator Differential protection;
- overall Differential protection (overlapping the generator circuit breaker);
- transformer over excitation;
- rotor earth fault alarm (generator breaker open);
- fuse failure detection (Duplicate PT's);
- station service transformer protection (Overcurrent);
- excitation transformer protection (Overcurrent);
- 15 kV Generator breaker fail.

Stator/rotor over temperature (RTD direct and trip logic) will also be provided.

Some or all of these protections will be available within the multifunction relays but the actual elements activated in each relay will be chosen to match the operational requirements at the design stage.

The generator and the transformer bank will be protected by separate differential zones overlapping at the generator circuit breaker to protect the main generator, the main transformer bank, the isolated phase bus, the station service auxiliary transformer, and excitation transformer taps.

A non-directional low impedance distance relay or a voltage restrained time over current relay is used for overall backup protection.

In the event of a blown primary or secondary potential transformer fuse, a voltage balance relay blocks the loss of excitation relays, the low impedance distance relay or voltage restrained relay and transfers the generator voltage regulator from automatic to manual control.

"Volts/Hertz" relays protect the main transformer, the station service auxiliary transformers and the generator against over-fluxing.

The main transformer protection also includes gas pressure, thermal and ground relays. Each of these protective relays and other protective devices are able to initiate generator shutdown and to trip the generator field breaker and the generator circuit breaker as appropriate for the type of fault and its location.

Certain mechanical protections which could result in uncontrolled overspeed, if the circuit breaker were opened prematurely (governor oil pressure, etc.), will trip the machine mechanically only and then check that the power flow is minimum before it trips the main circuit breaker.

Other mechanical protections which require an electrical unit shut down such as overspeed, bearing failure, etc., and any cross trips from the governor or AVR will also be integrated into the protection system. Equally, the lock-out trip logic will prevent start sequence operation, field breaker closure and governor start up as appropriate.

11.6.7 General

External communication links for protection, supervisory and remote control will be provided through an optical fibre ground wire (OPGW) system (OC48) with a line-of-sight 7 GHz synchronous optical network (SONET) digital microwave system (OC3) utilized as a backup redundant link. Each system will be a completely stand-alone with separate multiplex systems and associated equipment.

Included in the Muskrat Falls project is the necessary microwave equipment and infrastructure to extend the microwave system into the Muskrat Falls powerhouse from a site in close proximity to Muskrat Falls (this assuming that the microwave link from Gull Island to the Labrador Regional Control Center is already built as part of the Gull Island project). Also the terminal equipment for the OPGW termination at Muskrat Falls and a similar cost for the OPGW termination at the HVDC Converter station is provided for in the Muskrat Falls study. All internal plant communication systems are addressed in Section 11.6.7.3.

11.6.7.1 *Microwave System*

The microwave system at the Muskrat Falls Plant will consist of a link to the proposed Labrador Regional Control Centre (LRCC) at Goose Bay and a separate link to the HVDC Converter Station. The Microwave system would generally fit the following criteria:

- Terminal equipment would be OC3 based on the ALCATEL 1603/SE Sonet multiplexer system. The terminal equipment will interface with the Internal Plant Communication Systems via a channel bank (NORTEL DE4). This equipment will operate independently of the Fibre Optic OPGW OC 48 terminal equipment to form the second redundant tele-protection ring.

- Radio equipment will generally be based on a 7 GHz line of sight microwave system using frequency diversify ALCATEL MDR-43085-D radio equipment. Space diversity will only be used if necessary to provide a 99.9999% path link reliability factor.

- The plant terminal equipment will be located in an air conditioned communication room (6mx6m) with computer room access flooring and all ancillary equipment (i.e. battery bank, lights, racking, etc.).

- Two steel foundation structures will be provided on the intake gatehouse roof to mount the two antennas necessary for the microwave link to the microwave system extended towards the HVDC Converter Station and the microwave link to the Regional Control Center at Goose Bay.

11.6.7.2 *Fibre Optic System*

The fibre optic communication system will operate in two directions using the high voltage transmission line OPGW fibre for connection to the HVDC Converter Station and via a pole attached dielectric self supporting (ADSS) fibre cable to the Regional Control Centre (RCC).

All fibre optic systems will operate using an OC48 transport system and will terminate in an ALCATEL Model 1648 Multiplexer / NORTEL DE4 channel bank configuration.

The fibre optic connection to the overhead line OPGW will be sized to match the transmission line OPGW cables. The transmission line OPGW cables will be extended to the communication room.

All equipment will be stand-alone and will form the primary communications channels for tele-protection, control and communications

11.6.7.3 *Plant Communications*

- Voice

Telephones with speed dialing capability will be located in the plant control room, in all work areas at each auxiliary system panel, at each local generator control panel, at each switchyard control and relay panel area, and at selected ancillary locations.

The telephones will be linked through a PABX system (NORTEL 61C) to the master NLH communication center. Consideration will also be given to a microcell type private cell phone system or to a pocket pager system to allow for continuous operator communication through the station.

- Data

Interior data communications will be provided by fibre optic cables terminated in Optic Fibre Distribution Frames (ODF's) and routed to the protection, control and communications multiplexer systems as appropriate.

Local fibres will be 4 fibre pairs in heavy duty sheaths and installed in rigid galvanized conduits throughout the station to the various local panels. While fibre optic LAN connections are preferred within the powerhouse and switchyard short lengths of suitably protected twisted shield pair may be used to interconnect local data systems.

The data system will also be used for administrative purposes and for all operational data requirement.

11.6.7.4 *VHF Mobile Radio*

A VHF mobile radio system will be provided to facilitate communications at the hydro facilities and along the 230 kV transmission lines.

The cost of the VHF Mobile Radio System were provided by the Client (Hydro) and is included in the Gull Island Study.

11.7 PLANT MECHANICAL

Mechanical systems of the powerhouse include the intake mechanical equipment, the powerhouse cranes, the draft tube gates, their hoisting mechanism, the elevator and the mechanical auxiliary services described in the following sections.

Refer to the applicable section of the Design Criteria presented in Appendix B for the governing design parameters for the mechanical equipment and services.

11.7.1 Intake Mechanical Equipment

The intake of each unit is divided by two piers into three water passages each provided with a trash rack, gains for a sectional bulkhead gate and an intake gate.

The intake gates are of the vertical lift roller gate type with an upstream seal, 7.33 m wide by 16.3 m high (clear opening). Each gate is operated by a cable hoist located in an enclosed, heated gate house.

Gains are provided upstream of the intake gates for the installation of a set of three bulkhead gates for inspection or maintenance of intake components of one unit at a time. One set of three bulkhead gates is provided for the maintenance of the 12 head gates. Each bulkhead gate is 7.33 m wide by 17.8 m high (clear opening). They are installed/removed in three sections of approximately 37 t each, and are designed to be handled with a mobile crane. It has been assumed that the mobile crane can be rented from a supplier in Happy Valley-Goose Bay.

A set of three trashrack sections is provided at the entrance of each intake. Each trashrack section is 7.33 m wide by 22.0 m high. The handling and cleaning of the trashracks is performed using a mobile crane.

The approximate mass of the different components are as follows:

Equipment	Number	Mass (each)
Intake gates	12	110,000 kg
Intake gate guides	12 sets	28,000 kg
Bulkhead gates	3	110,000 kg
Bulkhead gate guides	12 sets	15,000 kg
Trash racks	12 sets	84,000 kg
Trash rack guides	12 sets	17,000 kg
Hoists	12	30,000 kg

The intake will be equipped with the following auxiliary systems:

- Gate well heating
  - Each intake gate well is protected against ice formation by the use of heaters.
- Gatehouse
  - The gatehouse is a continuous tunnel-like enclosure located downstream of the roadway across the intake. Sections of the roof above each hoist are removable for hoist service.
  - The gatehouse is heated with thermostatically controlled electric space heaters.

11.7.2 Powerhouse Mechanical Equipment

11.7.2.1 *Powerhouse Crane*

The two powerhouse overhead bridge cranes each have a main hook capacity of 400 tonnes and an auxiliary hook capacity of 25 tonnes. The cranes are used to assemble or dismantle the turbines and the generators. They are provided with an equalizing beam to double capacity for handling the assembled rotors.

The two hoists are controlled from a cab located on the upstream side of each crane. Power is supplied to the crane by insulated conductors mounted near the upstream or downstream rail through shoe type collectors. The controls are capable of raising and lowering loads with precision during installation or dismantling of equipment.

Operator access to the crane cabs is provided along the upstream side of the powerhouse.

11.7.2.2 *Draft Tube Gates*

Each draft tube, with a central concrete pier, is equipped with two sets of guide slots to receive draft tube sectional bulkhead gates, permitting the inspection and maintenance of one unit at a

time. Four draft tube bulkhead gates are provided to dewater two units. Each draft tube bulkhead gate has a mass of approximately 118,000 kg and is in four sections. The gate sections are made of fabricated steel and are designed for the maximum tailwater elevation. They are supplied complete with elastomer type synthetic seals and leaf springs to assist the seal closure, and dogging devices for securing sections on top of the gate slot when required. The top section is equipped with a filling valve operated by the lifting beam.

11.7.2.3 *Draft Tube Gate Bridge Crane*

One 40 t electric bridge crane is provided with a two hook follower beam to handle the draft tube gate sections. The crane trolley and bridge travel is controlled by pendant push-button controls.

11.7.2.4 *Powerhouse Elevator*

An elevator is provided at the south end and upstream side of the powerhouse. It services all powerhouse floors from the turbine floor to the powerhouse roof. Its capacity is approximately 2,000 kg.

11.7.3 Mechanical Auxiliary Systems

11.7.3.1 *General*

This Section describes the mechanical auxiliary systems of the Project. These systems are:

- raw water system;
- generating unit cooling water systems;
- fire protection systems;
- service water system including filtered water and potable water systems;
- sanitary drainage and sewage treatment system;
- unit dewatering and filling system;
- clear water drainage system;
- oily water drainage system;
- lubricating/hydraulic oil handling system;
- low pressure compressed air system (service air); and
- high pressure compressed air system (governor air).

11.7.3.2 *Raw Water System*

All of the powerhouse water requirements will come from the raw water main (18" dia.) fed from four taps, one at each unit, taken upstream of the generating unit between the intake bulkhead and service gates.

Each raw water intake pipe (12" dia.) will be provided with a self-cleaning motorized strainer (norm. 130 L/s, emerg. 175 L/s) capable of retaining particles larger than 0.8 mm. The strainers

are sized so that one can be out of service while the three others fulfill the total powerhouse needs (525 L/s) which are:

- generator air cooling;
- turbine and generator bearing cooling;
- fire protection;
- service water, including:
  - compressor cooling;
  - HVAC systems cooling;
  - filtered water for turbine seals;
  - potable water.

All raw water piping and equipment will be insulated to prevent condensation.

11.7.3.3 *Generating Equipment Cooling Water System*

The purpose of the generating equipment cooling water system is to cool the generators and the generating unit bearings. Water for unit cooling is fed from the raw water header, with an independent (10" dia.) branch for each unit. A maximum head loss of 100 kPa is allowed for the generator coolers. The cooling water system will be designed to meet with applicable codes and regulatory requirements for aquatic life.

Cooling water supply to each generating unit (106 L/s) is controlled (on/off) by a motorized shutoff valve located in the water discharge line which leads to the tailrace at each unit through the downstream wall of the powerhouse.

11.7.3.4 *Fire Protection Systems*

Fire protection systems are provided for various equipment and rooms exposed to fire hazard. Major equipment, such as generators and oil filled transformers are protected by automatic water spray fixed systems (deluge). Governor areas and the oil storage room are equipped with automatic water sprinkler systems. Standpipe and hose stations are installed at strategic points throughout the powerhouse. Other service rooms are protected according to the prescriptions of the CNB Code. Both local and remote alarms are provided.

The design of the fire protection systems is in accordance with the NFPA Standards.

The (10" dia.) fire water supply comes from a pick-up point on the raw water distribution header. The fire protection water demand (107 L/s) is calculated based on the simultaneous operation of one transformer water spray fixed system and two 2 1/2 inch hoses.

Two pumps, each sufficient to deliver 100% of the fire protection water needs (107 L/s @ 700 kPa), are provided. One jockey pump (1 L/s @ 760 kPa) is provided to continuously maintain the pressure in the system. Downstream of the pumps, the distribution pipe is 8" dia. A test loop, complete with flow controls, is provided for pump testing. Power supply to the fire pumps is taken

from the powerhouse essential service board which is fed by a diesel generator in case of emergency.

The control room, telecommunication room and computer room are protected with a clean agent fire extinguishing system using Inergen as the extinguishing gas.

11.7.3.5 *Service Water System*

The service water requirements for the powerhouse (74 L/s) is supplied from an 8" dia. pick-up point on the raw water header.

Three centrifugal pumps (37 L/s @ 450 kPa), each sized for 50% of the total service water needs (74 L/s), are provided to allow proper water distribution (2 running/1 stand-by).

Water for domestic and sanitary services, and for turbine seals, is supplied through the potable water treatment system from the service water system. The treatment includes sand filtration and bacterial control treatment (chlorine or UV). Filtered water is fed to the turbine seals before the bacterial control treatment.

Three sand filters, with any two capable of handling 100% of the potable water needs, are installed. The filters are designed to retain all particles larger than 25 microns. To ensure proper filtration, the filters are designed to handle a maximum of 2.8 L/s of water per square meter of filtration surface. Sand filters are equipped with stainless steel pipes and accessories.

Domestic water is supplied to drinking fountains, kitchen sink, service sinks, lavatories, showers and toilets, with hot water supplied for appropriate uses. Portable emergency eyewash and showers are provided in the battery room.

Water needs for the potable water and the turbine seals are estimated to be respectively 3 L/s and 4 L/s (1 L/s per generating unit).

11.7.3.6 *Sewage Treatment System*

A conventional septic tank, followed by a metering tank and two circulating filter beds, treat sanitary waste discharges to proper environmental standards. Treated effluent is discharged to the tailrace. The septic tank is (3.1 m (l) x 1.3 m (w) x 1.3 m (h)) and is designed to treat sanitary discharges based on a crew of 15 employees and a daily water use of 0.13 m^3 per person.

The septic tank output flows by gravity to the metering tank (1.6 m (l) x 1.3 m (w) x 1.6 m (h)). The latter is equipped with two circulating pumps (3 L/s@ 60 kPa), 1 running/1 stand-by, to feed the sanitary water to the circulating filter beds (each = 3.75 m (l) x 2 m (w) x 1.7 m (h)) for treatment. Approximately 20% of this circulated flow is rejected, on a steady basis, to the evacuation pit where two pumps (3 L/s @ 300 kPa), 1 running/1 stand-by, direct the treated water to the tailrace.

For occasional pumping of the septic tank to a sanitary tank truck, connections for a portable pump are provided and the discharge piping is routed to the main entrance.

11.7.3.7 *Dewatering System*

To prevent powerhouse flooding, a completely closed system is installed to dewater the turbine water passages to the desired level below the draft tube mandoor and maintenance platform. This system drains the concrete semi-spiral, runner and draft tube cone of the unit after lowering of the intake gates and the draft tube gates. Once the water is lowered to the required level, leakage coming from the intake and the draft tube gates is handled by the powerhouse drainage system. Occasional complete emptying of the draft tube is also possible.

Emptying of the upper intake water passages is carried out by water release through the turbine wicket gates down to the tailwater level. Below this level the dewatering pumps, located in the dewatering/drainage sump take over to lower the water level to below the draft tube cone access man-door to allow access for inspection of the turbine.

The dewatering system includes a stand pipe (30" dia.) extending to below the bottom of the draft tube at each generating unit. A 30" dia. header is provided to deliver water from each draft tube to the dewatering pumps located in the dewatering/drainage sump. When it is required to completely empty the draft tube, a portable submersible sump pump (100 L/s) is placed in the stand pipe with its discharge hose connected to the 30" dia. header. The water is pumped into the header and thence to the dewatering pumps.

Three vertical turbine pumps (2 running/1 stand-by) in the sump are dedicated to the dewatering system. The pumps are enclosed in vertical pipes connected to the header and the discharge from the pumps is directed to the tailrace downstream of the draft tube gates. Each pump (204 L/s @ 400 kPa) is sized for 50% of the total capacity to allow the dewatering of one draft tube in eight hours.

The dewatering pumps also act as emergency pumps to the normal station drainage system, in case of failure of the station drainage pumps, and are provided with automatic level controls in the sump for this purpose. During the dewatering operation, they are controlled (start/stop) manually.

To evacuate leakage from the intake and draft tube gates, a portable sump pump (30 L/s @ 400 kPa) is used. This pump also serves to totally empty the dewatering/drainage sump when required.

11.7.3.8 *Drainage System*

The drainage system is provided to drain the used water of the following services to the tailrace:

- powerhouse seepage water;
- leakage from the turbine seals;
- fire protection water;
- leakage from intake and draft tube gates when a unit is dewatered;

- floor washing water;
- backwash from motorized strainers;
- backwash from sand filters;
- cooling water from compressors; and
- cooling water from HVAC systems.

All of the floor trenches drain towards the dewatering/drainage sump. The battery room is not equipped with a floor drain. In case of acid spill, which is usually small, it is contained within the room, mopped up and removed for disposal.

From the sump, drainage, water is pumped to the tailrace by three dedicated vertical turbine pumps (2 running/1 stand-by) dedicated to the drainage system. Each pump has a capacity of at least 50% of the total drainage flow. Pump operation is based on level control. The sump (7.5 m x 7.5 m) is sized to allow a minimum of 15 minutes of drainage pump running time per cycle. As noted above, a portable sump pump (30 L/s @ 400 kPa), is used to dry out the sump for repair and maintenance.

An oil interceptor is incorporated at the entrance of the dewatering/drainage sump to intercept any oil before it enters the sump.

11.7.3.9 *Oily Water Drainage System*

The oily water drainage system is designed to intercept any oil leakage or accidental oil spill from the equipment. The equipment exposed to oil spill are, the main oil filled transformers, the governor equipment and the oil storage room.

Each main oil filled transformer is installed above a gravel bed of sufficient volume to collect oil and fire protection water from the transformer. Each bed is drained to a 15" dia. collector pipe that carries the flow of oil and water to an oil separator (13 m (l) x 4 m (w) x 2.6 m (h)) sized to retain the oil volume of one transformer while allowing the rain or fire protection water to continue to the tailrace. The oil separator tank is located below the cable tunnel which runs under the transformer deck at El. 18.0 m.

The oil storage room and the governor system areas are equipped with pits of sufficient capacity to collect any possible oil spill and to separate and evacuate clarified water to the drainage system. Drainage water from these areas is then piped to the drainage sump.

Spilled oil will be collected and disposed of in an environmentally approved manner.

11.7.3.10 *Lubricating/Hydraulic Oil Handling System*

The powerhouse is equipped with its own complete system of two storage tanks, one portable transfer pump, a portable purifier equipped with a pump, supply and return piping with valves necessary to transfer or purify the oil from the generating units.

The system includes provision for:

- filling of the "new oil" storage tank from a tank truck;
- moving oil from the "new oil" storage tank through a portable purifier either back to the "new oil" storage tank, or to a generating unit;
- transferring oil from the "new oil" storage tank through the transfer pump to a generating unit;
- transferring used oil from the "used oil" storage tank to a tank truck for disposal, by means of the transfer pump;
- recirculating oil from a generating unit through the portable purifier back to that generating unit; and
- transferring used oil from a generating unit through the portable transfer pump to the "used oil" storage tank.

Only one operation can be performed at one time.

The lubricating oil storage tanks hold sufficient oil to fill or empty the governor oil system and all bearings of one generating unit. At Muskrat Falls, the size of the units justifies the use of two 22 m^3 tanks. These tanks are installed in an adequately ventilated room over a retaining pit. The volume of the pit is enough to contain the oil of the two storage tanks.

11.7.3.11 *Low Pressure Air System (Service Air)*

The rated pressure of the low pressure air system is 860 kPa. It provides compressed air for maintenance and pneumatic repair tools. The system is designed to feed six grinders, two drills and two screwdrivers simultaneously.

Two water cooled oil lubricated, screw type compressors, each having a capacity of 220 L/s @ 860 kPa (1 running/1 stand-by), are provided. Each compressor has its own intake filter, oil filter and water separator. Both compressors feed into a service air receiver, which has a capacity of 2.7 m^3.

For air distribution, ¾" dia. quick connect outlets are provided at the following locations:

- main floor : two per generating unit;
- main entrance : two;
- turbine floor : one per generating unit;
- cone/spiral case floor : one per generating unit;
- workshops and mechanical rooms : depends on room size, minimum one per room;
- compressor room : one;
- septic tank room : one;
- oil separator : one;
- battery room : one;
- oil storage room : one;
- service water treatment room : one.

In addition, a 2" dia. connection is provided at the turbine pit and draft tube entrances of each generating unit.

The low pressure air system also provides a back up supply to the generator brakes.

11.7.3.12 *High Pressure Air System (Governor Air)*

The high pressure air system serves the governor accumulator tanks and provides air to the generator brakes. The system includes two compressors and two receivers at 6.9 MPa for the governor system, one pressure reducing station, a brake air receiver at 860 kPa and associated piping.

Two air cooled, oil lubricated, piston type compressors at 6.9 MPa provide the required compressed air. Each is sized to be able to re-establish in four hours the initial pressure in the governor air receivers after a governor accumulator tank has been refilled.

The receivers for the governor air are sized (2.7 m^3 each) to rebuild the pressure in the governor accumulator tank to the nominal working pressure of 6.3 MPa, with the help of the compressors during normal operating conditions. This operation should not exceed 2 hours.

The receiver dedicated to the generator brake system has sufficient volume (2.7 m^3) to supply the air required for stopping all units at once, without the help of the compressor.

11.7.4 Heating, Ventilating and Air Conditioning

11.7.4.1 *General*

The HVAC systems are provided to maintain a healthy atmosphere and acceptable temperature levels in the various areas of the powerhouse. In case of a fire, these systems are used for both smoke exhaust and pressurization of staircases and other escape passages to permit evacuation of the operating personnel. The control area is air conditioned. Its outside air supply is independent and cannot be interrupted.

Basically, air circulation is automatically controlled and is not influenced by natural air movements. Local areas exposed to harmful gases or odors are maintained at a negative pressure. Outside air intake will be located away from the exhaust. Outside air supply will be ducted and insulated. HVAC systems serving energy production equipment or area have standby provisions.

11.7.4.2 *Ventilation System*

- Ventilation Rates
  - The minimum outside air ventilation rate is one air change per hour, based on the total volume of the powerhouse. On the working level volume (taken as the overall floor area multiplied by a height of 3.6 m), this represents approximately 2 air changes per hour.

- To avoid inside air stagnation, the total air circulation rate, including outside air, is set at 6 air changes per hour, based on the working level volume. In some areas, air circulation rates may be higher and will be governed by the internal cooling loads.

- Cooling and Heating Loads

  - The heat dissipation from internal sources creates a cooling load estimated at approximately 700 kW for the powerhouse and 85 kW for the service area.

  - The building heat load is estimated at approximately 750 kW during the winter. This load can be compensated by the equipment heat dissipation in some areas. However, during plant shut-down, the HVAC systems are designed to compensate for a heat load of approximately 550 kW.

- HVAC Equipment General Arrangement

  - Due to the size of the powerhouse, to minimize outside air requirements and to avoid cumbersome duct runs, the powerhouse cooling is done using local water cooled air handling units.

- Battery Room Ventilation

  - The battery room is ventilated and maintained at a negative pressure.

  - The minimum air flow will be based on the most demanding of the following:
    - Q (L/s) = 0.025 x no. of cells x charge current;
    - Q (L/s) = 12 air changes / hour.

  - Two spark proof exhaust fans (one standby) are provided for this service. The room air is exhausted outside through a dedicated duct. If electric motors need to be located inside the battery room, they will be explosion proof.

- Normal Exhaust

  - Each of following rooms or areas also have an independent exhaust system. A dedicated exhaust duct is provided for each of the following system:
    - oil storage room;
    - lunch room and kitchen;
    - toilet/locker room and septic tank.

- Emergency Ventilation Systems

  - These systems are provided for the following purposes :

    - pressurization of staircases and other escape passages to permit evacuation of the operating personnel;

- exhaust smoke outside and prevent it from migrating to the areas not affected by the fire;

– The smoke exhaust system is provided for the powerhouse main floor.

– The make-up air will flow inward to compensate for the smoke exhausted and to keep the escape passages full of clean air and free of smoke. Fans used for the normal ventilation are planned to be used in case of emergency.

11.7.4.3 *Air Conditioning System*

A dedicated air conditioning system is provided for the control room, the computer room and the telecommunication room. Two air conditioning units (one stand-by) and a separate outside supply are provided to increase reliability. The relative humidity is maintained between 30 and 55 %.

- Noise Criteria

  – In the powerhouse, the noise coming from the turbine/generator units generally obscures that from the HVAC equipment. A maximum level of 85 dbA from the HVAC equipment is expected to be met.

  – In the other areas, the HVAC systems will be treated acoustically to meet the neighboring local noise criteria.

SECTION 12.0

230 kV SWITCHYARD

SECTION 12.0

230 kV SWITCHYARD

12.0 230 KV SWITCHYARD

12.1 SWITCHYARD ARRANGEMENT

The Switchyard is located on the roof of the powerhouse and contains the following: 230 kV SF_6 circuit breakers, line disconnects with ground switches, potential transformers, current transformers and associated bus work and steel structures all as shown on Plates 22 and 23.

From the pull-off structures on the roof of the powerhouse, two 230 kV transmission lines exit the switchyard to the south and on to the HVDC Converter some 65 km upstream.

Provision for a third 230 kV transmission line or cable termination for a transmission line to Goose Bay has been allowed for with the cost of the steel structure only provided for in the estimate. The location of future equipment is shown on the drawings.

The high voltage side transformer protection for the generator unit transformer is provided by the 230 kV line breaker and tie breakers in the switchyard. The transformers are connected between these two circuit breakers via a motorized disconnect switch which allows the transformer to be automatically isolated for a transformer fault and the transmission line to be returned to service immediately. Ground switches are provided for transformer grounding and transmission line grounding. Current transformers and potential transformers are provided for relaying, metering and synchronizing. All breakers and motor operated disconnects are remotely controlled from the plant control system.

Terminal structures are provided to connect the 230 kV lines leading to the HVDC Converter Station. The overhead ground wires of these lines are connected to the grounding grid at the Muskrat Falls switchyard and should be also connected to the grounding grid at the HVDC Converter Station and at each line tower, to provide adequate grounding protection for the 230 kV station on the powerhouse roof and the powerhouse ground grid.

12.2 SWITCHYARD ICING

There is current practice of locating HV Switchyards on the powerhouse roof close to the tailrace and spillway structure even in areas with very cold climates like the Northwest Territories, northern Quebec, and Labrador.

In general, spillways do not operate in winter as they are mainly operated during the spring and summer floods. If they are required to be operated in the winter, only the gates further remote from the switchyard should be used first.

During the cold period, the river is normally covered with ice and the unfrozen area in the vicinity of the tailrace is too small to create an icing problem in the switchyard; furthermore, in the case of Muskrat Falls, the prevailing winds are generally from the west and the mist would be blown away from the switchyard.

Recent information obtained from James Bay LG-1 engineers confirm that no particular measures were taken during the design stage and that no excessive buildup of ice or freezing has been observed to date in the switchyard which is located on the roof of the powerhouse. LG-1 units were put in operation in 1994-1995.

12.3 SWITCHYARD DESIGN CRITERIA

Refer to the Design Criteria presented in Appendix B, Section 7.0 Electrical, for the design parameters of the equipment in the switchyard.

12.4 SWITCHYARD PROTECTION AND LOCAL CONTROL

12.4.1 General

The 230 kV lines from Muskrat Falls to the HVDC Converter Station switchyard will have microwave and fibre optic communications for full redundancy. The line protection system will have a fully digital optic fibre differential protection as the main protection and a digital line distance relay for the backup system. The advantage of the digital system is the speed and accuracy, and very high sensitivity for ground faults which is necessary for the ground conditions in Labrador. It also has the ability to handle series compensation if required. This would also keep the 230 kV and 735 kV protections similar, reduce spares and maintenance training. In addition to the phase by phase digital optical fibre differential protection, these lines have at least three zones of phase and ground distance, ground directional (connected in a comparison mode) and optic fibre teleprotection systems. A microwave backup also is required for the teleprotection. The lines are equipped with both three pole and single pole auto-reclose facilities, including synchro-check and/or automatic synchronizing facilities. A link into the station control data is provided by the bay DAU interfaces in the relay rooms. The transformer mechanical alarms and the breaker and disconnect status are similarly read by the bay DAUs and transmitted to the central control. Each relay panel is equipped with a minimal direct control and alarm display to enable an operator to control any of the high voltage equipment from the switchyard if it is necessary to evacuate the powerhouse. These facilities are also used during local bay maintenance or if it is necessary to maintain or upgrade the control computer system.

The 230 kV transmission lines at the HVDC Converter Station should have the same type of line protection and integral circuit breaker fail protection tripping which will initiate tripping to Muskrat Falls via a direct transfer trip circuit. Similarly, the switchyard at Muskrat Falls will have bus differential protection zones for the sections of bus between the line breaker, the tie breakers and, the transformer high side bushings.

This protection overlaps with the line and transformer protection and cover any "small zone" faults between each circuit breaker and its associated current transformers (CTs). The above protection, in conjunction with the conventional circuit breaker fail protection, ensures that no fault single contingency is left to be cleared by remote, zone 2 or 3 or over current backup systems. If the fourth breaker for L3 to Goose Bay is added a single fault and a circuit breaker fail will not cause a complete loss of the plant.

All the 230 kV circuit breakers are provided with circuit breaker fail protection to detect if a circuit breaker has tripped correctly on fault. They are also provided with pole discrepancy protection to ensure that full three pole tripping occurs if one pole does not function during normal switching.

Breaker fail protection requires direct transfer trip signals over the fibre optic cable system and via the backup microwave system to ensure that the respective circuit at the HVDC Converter Station switchyard is isolated.

12.4.2 Transformer Protection and Local Control

The main protection will be a digital protection package with a separate digital package for backup. The listed protection will be covered by the main multifunction relay and backed up as required by a second multifunction relay or additional discrete relays as required. Independent dc or dual dc powered relays tripping into dual trip logic will be used powered by the group A and group B battery systems. The mechanical trips (Buchholz, WTT etc.) and differential protections will be in separate systems. The protection will consist of the following:

- a digital transformer differential and transformer monitor relay;
- digital overcurrent relays for high side phase and neutral overcurrent protection;
- alarm interface and trip logic for transformer mechanical protections; gas, surge (Buchholz), winding temperature, pressure relief etc. This will be separate from the differential protection to provide the required A and B trip logic;
- digital circuit breaker fail protection tripping into the bus protection logic;
- local standby direct wire control (125Vdc), synchronizing logic, local indication and alarm unit with repeat and interposing relays to interface with the DCS DPU; and
- a digital instrumentation and/or transducer system for the Transformer bay with 4-20 mA and/or LAN interface to the local DCS system DPU; and
- DCS Data Processing Unit (DPU) for breaker and disconnect status, measured and alarm interfacing to the DCS system (Bay Box).

As an open ring configuration is proposed, the transformer high side disconnect will open automatically on a lockout trip to allow the line and tie breakers to be re-closed.

12.4.3 Switchyard Bus Protection and Local Control

A digital bus protection system will be provided zoned to suite the final bus configuration. It will require one CT core per circuit and be connected to provide phase by phase protection, CT open circuit alarms and internal check and block logic. The other facilities will be:

- alarm interface and trip logic to trip all circuit breakers connected to that section of bus;
- trip logic to receive circuit breaker fail input to trip all circuit breakers connected to that section of bus;
- trip logic to interface with the trip circuits to and from the transformer (or line) disconnect for isolation purposes;
- digital overcurrent relays for the bus tie circuit breaker phase and neutral overcurrent protection. It should be noted that although phase and neutral inverse and instantaneous trip facilities will be provided these may only be used during commissioning and may be selectively deactivated during normal operation;
- digital circuit breaker fail protection for the bus tie circuit breaker(s) tripping into the bus protection logic;
- local standby direct wire control (125 Vdc), synchronizing logic, local indication and alarm unit with repeat and interposing relays to interface with the DCS DAU;
- a digital instrumentation and/or transducer system for the tie breaker and bus voltages with 4-20 mA and/or LAN interface to the local DCS system DAU; and
- DCS Data Acquisition Unit (DAU) for breaker and disconnect status, measure and alarm interfacing to the DCS system (Bay Box).

12.4.4 Transmission Line Protection and Local Control

The transmission line protection will consist of two systems, a digital fibre differential protection scheme and a digital distance protection scheme and backup systems, including the necessary control facilities for circuit breaker fail, tripping and auto reclosing, distance to fault measurement system and data storage, as follows:

- A first main protection consisting of a digital optic fibre differential protection using two directly connected fibres or equivalent multiplexed channels, in the OPGW, with the following facilities:
  - independent phase to phase differential protection;
  - standby direct transfer trip facilities for CB fail etc;
  - facilities for single pole tripping and auto-reclose interfaces;
  - capability to protect series compensated lines; and
  - channel monitoring and fault data storage.

- A second main protection consisting of digital distance protection fed from line CVT's, with the following facilities:

  - tele-protection logic for permissive under or over reach and ground fault direction comparison logic;
  - three zone distance, phase and ground with single pole trip logic;
  - ground directional operating in a directional comparison mode for three pole tripping only;
  - time delayed non-comparison backup;
  - distance to fault measurement, PT monitoring and data storage;
  - a minimum of three zones of measurement; and
  - facilities for single pole tripping and selectable single and/or three pole auto-reclose interfaces.

- Auto-reclose logic with the following facilities:

  - single pole high speed reclose (500-1000ms);
  - three pole delayed reclose with synchrocheck (5-15s);
  - synchronizing logic with selection for dead-line/live bus, live bus/dead-line, and/or synchronize close; and
  - synchrocheck and or automatic (system) synchronizing relays.

- Digital overcurrent relays for phase and neutral overcurrent protection.

- Alarm interface and trip logic.

- Digital circuit breaker fail protection tripping into the bus protection logic.

- Local standby direct wire control (125Vdc), synchronizing logic, local indication and alarm unit with repeat and interposing relays to interface with the DCS DAU.

- A digital instrumentation and/or transducer system for the line bay with 4-20mA and/or LAN interface to the local DCS system DAU.

- DCS Data Acquisition Unit (DAU) for breaker and disconnect status, measured and alarm interfacing to the DCS system (Bay Box).

12.4.5 Teleprotection System

Teleprotection system using fibre optics and microwave are required between the Muskrat Falls switchyard and the HVDC Converter Station. The facilities required are as follows:

- First main teleprotection signaling with four channels using the optic fibre as data medium as follows:

  - Channel 1 Direct transfer trip for CB fail etc.;

- Channel 2 Permissive transfer trip for distance protection;
- Channel 3 Directional comparison signal for directional ground; and
- Channel 4 Generation Shed.

- Second main teleprotection signaling with four channels using a channel on the microwave multiplex system as follows:

  - Channel 1 Direct transfer trip for CB fail etc.;
  - Channel 2 Permissive transfer trip for distance protection;
  - Channel 3 Directional comparison signal for directional ground; and
  - Channel 4 Generation Shed.

12.4.6 Switchyard Maintenance

The access for personnel to the switchyard for purposes of bus line and equipment maintenance is by the elevator. Access for major maintenance and equipment required for the maintenance will be from the intake deck located 8 m above the powerhouse roof. The maintenance equipment will either be placed on the roof by means of a suitable crane from the elevation 45.5 m or stored on the roof in a suitable location and enclosure. Equipment for maintenance may include an A-frame gantry crane and/or a portable hydraulic lift, depending on the maintenance being performed.

The spare transformer will be located on the transformer deck as shown on Plates 16 and 22. All the electrical spare parts will be stored in the camp warehouse.

A final recommendation on equipment required will be reviewed during detail design.

SECTION 13.0

CONSTRUCTION FACILITIES

SECTION 13.0

CONSTRUCTION FACILITIES

13.0 CONSTRUCTION FACILITIES

13.1 ACCESS

Access to the site will be by air and by sea through facilities at Happy Valley-Goose Bay and then by road for a distance of 32.5 km to Muskrat Falls. Road access is also available year round via the Quebec North Shore through Baie Comeau, Labrador City and the Trans-Labrador Highway to the project site.

13.1.1 Airport

Air access to the site for personnel and air freight will be through the airport at Happy Valley-Goose Bay. The airport is a well equipped modern facility and is capable of handling the largest aircraft in present day service. Presently the airport is serviced by Dash 8 aircraft on a scheduled basis for both freight and passenger service. Additional flights or larger aircraft will be added to the scheduled service at Goose Bay if passenger or freight volumes warrant more.

13.1.2 Road Access

A year-round vehicular access is available from central Canada via the Quebec north shore region (Baie Comeau) through to Labrador City and the Trans-Labrador Highway to the project site. This route has a total length from Labrador City to site of about 530 km which is comprised of sections, namely; Labrador City to the intersection with the Esker Road (approximately 200 km), intersection with the Esker Road to Churchill Falls (approximately 40 km), Churchill Falls to Gull Island (approximately 225 km), and Gull Island to Muskrat Falls (approximately 64 km). Access is also available from Happy Valley-Goose Bay to Muskrat Falls (approximately 32.5 km). Each section is briefly described below.

a) Labrador City to Intersection with the Esker Road. Most of this road has been constructed since 1990. The road is designed to RCU 80 standards (speed limit of 80 km per hour). There are two bridges en route; one across the Ashuanipi River designed for MS250 loading, and the second spanning the Ossokmanuan Reservoir crossing area. The Ossokmanuan bridge consists of a 2-lane, 3-span structure with a long approach to each abutment. The bridge was designed for CS600 loading and was also checked and found acceptable to withstand loading produced by the convoy assembly which transports transformers to Churchill Falls.

b) Intersection with Esker Road to Churchill Falls. This road was built as part of the Churchill Falls project and, until recently, was maintained by CFLCo. The road is now owned and maintained by the Department of Works, Services and Transportation, as part of the Trans-Labrador Highway. It equals or exceeds the RCU 80 standards. Some road re-alignment is planned for a section of the road near the Twin Falls turnoff.

c) Churchill Falls to Gull Island. Some reconstruction of this road has been carried out and the remaining reconstruction is planned for completion by the end of 1999. To date, all major river crossings are spanned with new bridges to CS600 load standard. The

remaining older, smaller bridges are being replaced with large culvert assemblies. The road section will be upgraded to RCU 80 standards.

d) Gull Island to Muskrat Falls. This section of the road has been upgraded and reconstruction was completed in 1997. The reconstruction program consisted of the construction of new bridges (MS250 or CS600 capacity) at all major river crossings, installation of new culverts, road re-alignment and upgrading where required, finishing up with a final road section meeting RCU 80 standards (9.0 m top width).

e) Happy Valley-Goose Bay to Muskrat Falls. This section has also been upgraded with reconstruction completed in 1997 to the same standards as the road from Gull Island to Muskrat Falls (see (d) above). Local widening of the Trans-Labrador Highway will be required at selected locations between Happy Valley-Goose Bay and Muskrat Falls to permit public vehicular traffic to pass any slow moving, heavy lift convoys transporting heavy and/or large equipment items to the project. The estimate does not include any allowance for the construction of these passing areas as it is assumed that these will have been constructed for the Gull Island Project.

Construction Roads

The existing access road from the Trans-Labrador Highway will be extended and upgraded to become the main access road to the site. The existing road extends to the North Spur pumpwells and will be upgraded along its approximately 3.0 km length. This road will be extended 1.8 km to the end of the north dam.

Various construction roads to the areas of work or quarries will be required. These site construction roads are included in the unit and lump sum prices for construction.

13.1.3 Harbour

Equipment can be shipped via marine transport by ship and barge to the port of Happy Valley-Goose Bay. This port is ice free and navigable for a maximum of seven months a year.

The harbour facilities at Happy Valley-Goose Bay are approximately 40 km from the project site and any equipment arriving by marine transport must be trucked via local roads in the town and the Trans-Labrador Highway to the site.

The port will be utilized as the destination point for heavy lift and bulk cargo items. Dock cranage is not available at the wharf and therefore mobile cranes will be utilized for the lifting of cargo arriving at Goose Bay.

A small marshaling yard and dry storage building will be utilized at the port as a receiving area for materials, arriving by marine transport. It is assumed that these facilities will be in place from the Gull Island project operations and hence no cost for these facilities are carried in the Muskrat Falls Capital Cost Estimate.

13.2 CONSTRUCTION FACILITIES

13.2.1 Camps

Type of Accommodation

a) Muskrat Falls. The construction camp will be located between the Trans-Labrador Highway and the Churchill River, on the left (west) side of the access road. The construction camp location and general layout are shown on Plates 24 and 25 respectively.

The camp is designed for 1,215 single status personnel. The camp will consist of ten prefabricated modular, two storey units accommodating 100 persons each. These units will be designated as male and female units and contain single rooms and common washrooms. There will also be five single storey units for management or supervisory personnel. These units will contain single rooms with a shared washroom between two rooms.

The camp will be constructed in two phases to support the project construction manpower schedule. The first phase will be designed to accommodate 550 people and will be constructed as soon as possible in the fall of the first year of construction. Work on the diversion tunnels will be in progress prior to the first phase of the construction camp being ready for occupancy. It is planned that workers for the initial mobilization and construction phase of the river diversion will be housed by the Contractor in existing facilities in Goose Bay. The second phase of the construction camp will be completed during the summer of Year 2.

b) Happy Valley-Goose Bay. Presently, in excess of 350 housing units are available at Canadian Forces Base (CFB) Goose Bay. These units were formerly used to house armed forces personnel. Approximately 100 of these units will be utilized for family accommodations of management and personnel who will commute daily from site. The existing infrastructure in Happy Valley-Goose Bay is considered sufficient to handle the increase in population associated with the project.

The estimate includes the rental of 100 existing housing units including utility charges and the purchase of basic furniture and major appliances.

13.2.2 Recreation Facilities

Adequate recreation facilities will be provided. It will be consistent with other recent large construction projects in remote areas. The recreation facilities proposed are as follows:

- Recreation Centre comprised of prefabricated modular units totalling 960 m^2. This building will contain TV rooms, commissary, games room, library/meeting rooms and camp administration;

- Gymnasium/Fitness Centre totalling 1814 m^2;

- Bar/Restaurant comprised of prefabricated modular units totalling 430 m^2; and
- Outdoor recreation area for summer sporting activities such as softball and soccer.

Sporting and recreational activities will be coordinated by the recreation staff.

13.2.3 Camp Services

Potable and fire water for the camp will be pumped from the existing pumpwell F-3 on the north spur. Storage capacity will be provided for every day use and to provide adequate flow for fire fighting. All water distribution mains will be buried and heat traced. Sewer mains will also be buried with sewage treated in a prefabricated sewage treatment plant.

Electrical power will be provided from a local substation connected to the existing 138 kV transmission line running between Churchill Falls and Happy Valley-Goose Bay. Power is distributed around the camp on an overhead distribution system.

All building heating will be electric. Two diesel generators of 500 kW each will provide back-up essential power to the camp in the event of a power failure.

13.2.4 Camp Operation

The estimate is based on the assumption that the catering contractor will perform catering, housekeeping and janitorial services for the construction camp, recreation facilities and offices. This contractor will also operate the commissary, postal agency and tavern.

All building maintenance, snow clearing and utility operation and maintenance for the camp, recreation facilities and offices will be performed by other service contractors.

Medical services in case of illness or injury will be provided in the clinic. The clinic will be 45 m^2 in area and will be staffed 24 hours a day by nurses and daily with a medical doctor.

Medical emergencies which cannot be handled at the clinic will be taken to the Melville Hospital in Goose Bay and serious cases will be flown to a more advanced treatment centre.

13.2.5 General Support Facilities

The estimate includes a security force comprising five persons on duty at all times. Also included in the estimate is a fully equipped fire station and fire fighting personnel.

The following buildings will also be provided at the camp in support of construction:

- a 330 m^2 fire station and ambulance shelter;
- a 180 m^2 security building which will contain the site security office and the clinic;
- a 820 m^2 office building for Management and Engineering;
- a 150 m^2 laboratory;

- a 1,860 m^2 warehouse for the Owner's and Manager's use;
- a 195 m^2 workshop for camp maintenance; and
- a 100 m^2 administration/visitor information building.

The estimate includes site preparation and grading of 60,000 m^2, to be allocated to contractors for their workshops, warehouse and offices and 9,100 m^2 of graded area for open storage by the Owner or Manager. The Contractor's area will be provided with the necessary utility servicing.

Contractors will be responsible for the delivery, storage and distribution of their own fuel requirements at the project site. There is sufficient storage in Goose Bay and with the improvements to the Trans-Labrador Highway, it is unnecessary for the Owner to be responsible for fuel storage and facilities at the site.

Bulk cement will be delivered to and stored in Goose Bay and the project site by the cement suppler.

13.2.6 Construction Communications

Construction communication facilities, for the construction site, will generally include the following systems:

- digital telephone services for offices, support services and pay telephones. Negotiations will be undertaken with the local service provider for these services;

- cable television will be provided via a satellite receiver with distribution to all accommodation modules and recreational areas;

- digital data services will be provided in conjunction with the telephone system;

- mobile communications will be provided by a localized cellular style VHF system with a base station and unique call number vehicular remotes, along with a limited number of portable units. Coverage under this study is limited to the site, i.e. from construction camp to powerhouse to dam structure; and

- provisions will be included for video conferencing and other broadband facilities at the site.

13.2.7 Alternate Location

The cost estimate is based on locating the construction camp at the project site. If the camp location is selected in Happy Valley-Goose Bay, the Town's infrastructure is capable of handling an increase in population associated with a construction force of approximately 1,300 persons.

Savings in reduced construction costs associated with utilizing existing infrastructure and recreation facilities may, therefore, be realized by locating the camp in Happy Valley-Goose Bay. These cost savings would have to exceed any extra costs that occur as a result of locating the camp in the Town as opposed to the project site. Extra costs would be

infrastructure extension and upgrading costs, utility usage charges, building renovation costs and user fee contributions to recreation and other public service facilities. Other extra charges to be considered are transportation costs for bussing the workers to and from the project site and any travel time premiums that the workers would receive because of this travel.

The travel time premium is the most significant cost item to be considered in the selection of the construction camp location. If the workers are paid for their travel time the cost impact will certainly eliminate Happy Valley-Goose Bay as the alternate location. If worker travel time is not a cost item to the project, further study into locating the camp at Happy Valley-Goose Bay is recommended. The impact, if any, of workers travel time is unknown at this time. This item would be included in the negotiations of the project site labour agreement and its impact will only be known once these discussions have concluded.

It is anticipated that an order of magnitude cost savings in the range of $ 10,000,000 will be realized if the camp is located in Happy Valley-Goose Bay.

SECTION 14.0

CONSTRUCTION MATERIALS

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14.0 CONSTRUCTION MATERIALS

There are several classes of materials required for construction of the various components of this project. They include the following:

- impervious core material;
- granular filter material;
- rockfill;
- riprap and armour stone; and
- concrete aggregates.

Several borrow sources in the study area were explored to verify the potential of utilizing the native materials for various construction applications depending on their properties.

Results from the 1980 Engineering Report were used as a basis for this study, in particular for identifying target exploration areas. For consistency, the names of exploration areas (i.e. T-1, T-2, etc.) identified in the 1980 Report were kept for the present study. The locations of the material deposits and test pits excavated during the 1998 site investigation program are presented on Plate No. 24 in Volume 1 and Plates 29 to 32 in Volume 2, Appendix A.

A large proportion of till from deposits T-2 (west) and T-4A, and granular deposit GR-4 and all of the materials from deposits GR-1, 2, 3 and 6 were used during the Trans-Labrador Highway construction.

Maximum use will be made of suitable excavated material during construction to minimize the volume of materials to be disposed.

Refer to Volume 1, Appendix A for a list of material types and quantity requirements. The characteristics of the construction materials are described in the following sections and in more detail in Volume 2, Section 5.0.

14.1 IMPERVIOUS CORE MATERIAL

The impervious core material required for the freeboard dyke and cofferdams will come from glacial deposits referred to as glacial till or moraine.

In the Muskrat Falls region, glacial till is abundant. However, many of the deposits tend to be low in fines or are located in low lying wet areas or on wet hillsides. Care must be taken in designating a borrow area to ensure that the deposit is at or below its optimum moisture-density relationship.

In the 1979 investigations, three separate areas were identified as potential sources for the "impervious" core material. These are shown on Plate No. 24, as deposits T-2, T-4A and T-4B. Several other deposits investigated, including T-1, T-3 and the central and eastern parts of T-2 were determined to be too shallow and too wet for economic exploitation.

During the construction of the Trans-Labrador Highway more than 40% of deposit T-2 (west) has been used. In 1979 this deposit was one of the most promising till sources, comprising dry glacial till at a location less than 1.5 km from the Muskrat Falls access road. The deposit identified as T-4A in 1979 has also been largely exploited over more than 25% of its surface area.

The glacial till in the project area tends to vary widely in fines content from as low as 11% to over 40%. The fines have little or no clay size particles. For the present study, both the T-2 (west) and T-4 (divided into T-4A and T-4B) were explored in detail due to their close proximity to the project and their favorable fines and moisture content discovered during the 1979 and 1998 investigation programs. T-1, T-2 (east) and T-3 were explored further due to their relatively close proximity to the site but these till sources would most likely require artificial drying prior to use and thus are considered uneconomic to exploit.

The optimum moisture content and the corrected moisture content of the till deposits, based on the Standard Proctor test (ASTM D-698, methods A or C) was found to be between 6 to 9 percent and 7 to 10 percent respectively. The natural moisture contents in the materials above the water table are ± one percent of the optimum at deposits T-2 (west) and T-4A, and + 2 to 3 percent at deposit T-4B. The water table was observed to lie at a depth of about 5.6 m and deeper in the western section of the T-2 deposit. It was not observed within 6.0 m at deposit T-4B (long drumlin).

Deposit T-2 is estimated to contain a total useable material quantity of 200,000 m^3 compared to 100,000 m^3 for T-4A and 200,000 million m^3 at T-4B. Based on the high fines content, lower moisture content, and generally shorter haul, deposits T-2, T-4A and T-4B are recommended as prime sources of till.

Screening of oversized particles will be required for all till deposits.

It is recommended that a deep trench be excavated at the time of the pre-bid meeting at the site to give the potential contractors a better understanding of the borrow pit potential.

It is noted that some restrictions may have to be imposed on borrowing methods, such as mandatory borrowing from near vertical slopes in till and placing a limit on the axle load while hauling the material within the pit.

14.2 FILTER AND CONCRETE AGGREGATE SOURCES

Granular filter material required for the freeboard dykes and cofferdams and for the granular drains required to stabilize the slopes on the north spur may be obtained from deposits GR-4 and GR-5. The locations of these deposits are shown on Plate No. 24.

One of the most consistent deposits in terms of gradation is deposit GR-4. This deposit is generally shallow, in the order of 2 m deep but has a relatively constant gradation over a large area. The deposit comprises well graded sand with around 20% gravel. Within the study area this deposit is the best available in terms of fine aggregate for concrete production. However,

more than 50% of this deposit was used for the Trans-Labrador Highway construction and the remainder of this deposit is privately claimed.

One test pit was completed in the existing borrow pit. Visually, the sand and gravel soil is sound, washed, durable and with round to sub-round particle shapes which are considered to be favorable material for concrete aggregate or filter material. The aggregates along the Churchill River are not known as a particularly troublesome aggregate and deposit GR-4 appears to be a most promising source. However, the limited volume of, 200 000 m^3 and its distance from the Muskrat Falls site may preclude this deposit from development.

Deposit GR-5 is similar, in terms of grain size, to deposit GR-4. Based on limited test pitting and field examination, it was concluded that GR-5 could be extended to the west, thus increasing its size by a factor of 2.5 with an approximate volume of over 2 million m^3.

At this time, there is no access trail or bush road to the GR-5 area and it was not exploited for the new Trans-Labrador Highway. Access to this deposit will require construction of a road 1.5 kilometers long from the new Trans-Labrador Highway. The haul distance from the point of utilization at Muskrat Falls is approximately 12.5 km.

Materials for granular filter can be obtained from various sources at the site. The native till in the project area typically contains less than 10% cobbles and small boulders with a relatively low fines content in the range of 16 to 28%. Occasionally, the fines content may be as high as 45%. However, processing will be required, including both crushing and washing. The coarse or transition zone filter required may be derived from the crushing of quarry rock or excess shot rock from the various excavations at the site.

The ideal condition of one large borrow area to supply natural sand and gravel for concrete aggregate and filter material may not be possible at Muskrat Falls, although GR-5 has potential to provide both aggregate and granular fill requirements. Although several sources of sand and gravel have been identified and subjected to shallow test pitting in 1979 some of them (deposits GR-1, 2, 3 and 6 and 50% of GR-4) were used during the Trans-Labrador Highway construction. Additional investigation as outlined in Section 17.2.2.1 will be necessary in final design to finalize sources of granular fill.

14.3 ROCKFILL

Quarry run rockfill will be required for the cofferdam, and for the rockfill berms to stabilize the slopes of the spur of land. The rockfill material in the required gradation will generally be obtained from the open cut and tunnel excavations.

Four potential quarry sites, Q-6 and Q-7 from the 1998 investigation and Q-4 and Q-5 from the 1979 investigation, are located on the north and south shores of the River and shown on Plate No. 24. The quarry-run rock will only be exploited if the rockfill material from the open cut and tunnel excavations fall short of the quantity required. To provide rockfill for armouring the south bank prior to upstream cofferdam closure, quarry Q-4 will have to be opened to provide a moderate volume, 3,000-4,000 m^3, of rockfill.

If an additional rockfill source is necessary during construction, a quarry could be opened in the riverbed between the falls. The bedrock from this area should have the same properties as the remainder of the site.

14.4 RIPRAP AND ARMOUR STONE

The total quantity of riprap and armour stone required for the Project will be low and may possibly be provided by selective sorting of shotrock from the required excavations. However, as joint spacing in all rock in the Project area is relatively close, it may be difficult to produce sufficient large size rock for the cofferdam closure. In this case, it would be necessary to utilize one of the identified nearby quarries along the Trans-Labrador Highway.

14.5 RCC AGGREGATE SOURCES

Sand and gravel materials are required for the RCC dams. Deposits S-3, S-5 (Plate 24) and north spur deposits, typically contain fine to medium - grained sand in the main terrace (Terrace 1 on Plate 8) which covers most of the broad valley. Quantities are generally unlimited in terms of project requirement but it should be noted that occasional silt or even clay strata can be encountered which would require washing.

Laboratory analysis shows that the existing in-situ soils have a fine sand content in the order of 41% (S-5-2) to 98% (S-5-1). The estimated volume of these deposits is more than 7.5 million m^3, of which less than 3% of this volume is gravel. Gravel sizes would be provided by crushing shotrock from required excavations or quarries.

Gravel material is available at deposits GR-4 and GR-5. As explained in Section 14.2, these deposits are considered to be favourable materials for RCC dams and concrete aggregrate. But, access to deposit GR-5 will require the construction of about 1.5 km of road and further investigations for final acceptance will be necessary to prove volumes and the quality of this material.

SECTION 15.0

CONSTRUCTION PROGRAM

SECTION 15.0

CONSTRUCTION PROGRAM

15.0 CONSTRUCTION PROGRAM

15.1 CONSTRUCTION SCHEDULE

15.1.1 General

In accordance with the Terms of Reference for the study, an optimum Project Master Schedule was developed; the goal being to have the shortest, feasible schedule. The shortest schedule would provide the least cost with respect to construction and financing costs and feasible meaning that the Project is constructable within the time frame.

A key requirement of the optimum schedule would be to achieve river closure as soon as possible after the spring flood has passed. An optimum schedule was developed with a Project release of January 1 which would allow river closure by the end of July of Year 2. With this Schedule, first power would be available by October 1 of Year 5, 57 months after Project release. This schedule is shown on Plate 28 and is described in Section 15.2.

This schedule, with a June Project start, was compared to a schedule with a January 1 Project start, as requested in the Terms of Reference. The June start schedule offers the following advantages:

- summer mobilization for the critical diversion contract with lower cost due to better productivity in better weather conditions;
- summer closure of the river which would avoid any ice management problems, and would enable closure against lower river flows, thereby reducing the risk of delay and additional costs associated with a January start;
- summer closure would reduce potential delays in this critical schedule activity with less impact on downline contractors' schedules and costs and the first power date;
- A potential cost saving, since summer closure of the river would have less impact on lost revenue at the Churchill Falls Plant, due to reduced plant output to decrease river flow, at a time when the plant load demand would be less. This cost saving would be in the order of approximately $10 million to $15 million.

Based on the foregoing, with a reduced potential for additional costs and risks, SNC-AGRA recommends that the Project schedule be based on a Project start of June 1.

15.1.2 Recommended Schedule

The Project schedule for the Muskrat Falls Hydroelectric Development is shown on Plate 28, entitled Project Master Schedule. The schedule is based on a Project start date of June 1 of Year 1. First power would be October 1 Year 5, or 57 months after Project release (January 1 of Year 1). The final unit would come on line April 15 of Year 6. Some demobilization of the camp and other facilities would continue for a few months after this date.

The critical path for the project is through the upstream diversion portal and diversion tunnel, construction of the upstream cofferdam and diversion of the Churchill River through the diversion tunnels, the intake/powerhouse excavation, concrete of the service bay and unit 1, the erection of the service bay structural steel, the installation of the main powerhouse cranes, the installation of the first turbine-generator and the commissioning of the units.

Initial schedule constraints identified in the 1980 report have been reduced. The completion of the Trans-Labrador Highway to RCU 80 standards allows all-weather road access from central Canada to Happy Valley-Goose Bay. The permanent road connection to Happy Valley-Goose Bay allows quicker access to existing infrastructure. This includes such items as fuel storage, hospital, port and airport facilities.

In addition, due to the reduced activity at the air base, it is possible to make use of some of the accommodations at the base. The combination of the above factors allows construction of the diversion facilities to start prior to the first phase of the construction camp becoming operational.

15.1.3 Preliminary Work

To facilitate the schedule as much as practical, the following preliminary work is recommended:

- Tender and award of a contract for the "turbine-generators". This contract would be awarded about 2 ½ months prior to Project start (June 1 of Year 1). This will ensure the earliest start date for machine erection and allow for early input of the turbine-generator engineering into the powerhouse civil design.

- Tender and award of a contract for the 6/8 MVA construction power transformer. This contract would be awarded 2 ½ months prior to Project start (June 1 of Year 1). This will ensure delivery of this long delivery item in time for the completion of the construction power facilities.

- Tendering of a "preliminary works" contract. This contract could be awarded immediately after Project start (June of Year 1), allowing the successful contractor time to order materials to permit installation to commence in the fall of Year 1. This contract would be responsible for the supply and installation of the substation off the existing 138 kV line and the site distribution system. The contract would also include the supply and installation of material, equipment and buildings required for the first and second phases of the construction camp and main site roads.

- Tender and award of the "Diversion and North Spur Stabilization" contract immediately after Project start (June of Year 1). This will allow the contractor to mobilize and start work as soon as possible in the fall of Year 1. This work is essential to achieve river closure after the Year 2 flood.

15.2 CONSTRUCTION SEQUENCE

15.2.1 General

No special or unusual methodology is deemed necessary to construct this Project as all facilities are considered conventional. The methods and equipment utilized in the construction planning have been chosen to achieve the proposed schedule, while providing for efficiency and standardization of equipment fleets, timely access to the work areas and minimization of interfaces between contractors.

This section outlines the methodology and sequence of construction activities for each facility to achieve the recommended construction schedule.

15.2.2 Diversion

The diversion facilities are the first major work to be carried out and are on the critical path for project construction. Initial work will consist of constructing road access to the top of the inlet portal at El. 45.5 m.

During the geotechnical investigations, the main access road would be constructed as far as the adit portal. The exploratory adit would be built at the location of the construction adit. This work would be done during the engineering phase of the Project, prior to Project release.

Power will be supplied to the diversion contractor from the existing 2 MVA, 138/25 KV substation uprated to 2.6 MVA with the addition of cooling fans installed on the transformer. The amount of power available should be limited to 2.5 MVA until permanent construction facilities and the new 6/8 MVA transformer are installed.

Subsequent to clearing and excavation of overburden material, rock excavation will start on the inlet portal face at El. 45.5 m and proceed to El. 12 m, the invert level of the heading for the diversion tunnels. Initially, a rock plug will be left at the upstream end of the excavation to protect against flooding during the initial excavation. Initially the upstream end of the inlet portal will be excavated to grade. Subsequent to removal of the rock, but prior to opening the excavation right through to the inlet structure, a till plug will be placed across the excavation. This till plug can be more quickly removed at the time of diversion compared to a conventional in-situ rock plug.

The top heading of both tunnels will be advanced in parallel 50 m in from the inlet. A construction adit would be driven from the south side of the rock knoll. The adit would access both diversion tunnels. The adit is necessary to provide faster access to the downstream section of the diversion tunnels and allows an early start to inlet portal concrete without heavy traffic passing through the structure.

When the construction adit is completed, the tunneling operation shifts from the inlet portal to the mid-section of the diversion tunnels. The remainder of the upstream portal excavation below El.

12 m, would then proceed to the final grade of El. 3.5 m. The tunnel bench would then be advanced 50 m in from the portal.

When the adit is in use, the tunnel top headings would be driven in both directions simultaneously. Being shorter in length, the upstream drive would be completed first. The remaining section of the upstream benching between the adit and the portal would be excavated through the adit.

As soon as access is available at the downstream portal, the tunnel top heading operation would be moved to the downstream end of the tunnel. Upon completion of the tunnel excavation from the adit, the concrete plugs which close the construction adit would be built from the adit while the excavation is being completed in the tunnel from the downstream portal.

At the point the tunneling operation ceases at the inlet end, the inlet structure concrete work can begin. The inlet portal is a concrete structure 27 m high. The portal structure would be built in 11 lifts with a maximum height of 3 m. Each lift would be completed on a seven day cycle except for the roof of the water passage. This more complicated pour would require three to four weeks. Once the concrete structure is completed, the gate guides would be installed, aligned and concreted. Concurrently with the guides installation, the structural steel superstructure and operating platform would be erected on top of the concrete. Finally the operable gates would be installed. After testing, the gates would be left open and dogged in position to allow water to pass through the tunnels. The hoists would be completed and tested prior to mid-July of Year 2. Two gates are required for controlling the water level upstream of the Project. The remaining two gates would remain in the open position.

Concurrent with the upstream portal excavation and the initial tunnel operations, an access road will be constructed along the river bank to the downstream portal to permit the downstream portal excavation to begin. The road would be built as quickly as possible to provide immediate access. However, it will be necessary to raise the road to El. 20 m so the road will not be flooded as a result of increased water levels caused by ice damming downstream in the winter of Year 1/Year 2.

The downstream portal excavation will be time consuming. There is limited access to the upper benches. Since the downstream portal excavation is undertaken during the period when flooding as a result of downstream ice damming could occur, an initial bench will be cut at El. 14 m to provide access to the portal area. The upper bench or two of the portal excavation would be completed by drilling with jack-leg mounted rock drill. A small bulldozer or excavator would be winched up to the first bench and used to push the blasted rock over the side and down to the access bench at El. 14 m. This operation would continue level by level to El. 14 m. The excavation would continue behind a rock plug with a crest elevation of 7 m. At this point the access road and portal excavation would be lowered to El. -4 m, the invert level of the tunnel top headings.

As mentioned above, the tunnel top headings will be completed from the downstream portal excavation. Once the top headings are complete, the open cut excavation would be ramped down from El. -4 m to the tunnel invert at El. -12 m. The tunnel benches would then be

completed from the portal. Due to the limited access in the downstream portal, the excavation would then be backed out of the portal area. The upstream portal cofferdam and downstream rock plug will be lowered and breached to allow the river to start to flow through the tunnels. The rock plug and cofferdam would be completely removed prior to the high river flows.

Rockfill from the portals and tunnel excavation will be used to:

- provide the rock stabilization needed on the upstream and downstream faces of the North Spur;
- construct the access road along the river to the downstream portal; and
- be stockpiled for the upstream cofferdam. Some rock may be placed in groins partially closing the river prior to full diversion.

Upon completion of the inlet concrete and gate dry tests, the tunnel and the breaching of the inlet and outlet plugs, the first phase of the main cofferdam will be completed across the river diverting the Churchill River through the diversion tunnels.

Prior to closure, the right bank in the area of the main cofferdam would be protected by armour rock. A minimum of 10 m of groins would be constructed from the south shoreline bank. This is required to prevent major overburden scour during final closure. The 3,000 m^3 of rock would come from Quarry 5. Access to the right bank would be provided by a barge during the previous season.

The river would be partially closed by rock groins from the north bank. The groins would be extended across the river to effect closure. Filter material and impervious glacial till material will be end dumped between the two rock groins effecting the cofferdam seal. Other than the initial rock protection placed on the south bank, the cofferdam would be constructed from north bank borrow pits and stockpiles. Final closure would be achieved during the month of July of Year 2. Final closure of the river may be assisted by controlled flows from the Churchill Falls plant. This will reduce the time required to close. The cofferdam would be raised to its final elevation prior to the next flood.

15.2.3 Intake/Powerhouse

With the river diverted, the downstream cofferdam will be constructed on top of the lower falls. The river between the two cofferdams will be self-draining for the most part. The pool between the two falls will be dewatered by pumping. Maintenance pumping will be required to handle seepage through the cofferdams, precipitation and run-off.

The area of the powerhouse excavation has been scoured over the years by the annual flood. There is little in the way of overburden excavation in this area. The rock surface is above the downstream water level. Rock excavation for the intake/powerhouse can begin as soon as the water has drained from between the two cofferdams and an access ramp constructed. The bottom elevation of the powerhouse excavation is at El. -32 m in the draft tubes and -10 m in the

tailrace. A rock plug will be required in conjunction with the downstream cofferdam to maintain the works in a dry state. The downstream rock excavation, being a critical activity, will start in mid-July of Year 2 and proceed through the winter of Year 3. This will permit the concrete to start in the winter of Year 3.

Concreting would start in the Unit 1 draft tube and in the service bay sump area. This work would start about a month before the final rock excavation is completed in Unit 4. Each successive unit would start at one and one half month intervals. Intake concrete would start in parallel with the draft tube concrete. Concrete operations would continue until the Christmas shutdown. Concrete placing would resume in February. Winter placing will require the appropriate temporary protection and heating to provide a reasonable level of productivity.

Concrete placing would also start in the spring of Year 3 on the south transition structure and the gravity section upstream of the erection bay. The transition structure interfaces with the south RCC dam. The transition section must be finished prior to placement of the roller compacted concrete in the south RCC dam.

As soon as the powerhouse upstream and downstream walls in each unit bay are complete, structural steel erection would start with the main frame. First steel would be erected in the service bay. This will allow the installation of the powerhouse bridge crane on the crane rails as soon as possible. As the intake concrete is completed, the smaller steel members for the mezzanines would be erected.

The cladding and roof deck would be installed as quickly as possible to provide a weather tight working environment. A temporary partition would be installed at the open end of the unit bay to temporarily complete the enclosure. Steel erection would continue as the concrete in each unit bay is completed. The steel would be completely erected and the powerhouse completely enclosed by the end of July of Year 4.

The first unit will be ready for the erection of the turbine embedded parts in November of Year 3. When these are erected, the Phase II concrete, including the concrete semi-spiral case, would be completed. The erection of the turbine-generator for the first unit would start in September of Year 4. The unit would be ready for testing and commissioning in August of Year 5. The unit would be ready for first power October 1 of Year 5. Thereafter the installation of the turbine-generators proceeds with each unit coming on line at two month intervals.

As each intake is completed, the gate guides would be installed and concreted. Once access is available over the south dam, the intake gates would be erected from the intake deck. The draft tube gates and guides would be installed after the powerhouse structural steel is erected in each unit bay.

After the structural steel for two unit bays has been completed and access over the south dam is available, work would start on the substation structure. As the structure is completed, the electrical equipment would be installed and commissioned. The substation on the powerhouse roof will be completed in Year 5.

The intake/powerhouse will require a total of 38 months from the start of excavation to first power. From first concrete to first power will require 32 months.

15.2.4 Spillway and North RCC Dam

The spillway structure excavation will be excavated in conjunction with the intake/powerhouse excavation. The excavation is not on the critical path.

Concreting operations for the spillway structure would start in Year 3 or 4 but for the purposes of the schedule is done in Year 4. Depending on camp capacity and concrete production capacity, this work could be spread out over two seasons to lower the peak for the batch plant and camp demands. The main constraint would be the interface with the left abutment wall. This work should be started after the north RCC dam is sufficiently advanced so as not to cause interference with the concreting operations. The second consideration is the completion of the rock excavation in Unit 4 of the powerhouse. The right abutment concrete should not start until rock excavation is completed.

When the upstream section of the spillway piers, rollway and abutments are completed, the gate guides would be installed and concreted. The gate hoist superstructure and enclosure would be erected after completion of the spillway bridge. Until the north RCC dam is completed, access for the gate installation will be over the south RCC dam and the Intake. The three gates would be installed and tested by the end of Year 4.

The north RCC dam is a structure built with roller compacted concrete (RCC). As such, the structure can only be constructed during the summer months. As well, RCC cannot be placed in heavy rainfall. This limits the placing season from June to September.

The dam concrete would be placed in 30 centimetre lifts. Placing would continue 24 hours per day, seven days per week until the RCC is completely placed. Once the RCC is completed, the bridge piers would be constructed to allow the bridge to be installed to provide access onto the spillway and intake from the main access road. Placement of the structural concrete on the rollway crest and the upstream and downstream faces would follow RCC placement.

Once the dam crest concrete is completed on the north RCC dam, the rubber dam would be installed and tested.

The north RCC dam can be constructed either before or after the conventional concrete of the spillway. The decision would be left to the contractor to allow optimization of the construction plant and other resources.

15.2.5 South RCC Dam

With the completion of the south transition section in the summer of Year 3, the south RCC dam will be constructed in the summer of Year 3 or Year 4. For the purposes of the schedule, the work is shown in Year 3.

The south RCC dam should be constructed as soon as possible to provide access to the top of the intake. While not absolutely necessary, this will simplify the installation of the intake gates and the powerhouse roof substation.

15.2.6 North Spur Stabilization

The North Spur Stabilization program must be substantially completed prior to any impounding. This includes the partial impounding of the reservoir by the main cofferdam. Therefore, the majority of the work must be complete by the end of October of Year 2.

The stabilization work would be undertaken as part of the diversion contract. The construction of the various berms would be done in parallel with the rock excavation in order to minimize the rehandling of the rock. The pumpwells and crest unloading will be completed by October of Year 2.

15.2.7 Impounding

The final civil work will be the removal of the tailrace rockplug and the various cofferdams. The upstream cofferdam will be lowered 5 m to El. 26 m. This can be done when all the water retaining structures are completed and the spillway gates and the intake gates or stoplogs are completed. This would be achieved by the end of February of Year 5.

The water would continue to flow through the diversion tunnels until the downstream cofferdam and tailrace plug are removed. The downstream cofferdam and tailrace plug can be removed when all the spillway gates and hoists and all the draft tube gates and/or the temporary draft tube bulkheads are installed. This would be by the end of December of Year 4.

When the downstream plug is breached and the upstream cofferdam lowered, the diversion tunnel gates would be closed and the water passed through the spillway.

The diversion tunnel will drain to the normal tailwater level. Concrete plugs would then be constructed and grouted. Access to the work will be through the air vents provided in the inlet structures.

15.3 CONSTRUCTION PACKAGING

15.3.1 Type of Contract

The selection of an appropriate contracting strategy for the project depends upon the degree of uncertainty associated with scope, performance, schedule and cost. Whenever any work is to be contracted from outside the Client organization, a suitable contract needs to be formulated in detail.

The selection of the right form of contract requires:

- the identification of risks; and

- the determination of how the risks should be shared between the parties.

Contracts are generally identified first by the type of work and then by the methods of reimbursement:

a) The types of work are:

- on site construction;
- supply of materials or equipment;
- installation of materials or equipment;
- services only; and
- combination of any of the above.

b) The generic methods of reimbursement are:

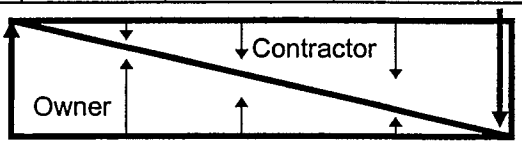
- actual cost plus various types of fee (CP);
- fixed price (FP) with various types of incentive;
- variable price (VP) with various types of incentives; and
- combination of the above on different parts of the project.

The type of contract selected is based on how much is known about the work to be undertaken.

- Work that is essentially undefined, and/or cannot be scheduled, is obviously highly uncertain, and therefore risky and is unlikely to proceed except on some form of cost reimbursable basis.
- Work that is fully described, that can proceed without interruption, be reliably scheduled and estimated and can be tendered and awarded on a fixed price basis.

Fig. 15.1 shows the relationship between different types of contracts and the allocation of risk.

FIGURE 15.1
TYPES OF CONTRACTS AND RISK SHARING

Scope of work information	very little		partial		almost complete
Risk and uncertainty	high		moderate		low
Type of probability	subjective				objective
Types of contract	CPPF	CPFF	CPIF	FPPI	FFP
Consequent risk allocation	100 % / 0 %				0 % / 100 %

Contracts range from those that are cost-based to those that are price-based.

Cost Plus Percentage Fee (CPPF) does nothing to encourage efficient working, and trust between Owner and Contractor is a key element.

Cost Plus Fixed Fee (CPFF) is somewhat of an improvement, since the fee is fixed and the more the work costs, the less is the effective mark-up received by the Contractor.

Cost Plus Incentive Fee (CPIF) is a risk sharing between the Contractor and the Owner on the unknown, believing that the Contractor has a better grasp on it.

Variable Price(VP) is between the two extremes. The type of contract provides some degree of risk sharing. An example of this type of contract is the Unit Price contract. This system provides a fixed unit price for a specified quantity of work where it is difficult to precisely define the quantity of work. The unit price for each quantity of work covers all costs, overheads, profits, and contingencies for all other factors except the quantity of work.

Fixed Price Plus Incentive (FPPI) is an incentive to the Contractor to find a better way of doing the job.

Firm Fixed Price (FFP) or Lump Sum (LS) contracts are at the other end of the scale. This is a system in which a Contractor offers to perform a specified amount of work for a fixed price. In such contracts, payments cover all costs, overheads and profit and the Contractor includes in the price all necessary contingencies for risks specified and/or implied.

A variation of the non lump sum contracts is the Target Price (TP). The target price tries to provide the Owner with some idea of what the maximum cost will be. The Unit Price (UP) contract can be considered a specialized form of the Target Price contract. The maximum target being tied to an estimated or guaranteed total quantity.

The unit price can be fixed or a function of the quantities. This is often done when the actual measured quantities vary by more than a reasonable amount. For example if the quantity is significantly lower than the bid quantity, then the contractor will not recover the fixed costs and

profit allocated to the unit price. If the quantity rises, either the contractor will make a windfall profit or it will be necessary to work overtime and/or bring additional resources to the project.
A point frequently overlooked on unit price contracts is that the unit price used may be based on important underlying factors not stated in the contract. A price per cubic meter of fill might be based on a certain quantity available from a convenient source; an increase in the quantity could result in excessive costs if other sources have to be used.

In writing a unit price contract, permitted changes without adjustment in the unit price, and factors that will require a change in the unit price, should be specified. Failure to make a provision for a quantity adjustment in unit price work is a prime condition for claims.
As mentioned previously, one factor influencing contract type is the concept of risk sharing. Cost plus fee contract places all the risk on the Owner. There is often no way with this type of contract to control what the final cost will be prior to the completion of the work.

At the other extreme, is the Fixed contract price. In this type of contract the contractor assumes all the risk associated with completing the work. This cost does not disappear. The contractor will make an assessment of the various risks he faces and apply an appropriate contingency.

For the Owner the advantage is that the final price is known. The main disadvantages for the Owner are:

- if the actual conditions are significantly different from the tendered conditions, there will be a claim; and
- if the risk condition does not materialize, the Owner has already paid the cost.

It is often the case on large projects such as this, that some form of risk sharing is in order. An example of this is escalation clauses. This type of clause allows the contractor some relief from inflation over a two or three year contract.

15.3.2 Construction Packages

The following considerations are made in the formulation of contract packages:

- nature of work (industry practices or degree of specialization);
- optimal size and content (too large and competent smaller Contractors may be excluded; too small and competent larger Contractors may not be interested.);
- optimization of effort and resources (must avoid duplication);
- local conditions (the effects of local support and infrastructure on costs);
- schedule constraints (network logic); and
- availability of funding (Owner's cash flow).

Table 15-1 gives the selected construction packages, the work comprised in those packages as well as the type of contract proposed at this stage. The proposed contract packages are but one possible alternative. The detailed scope and type of contract packages will be reviewed and developed further at the next phase of the project.

TABLE 15-1
CONTRACT PACKAGES

NO.	DESCRIPTION	SCOPE	TYPE OF CONTRACT
1	Preliminary Works	Supply and installation of camp buildings and temporary utilities, Construction power, Main Site Roads	UP/LS
2	Clearing	Reservoir and site.	UP
3	Diversion Works and North Spur Stabilization	All civil work for portals and tunnels, including concrete, installation of the embedded parts for the gates. North Spur Stabilization. Upstream cofferdam. Clearing excluded. Installation of hoists and gates.	UP
4	Diversion gates (if necessary)	Supply of embedded parts, hoists, gates, and stoplogs. Note: this contract would only be required if the Gull Island diversion gates were not available.	LS
5	Main Civil Works	All work for installation of downstream cofferdam, excavation of powerhouse and spillway structures. All civil work for construction of powerhouse, intake and spillway structures including foundation clean up, supplying and placing all concrete, superstructure steel, cladding, as well as embedded parts and supply and installation of powerhouse architectural works. Construction of north and south RCC dams, and construction of permanent powerhouse access road.	UP/LS
6	Balance Of Plant Mechanical Electrical Works	Supply of embedded parts, hoists and gates. Installation of gates and hoists for the intake, draft tubes and spillway. Supply and installation of the powerhouse cranes. Supply and installation of all mechanical and electrical systems. Supply and installation of building services.	UP/LS
7	Turbines/generators	Supply and installation.	LS
8	Camp operation	Operation and maintenance of camps, including catering and camp services. Decommission camp	UP/LS

NO.	DESCRIPTION	SCOPE	TYPE OF CONTRACT
9	Switchyard	Supply and installation of all electrical equipment, transformers and substation superstructure.	LS
10	Management and Engineering	Final design, project management and construction management.	CPIF

15.3.3 Owner Furnished Equipment and Materials

As indicated above, long lead time permanent or temporary equipment, such as turbines, generators and the construction power transformer will be included in the appropriate contract package. However, if it is deemed appropriate, specific equipment such as the main transformers could be separate supply contracts.

With the completion of the Trans-Labrador Highway and the increased infrastructure in Goose Bay, each contractor is assumed to handle its own bulk materials such as fuel, cement and reinforcing steel. However, for some commodities such as fuel, it may be appropriate to have a commercial concession at either the site or Goose Bay.

15.4 LABOUR SCHEDULE

A summary of the required labour force is shown in Figure 1.2. It indicates a peak workforce of 1,315 in the summer of Year 3. It is assumed that management staff will be living in Goose Bay, therefore, the peak load in the construction camp is estimated at 1215. Based upon this labor schedule the total number of person days of site labour is 1,150,000.

SECTION 16.0

COST ESTIMATE

SECTION 16.0

COST ESTIMATE

16.0 COST ESTIMATE

16.1 ASSUMPTIONS

The cost codes used for the 1980 and 1989 Muskrat Falls reports, as well as the 1997 Gull Island report, are maintained for this Final Feasibility Study.

The capital cost estimate is based solely on the scope of the final layout selected and does not rely on any scope or costs from previous estimates. The estimate reflects improvements that have taken place in the construction industry since the 1980 report. In particular the cost estimate includes improvements and/or productivity gains in:

- design and operation of hydraulic drills used for rock excavation and rock tunneling;
- design and operation of large backhoe's used for excavating and loading material. In many situations a large backhoe is replacing the more traditional front-end loader and bulldozer combination;
- water resistant, bulk loaded emulsified explosives;
- concrete batching and placing methods; and
- Roller Compacted Concrete (RCC) batching.

The estimate, the incurred cost flow and labour force are presented in detail in Appendix A.

Volume 3 presents the detailed backup of the capital cost estimate.

The cost estimate is based upon the salient project data included in Section 7.3.

16.1.1 Basis of the Estimate

The basis of the estimate is as follows:

- all costs are expressed in June, 1998 Canadian dollars.

The following cost items are excluded from the estimate:

- escalation after June 30, 1998;
- interest during construction;
- construction "all risk" and other insurances which are provided by the Owner;
- any applicable sales taxes including HST and GST;

- land acquisition, easements, rights-of-way, compensation to land owners or lease holders and other similar items of cost;
- the Owner's operational and pre-operational expenses including supervision, operator training, staff build-up for the operational phase of the Project, operating supplies and plant start-up;
- cost reduction due to further optimization studies;
- the Owner's general and administrative contingency and expense items, including interest during construction (IDC) and working capital requirements;
- permits and licenses;
- engineering, field exploration and construction performed prior to the preparatory activities (previous studies, etc.);
- environmental and/or fisheries requirements that are more stringent than those described in the 1980 report;
- Environmental Baseline Studies and Environmental Impact Statement (EIS);
- archeological study;
- changes and/or additions to laws and in the regulations by governments and/or their agencies after June 30, 1998;
- 230 kV transmission lines including OPGW; and
- aboriginal land claims.

16.1.2 Field Investigation

The 1998 Field Investigation program has augmented previous investigations undertaken in 1974 and 1979. There now exists a large body of information upon which to base the design and capital cost estimate for this Final Feasibility Study. Additional field work necessary to move to final design of the project is outlined in Section 17.0.

16.1.3 Support Facilities and Transportation

16.1.3.1 *Construction Roads*

For estimating purposes, it is assumed that the Trans-Labrador Highway will be available for year-round use for all project vehicular transportation requirements, and that all normal road maintenance, including snow clearing, will be performed by the Provincial Department of Works, Services and Transportation. Accordingly, no allowance has been included for use or

maintenance of this road. No allowance has been made for the cost of widening this road at selected locations between Happy Valley-Goose Bay and Muskrat Falls to provide pull-outs to allow public vehicular traffic to pass around any slow moving, heavy lift convoys. The cost for this road widening is already included in the estimate for the Gull Island Hydroelectric Development which is expected to precede the Muskrat Falls Hydroelectric Development.

The existing access road to the site will be upgraded to RLU70 design standard. A new permanent access road will be constructed around the south side of the rock knoll connecting to the bridge deck over the north dam and then over the south dam to the repair bay entrance of the powerhouse.

A temporary access road will be necessary to connect the upstream portal of the diversion tunnel with the adit and downstream portals. This road, constructed high enough to avoid winter flooding problems, will be the main access to the site for most of the construction period. Additional construction roads will access the south dam and the powerhouse excavation as well as concrete and RCC batching plants, located on the south shore near the south RCC dam.

Site construction roads are included in the direct costs estimated for construction.

16.2 PRICING DATA

16.2.1 Basic Rates

16.2.1.1 *Labour*

It is anticipated that the Owner will enter into a Project agreement stating the conditions of work, wage rates and benefits. For the purpose of this estimate, existing agreements in Newfoundland and Labrador have been reviewed and a blended rate for wages, conditions and benefits has been adopted for use in all the current Churchill River hydroelectric development studies.

The labour rates and conditions adopted and used for this estimate are based on:

a) basic rate, overtime premium, vacations and holidays, social benefits, workers compensation, pensions, employment insurance and other benefits; and

b) a 60 hour week, 50 hours paid regular time and 10 hours double time.

16.2.1.2 *Construction Equipment*

The construction equipment hourly rates are based on new equipment purchase cost and the depreciation period in hours suggested by manufacturers. A standard listing of equipment rates has been adopted for use in estimating all current Churchill River hydroelectric developments.

The hourly equipment rental rates utilized in the estimate include: purchase cost, financing, insurance, fuel and lubrication, normal maintenance, major overhaul, tires and other fixed or variable costs.

16.2.1.3 *Construction Materials*

Construction material prices were obtained from Newfoundland and central Canada suppliers with the lowest of these prices utilized in the cost estimate. A standard listing of material rates has been adopted for use in estimating all current Churchill River hydroelectric developments.

16.2.1.4 *Permanent Equipment*

The supply and installation costs for the turbines and governors, generators and exciters, main transformers and main switchgear were obtained based on manufacturer estimates and SNC-AGRA experience.

The costs for intake gates, diversion gates, spillway gates, trash racks and embedded parts, etc. were taken from an in-house data bank and adjusted to reflect the site conditions.

The costs for transformers, main switchgear and auxiliary mechanical and electrical systems were obtained from an in-house data bank.

16.3 ESTIMATING METHODOLOGY

To prepare the detailed capital cost estimate, included herein, the entire scope of work was analyzed and separated into identifiable components. For each component, the construction labour team, the construction equipment fleet and the construction materials were selected to achieve the best productivity and to complete the task within the construction schedule.

Normal market conditions were assumed (no shortage and no over supply situation for labour, equipment and supplies).

16.3.1 Bid Unit Costs

The unit costs presented in the cost estimate represent an estimate of what a qualified contractor would tender for the work in a competitive bidding situation. The estimated bid unit costs are composed of both direct and indirect costs.

16.3.2 Direct Unit Costs

The direct costs include all expenses incurred in the field for a specific construction activity and include the construction labour crew, construction equipment and construction materials.

16.3.3 Indirect Costs

The indirect costs include the expenses incurred by the Contractor which do not directly apply to specific construction activities, but are required and necessary to support and supervise the construction work.

The following expenses are considered as indirect costs:

- construction supervision;
- field engineering and administration;
- site offices, warehouses, shops, garages, etc. and operation;
- support vehicles, equipment and operation;
- construction power;
- transportation of employees;
- general consumables;
- small tools;
- security, safety, health and first aid services;
- mobilization and demobilization;
- bonds and insurances;
- head office overhead;
- short term financing;
- contractor's profit; and
- contractor's contingency.

These indirect costs are evaluated for each contract package and transformed into a contractor's mark-up which is applied to the direct unit costs to obtain the bid unit costs.

16.3.4 Computerized Estimation System

The capital cost estimate is presented in a computerized format.

- The coding system of the 1980 and 1989 Muskrat Falls Capital Cost Estimates, as well as the 1997 Gull Island Capital Cost Estimate has been re-used.

- The labour rates, construction equipment rates and the construction material unit costs, as of June, 1998 are introduced into the program data bank.

- The resources required to achieve a specific construction activity are itemized on a work sheet in conjunction with an estimated rate of production to arrive at the estimated direct unit cost.

- The direct unit costs are multiplied by the contractor's mark-up to produce the bid unit cost.

- All the construction activities are summarized per structure in accordance with the code of accounts.

16.3.5 Risk Evaluation and Contingencies

Only risks mitigated by contingencies are presented hereafter.

16.3.5.1 *Contractors' Contingencies*

The contractors' estimates were developed assuming a reasonable construction period and that sufficient information is available to properly evaluate the work and to develop competitive estimates. Therefore, the contingencies carried are an assessment of risk elements within these criteria and not for scope changes.

The contractors' contingencies are included in the direct construction cost estimates. They are based on the normal risk exposure that the contractor faces due to weather restraints, labour availability and productivity and normal contract clauses on risk sharing.

16.3.5.2 *Project Contingency*

The Project contingency reflects the quality and amount of information available to the Owner and Manager for the preparation of estimates and schedules. It does not include any impact of cost increases due to inflation and/or market conditions at the time the project is in progress.

The method of evaluation is an item by item review of available information resulting in a contingency which represents an assessment of the Owner's exposure. The geological and geotechnical ground conditions, weather, dewatering, construction labour are examples of items reviewed in the discussion of the contingency. The contingency was not reduced to account for potential optimization.

When the Project is authorized, it is recommended that further analysis be carried out to review some of the exposure expressed under the Project contingency, including the impact on the

market of other possible large construction undertakings at the same time and their effect on the Project.

The Project contingency in this estimate does not allow for the possibility of more stringent environmental requirements than those described in this report. Default by Suppliers or Contractors was not considered.

The contingency carried in the estimate, (code 9800) is the maximum of three approaches:

- based on risk evaluation as per Section 16.3.5.3;
- based on specialist appreciation of their respective items;
- based on a standard 10 % of construction cost.

The latter has been included in the Project estimate.

16.3.5.3 *Risk Evaluation*

A risk evaluation was carried out to highlight the items that still have an element of uncertainty due to lack of field information and time for optimization. The gathering of the required information and optimization are recommended for the next phase of engineering. The information to be gathered and the optimization studies planned are described in Section 17.0 of the report.

The risk items are identified and evaluated and presented in Table 16.1 entitled "Risk Evaluation". The table presents the exposure value, which is a judgmental evaluation of the portion of the item cost exposed to the identified risk, and the probability of occurrence is established to arrive at a suggested contingency value.

16.3.6 Insurable Risk

The of risks which are insurable can be selected from a list of possible risks to the project and duly insured.

For example:

a) Direct property damage to:

- installation during construction;
- equipment, in transit or handling, etc.;
- project materials, including theft; and
- contractor's property.

TABLE 16.1
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
RISK EVALUATION

RISK EVALUATION		COST VALUE (X $1000)	EXPOSURE %	EXPOSURE Value (X $1000)	PROB. %	CONTINGENCY (X $1000)
DIVERSION						
Tunnels	Rock Quality	23,000	50%	11,500	33%	3,800.0
Cofferdam	Quantity Variation	3,800	25%	950	50%	475.0
Inlet Gates	Gate Jammed	600	100%	600	10%	60.0
RCC DAMS						
RCC	Quantity	30,000	25%	7,500	25%	1,875.0
Foundations	Rock Quality/Foundation Prep./Grouting	3,000	50%	1,500	50%	750.0
Roller Compacted Concrete (RCC)	Cement Content	1,000	100%	1,000	50%	500.0
SPILLWAY						
Foundations	Rock Quality/Foundation Prep./Grouting	500	50%	250	50%	125.0
Structure	Change in Design	1,000	100%	1,000	25%	250.0
Structure	Dimensions	33,000	20%	6,600	25%	1,650.0
Mechanical/ Electrical	Design Changes, Guides Installed Incorrectly	8,200	10%	820	20%	164.0
NORTH SPUR						
Excavation & Fill	Quantity Variation	18,000	40%	7,200	40%	2,880.0
POWERHOUSE						
Powerhouse/ Intake Structure	Rock Quality/Foundation Prep./Grouting	10,000	50%	5,000	50%	2,500
	Overbreak/Backfill Concrete	36,000	20%	7,200	30%	2,160.0
	Modified Cement	10,000	100%	10,000	10%	1,000.0
	Turbines & Generators	145,000	10%	14,500	75%	10,875

RISK EVALUATION		COST VALUE (X $1000)	EXPOSURE %	EXPOSURE Value (X $1000)	PROB. %	CONTINGENCY (X $1000)
	Spare Parts	6,800	50%	3,400	50%	1,700.0
Tailrace Plug Excavation	Rock Quality	6,000	20%	1,200	30%	360.0
COMMUNICATIONS						
Microwave		1,900	50%	950	50%	475.0
CAMP & SUPPORT FACILITIES						
Camp and Support Facility		112,000	25%	28,000	25%	7,000
Trans-Labrador Highway	Additional Maintenance	6,300	50%	3,150	10%	315.0
MANAGEMENT		80,000	5%	4,000	15%	600.0
ENGINEERING		41,000	10%	4,100	20%	820.0
Field Investigation and Hydraulic Models		2,500	50%	1,250	50%	625.0
Contract Packaging		965,000	10%	96,500	25%	24,125.0
Labour	More Expensive Agreement	350,000	15%	52,500	15%	7,875.0
ENVIRONMENTAL						
Reservoir Clearing		37,600	20%	7,520	25%	1,880.0
Fisheries Compensation		5,000	100%	5,000	20%	1,000.0
TOTAL						75,839.0

b) Indirect consequential loss for:

- cost of removing direct loss debris (clean up);
- equipment replacement;
- liquidated damages;
- increased financing.

c) Legal liability for:

- public bodily harm;
- property damages arising from the negligence of others;
- personal injury arising from the negligence of others;
- damage to the project entity due to design errors, execution errors and project failure to perform as specified.

d) Personnel costs related to:

- employee bodily injury;
- cost to replace employee.

16.3.7 Cost Flows

All construction activities are dated in accordance with the Project Master Schedule to generate the following monthly cost flows:

- the general project cost flow; and
- cost flows for construction labour, construction equipment and construction materials at direct cost level and at bid cost level.

16.4 OPERATIONS AND MAINTENANCE

16.4.1 General

Operations and maintenance costs (O&M) have been estimated in order to complete the economic evaluation of the Project.

The cost estimate is based on the establishment of an "operations and maintenance" team centered in Happy Valley-Goose Bay, Labrador, whose duties will include the provision of all operations and maintenance requirements for both Gull Island and Muskrat Falls plants.

This estimate includes direct costs attributable to Muskrat Falls plus the Muskrat Falls Project portion of those general technical, managerial and overhead costs to be shared by both Muskrat Falls and Gull Island plants.

16.4.2 Assumptions

The O&M cost estimate is based on the following assumptions:

- the organization and cost evaluation is modeled on large hydroelectric plants, and will include the all facilities up to the termination lines in the switchyard located on the roof of the powerhouse;
- the maintenance, administration, technical and managerial employees will work a 5 day week;
- operations staff will work four 12-hour shifts per week with 3 days off. Only one operator will be on duty at a time;

- operations staff will reside at site in the Owner's permanent accommodations during their 4 day work period;
- short-term maintenance crews will commute from Happy Valley-Goose Bay. If the duration for maintenance extends for more than one day, the crew will be housed at site in the Owner's permanent accommodations. Travel time from Goose Bay to site of 30 minutes each way has been allowed;
- the cost of the power station manager, technical and administrative support will be shared 30% by Muskrat Falls and 70% by Gull Island;
- a provisional fund, or interim replacement fund, will be established which will accumulate each year as a future provision for unscheduled major maintenance and purchase of major replacement items;
- costs for provision of the Regional Control Center in Goose Bay are not included in the capital cost estimate;
- costs for provision of a 10 persons permanent accommodations building at Muskrat Falls site are included in the capital cost estimate; and
- calculation of non-productive days per employee:

a) General

Annual holidays	25 days
Legal holidays	10 days
Sickness	8 days
Training	10 days
Others (including inclement weather)	4 days
Total non-productive days	57 days

b) The employees involved with maintenance (clerks), administration, technicians and management will be working 5 days a week (40 hours).

- Productive person-hours = 40 hrs/week x 52 weeks - 57 days x 8 hr/day - time allowed for transportation to Goose Bay (1 hr/week x 41 weeks) = 1,583 hours

- Maintenance at Muskrat Falls is assumed to require:

– Limited Maintenance	4 units at 3 days/unit/year	12 crew days
– Complete Maintenance	4 units at 15 days/unit/year	60 crew days
– Breakdown repair evaluation		24 crew days
– Total annual maintenance		96 crew days

16.4.3 Organization

The organization of the operations and maintenance team to cover both the Muskrat Falls and Gull Island projects is shown in Figure 16.1.

16.4.4 Permanent Accommodations

A permanent accommodations building will be provided for 10 single status personnel. This building will be pre-engineered and contain a floor area of 400 m^2. The building will contain single occupancy bedrooms with a shared washroom between two rooms. The building will also have a kitchen/dining area and a recreation/lounge area. Potable water will be supplied to the building from a drilled well and sewage will be treated in a septic tank and tile field.

16.4.5 Cost Estimate

In order to optimize the use of human resources and facilities, one organization complete with the necessary specialists will serve both Muskrat Falls and Gull Island Projects. It will be located at Goose Bay within the same facilities used by the Regional Control Center.

If a separate organization is required for Muskrat Falls, the cost of staffing and support facilities will increase by 15% to 25%.

Table 16.2 presents the validation of annual provision for interim replacement of unscheduled major maintenance and capital expenditures. It should be noted that the Operations and Maintenance Budget has been prepared with the inclusion of the expected level of annual operation and maintenance expenses for a project of this size (with some cost sharing with Gull Island Hydroelectric Development). For the feasibility level budget, an amount equal to $1.6 million per year has been included as an annual provision for an interim replacement fund to provide for the future funding of unscheduled major maintenance and capital expenditures.

The interim replacement amount for the feasibility level estimate has been determined somewhat arbitrarily, but with some consideration of a near-worst series of major maintenance and equipment replacement events over a 25 year period, using a real (non-inflation) interest rate of 3%.

Attached is Table 16.2 listing this series of major repairs and equipment replacement, and the estimated cost of each, in 1998 dollars. The sum of these events total $58,357,000.

ORGANIZATION - GULL ISLAND AND MUSKRAT FALLS POWER STATION

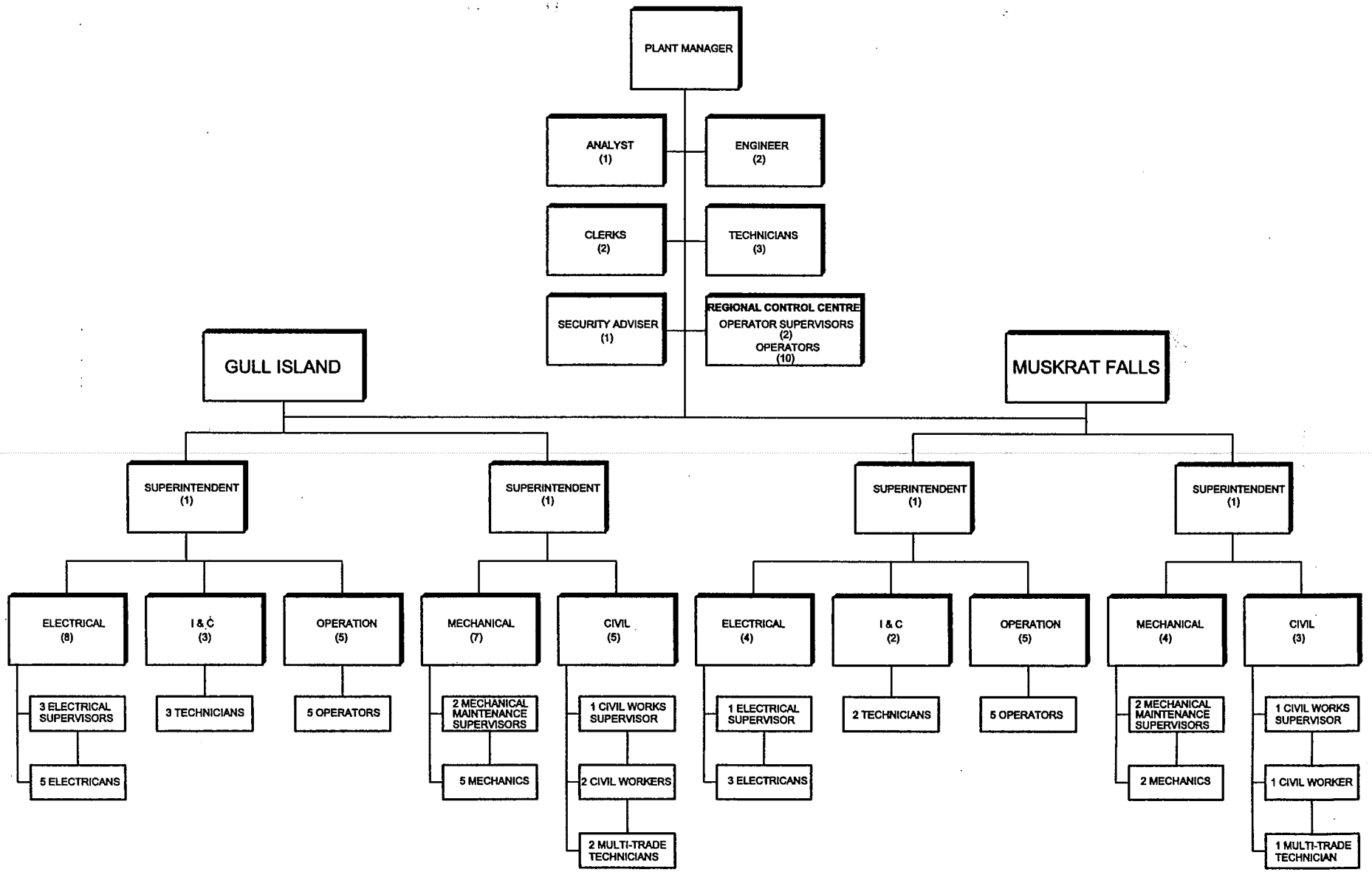


Figure 16.1

At the detailed design stage, when final field information details the actual materials to be used in the Project, and detailed information is available from likely suppliers relating to long term reliability of equipment, a detailed risk analysis could be produced which may very likely suggest that the annual provision for the interim replacement fund be reduced, yet still provide a future fund sufficient to cover a realistic series of major maintenance events.

The following Table 16.3 (Annual Trades Personnel Evaluation) shows the derivation of the number of maintenance staff shown in Table 16.4.

Table 16.4 presents the annual cost of operations and maintenance resources required on an annual basis as derived from Table 16.3. Table 16.4 also includes operations staff, supporting expenses and allowances for tools and future major maintenance expenditures.

The total annual operations and maintenance cost is estimated to be $5.9 million.

Table 16.2
Validation of Annual Provision for Interim Replacement of Unscheduled Major Maintenance and Capital Expenditures

Account	Area Activity	Total Cost $x1000	Occurrence in 25 years	Percent Applicable %	Cost Applicable $x1000
1112	Access Road	2,885	1	15%	433
2300	RCC Dams				
	Repair of concrete deterioration	1,000	1	100%	1,000
	Repair of bridge deterioration	500	2	100%	1,000
	Rubber Dam				
	Replacement	1,600	1	100%	1,600
	Rubber dam repairs	100	4	100%	400
2400	Spillway				
	Gate replacement	6,000	1	33%	1,980
	Major concrete repair	20,000	2	10%	4,000
	Control excessive rock erosion	1,000	2	100%	2,000
	Heater repairs	50	2	100%	100
	Hoist replacement	50	2	100%	100
2800	North Spur				
	Replace pumpwells	2,000	1	100%	2,000
	Repair of slopes	5,000	1	100%	5,000
3100	Intake				
	Trashrack replacement	1,000	2	100%	2,000
	Gate/hoist replacement	1,000	1	100%	1,000
	Trash removal	300	4	100%	1,200
3300	Powerhouse				
	Roof/siding replacement	3,000	1	50%	1,500
	Overhead crane replacement	3,800	1	50%	1,900
	Elevator replacement	400	1	50%	200
	Parking lot repairs	100	2	100%	200
3400	Turbines and Generators				
	Generator	17,000	1	50%	8,500
	Turbine	21,500	1	50%	10,750
3500	Electrical Ancilliary Equipment	11,000	1	25%	2,775
3600	Mechanical Ancilliary Equipment	12,000	1	25%	3,000
3700	Generator Transformers	4,170	1	100%	4,170
5200	230 kV Switchyard	3,840	1	25%	960
6500	Telecommunications	1,965	2	15%	590
	TOTAL				58,357
	Annual Provision for Interim Replacement of unscheduled major maintenance and capital expenditures.	1,600	at 3%, 25 yrs.		58,357

Note: All prices shown are 1998 base year estimates. Future value of annual provision for interim replacement contributions is determined using real interest rate of 3%, which excludes inflation.

Table 16.3
Annual Trades Evaluation - Hours Per Year
(exclude management and supervisory personnel)

Description	Disciplines			
	Mechanical	**Electrical**	**Civil**	**Instrumentation and Controls**
A) Plant O&M Labour				
Annual maintenance of 4 units	1,600	1,000	600	400
Maintenance of auxiliary systems (water, air, electrical distribution)	1,000	830	70	300
Auxiliary equipment (cranes, gates and others)	1,100	700	300	200
Maintenance waterworks intake and spillway	400	300	400	150
Servicing	1,600	1,500	200	300
Miscellaneous	1,000	700	700	450
Building maintenance, roads, dam, etc.			1,090	
Calibration and instrumentation maintenances and others				800
B) Switchyard O&M Labour				
Annual Maintenance	-	1,500	-	-
Total Production Persons For Plant & Switchyard - Hours/year (A + B)	**6,700**	**6,530**	**3,360**	**2,600**
Number of persons required per specialty	**4**	**4**	**3**	**2**

Note: The overtime rate is 6% and is included in this estimate. Most of the overtime results from unscheduled maintenance.

Table 16.3 presents the annual cost of operations and maintenance.

Table 16.4
Annual Cost of Operations and Maintenance Resources

Description	Cost $	% Utilization	Annual Cost
A) PLANT O & M LABOUR COSTS AND BENEFITS			
1 Plant Manager	102,000	30	30,600
2 Engineers	168,000	30	50,400
1 Analyst	78,000	30	23,400
1 Security Advisor	78,000	30	23,400
2 Clerks	92,000	30	27,600
3 Technicians for Operations (multi-trades)	210,000	30	63,000
2 Maintenance Superintendents	158,000	100	158,000
1 Electrician Supervisor	63,000	100	63,000
2 Electricians	120,000	100	120,000
2 Technicians (Inst. & Control)	140,000	100	140,000
5 Operator	330,000	100	330,000
2 Mechanical Maintenance Supervisors	126,000	100	126,000
2 Mechanics	120,000	100	120,000
1 Civil Works Supervisor	57,000	100	57,000
1 Civil Worker	55,000	100	55,000
1 Technician (multi-trades)	57,000	100	57,000
Sub-total (A) Includes Labrador Allowance			**1,444,000**
B) SWITCHYARD O&M LABOUR COSTS			
1 Electrician **(sub-total (B))**	60,000	100%	**60,000**
C) REGIONAL CONTROL CENTER LABOUR COSTS			
2 Operator Supervisors	146,000	30	43,800
10 Operators	680,000	30	204,000
Sub-total (C) Includes Labrador Allowance			**$247,800**
D) EXPENSES			
Vehicle[1]			109,300
Equipment (maintenance only)[2]			218,200
Services[3]			312,200
Meals and Accommodations[4]			630,000
Transportation Fees[5]			14,900
Sub-total (D)			**$1,284,600**
E) SUPPORT FEES[6] (E) = ((A) + (B) + (C)+ (D))x40% = (Sub-total E)			**$1,214,560**
F) ANNUALIZED FUNDS			
1 Maintenance Tools[7]			50,000
Annual Provision*			1,600,000
Total Operations and Maintenance Estimate of (A) + (B) + (C) + (D) + (E) + (F)			**$5,900,960**

* A provisional amount is included to be used for interim replacement of materials and/or equipment for major components and capital expenditures of the project.

[1] An average of 7% of the labour cost is used to estimate the vehicle costs (same as in some equivalent installations).

[2] For an equivalent installation, the cost is around 12 to 16% of labour costs (14% used in the calculations).

[3] Includes snow clearing, road maintenance and costs for caretaker, etc. Normally, this cost is from 15 to 20% of the labour cost for a similar installation. 20% is used in the calculations.

[4] $125/day/person is allowed to cover the cost of meals and accommodations.

[5] $0.70/km is allowed for the transportation of 4 persons in the same vehicle at the beginning and end of each work schedule. Transportation between the plant and accommodations facilities is included in this cost evaluaton.

[6] Support costs include all services required for the operation of this type of organization. These costs include purchase of services, vehicle, accounting services, etc. The support cost is estimated at 40% of the total cost.

[7] A one-time provision of maintenance tools in the amount of $500,000 is annualized to be approximately $50,000.

16.5 COST REDUCTIONS

Throughout the engineering design for this Final Feasibility Study, the preliminary design for many of the project structures was changed from the original layout of Variant 7, to effect cost reductions.

The design changes are consistent with changes in technology over the past twenty years and are based upon proven technologies and experience with existing projects.

In addition, other significant areas of cost savings from the original layout were identified related to project logistics and the construction camp.

As a result of the design and logistics changes noted above, the estimated capital cost in the Final Feasibility Study is lower than the cost for Variant 7, as presented in Table 6.3, "Relative Cost Comparison of Variants 5, 7, 10 and 11", Section 6.5. A detailed comparison of the present CCE with Variant 7 is shown in Table 16.5. The main reasons for the differences indicated in Table 16.5 include:

- reservoir clearing is greatly increased from the 1980 report as the entire reservoir is now being cleared versus partial clearing previously;

- optimizing the diversion tunnel configuration has allowed for smaller tunnels with less excavation;

- the combination of RCC overflow dams and a much smaller spillway in the Final Feasibility Study is $50 to $60 million cheaper than the rockfill dams and large spillway option in the original Variant 7;

- the installation and operation of the construction camp and related facilities, Cost Diversion 8000, is estimated to cost much less in the Final Feasibility Study than in the cost comparison of original Variant 7 and Final Feasibility Study layout. The main reasons for this are;

    - The costs used for Variant 7 were extracted from previous studies and cost estimates, in particular the 1980 and 1989 reports (References 21 and 35) and simply escalated to $1998. These costs were previously presented in Table 6.3, Section 6.5, wherein the relative costs of Variants 5, 7, 10 and 11 were compared.

    - The Capital Cost Estimate developed for the 1980 Report and in particular Cost Division 8000, Construction Facilities and Support, assumed an isolated site which required a major involvement by the Owner in the provision of infrastructure and support facilities. This was evidenced by the camp amenities and the costs included for transporting and distributing all major construction materials including cement, fuel, reinforcing steel, turbines, generators, gates, etc.

Table 16.5

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Cost Comparison of Original Variant 7 & Final Feasibility Study

Item	Description	Unit	Variant 7[(1)] (Div.Tunnels, PH & Spillway in River)			Final Feasibility Study			Difference
			Qty	Unit $	Cost	Qty	Unit $	Cost	$
1	**CONSTRUCTION COSTS**								
1000	**Permanent Support Facilities**				**2,200,000**			**$ 6,171,000**	**3,971,000**
2000	**Reservoir, Div., Dam & Spillway**								
	- 2100 - Reservoir Clearing	**Ha**	421	$ 10,000	**$ 4,210,000**	3,760	$ 10,000	**$ 37,600,000**	**33,390,000**
	- 2200 - Diversion / Water Control								
	- Access Roads		0	0	0		L.S.	$ 824,000	824,000
	- Portals - Overburden Exc.	m3	10,000	$ 15.00	$ 150,000	28,000	$ 9.05	$ 253,400	103,400
	- Portals - Rock Exc	m3	230,000	$ 35.00	$ 8,050,000	256,000	$ 24.43	$ 6,254,000	(1,796,000)
	- Inlet Structure Conc incl rebar	m3	11,000	$ 720.00	$ 7,920,000	9,460	$ 658.88	$ 6,233,000	(1,687,000)
	- Tunnel Plugs incl rebar	m3	20,000	$ 483.00	$ 9,660,000	4,100	$ 533.66	$ 2,188,000	(7,472,000)
	- Tunnel Excavation	m3	430,000	$ 70.00	$ 30,100,000	310,000	$ 74.79	$ 23,184,900	(6,915,100)
	- Gates, Guides etc	L.S.	-	L.S.	$ 8,286,000	670,000	$ 8.13	$ 5,447,100	(2,838,900)
	- Portal Inlet/Outlet Plugs	m3	50,000	$ 51.52	$ 2,576,000		L.S.	$ 1,032,000	(1,544,000)
	Sub-Total - 2200 - Diversion				**$ 66,740,000**			**$ 45,416,000**	**(21,324,000)**
	- 2300 - Dam & Cofferdam				$ -				-
	- Access Roads & Dewatering		- n/a				L.S.	$ 4,255,000	4,255,000
	- Cofferdam - Fill (Rock,Till, Filter etc)	m3	255,000	$ 25.00	$ 6,375,000	310,060	$ 12.09	$ 3,749,000	(2,626,000)
	- Dam Fill - (Rock, Till, Clean Up etc)	m3	340,000	$ 30.00	$ 10,200,000	- n/a		$ -	(10,200,000)
	- Concrete Gravity Dam	m3	22,000	$ 450.00	$ 9,900,000	- n/a		$ -	(9,900,000)
	- RCC Dams, Rubber Dam & Bridge		- n/a			148,200	$ 282.05	$ 41,800,000	41,800,000
	- Overburden Exc./Cofferdam Removal	m3	115,000	$ 15.00	$ 1,725,000	142,000	$ 8.39	$ 1,192,042	(532,958)
	Sub-Total - 2300 - Dam & C'Dam				**$ 28,200,000**			**$ 50,996,000**	**22,796,000**
	- 2400 - Spillway								-
	- Overburden Excavation	m3	30,000	$ 11.00	$ 330,000	- n/a		$ -	(330,000)
	- Rock Excavation	m3	75,000	$ 25.00	$ 1,875,000	8,500	$ 27.21	$ 231,285	(1,643,715)
	- Concrete incl rebar, cleanup, etc	m3	120,000	$ 665.00	$ 79,800,000	51,730	$ 531.18	$ 27,478,000	(52,322,000)
	- Spillway Bridge	t	- incl in Concrete		$ -		L.S.	$ 422,000	422,000
	- Gates, Guides, Stop Logs & Misc Stee	L.S.			$ 22,096,000		L.S.	$ 8,417,000	(13,679,000)
	Sub-Total - 2400 - Spillway				**$ 104,100,000**			**$ 36,548,000**	**(67,552,000)**
	- 2800 - North Spur Stabilization	**L.S.**			**$ 23,000,000**			**$ 18,016,000**	**(4,984,000)**
3000	**Powerhouse & Related Facilities**								-
	- 3300 - Civil Works incl Intake	L.S.			$ 218,050,000			$ 254,263,000	36,213,000
	- 3400 - Turbines & Generators	L.S.			$ 150,000,000			$ 145,000,000	(5,000,000)
	- 3500 - Electrical Equipment	L.S.			$ 23,000,000			$ 11,076,000	(11,924,000)
	- 3600 - Mechanical Equipment	L.S.			$ 10,000,000			$ 12,016,000	2,016,000
	- 3700 - Gen. Transformers/H.V. Switch.	L.S.			$ 34,000,000			$ 16,180,000	(17,820,000)
	- 3800 - Spare Parts & Special Tools	L.S.	- n/a					$ 7,000,000	7,000,000
	Sub-Total - 3000				**$ 435,050,000**			**$ 445,535,000**	**10,485,000**
6000	**Communications**	**L.S.**			**$ -**			**$ 1,965,000**	**1,965,000**
8000	**Construction Facilities & Support**	L.S.			**$ 264,000,000**			**$ 111,731,000**	**(152,269,000)**
	SUB-TOTAL - DIRECT COSTS				**$ 927,500,000**			**$ 753,978,000**	**(173,522,000)**
9000	**Management, Engineering & Others**								-
	- Management & Engineering	L.S.	15%		$ 139,125,000	15%		$ 121,000,000	(18,125,000)
	- Field Investigations	L.S.			$ 1,500,000			$ 2,500,000	1,000,000
	- Contingencies	L.S.	13%		$ 138,856,250	10%		$ 87,748,000	(51,108,250)
	- Taxes	L.S.			$ -			$ -	-
	Sub-Total - Manage/Engineer & Others				**$ 279,480,000**			**$ 211,248,000**	**(68,232,000)**
TOTAL - JUNE,1998$					**$ 1,206,980,000**			**$ 965,226,000**	**(241,754,000)**

Note: (1) Costs for Variant 7 are based on Table 6.3, Section 6.5

A comparison of Division 8000 costs in the 1980 report to the Relative Cost Comparison Variant 7 and the Final Feasibility Study is presented below in Table 16.6. The difference between the costs shown in the Final Feasibility Study and the Relative Cost Comparison (Variant 7) is also shown.

Table 16.6
Cost Comparison (Division 8000) - Variant 7 and Final Layout

Description	1980 Report ($1980)	$1980 escal. Variant 7 To $1998	Final Feas. Study ($1998)	Difference $
8200 - Muskrat Falls Site Site & General Services - Water, sewer, site prep, dismantle camp, etc.	3,460,000	6,750,000	7,910,000	1,160,000
Operate & Maintain Camp - Medical, catering, building repair, snow removal, etc.	47,730,000	93,000,000	56,970,000	-36,030,000
Camp Facilities - Commissary, kitchen, rec center, clinic, bar, etc.	7,950,000	15,500,000	7,800,000	-7,700,000
Camp - Dormitories	8,950,000	17,500,000	12,350,000	-5,150,000
Administration Buildings	3,700,000	7,400,000	5,400,000	-2,000,000
Fuel Storage & Distribution	1,360,000	2,650,000	0	-2,650,000
Personnel Transportation	4,450,000	8,700,000	4,700,000	-4,000,000
Electrical & Communications	7,300,000	14,250,000	9,300,000	-4,950,000
8300 - Happy Valley-Goose Bay - Operate & maintain hangar, housing, etc.	9,900,000	19,000,000	7,300,000	-11,700,000
8900 - Freighting Services	40,600,000	79,250,000	0	-79,250,000
TOTAL	**135,400,000**	**264,000,000**	**111,730,000**	**-152,270,000**

- The cost estimates shown in the 1989 Report on Supplemental Development Layouts accepted the 1980 cost estimate for Division 8000 and escalated the 1980 cost of $135.4 M for 3 units to a 1989 cost of $208 M for 4 units.

- The Relative Cost Comparison of Variant 7, (Table 6.3, Section 6.5) escalated the 1989 cost of $208 M to $264 M in 1998 and this was used as the cost for Division 8000.

- The Capital Cost Estimate developed for the Final Feasibility study did not rely upon any previous estimates or any escalated figures but was developed from scratch based upon the design as presented in the Report. Our analysis of the present situation for the Muskrat Falls Hydroelectric Development is much different than in 1980. The main reason for this is the completion, to RCU 80 Standard, of the Trans-Labrador Highway. All the costs for materials slated for Owner transport and distribution in the 1980 Report were obtained on an FOB Site basis for this CCE. Therefore the costs for this transportation and distribution are included in the direct costs for the applicable item and not in Division 8000.

- Other contributing factors to the lower cost of Division 8000 include;

  - 1980 CCE assumed a project duration of 5 ½ years versus the present schedule of 4 ½ years;

  - the unit costs for pre-engineered buildings and pre-fabricated trailers have not escalated as much as assumed in the Relative Cost Comparison of Variants (Table 6.3, Section 6.5). The inferred escalation was 95% whereas prices quoted for the present study were approximately 60% higher than in 1980.

- With the proximity of Happy Valley-Goose Bay some of the camp amenities were not included in the present CCE. These include:

  - Arena;
  - Men's and Women's Clubhouses;
  - Bank, Post Office and Barber Shop;
  - Cinema; and
  - Telecommunications Building.

16.6 POTENTIAL COST SAVINGS

Throughout the engineering for the Final Feasibility Study, many areas of cost savings were identified and included. The following addresses outstanding items which have been identified for potential cost savings and should be considered during final design.

Item	Description	Potential Cost Saving
1.	Re-use of the Gull Island Diversion Gates. The cost saving will be the difference between the cost of new gates, delivered to the site, and the cost of recovering the gates at Gull Island and delivering them to the site.	$2,000,000
2.	Reduce the peak camp size in Project Year 3 by rescheduling RCC dam construction over Year 3 and Year 4. This will reduce size of the construction camp by 100 persons.	$1,500,000
3.	Reduce the peak camp size in Project Year 3 by rescheduling intake concrete for Project Year 4.	To be reviewed in final design
4.	Reduce the peak camp size in Project Year 3 by providing suitable trailer park facilities.	$1,000,000
5.	Minimize the size of the construction camp at the site and house more construction personnel in Happy Valley-Goose Bay. The cost saving will include the transportation cost for personnel to and from the site. It does not include payment of labour costs for travel time. If payment for travel time is required, this potential cost saving is negated.	$10,000,000
6.	Selection of a narrow and deep unit as per the GEC Alsthom units at LG-1. This could reduce units spacing from 36 m to about 32.2 m and the powerhouse length by approximately 15 m.	$10,000,000
7.	Re-use Gull Island dormitories and buildings for Muskrat Camp if relative schedules permit.	$10,000,000
8.	Realize a salvage value of 10% on dormitories at end of project.	1,000,000
9.	Review the canal diversion scheme (Variant 11).	To be determined.
10.	Reduce flows from Churchill Falls during river closure. Use single rock groin.	1,000,000
11.	Including a credit for the estimated value of the cleared timber from the reservoir.	To be determined
12.	Staged integrated development of Muskrat Falls and Gull Island projects to minimize the cost of the engineering, management and equipment, and to utilize the labour force in the most effective way.	To be determined

13.	Investigation of a common transformer at Gull Island and Muskrat Falls projects.	$4,000,000
14.	With the present situation of falling commodity prices, currently most commodities are trading at 10 to 20 year lows, the possibility exists that the cost of the Project will decrease over the next few years.	To be determined
	Total Potential Cost Savings	**$40,500,000 plus items 3, 9, 11,12 & 14 to be determined**

16.7 MANAGEMENT AND ENGINEERING

16.7.1 Introduction

The Management and Engineering functions are those to be provided by an Engineering, Procurement and Construction Management firm acting on behalf of the Owner. This Project team will be responsible for the administration and supervision of all Project activities directly related to the physical execution of the Project and also for the following:

- the performance of engineering design;
- the procurement of Owner-supplied material, equipment and services;
- the soliciting, evaluation and recommendations of award for construction and installation contracts; and
- the supervision and administration of supply and construction contracts.

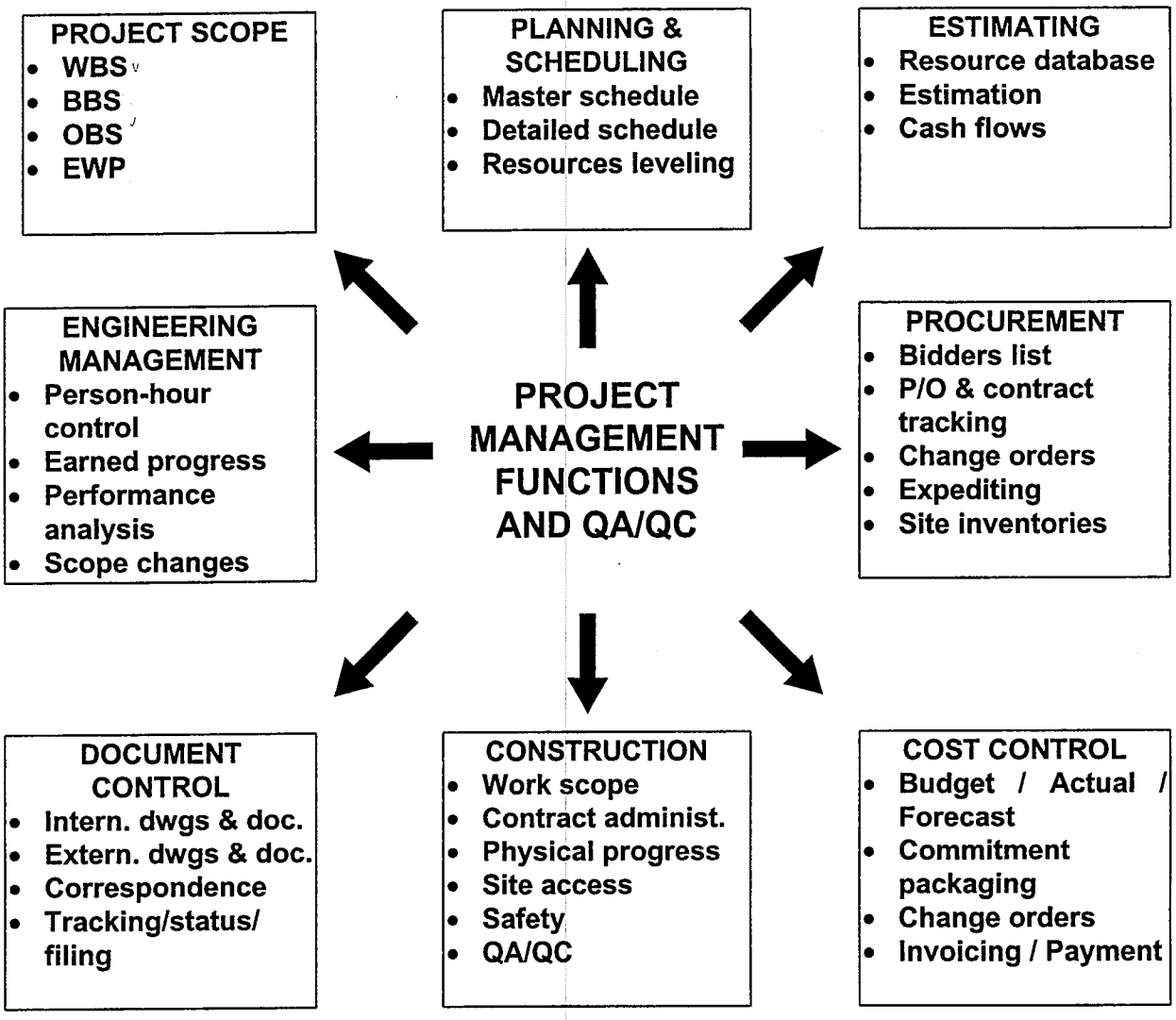
For all these activities and those integrated with them, there will be a Project Management Services function responsible for the implementation of cost, time and performance measurement, controls and reporting systems for the Project.

Detailed scope, methodology and cost estimates for Engineering, Procurement and Construction Management will be defined and implemented in the "Project Organization and Planning" phase.

The project management functions are shown in greater detail in Figure 16.2.

Allowances for Management and Engineering have been included in the CCE, code 9200. These allowances have been agreed between SNC-AGRA, Hydro-Quebec and Newfoundland and Labrador Hydro (NLH), and total approximately 16% of the direct costs (subtotal of codes 1000 to 8000). The actual amounts will be dependent upon the type of contract selected for the execution of the Project.

Figure 16.2
Project Management Functions



16.7.2 Project Function

16.7.2.1 *Project Scope*

The project management team will define in the project pre-release period the different coding structures that will be used to manage various data and establish the links between those structures. Among the basic coding structures are the following :

- WBS (Work Breakdown Structure) which will be used to identify the physical components of the Project. The WBS, used primarily by the estimating function, is the main breakdown structure to which the others are linked.
- BBS (Budget Breakdown Structure) reflecting the Owner's budget system.
- OBS (Organizational Breakdown Structure) used to establish organizational charts and to plan and monitor flow of information.
- EWP (Engineering Work Packaging) used to perform and monitor engineering work in accordance with the contract packaging.

16.7.2.2 *Planning and Scheduling*

Detailed engineering, procurement and construction schedules will be initiated in the project pre-release phase. Those schedules, which will take into account the contract packaging agreed with the Owner, detailed engineering and any additional information known at that time will be developed within the master schedule constraint. With information derived from the latest estimates, optimized resource leveling analysis will be conducted.

The scheduling of permits and environmental approvals will also have to be considered during the pre-release phase.

16.7.2.3 *Engineering Management*

This specific function addresses among other things the planning and control of engineering hours. Engineering person-hour control is achieved by measuring hours spent, earned value (added value and physical progress are other terms used for earned value) and hours required to complete the work. Progress is no longer based on spent hours versus a total budget; control should take into account Time, Cost and Performance Ratios.

Engineering management is best achieved in breaking down engineering work in packages (see Project Scope Section 16.7.2.1) which are broken down in deliverables of manageable size (eg: documents, drawings, reports) to which are attached budget values.

16.7.2.4 *Estimating*

Control estimates, taking into account detailed engineering, optimization studies, contract packaging and any additional information known at that time, will be done.

These estimates will be used to:

- monitor the budget and the cost forecast;
- validate bids; and
- perform more detailed cash flows and resource leveling.

16.7.2.5 *Procurement*

The procurement function takes care of the planning, monitoring and control requirements of contracts, purchasing, expediting, logistics, material controls and contract administration.

The function includes all the monitoring of procurement activities from the request for bids to the closure or completion of the contract or the purchase order. This includes among other things:

- preparation of approved bidders list;
- preparation of tender documents and issue of documents to tenderers;
- bid analysis and the preparation of a recommendation to award;
- preparation of award documents;
- contract administration including progress monitoring, change orders;
- expediting, inspection and logistics; and
- material control and site inventories.

16.7.2.6 *Document Control*

The document control includes the tracking, filing and monitoring of document status in close relation with engineering management of all documents related to the project. This includes:

- internal drawings and documents produced by the engineering group;
- external drawings and documents coming from the Owner, suppliers, manufacturers and contractors; and
- correspondence from any source.

16.7.2.7 *Cost Control*

This function addresses the monitoring of Budget, Scope Change, Actual Cost and Forecast at Completion and the processing of invoices and payments. This function is closely related to Project Scope (WBS and BBS) (Section 16.7.2.1), Planning and Scheduling (forecast) (section 16.7.2.2), Estimating (budget and forecast) (Section 16.7.2.4), Engineering Management (actual cost, scope change and forecast) (Section 16.7.2.3), Procurement

(commitment packaging, actual cost and forecast) (Section16.7.2.5) and finally Construction Management (actual cost, scope change and forecast) (Section16.7.2.8).

16.7.2.8 *Construction Management*

The Construction Management function addresses:

- planning of construction work at a very detailed level;
- contract administration including communication with the Contractors, contract changes, field work orders, contractor's request for change, field change orders and claims;
- monitoring and reporting of progress based on earned value or physical achievement;
- site access control including movement onto or off the site, of personnel, vehicles and equipment;
- management of all site services (catering, camp administration and maintenance, etc.); and
- health and safety plan, including maintenance of safety records for each contract and production of safety reports;
- implementation of the Environmental Protection Plan and environmental monitoring of construction activities.

16.7.3 Risk Sharing Management

More than any other contract, the contract between the Client and its Consultant for Management and Engineering, requires not only correct selection of the organizational structure and form but also attitudes of the parties involved to create a team spirit. Obviously this includes an equitable awareness and distribution of risk and the means for handling it in the event that it arises.

Characteristics of good contract risk management are as follows:

- strong emphasis on planning and good management practices designed to avoid unnecessary risks in the first place;
- awareness of the conditions of risk and their assignment;
- positive incentives linked to risk assignment; and
- flexibility for different assignment of different risk between the partners.

The Management and Engineering Contract has a tremendous impact on the success of the project as far as the following three objectives are concerned:

- performance;
- schedule; and
- cost.

It is recommended that the Contract for Management and Engineering includes a risk sharing for the above three objectives.

A positive incentive should be designed to create a team spirit, the incentive should be shared by all the personnel working on the project either from the Consultant or the Client in proportion to their contribution.

It is recommended to cascade this approach, when deemed justifiable to other contracts.

SECTION 17.0

CONCLUSIONS AND RECOMMENDATIONS

SECTION 17.0

CONCLUSIONS AND RECOMMENDATIONS

17.0 CONCLUSIONS AND RECOMMENDATIONS

17.1 CONCLUSIONS

The following conclusions result from the 1998 field investigation program and the engineering studies carried out for the Muskrat Falls Hydroelectric Development:

a) Civil

- Based on a review of alternative Project layouts, a layout derived from Variant 7 in the 1989 SNC Report was selected as the most economical development of Muskrat Falls. Individual structures were further optimized and, in some cases, different construction details were developed to reduce the capital cost. The layout, described in Section 7.0 and illustrated on Plates 10 and 11, includes:

  - tunnel diversion of river flows around the north side of the Muskrat Falls for two flood seasons. Design of the diversion is based on a 5% risk with a 1 in 40 year flood of 5,300 m^3/s. This will require a reduced outflow from the Churchill Falls Generating Station during the peak flood period, which could be achieved by short term operation of the Upper Churchill reservoirs in accordance with modified operating rules;

  - control of the upstream water level during winter to El. 24.0 m, to minimize frazil ice generation and the consequent ice damming and high water levels downstream. The volume of ice passing through the diversion is estimated to be <1% of the volume prior to diversion;

  - an integrated intake and powerhouse structure located in the river channel, above the lower falls, with a plant capacity of 824 MW;

  - a maximum reservoir operating water level of 39.0 m and a normal tailwater level of 3.0 m, providing a gross head of 36 m;

  - flood handling structures, comprising a three gate spillway at the north side of the intake/powerhouse and two concrete gravity (RCC) overflow dams, providing closure to the north and south river banks at the lower falls. A rubber dam, 2.4 m high and 167.5 m long, in five sections, is supported on the crest of the north dam. These structures will discharge the probable maximum flood (PMF) with a maximum reservoir elevation of 44.0 m;

  - permanent access to the development by road from either Happy Valley-Goose Bay, or Labrador City, via the Trans-Labrador Highway and a site access road. The access road crosses the dams, spillway and intake/powerhouse to approach the powerhouse at the south side of the Churchill River.

- The concrete semi-spiral cases required for this Project, to carry the water into the turbines, are very large but are not without precedent. Two other projects, Gezouba in China and

Iron Gate in Romania, have similar size or larger semi-spirals and have been operating without problems since constructed. The semi-spiral cases for this Project can be designed using current technology.

- The optimum summer plant capacity of 824 MW is based on a rated net head of 35.0 m and an average flow of 2,026 m^3/s which will provide an average annual energy output of 5.53 TWh/y at the HV side of the main power transformers.

- The derived plant capacity factor for the optimum plant output of 824 MW is 0.77.

- Concept optimization analyses found that the size and cost of a main gated spillway to pass the PMF could be substantially reduced if the north and south dams are designed as overflow dams, especially if roller compacted concrete (RCC) is used in the construction of the dams. The size and cost of the gated spillway can be further reduced by using a rubber dam crest on the north dam.

- Rubber dams and RCC construction for overflow dams are in common use in North America, and elsewhere in the world, as economical replacements for earth or rockfill dams and gated spillways.

b) Geotechnical

- The north spur, which is an important feature of the site, provides a natural dam for development of a reservoir on the River.

- Stabilization of the north spur will be required to prevent further slope failures along its upstream and downstream sides and prevent the potential loss of this important structure. The information gained from installation and operation of the interim pumpwell system since 1982 shows that control of the groundwater in the spur is the most cost-effective and reliable approach to spur stabilization. However, a combination of stabilization measures is necessary for the most effective and economical approach. Stabilization measures will require:

  - lowering the groundwater table in the downstream part of the spur;

  - provision of downstream erosion protection and downstream stabilizing fill in selected areas to improve local toe stability, and eliminate the potential for retrogressive failures due to presence of sensitive marine clays in the Upper Clay unit;

  - local topcutting of the spur and unloading of portions of the high steep side slopes to improve local stability;

  - provision of erosion protection and of a stabilizing upstream berm to prevent raveling of the upstream slopes from reservoir wave action and eliminate the potential for sliding in the unlikely (but possible) event of rapid drawdown during future project operation.

- It is not necessary, or feasible, to protect other areas of the reservoir rim from slope failures. The impoundment of the reservoir will generally improve the stability of slopes upstream of the dam and the frequency and size of slope failures will be less than under existing conditions. Small water level fluctuations during normal reservoir operation will not cause significant instability along the reservoir banks. However, erosion by wave action will cause beaching and local bank regression. Major slope regression is unlikely and reservoir slopes are expected to stabilize after a few years of operation. Surge waves from any slides triggered by rapid drawdown would not pose a serious threat to the project since they would occur into a lowered reservoir when extra freeboard is available.

- Downstream of the site, during and after construction, water level fluctuations will be reduced by the effective elimination of ice damming in the River below the falls.

- Seismic activity resulting from impoundment of the reservoir can, in certain cases, be induced depending on reservoir depth and area. The probability that the relatively small Muskrat Falls reservoir will initiate any significant seismic activity is very remote.

- The amount of sediments to be deposited into the future Muskrat Falls reservoir is unlikely to result in significant loss of storage or other problems and will be significantly reduced if the Gull Island Development is constructed first.

c) Mechanical

Four generating units, with one Kaplan and three Propeller turbines will provide near optimum energy output from the plant. Although four Propeller turbines would have a lower capital cost, the additional energy benefit with one Propeller replaced by a Kaplan would provide an economic payback in about two years. The mix of Kaplan versus Propeller turbines will be reviewed during final design.

d) Electrical

- The switchyard is a single bus arrangement, located on the roof of the powerhouse. It includes equipment for two 230 kV transmission lines which will pull-off from the south end of the powerhouse and thence connect to the HVDC Converter Station near Gull Island. It also includes sufficient space for a third line to Goose Bay.

- Two 330/440 MVA, 15/15-230 kV power transformers are provided at the downstream side of the powerhouse and connected to the four generators. This has reduced the cost of providing transformer capacity and the cost of the 230 kV equipment in the switchyard. The close proximity of the switchyard, transformers and powerhouse equipment eliminates the need for tie lines to a conventional switchyard remote from the powerhouse.

- The generator operating voltage has been selected as 15 kV.

- Unit breakers have been provided on the generator which has minimized the requirement for HV breakers on the roof of the powerhouse. This allows the station service loads to be

supplied from the HV system when the units are not operating. The necessity to switch station service when a unit is being shutdown has thus been eliminated.

e) Estimating

The proximity of Goose Bay and the improved condition of the Trans-Labrador Highway have improved the logistics of undertaking the project relative to previous estimates.

f) Construction

- A 1215 person construction camp is provided at the Muskrat Falls site. The camp contains all of the amenities required to comfortably accommodate the work force. Existing housing units in the town of Happy Valley-Goose Bay will be rented and upgraded to provide 100 family accommodations.

- The Project can be constructed using conventional construction methods and technology.

- Construction power is provided from the existing 138 kV transmission line between Churchill Falls and Happy Valley-Goose Bay and the existing substation located near the site. The substation is upgraded to provide 6 MVA of construction power.

- The flooded area is to be completely cleared, with the commercial wood stockpiled outside the reservoir limits.

g) Scheduling

The Terms of reference for the study required a Project schedule with Project release on January 1 of Year 1. However, it was determined that such a schedule for Muskrat Falls would have potential risks for delay and additional costs, as compared to a schedule with a June 1 Project release, due to a requirement for river closure by mid-November of Year 1. It was agreed to base the Project schedule on a Project release of January 1 of Year 1 and a Project start of June 1 of Year 1 which would provide first power in 57 months from Project release, i.e. on October 1 of Year 5. With this schedule, river closure would be after the spring flood of Year 2.

To achieve first power on October 1 of Year 5:

- the "turbine-generator" contract and the 6/8 MVA construction power transformer contracts must be tendered and awarded 2 ½ months following Project release of January 1 of Year 1;

- the "clearing" contract must be tendered and awarded 3 ½ months following Project release of January 1 of Year 1; and

- the "diversion and north spur stabilization", and the "preliminary works" contracts must be tendered and awarded immediately after Project start of June 1 of Year 1.

The total duration of the Project schedule, from award of the turbine-generator contract to first power and to full commercial operation would be 54½ months and 61 months, respectively.

17.2 RECOMMENDATIONS

The recommendations set out below follow from the results and conclusions of the study.

17.2.1 Recommendations for Project Development

a) Project Layout

The recommended layout of the Project will include:

- diversion facilities designed for a construction flood of 5,300 m^3/s, based on reduced outflows from the Churchill Falls Generating Station;
- a maximum reservoir operating water level of 39.0 m;
- an integrated intake/powerhouse structure located in the river channel, above the lower falls, with a plant capacity of 824 MW; and
- flood handling structures, including a main spillway and two overflow dams, located adjacent to the intake/powerhouse structure in the river channel, above the lower falls.

b) Diversion Facilities

Ice studies carried out by LaSalle Laboratories Ltd. (Reference 53), concluded that the diversion facilities should be operated to maintain a minimum upstream water level of El. 24 m during the winter seasons. This will eliminate frazil ice generation upstream of the site and thereby eliminate the problem with ice damming downstream of the site, and minimizing the cost of downstream cofferdams.

c) Flood Handling Facilities

- Design the flood handling structures for the PMF of 22,100 m^3/s, as established by others, with a maximum flood level of El. 44.0 m. Based on optimization of the flood handling facilities, the following structures and associated spill capacities are recommended:

– a three bay gated spillway	10,800 m^3/s
– a north overflow dam with rubber dam crest	5,800 m^3/s
– a south overflow dam	5,500 m^3/s
– total discharge capacity	22,100 m^3/s

- Three gates, 13.75 m wide x 20.2 m high, with a sill at El. 18.8 m, are recommended for the main spillway. The gate width is the same as used at Churchill Falls and Gull Island.

- The north overflow dam closes to the north river bank at the lower falls. It is recommended that this structure be constructed of RCC, with conventional concrete facings and crest, with the crest at El. 37.1 m. It is also recommended that the crest of the north dam include five independently inflatable/deflatable sections of rubber dam, between bridge piers, having a crest at El. 39.5 m.

- The south overflow dam closes to the south river bank at the lower falls. It is also recommended that this structure be constructed of RCC, with conventional concrete facings and top, with an ogee shaped crest at El. 40.0 m.

d) Power Facilities

- For optimum energy generation, installation of a plant with three Propeller turbines and one Kaplan turbine is recommended. An all Kaplan turbine installation will be reviewed during the final design.

- It is recommended that the intake/powerhouse be designed with concrete semi-spiral cases to carry the water to the turbines.

e) North Spur Stabilization

It is recommended that the north spur be stabilized by:

- lowering the groundwater table in the downstream part of the spur by the installation of additional pumpwells in the line of pumpwells installed in 1982, with the final pumpwell spacing selected based on the existing subsurface conditions and the performance of the existing pumpwell system;

- provision of toe relief drains and a major drainage trench for further lowering of the water table;

- provision of downstream erosion protection and downstream stabilizing fill in selected areas to improve local toe stability and eliminate potential for retrogressive failures due to presence of sensitive marine clays in the Upper Clay unit;

- local topcutting of the spur and unloading of portions of the high, steep downstream side slopes to improve local stability; and

- provision of erosion protection and a stabilizing upstream berm to prevent raveling of the upstream slopes from reservoir wave action and eliminate the potential for sliding in the unlikely (but possible) event of rapid drawdown during future project operation.

f) Construction

It is recommended that a 1215 person construction camp be provided for Muskrat Falls and that 100 housing units in Goose Bay be provided for family accommodations.

g) Schedule

A Project schedule with Project release on January 1 of Year 1 and a Project Start of June 1 of Year 1, is recommended. Based on this schedule, it is recommended that the "turbine-generator" and "construction power transformer" contracts be tendered and awarded 2 ½ months following Project release, the "clearing" contract be tendered and awarded 3 ½ months following Project release and the "preliminary works" and the "diversion and north spur stabilization" contracts be tendered and awarded immediately after Project start of June 1 of Year 1.

17.2.2 Recommendations for Preparatory Work

In order to meet the optimum Project schedule for the Muskrat Falls Development, it is recommended that detailed engineering be carried out in Project Years –1 and –2 for the following elements, as shown in the Project Master Schedule on Plate 28:

- Preliminary work, including camps and infrastructure, including access roads;
- Field investigations, including subsurface investigations and surveying at structure locations;
- Hydraulic model studies of the gate spillway and diversion facilities;
- North spur and diversion works;
- Main civil works;
- Mechanical-Electrical work;
- Turbines and generators;
- Switchyard; and
- Clearing.

Included in the detailed engineering is the preparation of tenders and award of:

- a contract for the turbines and generators about 2 ½ months following Project release of January 1 of Year 1;
- a contract for the construction power transformer about 2 ½ months following Project release January 1 of Year 1;
- a contract for clearing about 3 ½ months following Project release of January 1 of Year 1.
- a contract for the diversion and north spur stabilization immediately after Project start of June 1 of Year 1;
- a contract for preliminary works immediately after Project start of June 1 of Year 1; and

General

During negotiations for a comprehensive Project Agreement with trade unions, it is recommended that consideration be given to using Happy Valley-Goose Bay as a home base for the Project, with no payment for travel from Happy Valley-Goose Bay to the site.

17.2.2.1 *Investigations for Final Design*

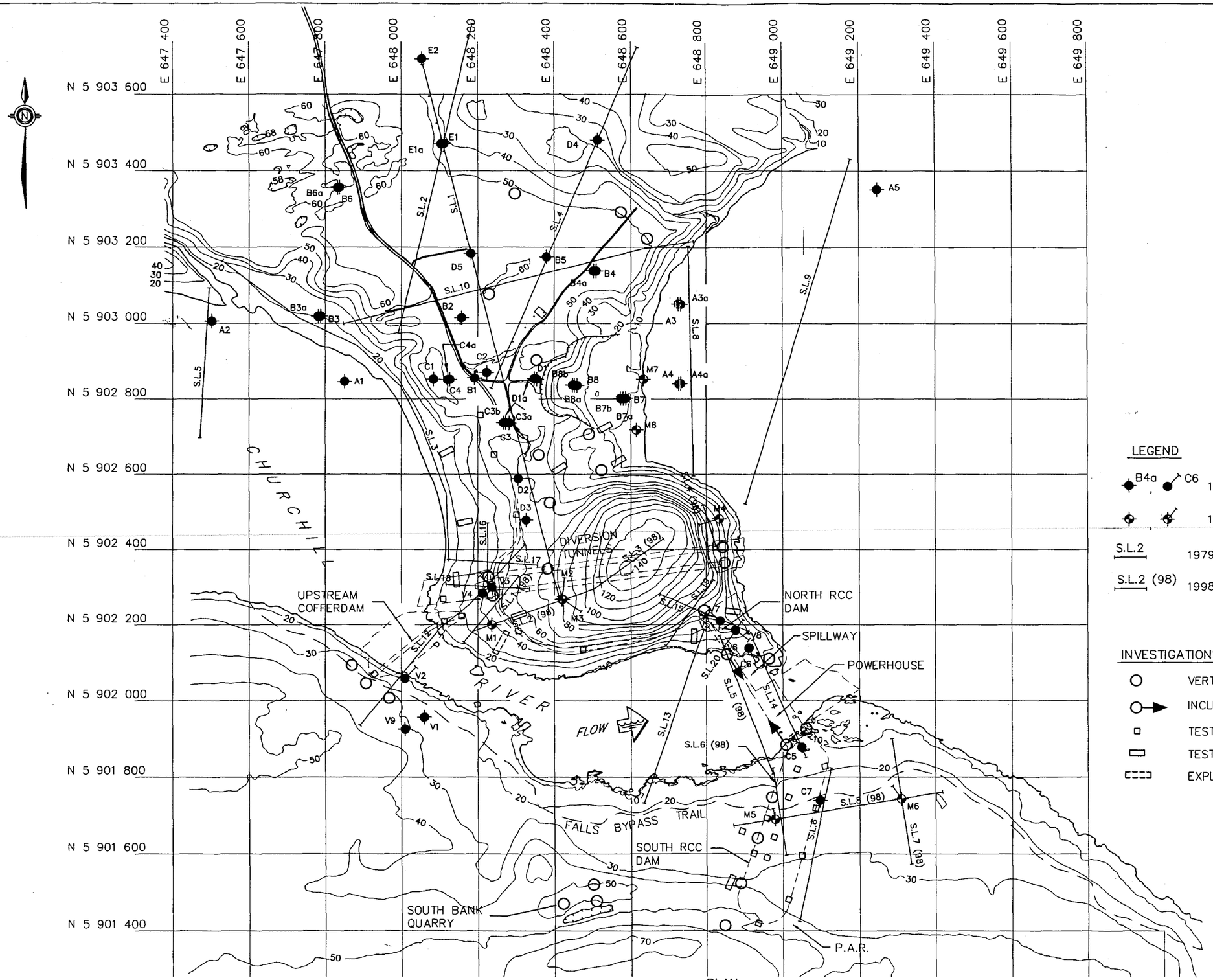
The investigations in the previous and in the 1998 engineering studies provided sufficient information on site topography, foundation conditions, bathymetry, construction materials, and groundwater conditions to allow the feasibility of the project to be determined, suitable layout developed, and schedule and costs determined. In final design, additional investigations and testing will be necessary for final optimization of the project layout, design of the project components, preparation of final costs and schedules, and provide data for bidders. Requirements for geotechnical investigations, field surveys, and hydraulic model testing are described below.

a) *Geotechnical Investigations*

Investigations for final design are discussed below, and are shown in Figure 17.1 and Table 17.1.

Table 17.1
Recommended Geotechnical Field Investigation Program

Location	Testpits/ Trenches	Drillholes	Drilling (Soil) (m)	Coring (NQ) (Rock) (m)
Spur	4	9	700	50
Diversion Tunnel	4	5	30	300
Upstream Cofferdam	8	4	20	60
North RCC Dam	4	2	10	100
Powerhouse and Spillway	0	4	0	350
South RCC Dam	5	4	40	100
Permanent Access Road	10	-	-	-
Granular Borrow	70	-	-	-
Impervious Borrow	20	-	-	-
Total	125	28	800	960



LEGEND

B4a C6 1979 BOREHOLE (VERTICAL, INCLINED)

1998 BOREHOLE (VERTICAL, INCLINED)

S.L.2 1979 SEISMIC LINE

S.L.2 (98) 1998 SEISMIC LINE

INVESTIGATIONS FOR FINAL DESIGN

- VERTICAL DRILLHOLE
- INCLINED DRILLHOLE
- TESTPIT
- TEST TRENCH
- EXPLORATION ADIT.

PLAN

0 50 150 250m

METRES

GEOTECHNICAL FIELD EXPLORATIONS FOR FINAL DESIGN

FIGURE 17.1

File name: FIG17-1.DWG Date: 1998/11/16 ● 09:22

- Spur

  - Nine additional drillholes and four test trenches are required to provide information on groundwater levels, pumpwell efficiency, and soil properties mainly at the north and south ends of the spur where only limited drilling data is available, and for installation of Casagrande piezometers for long-term monitoring of performance of groundwater control measures.

- Diversion Tunnels

  - Five additional drillholes at the tunnel portals and along the tunnel alignment to provide information on tunneling conditions and support requirements for final design and bid data, plus four testpits in the approach channel area to prove bedrock surface and provide data on overburden properties.

  - Excavation of a 3.5 x 3.5 m exploratory adit 50 m long for detailed geologic mapping to finalize tunnel spacing in final design, and allow inspection of tunneling conditions by bidders. The exploratory adit will be constructed along the alignment of the tunnel construction adit and will allow a three week earlier start on excavation of the diversion tunnels after project release. Earlier river closure will reduce cofferdam costs and allow earlier completion of the principal structures.

- Upstream Cofferdam

  - Excavation of testpits on both banks, and four drillholes on the right bank to prove bedrock elevation and establish abutment overburden properties for final design and bid data.

  - Three drillholes to prove a quarry for supply of rockfill to armour the south shore prior to upstream cofferdam closure.

- North RCC Dam

  - Two drillholes and four testpits to prove bedrock surface, overburden properties, bedrock quality and permeability for final design and bid data.

- Powerhouse and Spillway

  - Two angled drillholes along the axis of these structures to provide additional data on rock quality and permeability along the line of the grout curtain, and two vertical drillholes in the powerhouse/service bay areas to establish rock quality for rock cut and support design, and data for bidders.

  - Foundation resistivity survey for final design of powerhouse grounding grid.

- South RCC Dam

  - Four vertical drillholes along the dam axis to sample overburden and prove bedrock quality and permeability, and four testpits to prove bedrock and establish overburden properties for final design and bid data.

- Permanent Access Road

  - Testpits at 150 m spacing to establish soil properties and depth to bedrock (where overburden is shallow) for final design of the road and data for bidders.

- Granular Borrow

  - Additional testpits in Borrow Areas GR-2, GR-4 and GR-5, to prove project requirements for granular fill, and to provide an alternate source to processed shotrock for concrete aggregate and RCC aggregate, plus a 100% reserve to project requirements and bid data.

- Impervious Borrow

  - Additional testpits in Borrow Areas T-2, T-4A and T-4B to prove project requirements of select areas containing more than 20% fines plus 100% reserve for project requirements and bid data.

- Laboratory Testing

  - index testing on overburden samples;
  - gradation and compaction testing for borrow materials;
  - aggregate suitability and reactivity testing to confirm both gravel and shotrock aggregate sources are non-reactive, including mortar bar tests on selected gravel and shotrock samples; and
  - strength testing on undisturbed samples of Upper Clay from the spur for final design of stabilization measures.

The investigation program would require about eight months to conduct in Year -2, consisting of:

- two months to prepare exploration contracts, tender, evaluate and award;
- four months of field work to complete the program in the field; and
- two months to compile the results of the program for use in final design and for inclusion in bid data.

Three drills will be used, a truck mounted Mobile B-61 for north spur drilling, a heliportable Longyear 38 for north bank drilling, and a muskeg mounted CME-55 for south bank drilling. Bedrock coring to be done with NQ(3) barrels and the holes water pressure tested.

b) *Field Surveys*

During the 1998 field program, topographic and bathymetric surveys were carried out in particular areas to supplement new topographic mapping and to generally locate the major structures. It is recommended that further surveys be carried out prior to detailed design. These required surveys are:

- to establish the centreline location for each major structure;
- to establish centreline profiles and cross-sectional data for each structure, including the north spur, permanent access roads and campsite area; and
- to locate all new drillholes and test pits.

17.2.2.2 [redacted]

The following hydrology studies are recommended for Year -3, or Year -2 at the latest:

a) Dam Break/Flood Inundation Study: Newfoundland Department of Environment and Labour require that a dam break/flood inundation study, including preparation of a flood contingency plan, be carried out as a condition to obtaining a water license for a major hydroelectric dam. It is recommended that this study be coordinated with other environmental studies on the Lower Churchill.

b) PMF Study: The PMF Study on the Lower Churchill should be extended to the entire Churchill Basin to properly assess the contribution from the Upper Churchill to the PMF on the Lower Churchill sub-basin at Muskrat Falls and Gull Island. The scope of this study should also include a review of flood handling procedures at Churchill Falls, to take into account the existence of downstream developments.

   This Study could benefit from recent developments in the evaluation of PMF magnitudes in nordic climates.

c) Refinement of Construction Flood Handling Procedures:

   It is suggested that the modified flood handling procedures recommended in Section 4.0 be extended to meet the following objectives:

   - refine the proposed modifications to flood handling rules for Churchill Falls;
   - assess the requirements for energy exchange and capacity support between the Hydro Quebec System and NLH/Churchill Falls Systems, necessary to implement the modified flood handling rules. This study could provide useful input to negotiations between both utilities to decide on an appropriate operating protocol;
   - develop and test detailed operational rules and procedures required for real time appreciation.

17.2.2.3 *Hydraulic Model Studies*

Hydraulic model studies are recommended to refine the design of the following structures and are recommended for Year -3, or Year -2 at the latest:

- Gated Spillway
  - Unsymmetrical approach flow conditions will exist at the extremities of this structure, adjacent to the powerhouse and north dam. Special shaping of pier noses is required to correct this problem. This is best done via a hydraulic model study. The scope of this study should also include evaluation of spillway operation over a range of operating conditions, optimization of hydraulic design and determination of flow rating curves.
- Diversion Facilities
  - An overall model of the Diversion Facilities, cofferdam and tunnel, is recommended. This model would be used to investigate the following problems:
    a) transition from open channel flow to pressurized flow in the diversion tunnels to check for undesirable flow instabilities and to recommend mitigations, if required;
    b) optimization of overall hydraulic design of tunnel to facilitate construction, operation and final closure and also to produce flow rating curves;
    c) construction of upstream cofferdam, to recommend procedures for final closure, to determine extent to which rock groins can be pre-built into the river, and extent of south bank armouring to minimize overburden erosion during river closure.

17.2.2.4 *Engineering Design for Tender Documents*

In order for tendering to commence during the winter of Year -1, detailed engineering must begin during Year -2. The objective would be the preparation of all technical specifications for work of all disciplines, and the preparation of all tender drawings. These drawings would be sufficiently detailed to allow tenderers to prepare their bids. Later, after award of the contracts and the receipt of equipment suppliers' shop drawings, the tender drawings will be modified to include the necessary details for construction. Those drawings not requiring such modifications will be issued for construction as required following Project Release.

17.2.2.5 *Preparation of Tender Documents*

It is intended that tender documents for all major project elements be prepared, starting in Year -1, and continuing into Year 1, in time to receive and evaluate tenders prior to the date of anticipated Project start of June 1 of Year 1.

Contract packages will be as outlined in Section 15.3, Construction Packaging. A minimum number of packages will be prepared in order to both minimize problematic contract interfaces and the number of separate packages to prepare and manage during the short tendering period.

Standardized formats and content will be utilized.

17.2.2.6 *Tendering and Evaluation*

Tendering and evaluation of tenders will begin in Year -1 and continue into the first half of Year 1, such that all are completed in time for the Project Cost Estimate to be updated prior to the Project start date of June 1 of Year 1.

Some contract packages, particularly those involving the access road, initial camp, water and power supplies, to allow an early work start, will be awarded prior to Project start of June 1 of Year 1, and so must be scheduled accordingly.

The first major site work package on the critical path is the diversion tunnels, which will also have to be ready for award immediately after Project start of June 1 of Year 1.

17.2.3 Suggested Program (Year -3)

It was suggested that some investigation work and studies could be undertaken in Year -3 to provide more precise information to minimize some risks which could improve the financing aspect of the Project.

Such investigations and studies are outlined hereafter and could be further detailed at a later date, if required.

The suggested Year -3 Muskrat Falls Program was outlined in a preliminary draft e-mail document to the Client on January 11, 1999, which included descriptions and justifications of key items presented in the following sections.

17.2.3.1 Introduction

Section 7.0 of this report recommends the Project layout, and other sections confirm its technical and economic feasibility. Section 17.2.2 outlines a two year period prior to Project release during which time the following would be accomplished:

- establishment of the Project Management;

- completion of preliminary engineering;
- completion of field investigations and surveys;
- completion of hydraulic model studies;
- completion of detailed engineering for tender documents;
- preparation of tender documents;
- receipt and evaluation of tenders for all major Project components; and
- updating of Project cost estimate.

The schedule of activities requires that all preliminary engineering, along with the supporting site investigations, surveys and model studies be completed in Year -2. This is considered reasonable if the engineering is limited to the optimization and development of the selected layout and construction program, as defined in the report. However, the report identified some additional alternatives which required field information and/or studies which were beyond the present scope, but appeared to offer some additional savings of cost and time. These were:

- the search for granular and impervious materials beyond the vicinity of the site which would require minimum processing, offering savings in cost and time;
- the investigation of possible savings through the phasing of Gull Island and Muskrat Falls projects. Included here would be a review of common infrastructure requirements of the transmission line construction works (could be done with the Gull Island Hydroelectric Development suggested Year -3 program);
- the review of the Environmental Impact study to determine if potential environmental restrictions or mitigative measures might impact the project design;
- PMF study of the entire Churchill river basin (could be done with the Gull Island Hydroelectric Development suggested Year -3 program);
- Power system studies (could be done with the Gull Island Hydroelectric Development suggested Year -3 program); and
- Churchill River System control strategy (could be done with the Gull Island Hydroelectric Development suggested Year -3 program).

As these studies could result in significant changes to the Project, or by their nature take a longer time to complete than would be available in Year -2, it is suggested that they be completed or initiated prior to Year -2.

The following sections outline the studies suggested for 1999, including a description and scope of each study, the potential benefits, and the estimated resources and length of time required.

17.2.3.2 *Suggested Year -3 Geotechnical Field Investigation Program*

a) Justification

Significant volumes of glacial till and marginal volumes of aggregates have been identified by previous investigations. The revised dam construction calls for a significant increase in concrete aggregates. A broadened area should be studied to include identified but not thoroughly tested sources for these construction materials. An accurate inventory of these sources is required to confirm the required volume, with assurance of 100% extra.

b) Description

The Year -3 geotechnical field investigation program would comprise the studies as outlined in detail in Section 17.2.2.1. In general, the work will comprise additional geotechnical investigations in critical areas of the spur and at optimized structure locations. In addition to this, further acceptance testing and inventory will be performed on aggregate and impervious sources.

In total, 28 boreholes with 960 m of rock coring and 800 m of soil drilling are proposed. In addition, 70 materials and 55 geotechnical test pits will be excavated.

The duration of the field portion of this investigation will be of approximately 3 weeks duration and will require about 20 hours of helicopter time for drill moves.

17.2.3.3 *Schedule Modifications And Variant Studies*

A highway bridge across the river may be constructed prior to the Muskrat Falls Hydroelectric Development. If so, the impact of this bridge on the project should be reviewed.

The study of the impact of the bridge is divided into two separate studies. The first study would consider the impact on the project as it is currently designed. The second study would review the diversion sequence and the associated impact on the layout.

a) Current Layout

This study would have the potential of reducing the overall construction schedule significantly. The possibility of the bridge was not considered in the initial study, as per instructions from NLH. Since it is possible that the bridge would be constructed prior to Project release, the impact should be studied so NLH would have better knowledge of the benefits to the project.

The study of the current layout would require a review of the cofferdam requirements for the first season. There may also have to be some re-optimization of the layout to minimize the cofferdam requirements. The current road access and spillway bridge requirements would require review as these may be replaced by a road along the south shore of the falls. Input would also be required from the estimating group to determine the cost impacts of these modifications on the direct work and support facilities.

b) Layout Review

Immediate access to the south shore would imply that Variant 10 should be reviewed to determine if there would be possible cost savings. Early construction of the spillway on the south shore could provide an alternative river diversion through the spillway structure. Not only would a review of the hydraulics be required, but also a re-optimization of the plan layout, given this diversion scheme.

It is therefore suggested that this work be done in Year -3.

17.2.3.4 *Cost Savings Through Planned Phasing Of Gull Island And Muskrat Falls Project*

As the feasibility studies for both projects have been carried out without much regard to the implementation of both in combination, there may be savings to be realized if both are planned as a combined project. The benefits may arise from the combined use of infrastructure, contractor's equipment, the leveling of labor resources, for the base projects as well as for the related transmission facilities.

As the results of this study may influence the way in which infrastructure is developed as project support, it would be beneficial to have them identified prior to Year -2. This is work could be done with the Gull Island Hydroelectric Development suggested Year -3 program.

17.2.3.5 *Environmental Review*

No direct involvement was made during the feasibility study with the ongoing environmental impact assessment and the development of resulting actions or mitigation works. As some mitigative recommendations may involve changes to the project works, it is recommended that the current environmental assessment and findings be reviewed with the project staff. Requirements such as the need for a continuous riparian release during impounding have not been accounted for in the current design. To avoid surprises during preliminary design, such a review should be initiated prior to Year -2. This study would include in particular the concept and evaluation of an outlet to provide riparian flow during reservoir filling.

17.2.3.6 *Review Of PMF Study For The Churchill River Basin*

This study could be done with the Gull Island Hydroelectric Development suggested Year -3 program.

17.2.3.7 *Power System Studies*

This study could be done with the Gull Island Hydroelectric Development suggested Year -3 program.

17.2.3.8 *Churchill River System Control Strategy*

This study could be done with the Gull Island Hydroelectric Development suggested Year -3 program.

17.2.3.9 *Determination Of Cement Type For The Project*

a) Scope of the Study

The study would determine the following information:

- sources of cement in Newfoundland;
- the types of cement available;
- the production available;
- the chemical and physical properties of the cements which could be of interest for use on the project;
- the properties required for the cements to be used for the project; and
- if an acceptable cement is not available from Newfoundland sources, identification of other sources would be made.

b) Justification for the Study

- The properties of the cement to be used should be known at the start of the detailed engineering phase. There could be an impact on the construction methods and design to allow for particular cement properties.
- The cost estimates for the concrete depend to some extent on the type of cement.
- The availability of a suitable cement source must be established before calling for bids.
- If the modification of existing production facilities is a viable cost effective means of obtaining the cement, sufficient lead time should be available to make the modifications.

17.2.3.10 *Alkali Aggregate Reactivity Of Aggregates (AAR)*

a) Scope of the Study

The study would be an important supplement to the identification of concrete aggregate sources for the projects. The work would include the following activities:

- sampling of soil or rock from potential sources;
- determination of appropriate tests for AAR;
- AAR tests on the samples; and

- preparation of report on the study.

b) Justification for the Study

- Reliable tests for AAR normally require 6 to 12 months. Therefore it would be useful to know if a source of aggregate is unsuitable as soon as possible to enable alternative sources to be located if necessary.
- If it is necessary to modify the concrete mix design in order to accommodate an otherwise acceptable aggregate (addition of fly ash for example), the cost of the project may increase.
- The tests must be done sooner or later.

17.2.3.11 *RCC Dams Concrete Mix Proportions*

a) Scope of the Study

- Review of available or probably available material for the dams.
- Review construction methods best suited to this site.
- Optimization of the mix design for the RCC.

b) Justification for the Study

- The precision of the prices used for the RCC dams must be defendable.

17.2.3.12 *Rubber Dam Performance Evaluation*

a) Scope of the Study

- Collect technical data on rubber dams.
- Contact owners of existing rubber dams to obtain information on performance under similar conditions to those at Muskrat.
- Prepare a report on the findings.

b) Justification for the Study

- Due to lack of experience with this type of control structure in Canada, the confidence level in the acceptable performance of rubber dams must be demonstrated to the Client and to the financial institutions.

17.2.3.13 *Optimization Of The Number Of Kaplan Turbines To Be Installed In The Four Units Powerplant*

a) Description

Undertake the study of the following different mixtures of Kaplan and Propeller turbines to be provided at Muskrat Falls:

- two Kaplan and two Propeller turbines;
- four Kaplan turbines.

The analysis will include a preliminary selection of the design especially for the case of the four Kaplan turbines, a consultation with the major Canadian Manufacturers to confirm or improve the original selection and finally the evaluation of all the aspects associated with each possibility in order to select the optimal solution.

b) Justification

During the course of the last optimization process, the following points were found:

- the preliminary energy gains calculations when using Kaplan turbines indicated higher gains than initially anticipated;
- the cost difference between the Kaplan and the Propeller turbines as it was possible to compare from the reply of one of the manufacturer, was less than anticipated;
- the semi-spiral case width provided by the manufacturers were smaller than anticipated making it possible to shorten the powerhouse length.

PLATES

HYDRO NEWFOUNDLAND AND LABRADOR HYDRO

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

FINAL FEASIBILITY STUDY

PLATES / DRAWINGS

JANUARY 1999

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HYDRO **NEWFOUNDLAND AND LABRADOR HYDRO**

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

FINAL FEASIBILITY STUDY

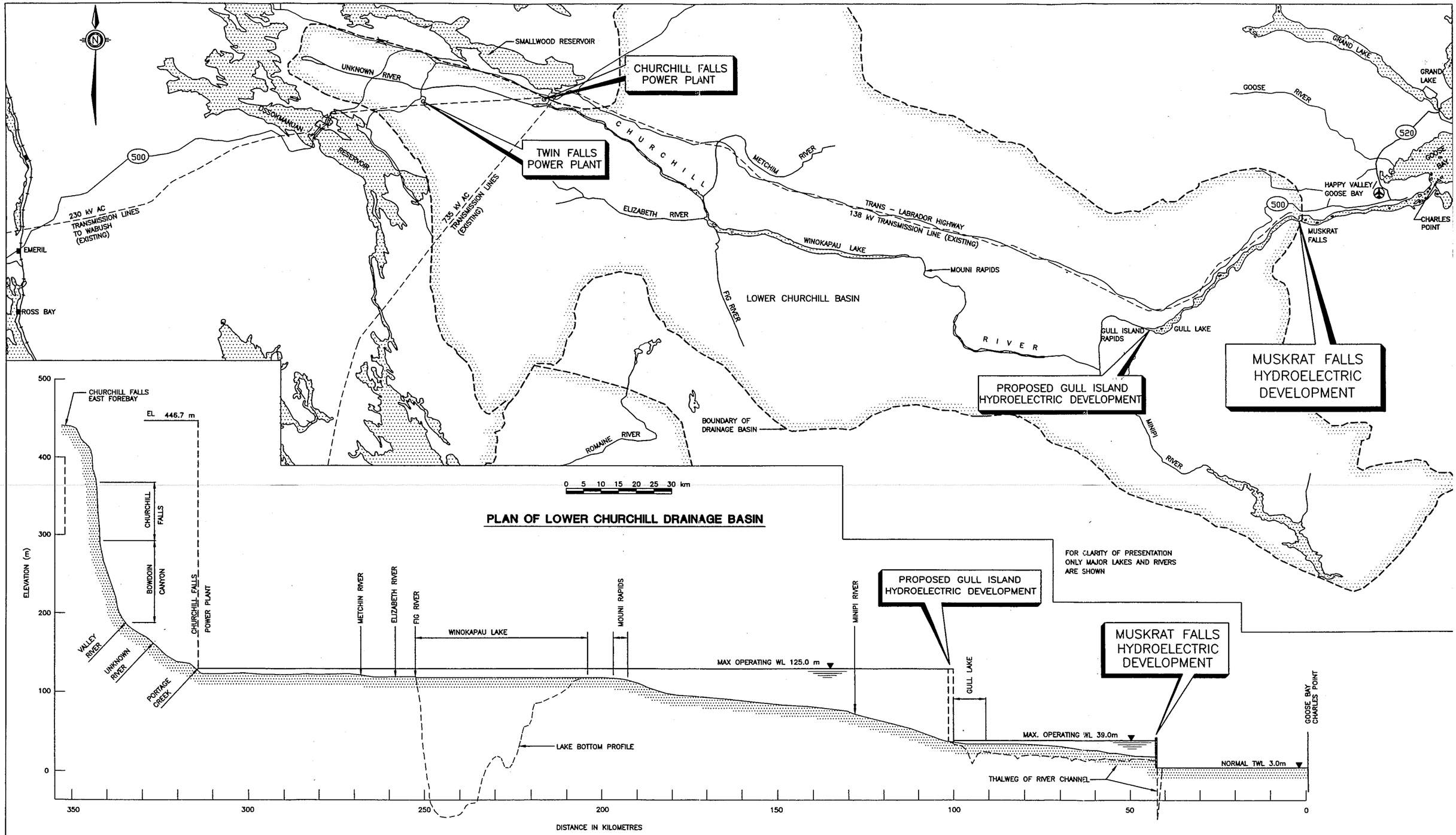
LIST OF PLATES / DRAWINGS

JANUARY 1999

PLATE No.	TITLE OF DRAWING	AUTOCAD No.
1	PROJECT LOCATION AND DRAINAGE BASINS	DC-721397-C-99-01
2	LOWER CHURCHILL RIVER - PLAN AND PROFILE	DC-721397-C-99-02
3	PLAN OF RESERVOIR	DC-721397-C-99-03
4	AERIAL PHOTOGRAPHY, MAPPING AND SURVEY 1998	DC-721397-C-99-04
5	ALTERNATIVE PROJECT LAYOUTS (VARIANTS 1-6) 1980	DC-721397-C-99-05
6	ALTERNATIVE PROJECT LAYOUTS (VARIANTS 7-11) 1989 and 1998	DC-721397-C-99-06
7	GEOTECHNICAL FIELD EXPLORATIONS 1979 & 1998 - LOCATION PLAN	DC-721397-G-99-01
8	SURFICIAL AND BEDROCK GEOLOGY	DC-721397-G-99-02
9	SECTIONS THROUGH BOREHOLES	DC-721397-G-99-03
10	PROJECT GENERAL ARRANGEMENT	DC-721397-C-99-07
11	ARRANGEMENT OF PRINCIPAL STRUCTURES	DC-721397-C-99-08
12	RIVER DIVERSION - PLAN AND SECTIONS	DC-721397-C-99-09
13	RIVER DIVERSION - TUNNEL PORTAL DETAILS	DC-721397-C-99-10
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15	GATED SPILLWAY - PLAN AND SECTIONS	DC-721397-C-99-12
16	INTAKE / POWERHOUSE - PLAN AND SECTIONS	DC-721397-C-99-13
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18	NORTH SPUR STABILIZATION MEASURES - PLAN AND SECTIONS	DC-721397-C-99-15
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21	STATION SERVICE SINGLE LINE DIAGRAM	DC-721397-E-08-02
22	230 kV SWITCHYARD GENERAL ARRANGEMENT SHEET 1	DC-721397-E-08-03
23	230 kV SWITCHYARD GENERAL ARRANGEMENT SHEET 2	DC-721397-E-08-04
24	CONSTRUCTION CAMP - ACCESS ROADS AND POTENTIAL BORROW AREAS LOCATION PLAN	DC-721397-C-99-17
25	CONSTRUCTION CAMP - GENERAL ARRANGEMENT	DC-721397-C-99-18
26	CONSTRUCTION CAMP - SUPERVISORS AND VISITORS ACCOMMODATION COMPLEX	DC-721397-C-99-19
27	CONSTRUCTION CAMP - TRADES ACCOMMODATION COMPLEX	DC-721397-C-99-20
28	PROJECT MASTER SCHEDULE	DC-721397-C-99-21

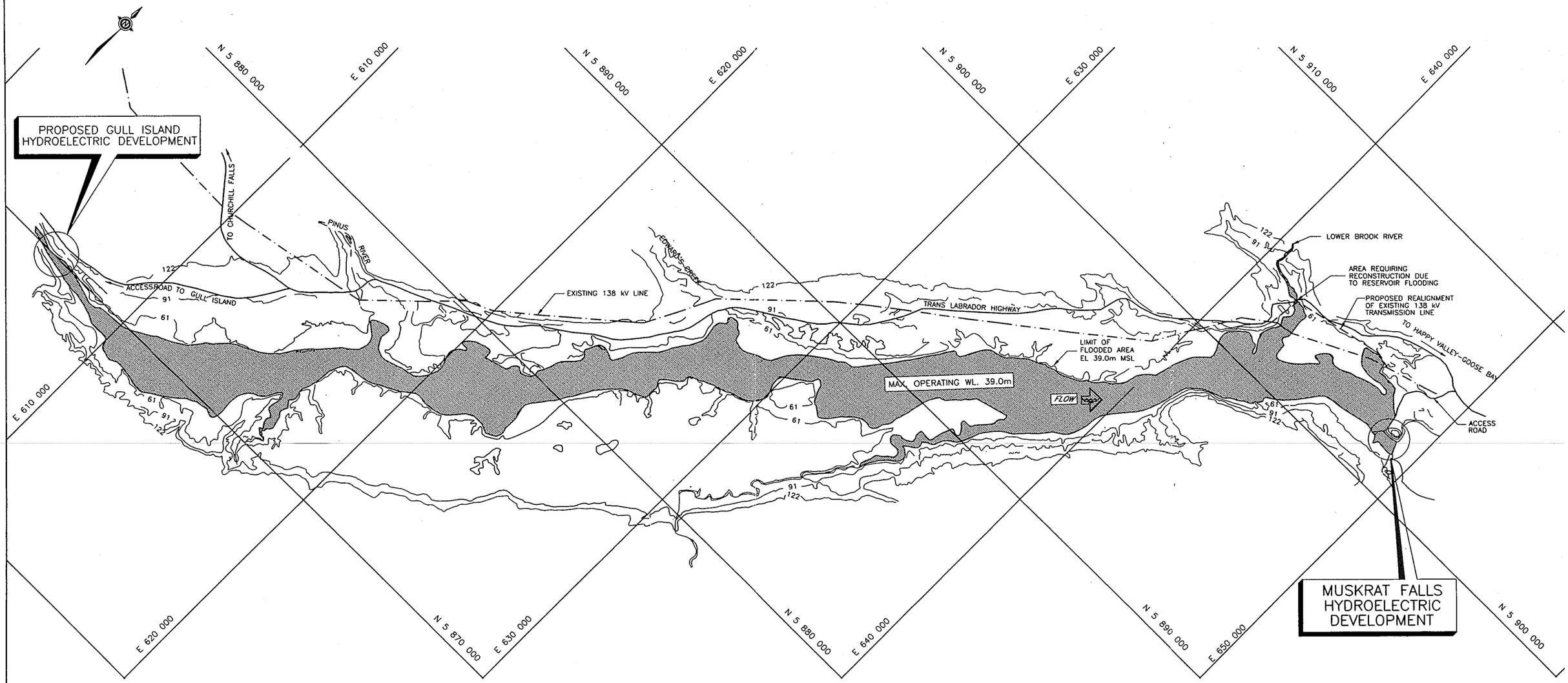
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SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 2
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title LOWER CHURCHILL RIVER PLAN AND PROFILE	
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File name: PLATE2.DWG Date: 1999/01/27 @ 21:33



0 1 2 3 4 5km

LEGEND

——— EXISTING ROAD

— · — EXISTING 138 kV LINE

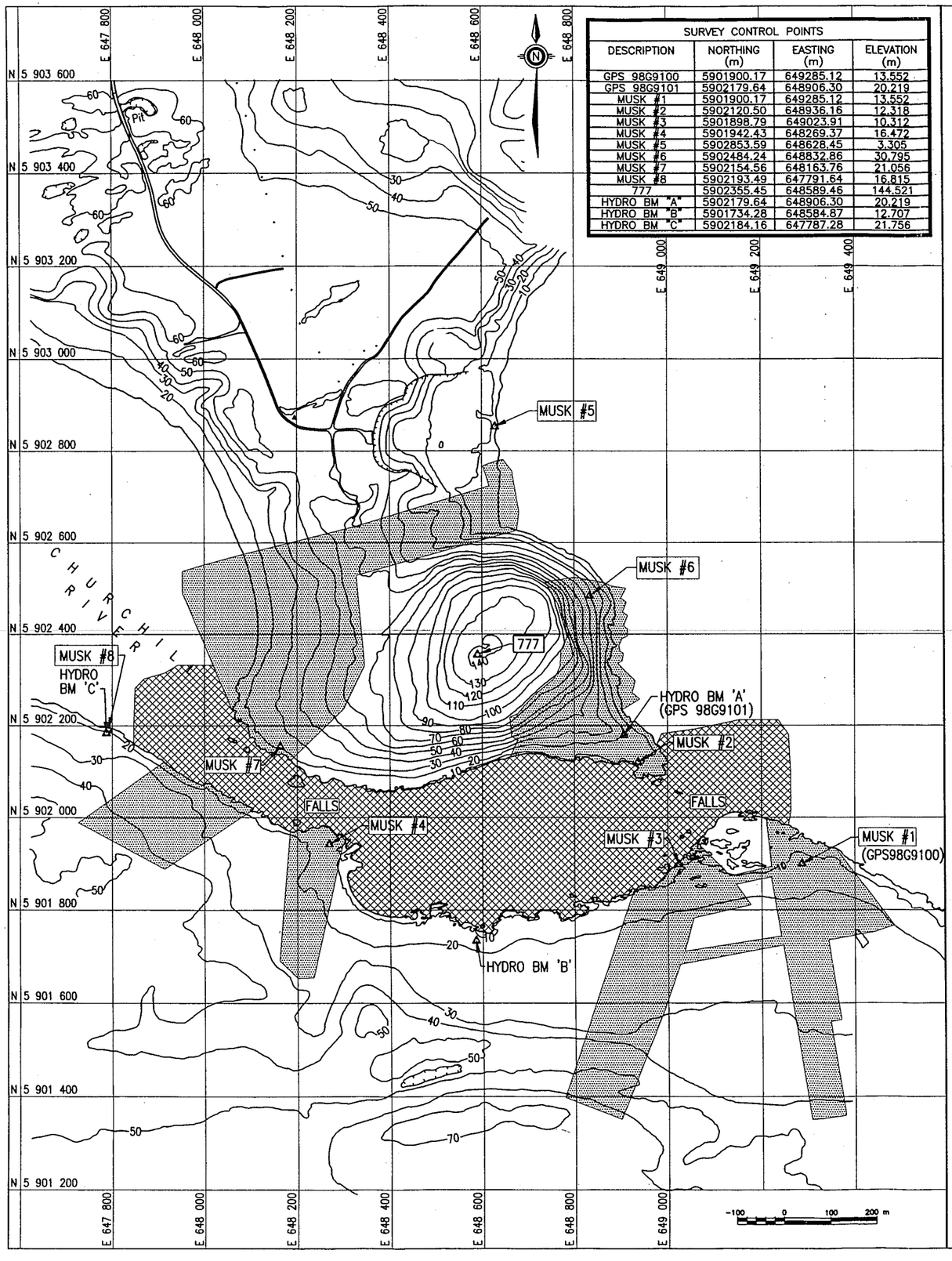
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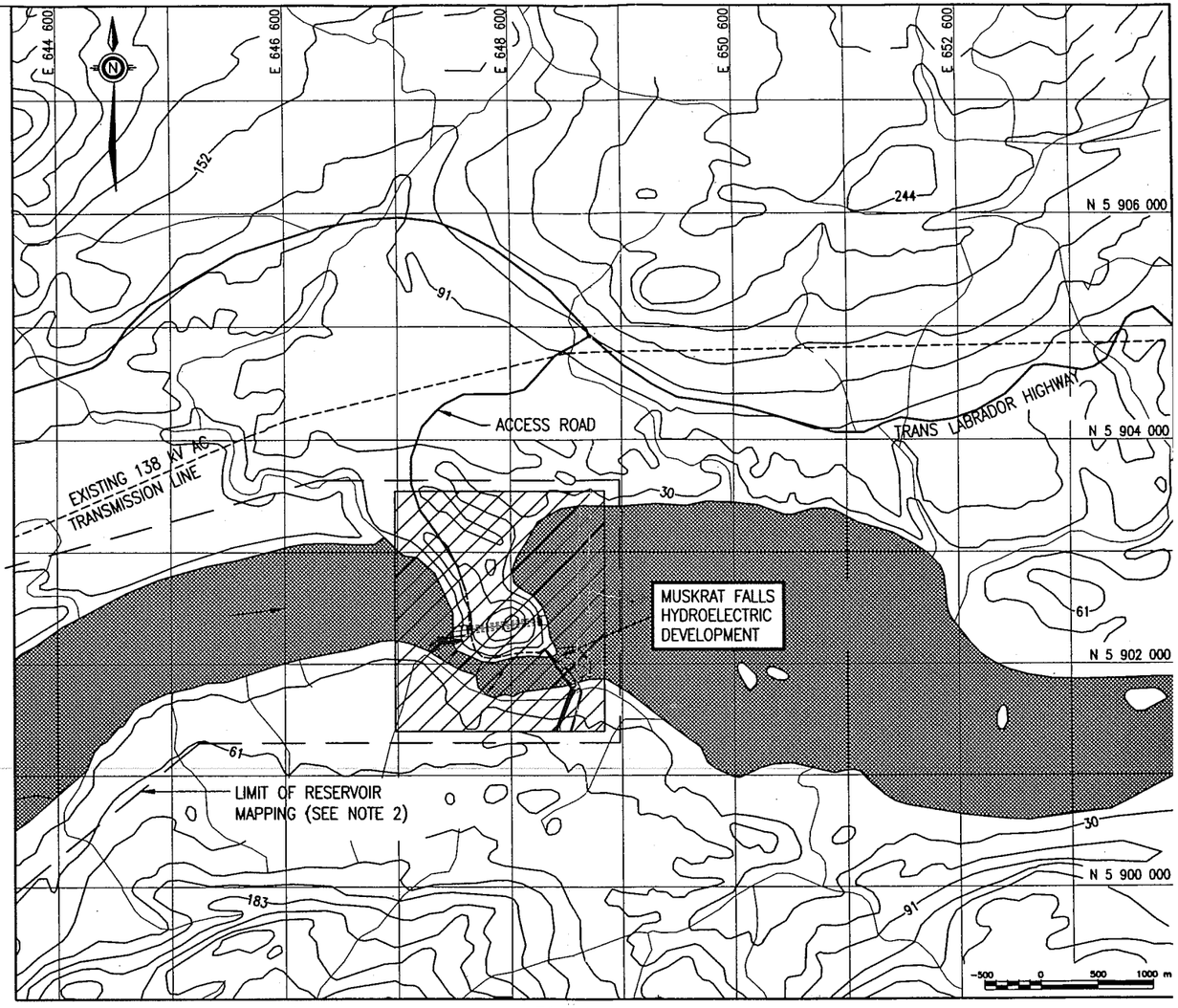
1. CONTOURS ARE IN METRES AND ARE STRAIGHT CONVERSION OF 100 ft. CONTOURS.
2. TOPOGRAPHIC GRID BASED ON NAD 83 SYSTEM.

SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 3
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title PLAN OF RESERVOIR	
Autocad DC- 721397 - C -99- 03		

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SURVEY CONTROL POINTS			
DESCRIPTION	NORTHING (m)	EASTING (m)	ELEVATION (m)
GPS 98G9100	5901900.17	649285.12	13.552
GPS 98G9101	5902179.64	648906.30	20.219
MUSK #1	5901900.17	649285.12	13.552
MUSK #2	5902120.50	648936.16	12.318
MUSK #3	5901898.79	649023.91	10.312
MUSK #4	5901942.43	648269.37	16.472
MUSK #5	5902853.59	648628.45	3.305
MUSK #6	5902484.24	648832.86	30.795
MUSK #7	5902154.56	648163.76	21.056
MUSK #8	5902193.49	647791.64	16.815
777	5902355.45	648589.46	144.521
HYDRO BM "A"	5902179.64	648906.30	20.219
HYDRO BM "B"	5901734.28	648584.87	12.707
HYDRO BM "C"	5902184.16	647787.28	21.756





KEY PLAN

LEGEND:

- GPR BATHYMETRIC SURVEY 1998 (NOTE 4)
- FIELD SURVEY 1998
- SITE MAPPING 1998 (NOTE 1)
- EXISTING ROAD
- PROPOSED ACCESS ROAD
- LIMIT OF RESERVOIR MAPPING
- EXISTING 138 kV AC TRANSMISSION LINE
- DIVERSION TUNNELS

NOTES:

1. SITE MAPPING PRODUCED @ 1.0 m CONTOURS BY GEODATA INC. FROM 1:8,000 SCALE PHOTOGRAPHS TAKEN IN 1998 BY AIRBORNE SENSING CORP.
2. RESERVOIR MAPPING EXTENDS UPSTREAM TO GULL ISLAND RAPIDS AND UP TO CONTOUR 42.0 m PRODUCED @ 2.0 m CONTOURS BY GEODATA INC. FROM 1:12,500 SCALE PHOTOGRAPHS TAKEN IN 1998 BY AIRBORNE SENSING CORP.
3. MAPPING TARGET CONTROL BY N.E. PARROT SURVEYS LTD. AND SNC-AGRA.
4. BATHYMETRIC CONTOURS @ 0.5m INTERVALS OBTAINED FROM GPR BATHYMETRIC SURVEY COMPILED BY GEOSCOTT EXPLORATION CONSULTANTS INC. SEPT. 1998.
5. GRIDS BASED ON NAD 83.
6. KEY PLAN TOPOGRAPHY DERIVED FROM 1:250,000 SCALE DIGITAL MAPPING. CONTOURS ARE IN METRES AND ARE STRAIGHT CONVERSION OF 100 ft. CONTOURS.

SNC-AGRA

Client: NEWFOUNDLAND AND LABRADOR HYDRO

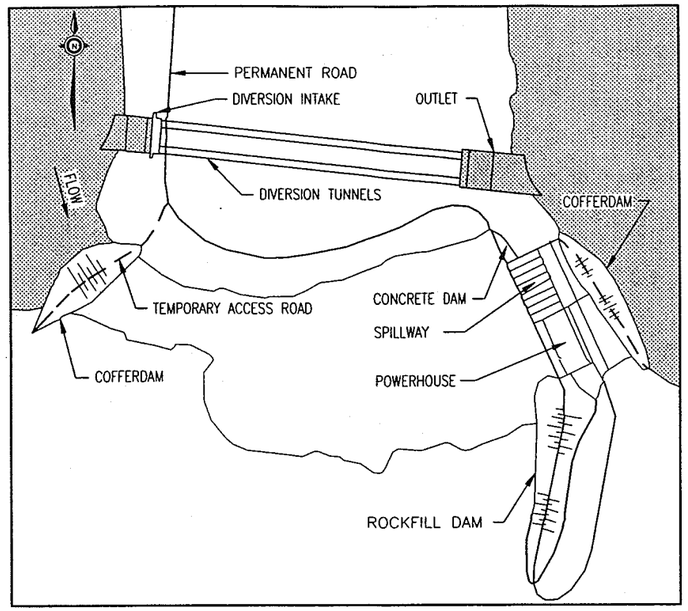
Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Date: JANUARY 1999

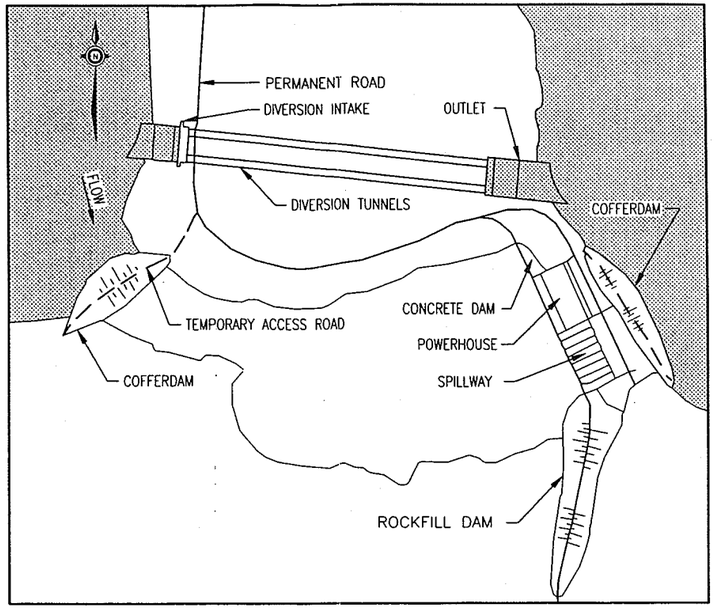
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Title: AERIAL PHOTOGRAPHY MAPPING AND SURVEY 1998

PLATE 4

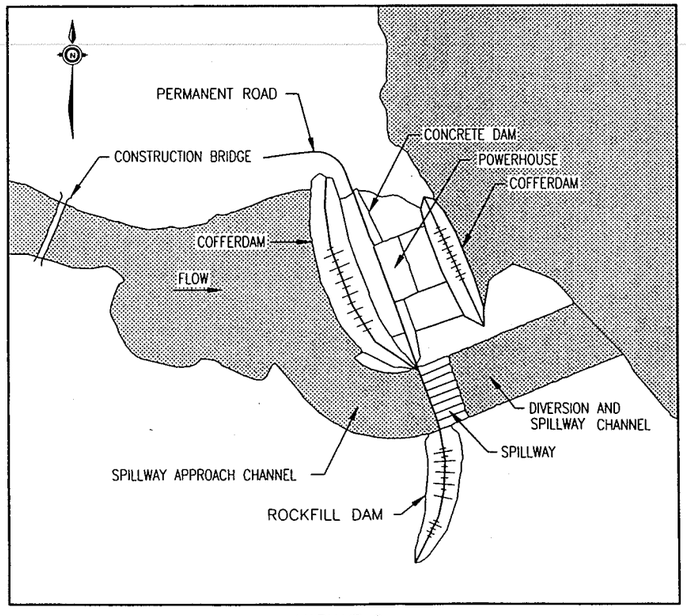


VARIANT 7 – VARIANT 3 (1988)
(TUNNEL SCHEME)

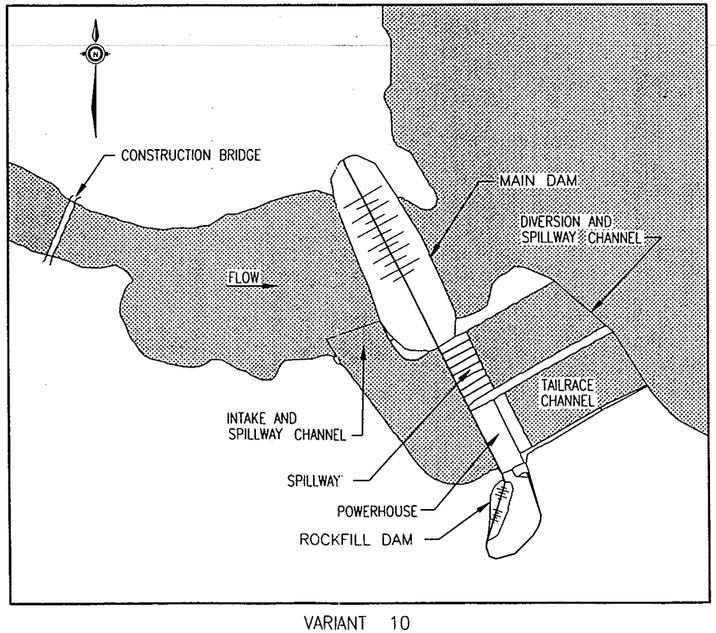


VARIANT 8
(MODIFIED TUNNEL SCHEME)

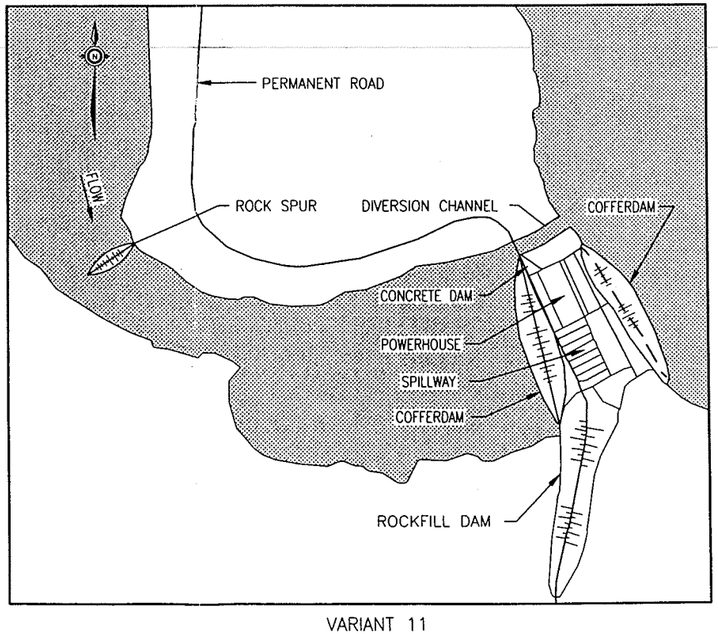
TUNNEL SCHEMES



VARIANT 9
(RIVER SCHEME)



VARIANT 10
(RIGHT BANK SCHEME)

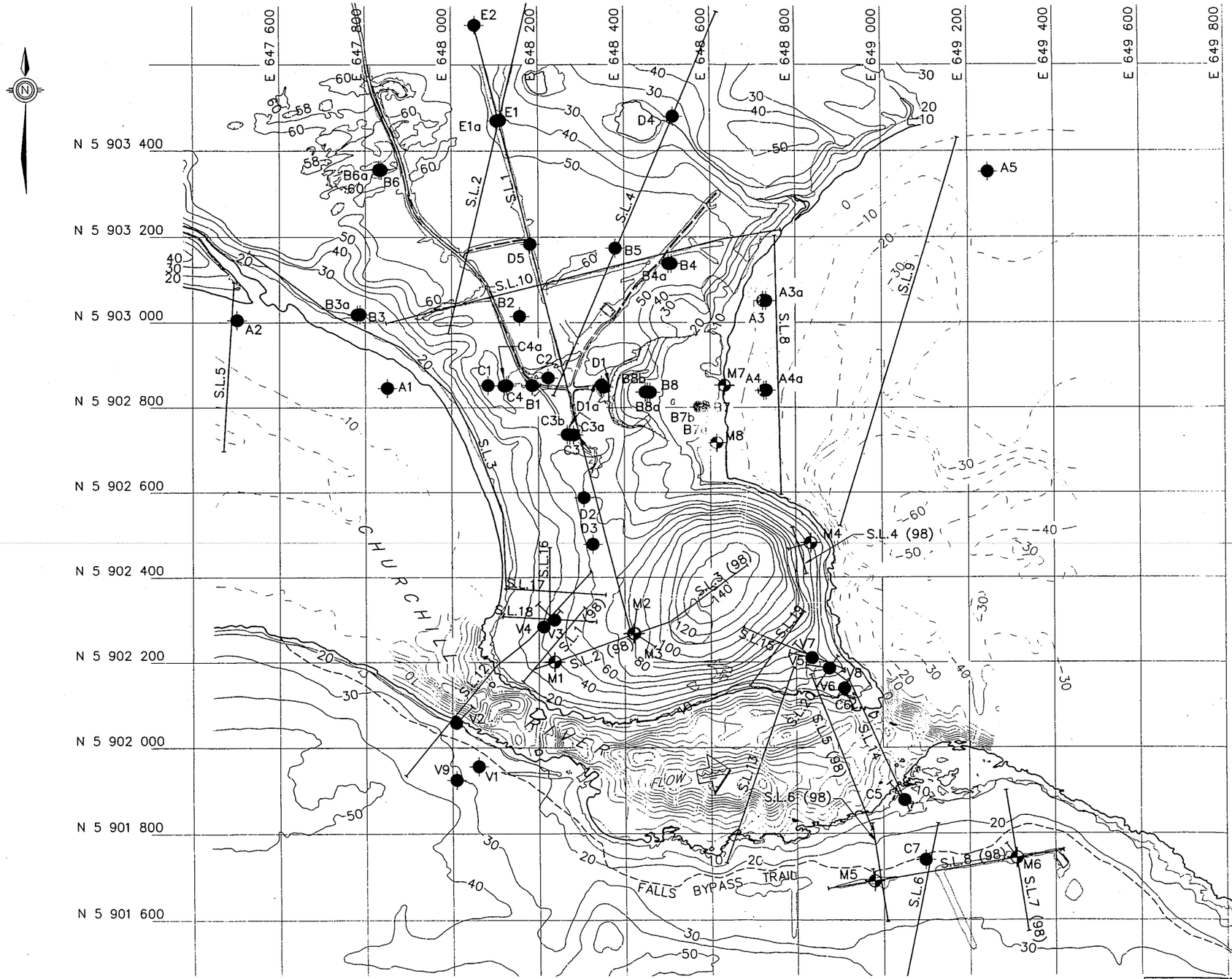


VARIANT 11

RIVER SCHEMES

SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 6
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title ALTERNATIVE PROJECT LAYOUTS (VARIANTS 7–11) 1989 & 1998	
Autocad DC- 721397 - C -99- 06		

File name: PLATE6.DWG Date: 1999/01/27 @ 21:47



LEGEND

B4a C6 1979 BOREHOLE (VERTICAL, INCLINED)

1998 BOREHOLE (VERTICAL, INCLINED)

S.L.2 1979 SEISMIC LINE

S.L.2 (98) 1998 SEISMIC LINE

NOTES

ABOVE WATER CONTOURS ARE BASED ON GEODATA DIGITAL MAPPING 1998.

BATHYMETRIC CONTOURS AWAY FROM FALLS AREA ARE ADJUSTED TO THE 1979 CORRECTED DATUM (M.S.L.).

TOPOGRAPHIC GRID IS BASED ON NAD 83 SYSTEM.

BATHYMETRIC CONTOURS IN RIVER AT THE FALLS ARE BASED ON GPR BATHYMETRY COMPILED BY GEOSCOTT EXPLORATION CONSULTANTS IN 1998.

0 100 200 300 400 500m

SNC-AGRA

Client NEWFOUNDLAND AND LABRADOR HYDRO

Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

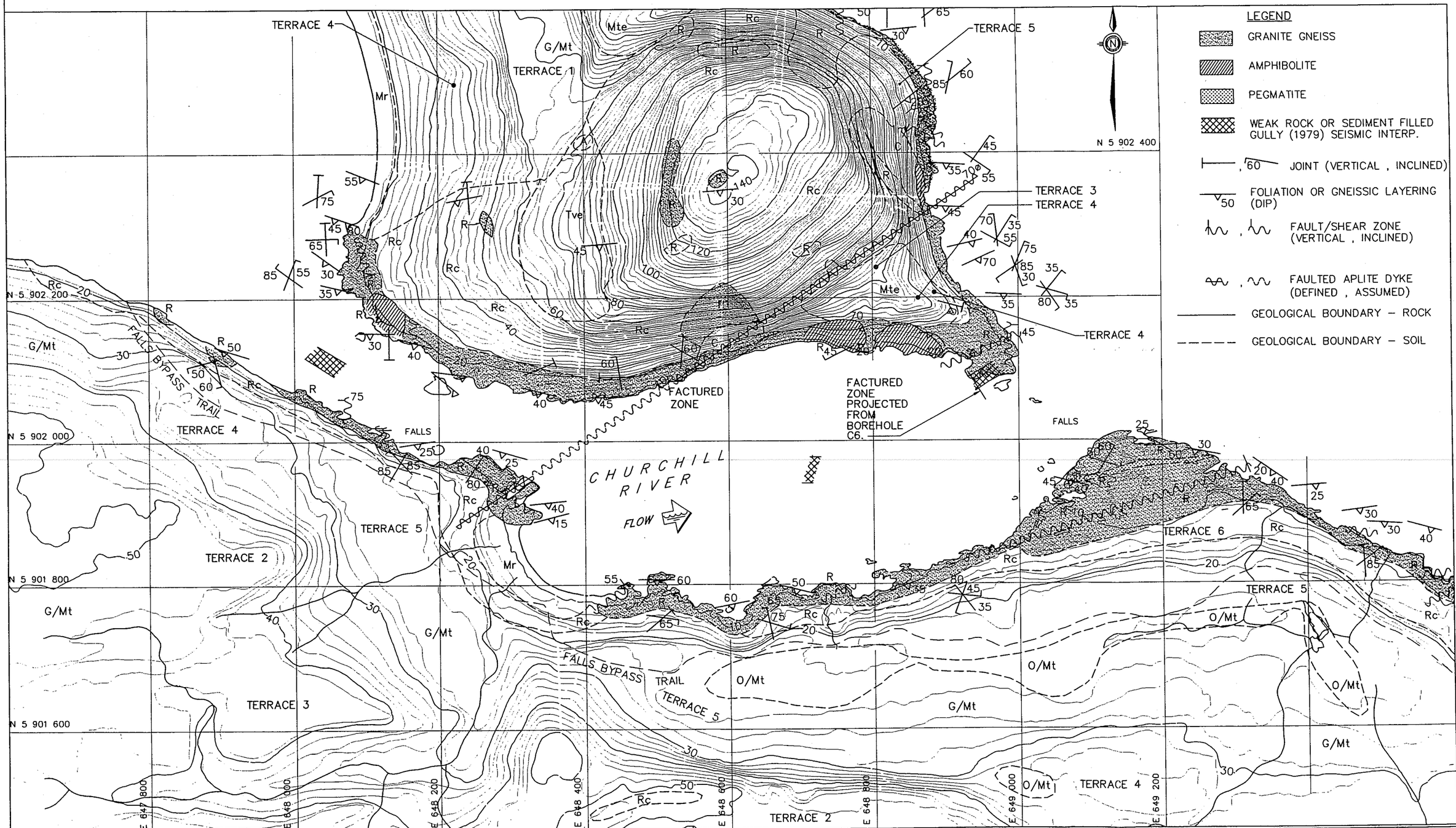
Date JANUARY 1999

Autocad DC- 721397 - G -99- 01

Title GEOTECHNICAL FIELD EXPLORATIONS 1979 & 1998 LOCATION PLAN

PLATE 7

File name PLATE7.DWG Date 1999/01/27 @ 21:55



LEGEND

- GRANITE GNEISS
- AMPHIBOLITE
- PEGMATITE
- WEAK ROCK OR SEDIMENT FILLED GULLY (1979) SEISMIC INTERP.
- JOINT (VERTICAL , INCLINED)
- FOLIATION OR GNEISSIC LAYERING (DIP)
- FAULT/SHEAR ZONE (VERTICAL , INCLINED)
- FAULTED APLITE DYKE (DEFINED , ASSUMED)
- GEOLOGICAL BOUNDARY – ROCK
- GEOLOGICAL BOUNDARY – SOIL

NOTES

- TOPOGRAPHIC GRID IS BASED ON NAD 83 SYSTEM.
- CONTOURS ARE BASED ON GEODATA DIGITAL MAPPING 1998.
- TERRACES ARE NAMED FROM OLDEST TO YOUNGEST.
- THERE ARE OTHER OLDER TERRACES THAT OCCUR OUTSIDE THE MAP AREA THAT ARE NOT INCLUDED IN THE NAMING

SURFICAL GEOLOGY LEGEND

MORPHOLOGICAL UNIT
- C – COLLUVIAL
- A – AEOLIAN
- G – GLACIOFLUVIAL
- M – MARINE
- T – GLACIAL TILL
- O – BOG
- R – ROCK

MODIFIER
- c – CONCEALED BY VEGETATION
- e – ERODED AND DISSECTED
- p – PLAIN
- r – RIDGE
- t – TERRACE
- v – VENEER

SNC-AGRA

Client: HYDRO NEWFOUNDLAND AND LABRADOR HYDRO

Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Date: JANUARY 1999

Autocad: DC- 721397 - G -99- 02

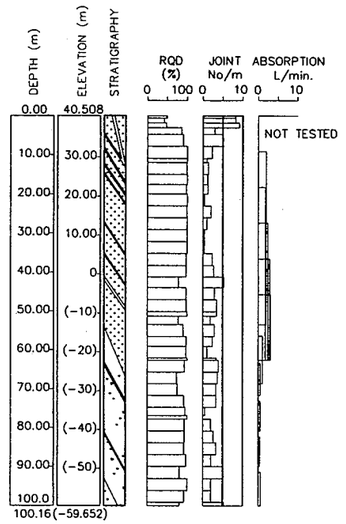
Title: SURFICIAL AND BEDROCK GEOLOGY

PLATE 8

File name: PLATE8.DWG Date: 1999/01/27 ● 22:01

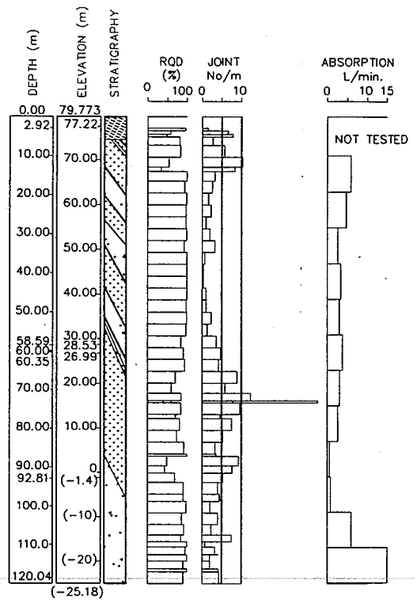
BOREHOLE M1–98

NORTH: 5 902 200.39
EAST: 648 236.20
ELEVATION : 40.508
AZIMUTH: –
DIP: 90° (VERTICAL)
DEPTH : 100.16



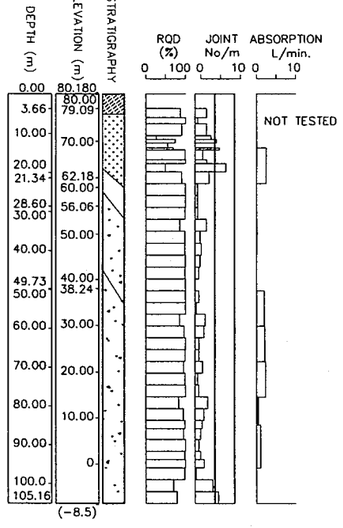
BOREHOLE M2–98

NORTH: 5 902 266.85
EAST: 648 419.96
ELEVATION : 79.773
AZIMUTH: 7°
DIP: 61°
DEPTH : 120.04



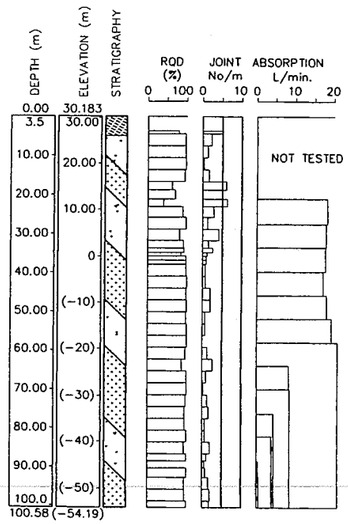
BOREHOLE M3–98

NORTH: 5 902 267.60
EAST: 648 421.90
ELEVATION : 80.18
AZIMUTH: 124°
DIP: 57°
DEPTH : 105.16



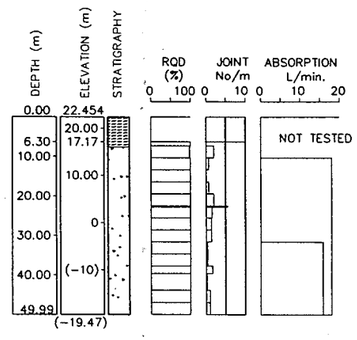
BOREHOLE M4–98

NORTH: 5 902 480.87
EAST: 648 834.02
ELEVATION : 30.183
AZIMUTH: 264°
DIP: 57.5°
DEPTH : 100.58



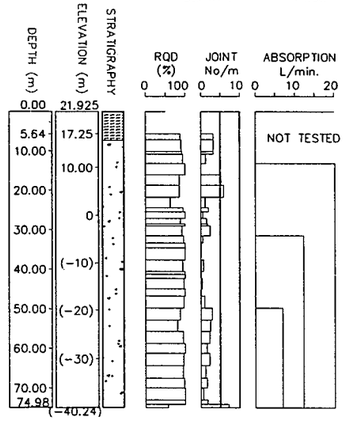
BOREHOLE M5–98

NORTH: 5 901 690.27
EAST: 648 978.59
ELEVATION : 22.454
AZIMUTH: 354°
DIP: 56°
DEPTH : 49.99



BOREHOLE M6–98

NORTH: 5 901 743.53
EAST: 649 309.28
ELEVATION : 21.925
AZIMUTH: 324°
DIP: 57°
DEPTH : 74.98



LEGEND

- OVERBURDEN
- GRANITE GNEISS
- AMPHIBOLITE OR AMPHIBOLITE GNEISS
- PEGMATITE

0 10 20 30 40 50 m

SNC-AGRA

Client: NEWFOUNDLAND AND LABRADOR HYDRO

Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

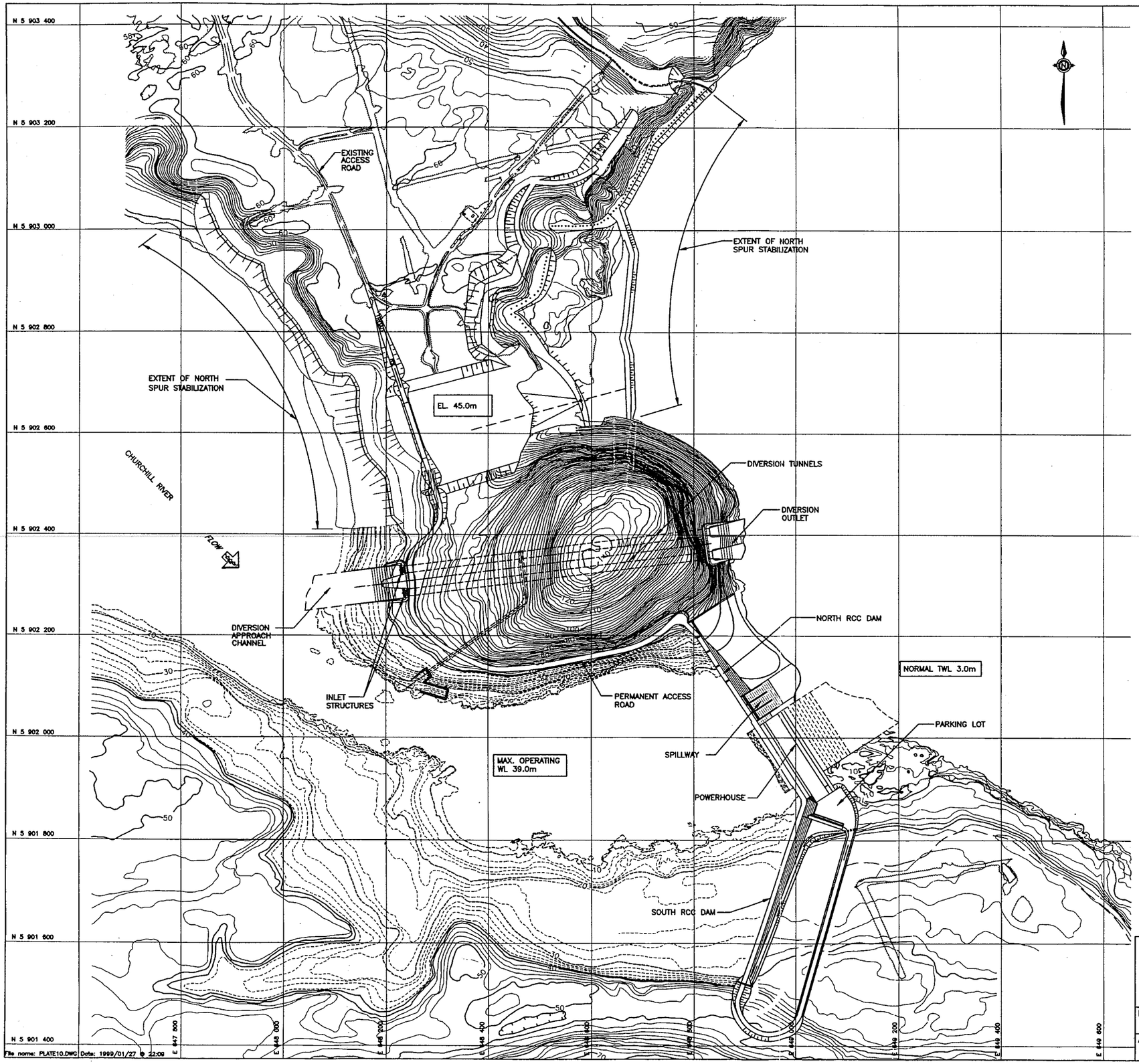
Date: JANUARY 1999

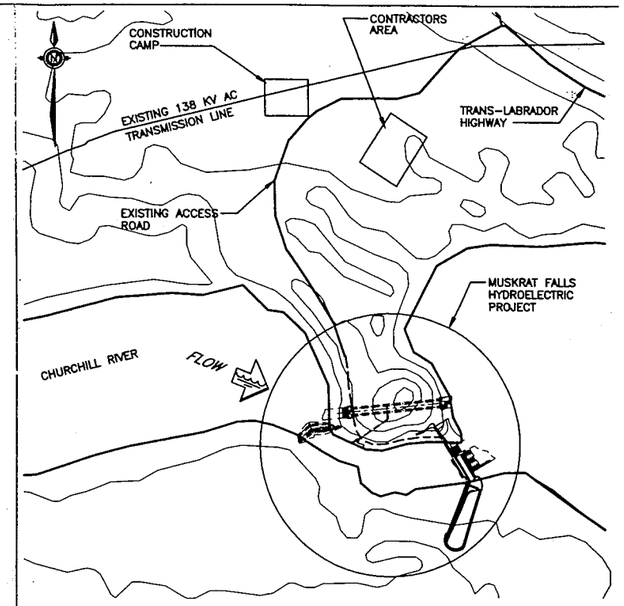
Autocad: DC- 721397 - G -99- 03

Title: SECTIONS THROUGH BOREHOLES

PLATE

File name: PLATE9.DWG Date: 1999/01/27 @ 22:03





KEY PLAN

N.T.S.

SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 10
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title PROJECT GENERAL ARRANGEMENT	
Autocad DC- 721397 - C -99- 07		

File name: PLATE10.DWG Date: 1999/01/27 @ 22:09

N 5 902 400

N 5 902 200

N 5 902 000

N 5 901 800

N 5 901 600

N 5 901 400

E 647 800

E 648 000

E 648 200

E 648 400

E 648 600

E 648 800

E 649 000

CHURCHILL RIVER

FLOW

INLET STRUCTURES

DIVERSION APPROACH CHANNEL

EL. 12.0

CONSTRUCTION ADIT

DIVERSION TUNNELS

DIVERSION OUTLET EL. -4.0

BEDROCK SURFACE

NORMAL TWL 3.0m

EL. 45.5

NORTH RCC DAM CREST EL. 37.1

RUBBER DAM: CREST EL. = 39.5m (INFLATED) = 37.1m (DEFLATED)

BRIDGE & ROADWAY EL. 45.5

SPILLWAY

POWERHOUSE EL. 1.0

MAX. OPERATING WL 39.0m

ROADWAY EL. 45.5

SLOPE DOWN 1 ON 3

EL. 14.0

EL. 5.0

EL. -28.5

EL. -10.0

SLOPE UP 1 ON 3

BRIDGE & ROADWAY EL. 45.5

SOUTH RCC DAM CREST EL. 40.0

SLOPE VARIES

TOE DITCH

ROAD

0 50 100 150 200 250m

File name: PLATE11.DWG Date: 1999/01/27 @ 22:12

SNC-AGRA

Client: NEWFOUNDLAND HYDRO AND LABRADOR HYDRO

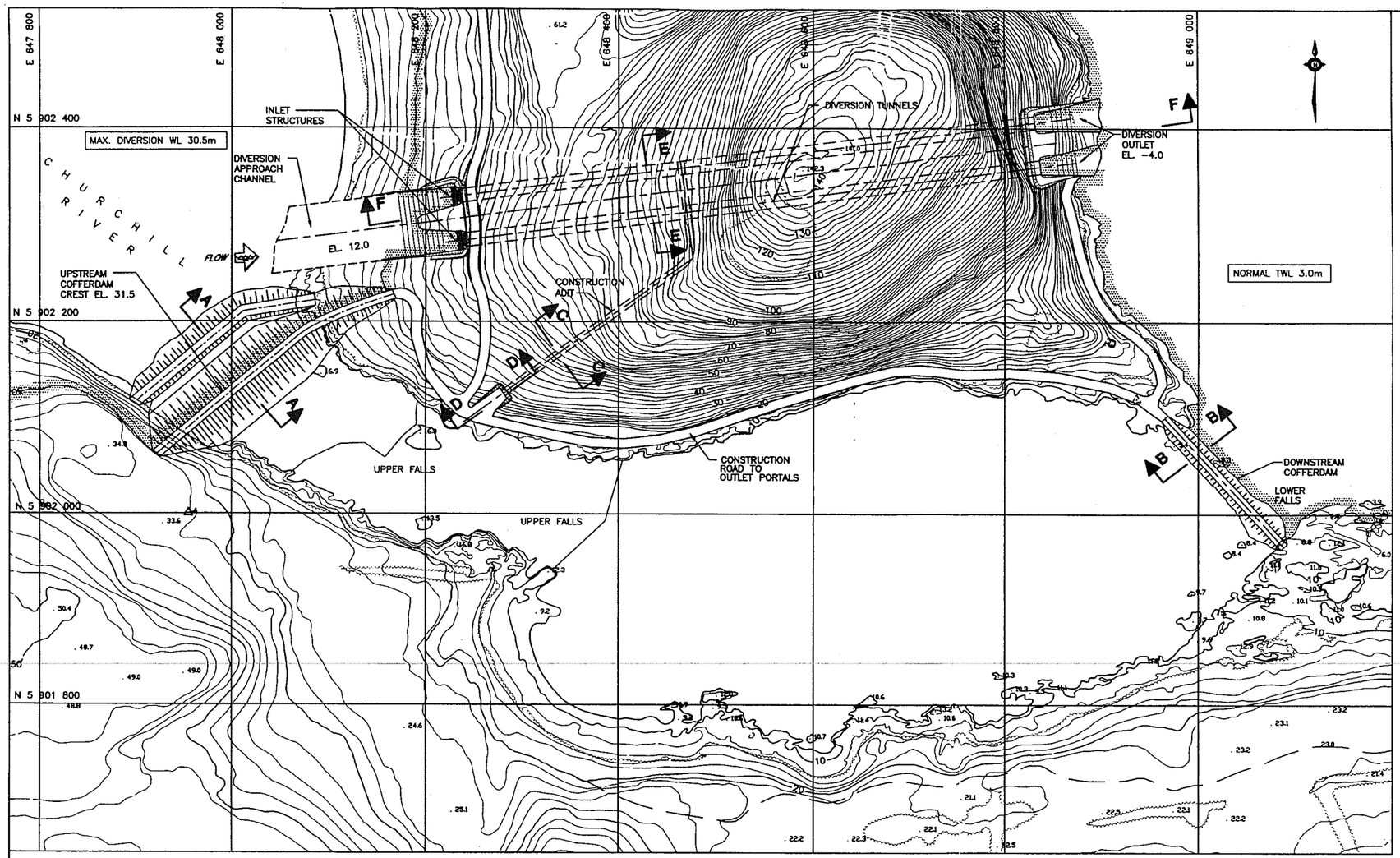
Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Date: JANUARY 1999

Autocad: DC- 721397 - C -99- 08

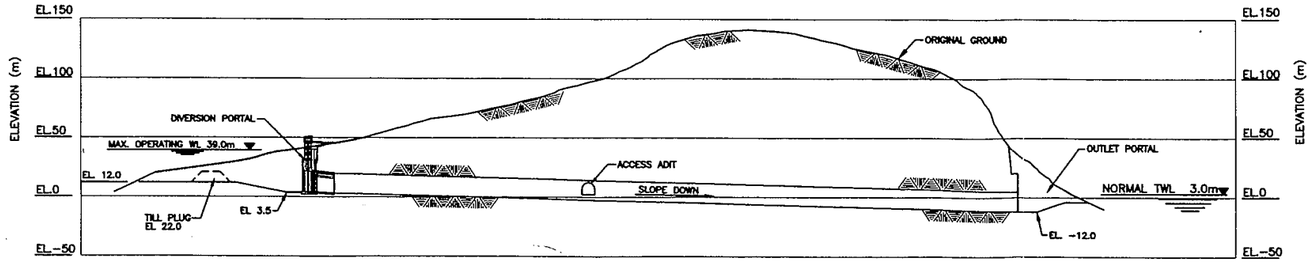
Title: ARRANGEMENT OF PRINCIPAL STRUCTURES

PLATE 1



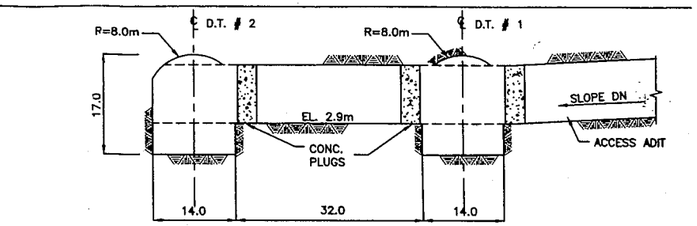
PLAN

0 50 100 150 200 250m

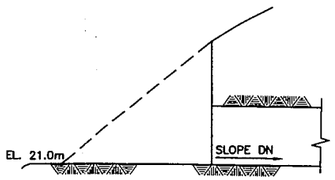


SECTION F-F
(PROFILE THROUGH DIVERSION TUNNEL)

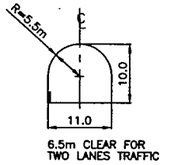
0 50 100 150 200 250m



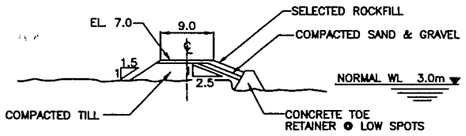
SECTION E-E



SECTION D-D



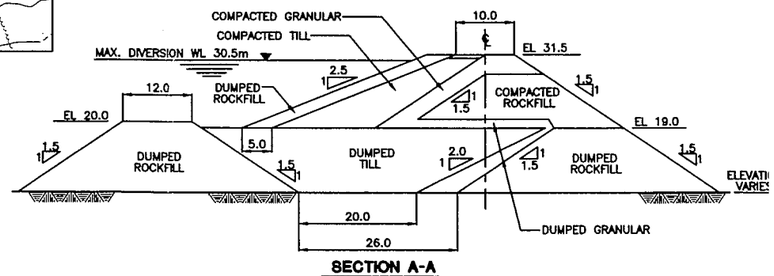
SECTION C-C



SECTION B-B

TYPICAL CROSS SECTION D/S COFFERDAM

0 10 20 30 40 50m

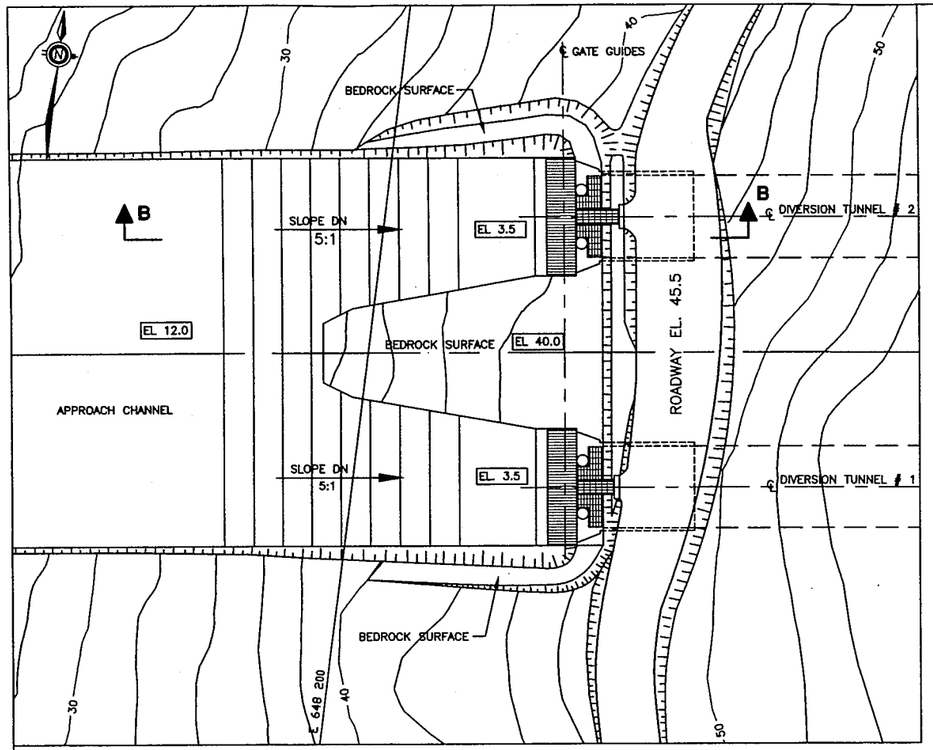


SECTION A-A
(TYPICAL CROSS SECTION U/S COFFERDAM)

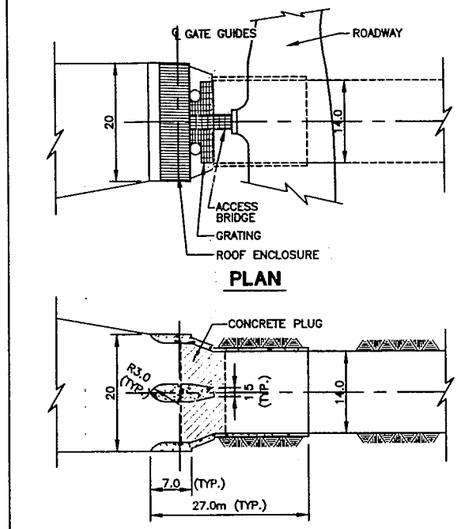
0 10 20 30 40 50m

SNC-AGRA	Client NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 12
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title RIVER DIVERSION PLAN AND SECTIONS	
Autocad DC- 721397 - C -99- 09		

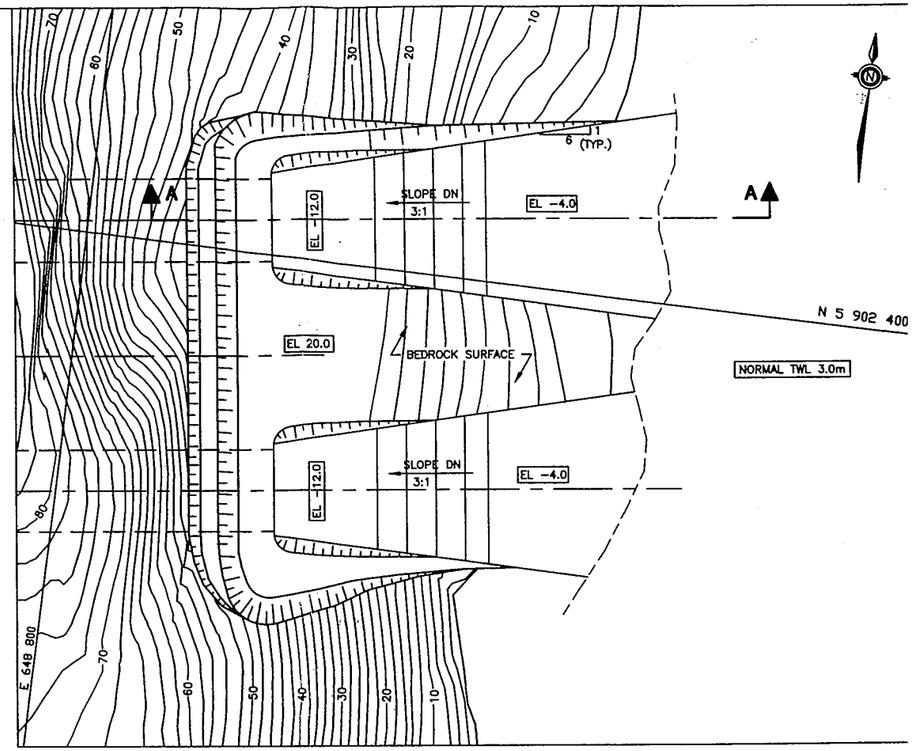
File name: PLATE12.DWG Date: 1999/01/27 @ 22:41



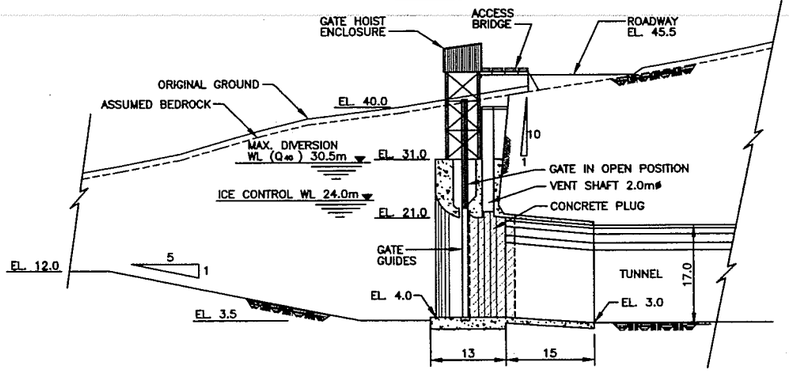
PLAN - INLET PORTAL



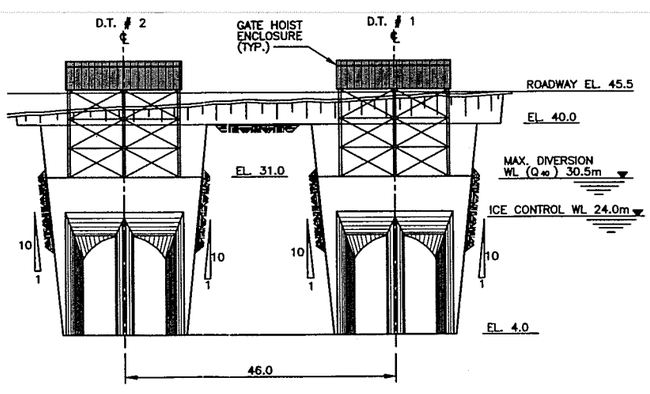
INLET PORTAL



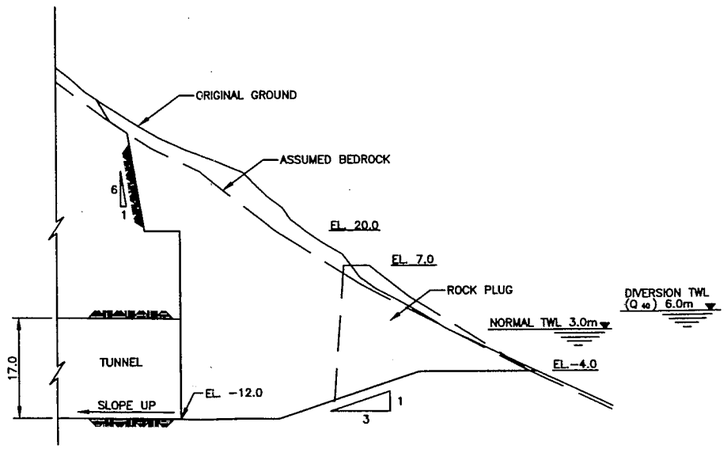
PLAN - OUTLET PORTAL



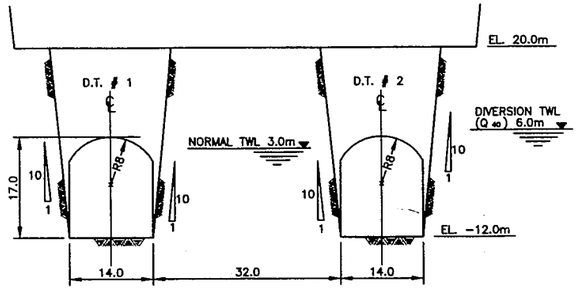
SECTION B-B THROUGH INLET PORTAL



ELEVATION - INLET PORTAL
(LOOKING DOWNSTREAM)



SECTION A-A THROUGH OUTLET PORTAL

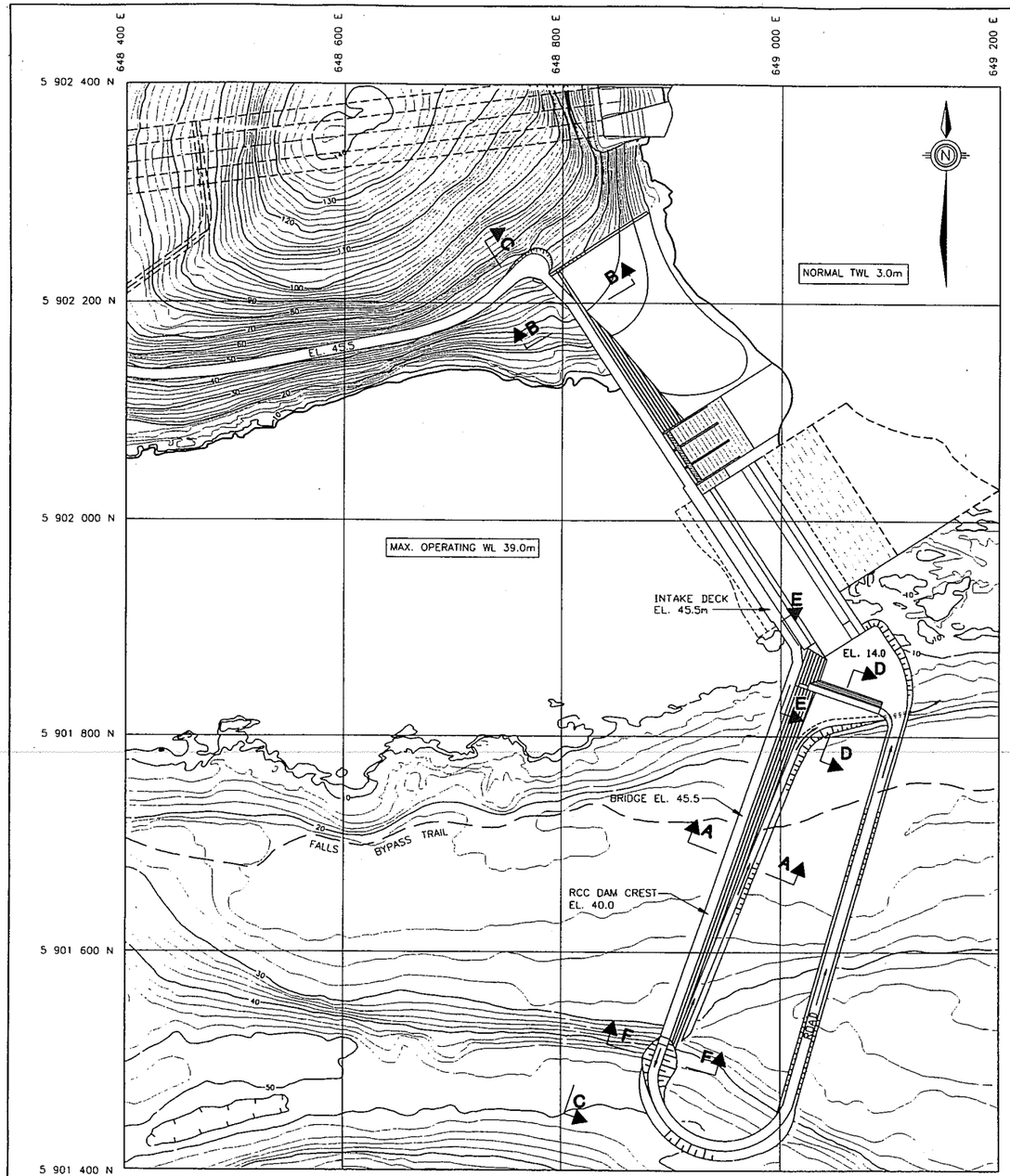


ELEVATION- OUTLET PORTAL
(LOOKING UPSTREAM)

0 10 20 30 40 50m

SNC-AGRA	Client NEWFOUNDLAND AND LABRADOR HYDRO	PLATE
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	13
Date JANUARY 1999	Title RIVER DIVERSION TUNNEL PORTAL DETAILS	
Autocad DC- 721397 - C -99- 10		

File name: PLATE13.DWG Date: 1999/01/27 @ 22:44



PLAN - DAM

0 50 100 150 200m

LEGEND:

RCC — ROLLER COMPACTED CONCRETE

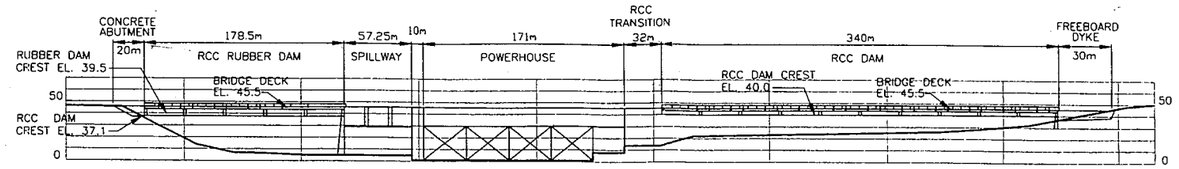
(1) — COMPACTED TILL

(2) — COMPACTED SAND - GRAVEL

(3) — TRANSITION

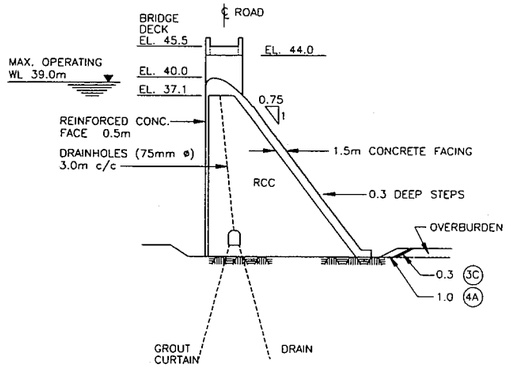
(4) — COMPACTED ROCKFILL

(RT) — ROAD TOPPING



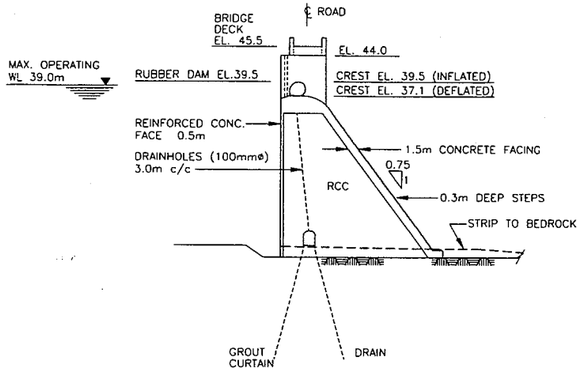
SECTION C-C

0 50 100 150 200m



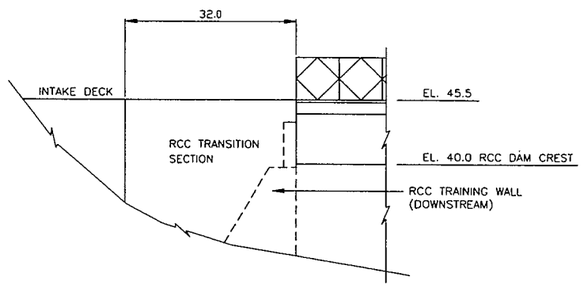
SECTION A-A

0 10 20 30 40 50m



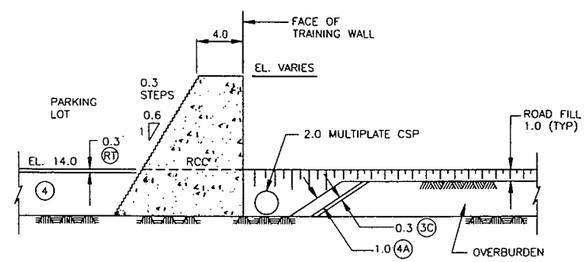
SECTION B-B

0 10 20 30 40 50m



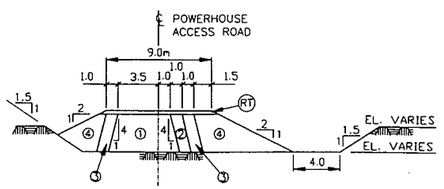
SECTION E-E

N.T.S.



SECTION D-D

0 5 10 15 20m

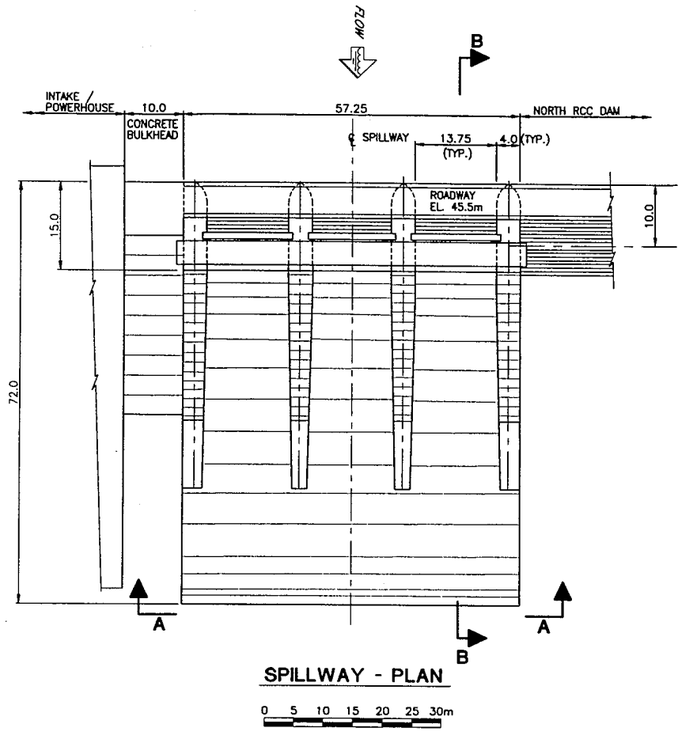


SECTION F-F

0 5 10 15 20m

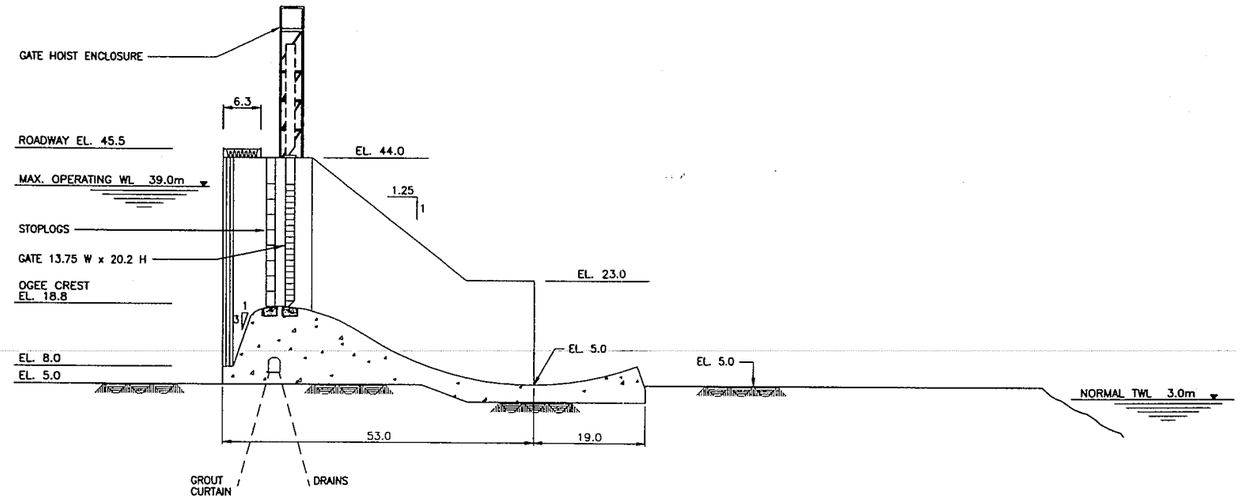
SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO		PLATE
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT		14
Date JANUARY 1999	Title ROLLER COMPACTED CONCRETE DAMS		
Autocad DC- 721397 - C -99- 11			

File name: PLATE14.DWG Date: 1999/01/27 @ 22:47



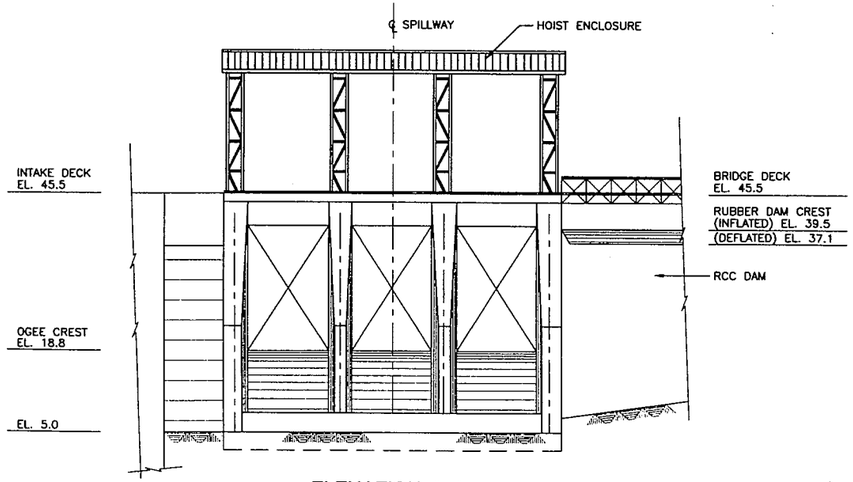
SPILLWAY - PLAN

0 5 10 15 20 25 30m



SECTION B-B

0 5 10 15 20 25 30m

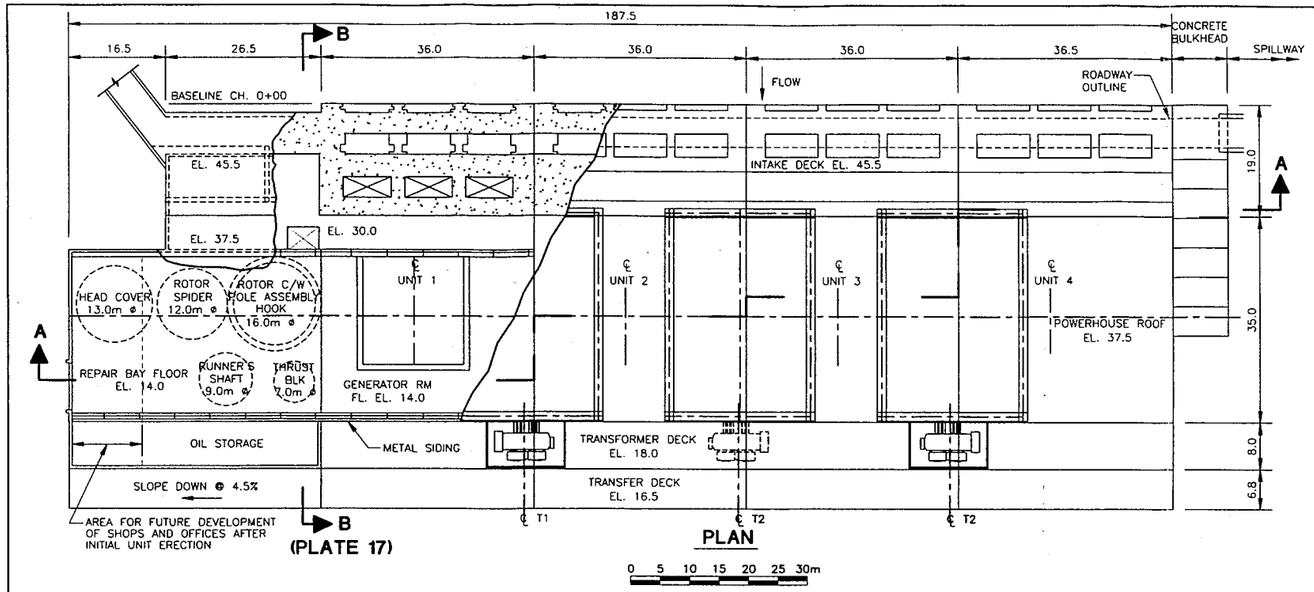


ELEVATION A-A
(LOOKING UPSTREAM)

0 5 10 15 20 25 30m

SNC-AGRA	Client NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 15
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title GATED SPILLWAY PLAN AND SECTIONS	
Autocad DC - 721397 - C - 99 - 12		

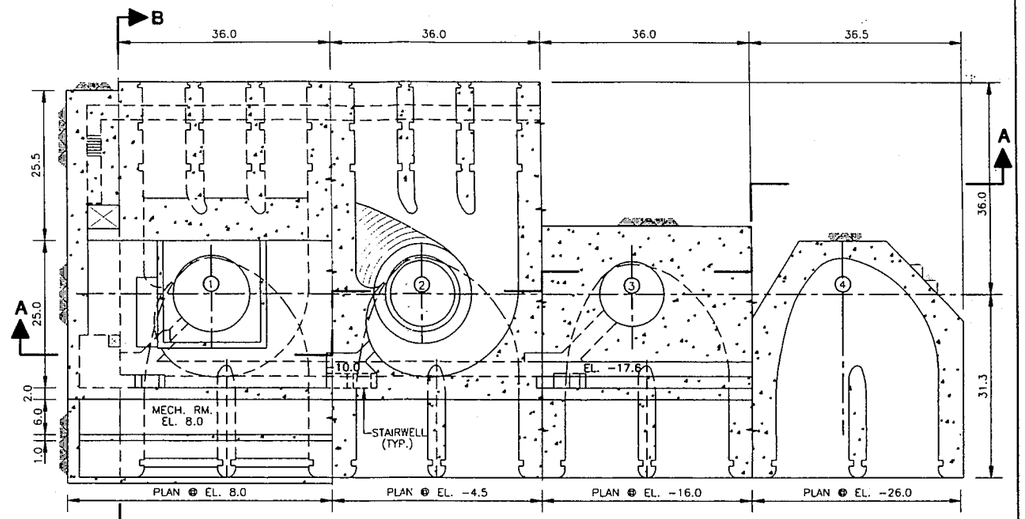
File name: PLATE15.DWG Date: 1999/01/27 @ 22:49



PLAN

0 5 10 15 20 25 30m

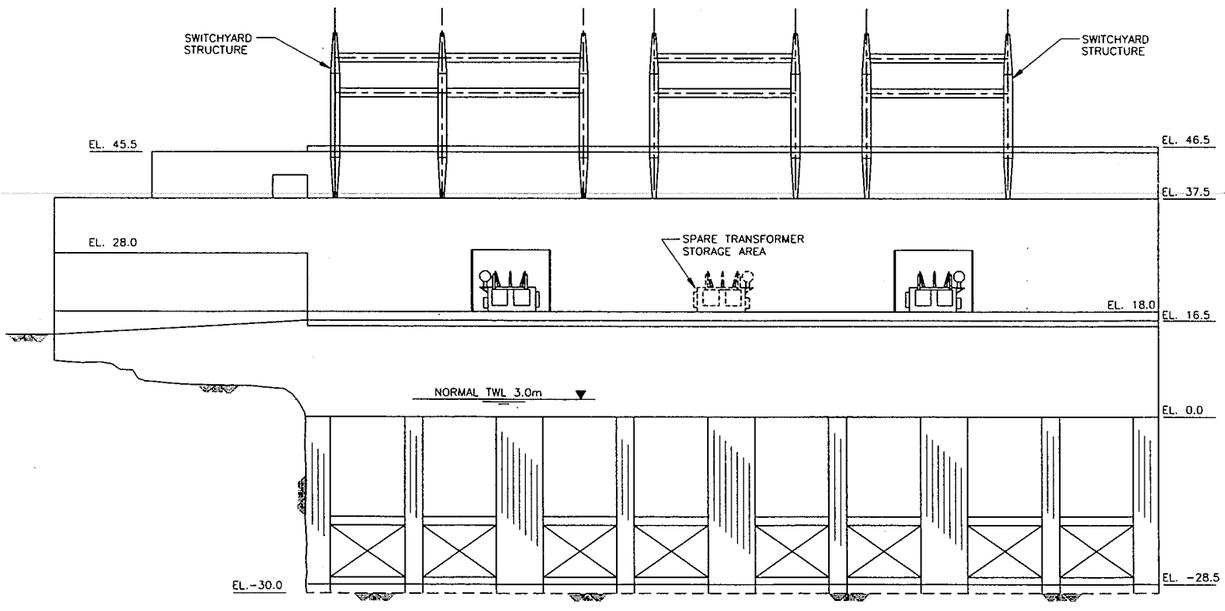
(PLATE 17)



(PLATE 17)

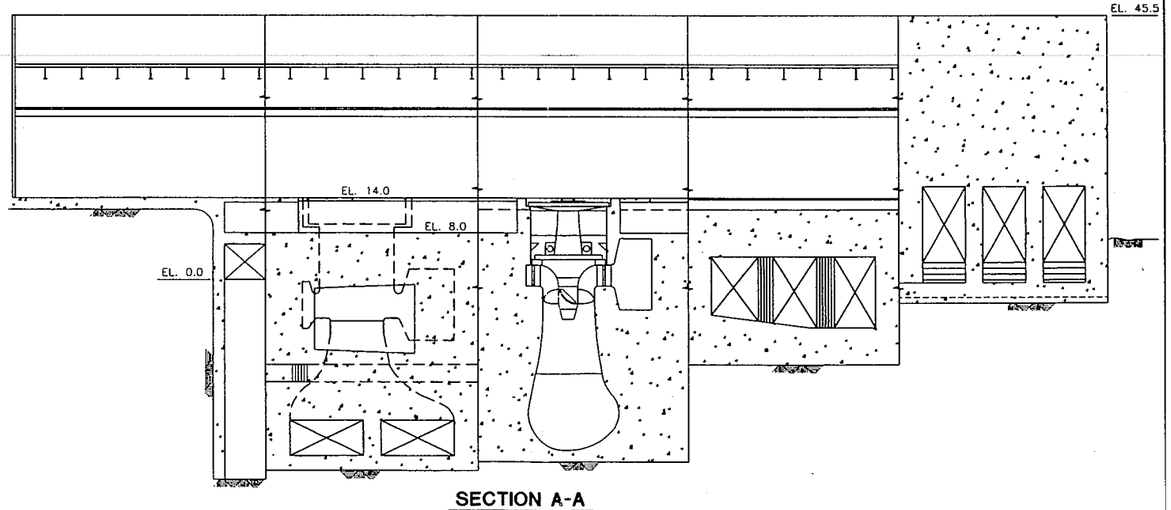
SECTIONAL PLAN

0 5 10 15 20 25 30m



DOWNSTREAM ELEVATION

0 5 10 15 20 25 30m

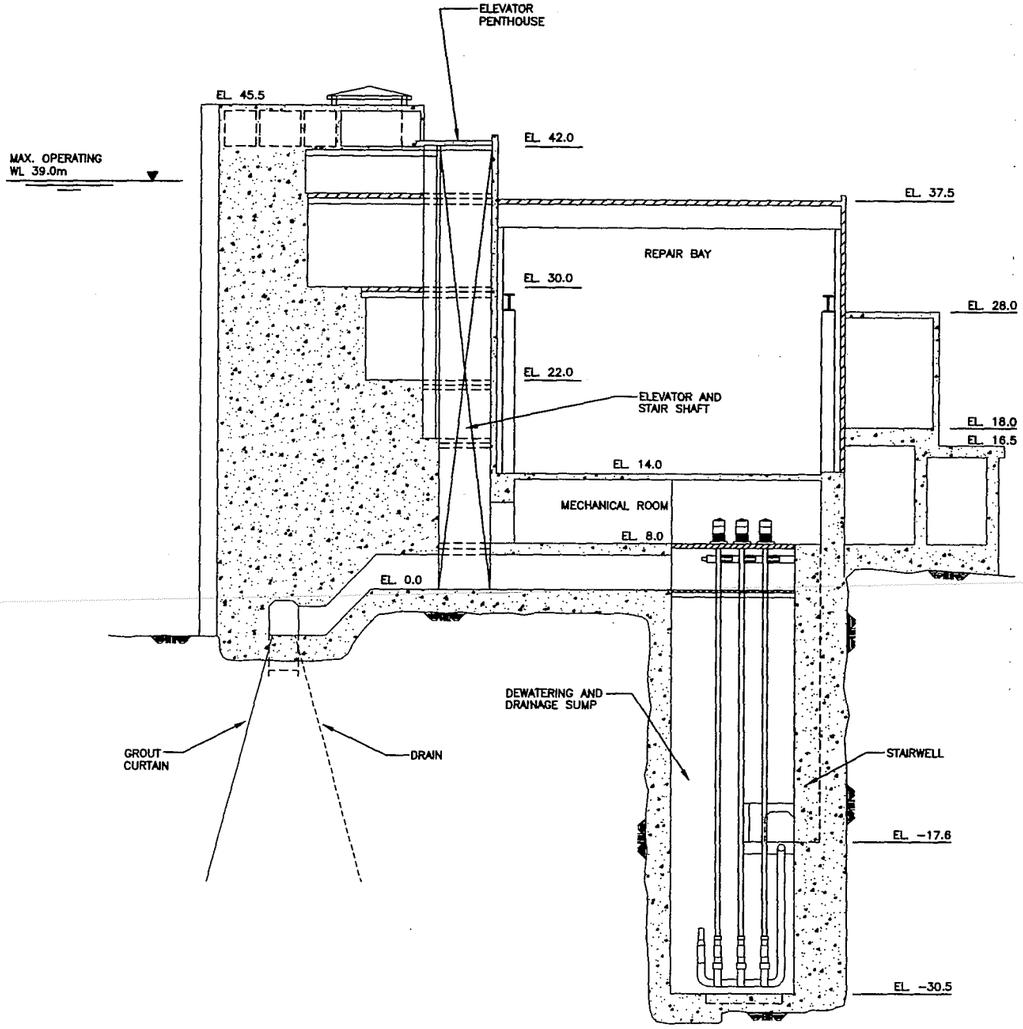


SECTION A-A

0 5 10 15 20 25 30m

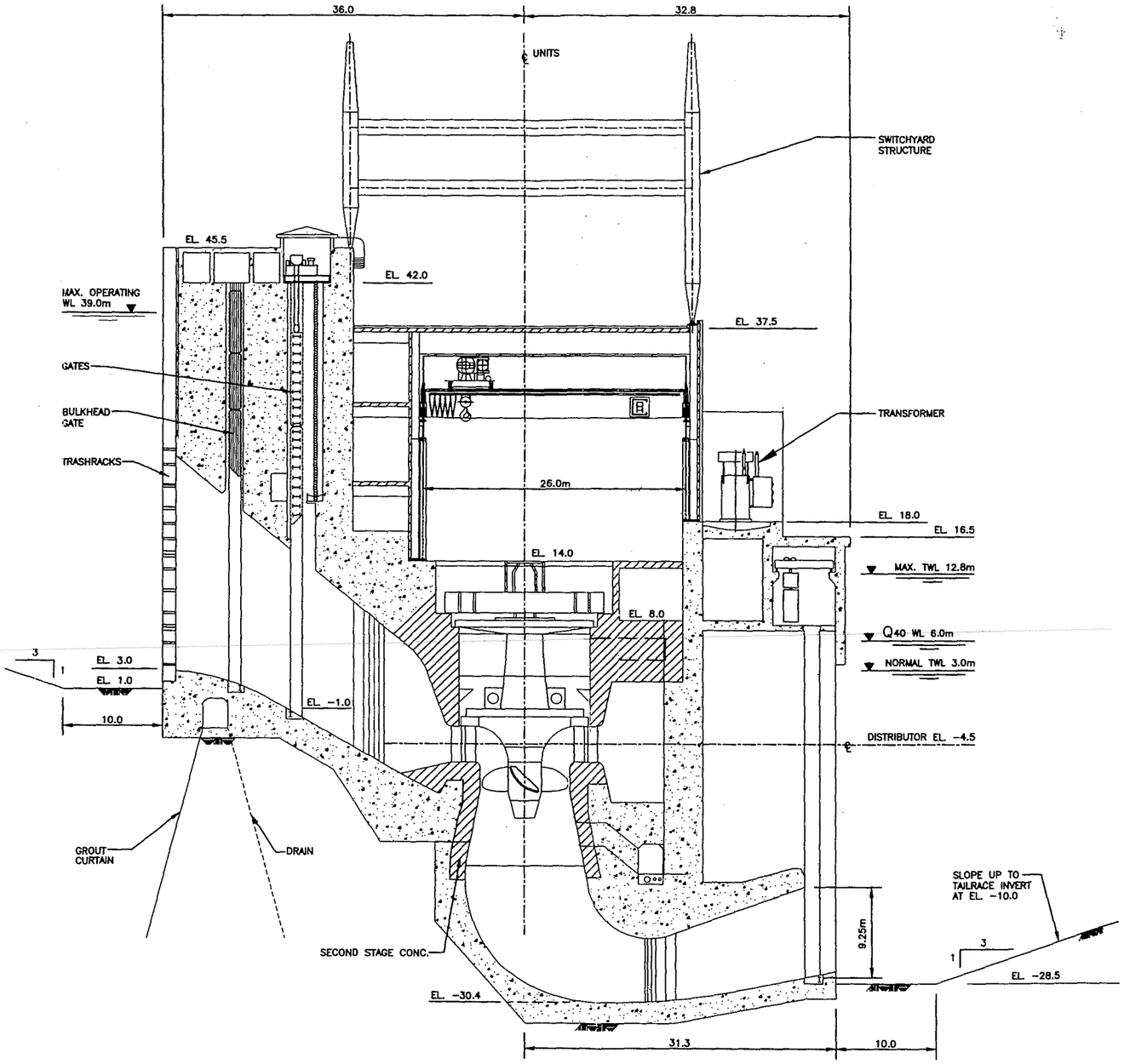
SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 16
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title INTAKE / POWERHOUSE PLAN AND SECTIONS	
Autocad DC- 721397 - C -99- 13		

File name: PLATE16.DWG Date: 1999/01/27 @ 22:51



SECTION B-B (PLATE 16)
THROUGH REPAIR BAY AND SUMP

0 5 10 15 20 25m

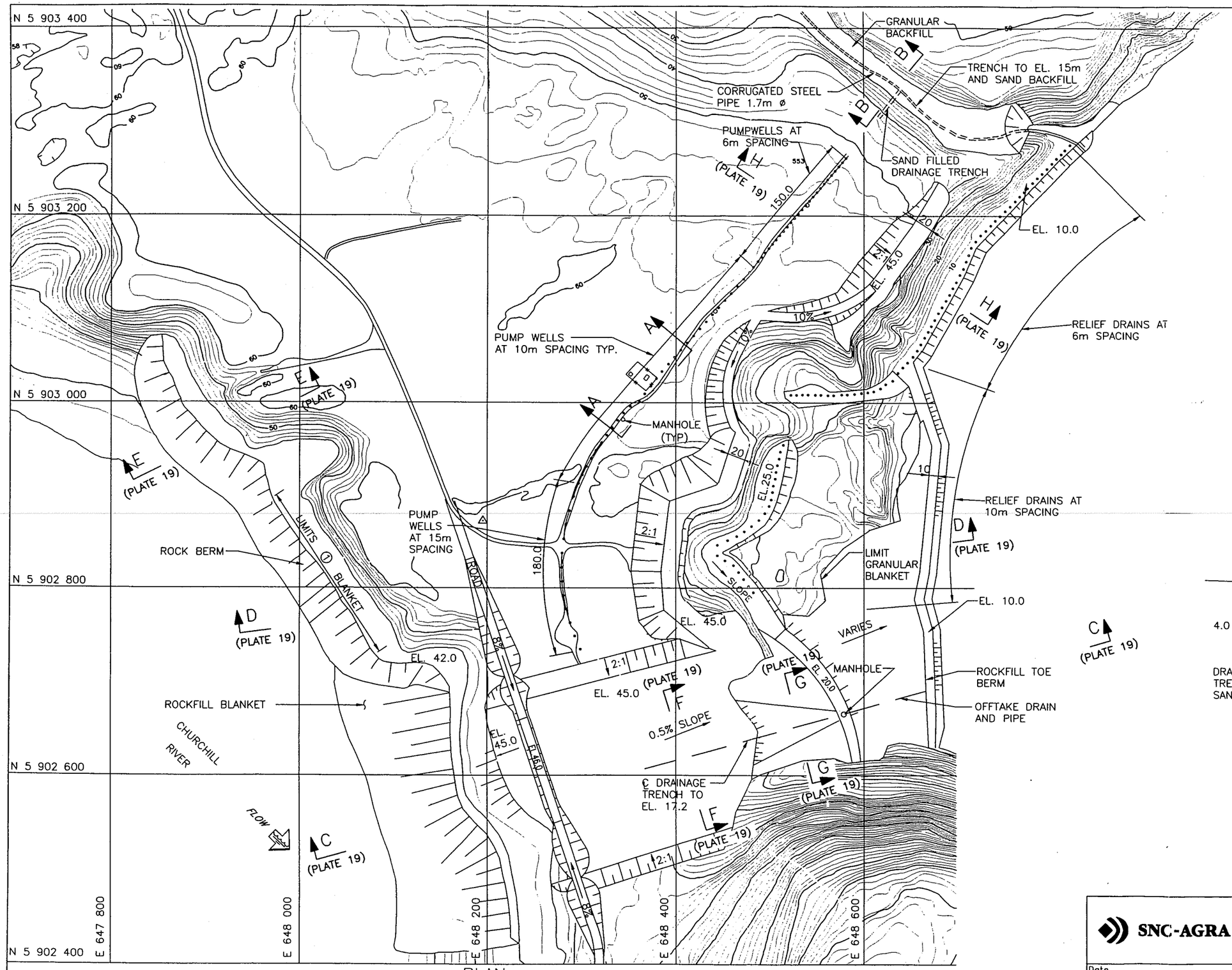


INTAKE / POWERHOUSE
TYPICAL CROSS SECTION

0 5 10 15 20 25m

SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 17
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title INTAKE / POWERHOUSE CROSS SECTIONS	
Autocad DC - 721397 - C -99- 14		

File name: PLATE17.DWG Date: 1999/01/27 @ 22:53

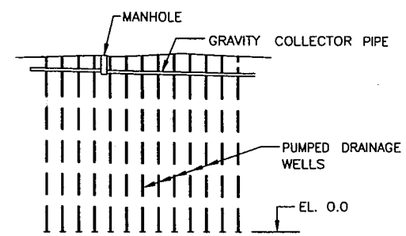


PLAN

0 50 150 150 200

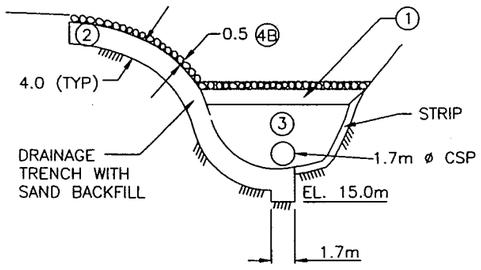
LEGEND:

① COMPACTED TILL
② FILTER SAND
③ GRAVEL
④B TUNNEL MUCK



SECTION A–A

N.T.S.

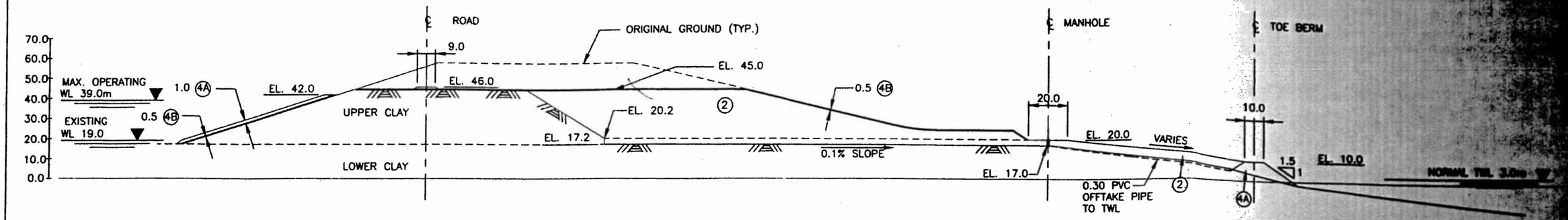


SECTION B–B

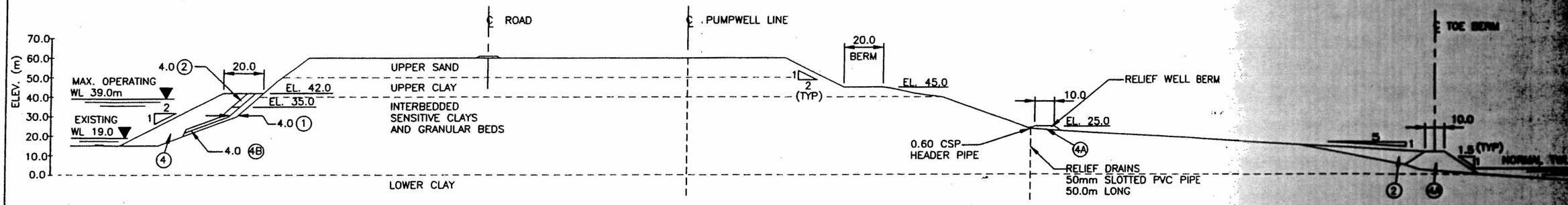
N.T.S.

SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 18
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title NORTH SPUR STABILIZATION MEASURES PLAN AND SECTIONS	
Autocad DC- 721397 - C -99- 15		

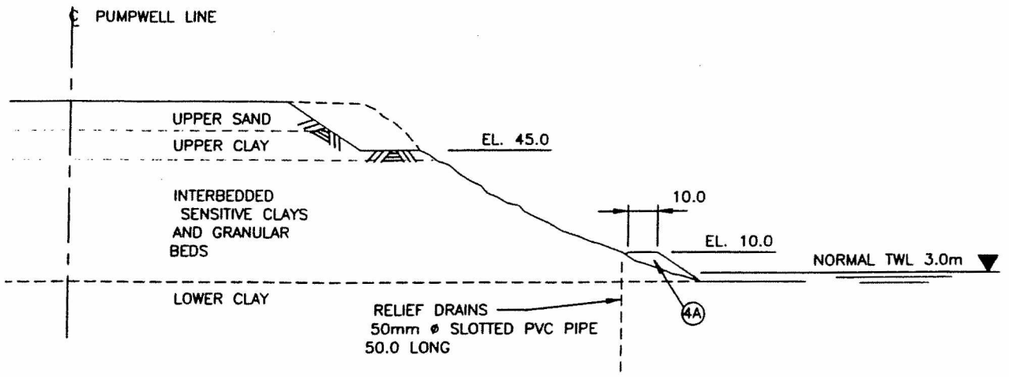
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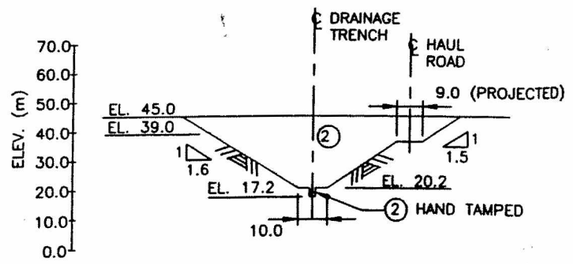
SECTION C–C



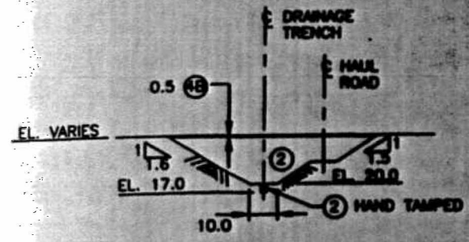
SECTION D–D



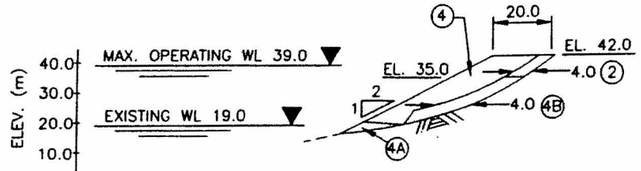
SECTION H–H



SECTION F–F



SECTION G–G



SECTION E–E

LEGEND:

(1)	COMPACTED TILL
(2)	COMPACTED GRANULAR FILL
(4)	COMPACTED ROCKFILL
(4A)	DUMPED ROCKFILL
(4B)	TUNNEL MUCK

0 50 100m

File name PLATE19.DWG Date 1999/01/27 @ 22:57

SNC-AGRA

Client NEWFOUNDLAND AND LABRADOR HYDRO

Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Title NORTH SPUR STABILIZATION MEASURES SECTIONS AND DETAILS

Date JANUARY 1999

Autocad DC- 721397 - C -99- 16

PLATE 1

$\frac{230kV}{\sqrt{3}}-115/\frac{115}{\sqrt{3}}$ V

2500-5/5/5/5/5A

230kV, 2500A

$\frac{230kV}{\sqrt{3}}-115/\frac{115}{\sqrt{3}}$ V

184kV

1200-5/5/5A

ONAN-165/165/330MVA
ONAF-220/220/440MVA

10000-5/5/5A

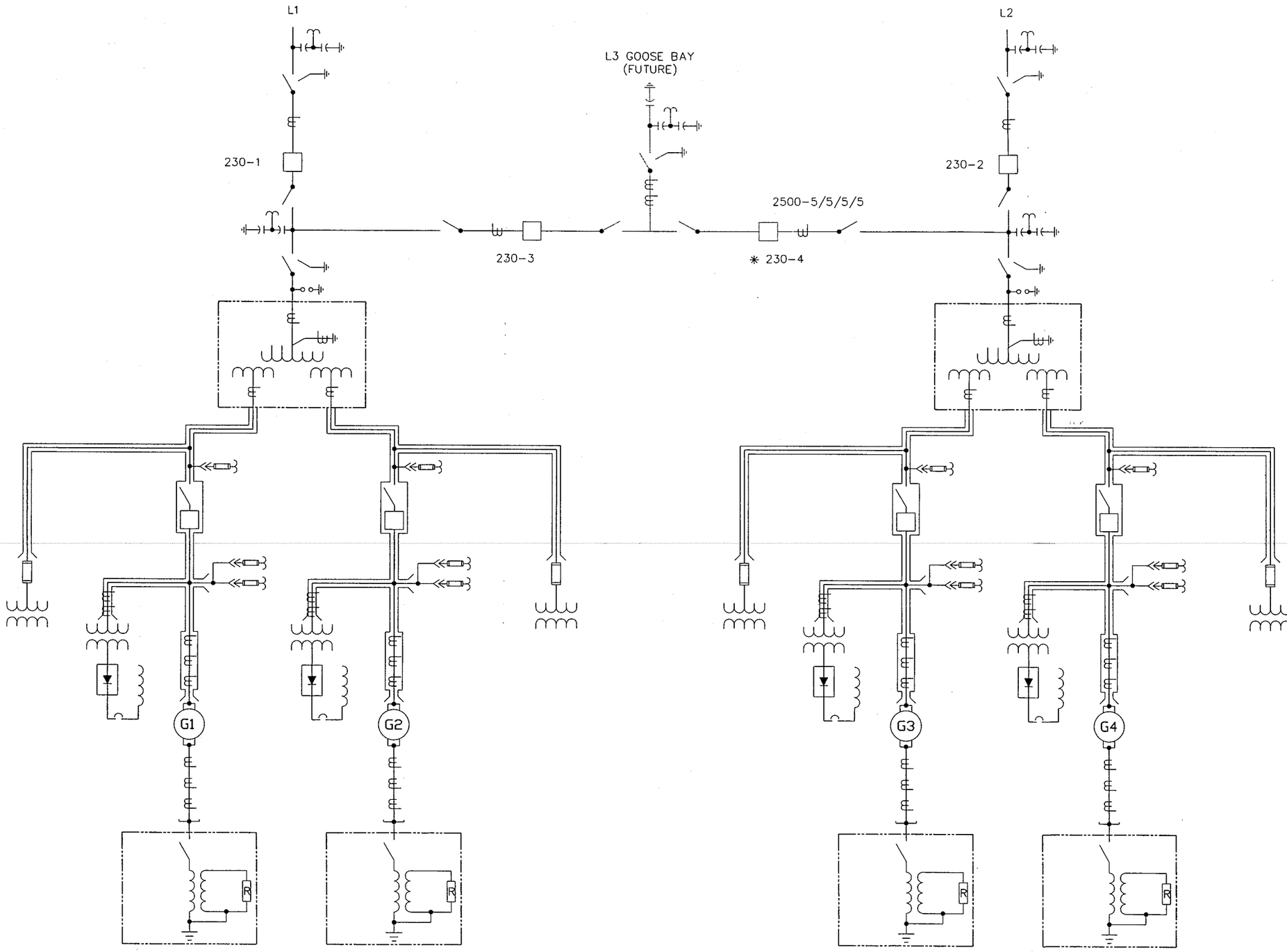
15kV-$115/\frac{115}{\sqrt{3}}$ V

10,000A
15kV

15kV-$115/\frac{115}{\sqrt{3}}$ V

220MVA, 15kV
0.95 P.F.
60hz
81.8 rpm

10000-5A



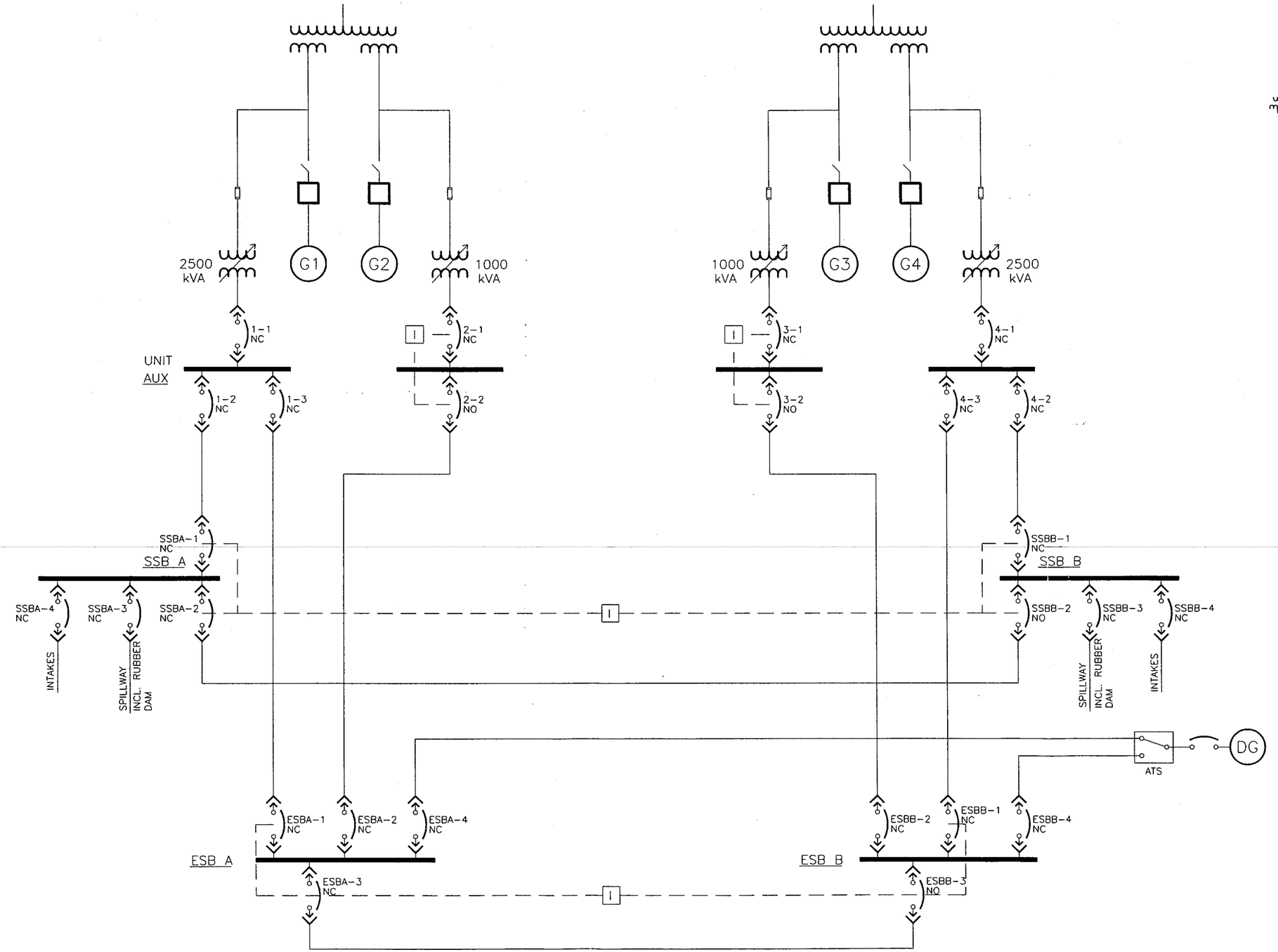
LEGEND

- TRANSFORMER THREE WINDING
- TRANSFORMER GENERAL
- FUSE
- FUSED POTENTIAL TRANSFORMER DRAW-OUT
- CURRENT TRANSFORMER
- ISOLATED PHASE BUS
- H.V. CIRCUIT BREAKER
- M.V. CIRCUIT BREAKER WITH DISCONNECT
- CAPACITOR VOLTAGE TRANSFORMER

* —— INDICATES FUTURE EQUIPMENT FOR L3 TO GOOSE BAY. BUS PROVIDED INITIALLY TO CONNECT L1 & L2.

SNC-AGRA	Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE 20
	Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	
Date JANUARY 1999	Title POWERHOUSE SINGLE LINE DIAGRAM MAIN POWER	
Autocad DC- 721397 - E -08- 01		

File name: PLATE20.DWG Date: 1999/01/27 @ 23:00



LEGEND

- TRANSFORMER THREE WINDING
- TRANSFORMER GENERAL
- FUSE
- L.V. CIRCUIT BREAKER DRAW-OUT
- INTERLOCK
- FUSED POTENTIAL TRANSFORMER DRAW-OUT
- M.V. CIRCUIT BREAKER WITH DISCONNECT
- DG — DIESEL GENERATOR
- NO — NORMALLY OPEN
- NC — NORMALLY CLOSED

SNC-AGRA

Client: NEWFOUNDLAND AND LABRADOR HYDRO

Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

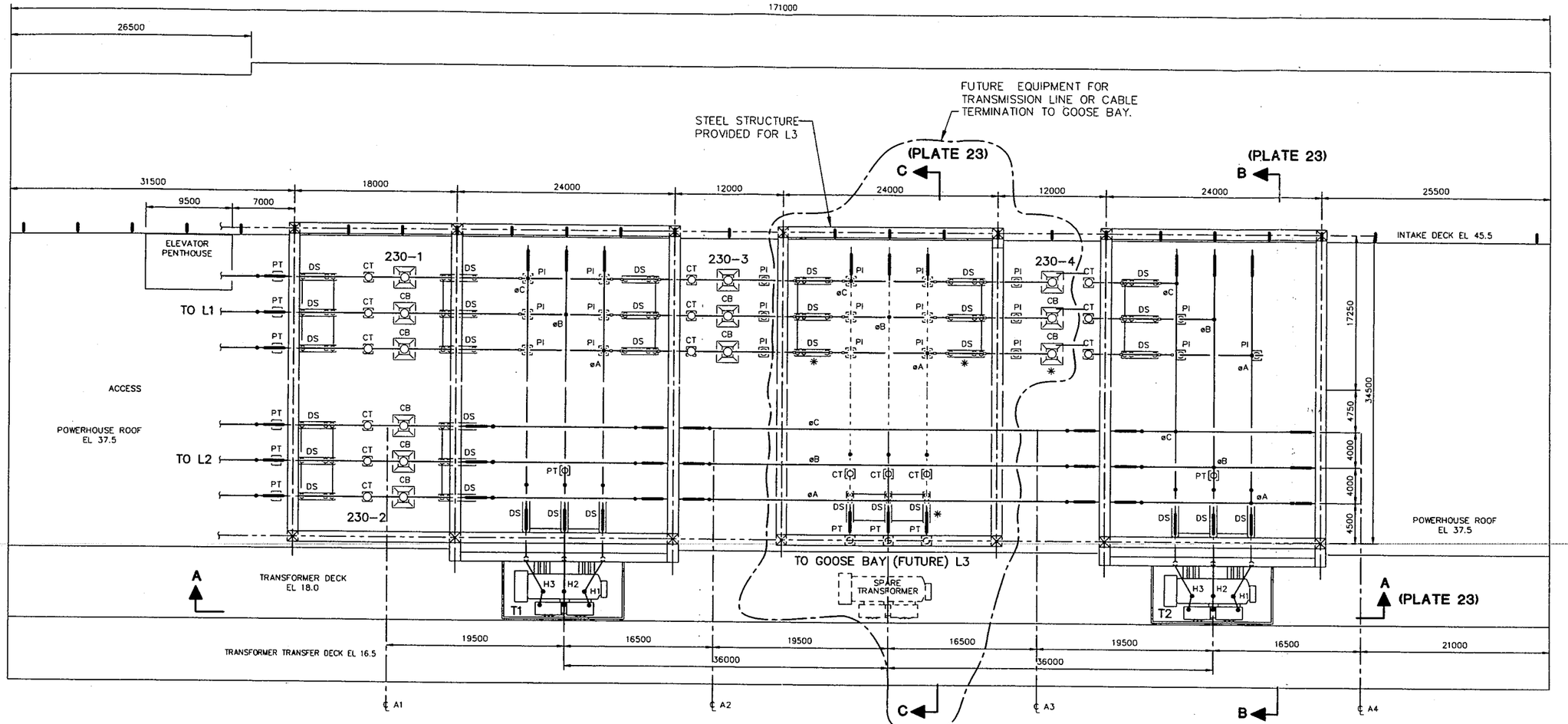
Date: JANUARY 1999

Autocad: DC- 721397 - E -08- 02

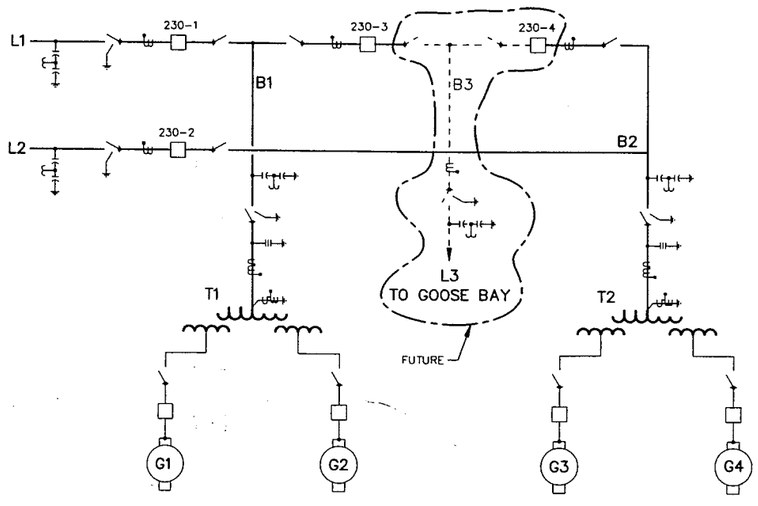
Title: STATION SERVICE SINGLE LINE DIAGRAM

PLA

File name: PLATE21.DWG Date: 1999/01/27 @ 23:01



PLAN



SINGLE LINE DIAGRAM

NOTE 1 * INDICATES EQUIPMENT REQUIRED FOR L3– TO GOOSE BAY NOT SUPPLIED UNDER PROJECT

NOTE 2 STEEL BOX STRUCTURE SUPPLIED AND INSTALLED UNDER THE PROJECT

0 5 10 15 20 25m

SNC-AGRA

Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO

Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

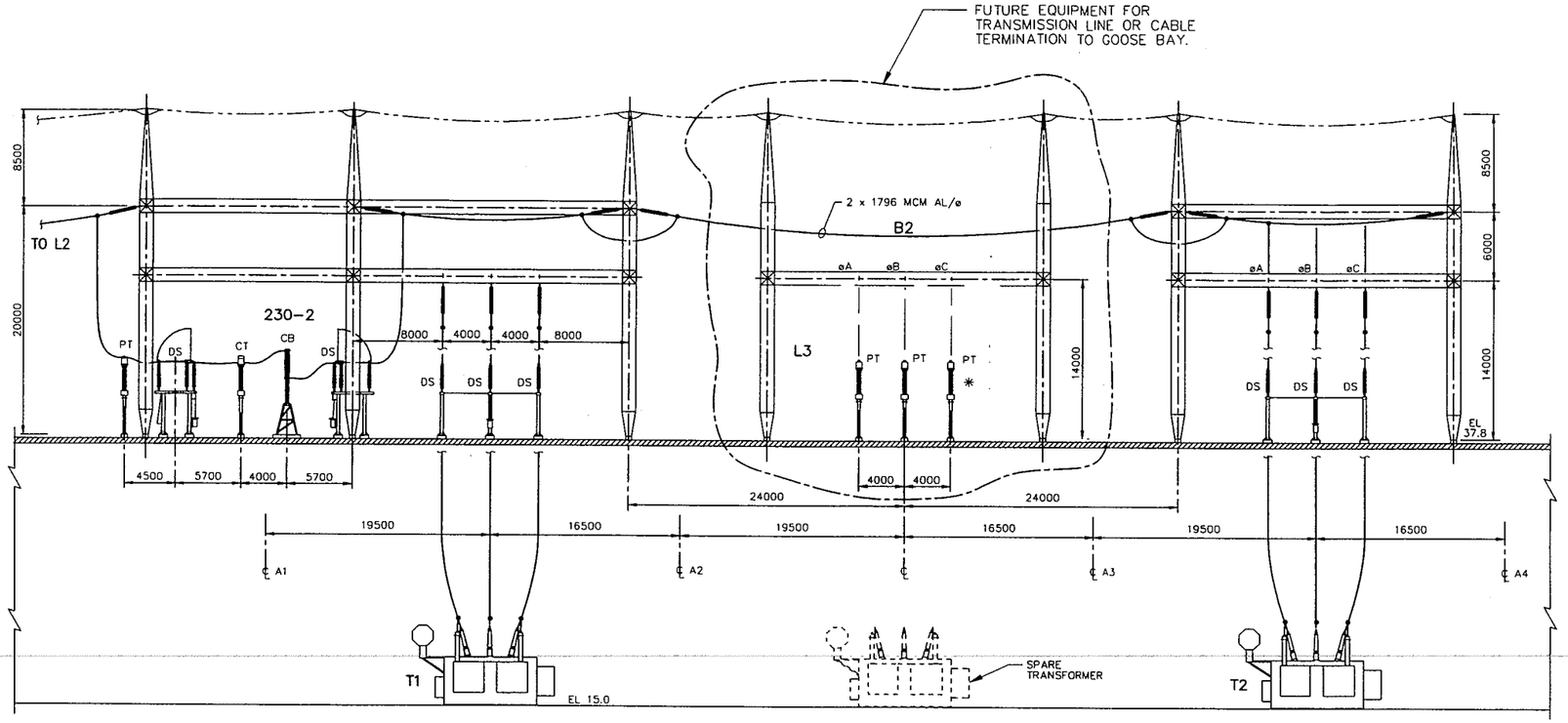
Date JANUARY 1999

Autocad DC- 721397 - E -08- 03

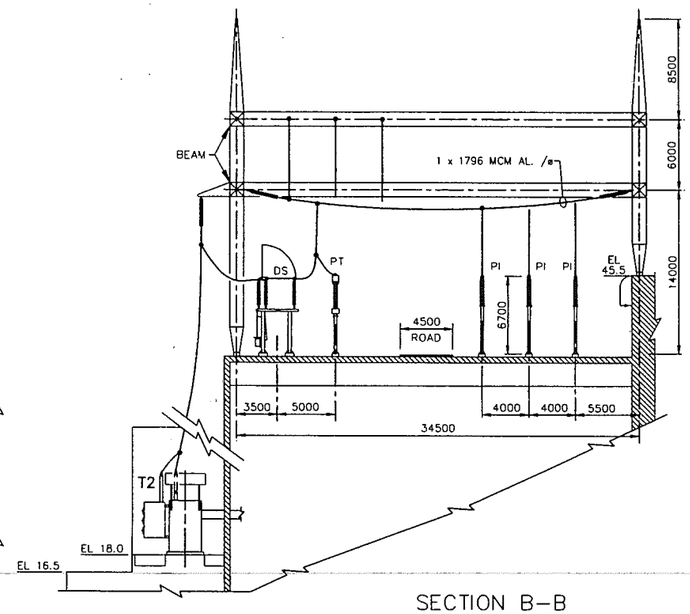
Title 230 kV SWITCHYARD GENERAL ARRANGEMENT SHEET 1

PLATE 22

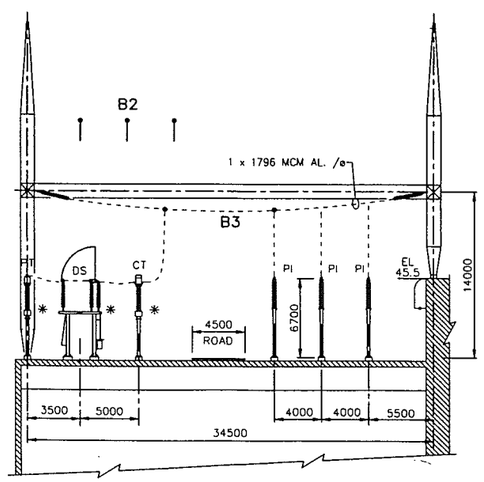
File name: PLATE22.DWG Date: 1999/01/27 @ 23:03



SECTION A–A
PLATE 22



SECTION B–B
PLATE 22



SECTION C–C
PLATE 22

LEGEND

* EQUIPMENT NOT PROVIDED (FUTURE L3)

------ CONDUCTOR NOT PROVIDED (FUTURE L3)

0 5 10 15 20 25m

SNC-AGRA

Client: NEWFOUNDLAND AND LABRADOR HYDRO

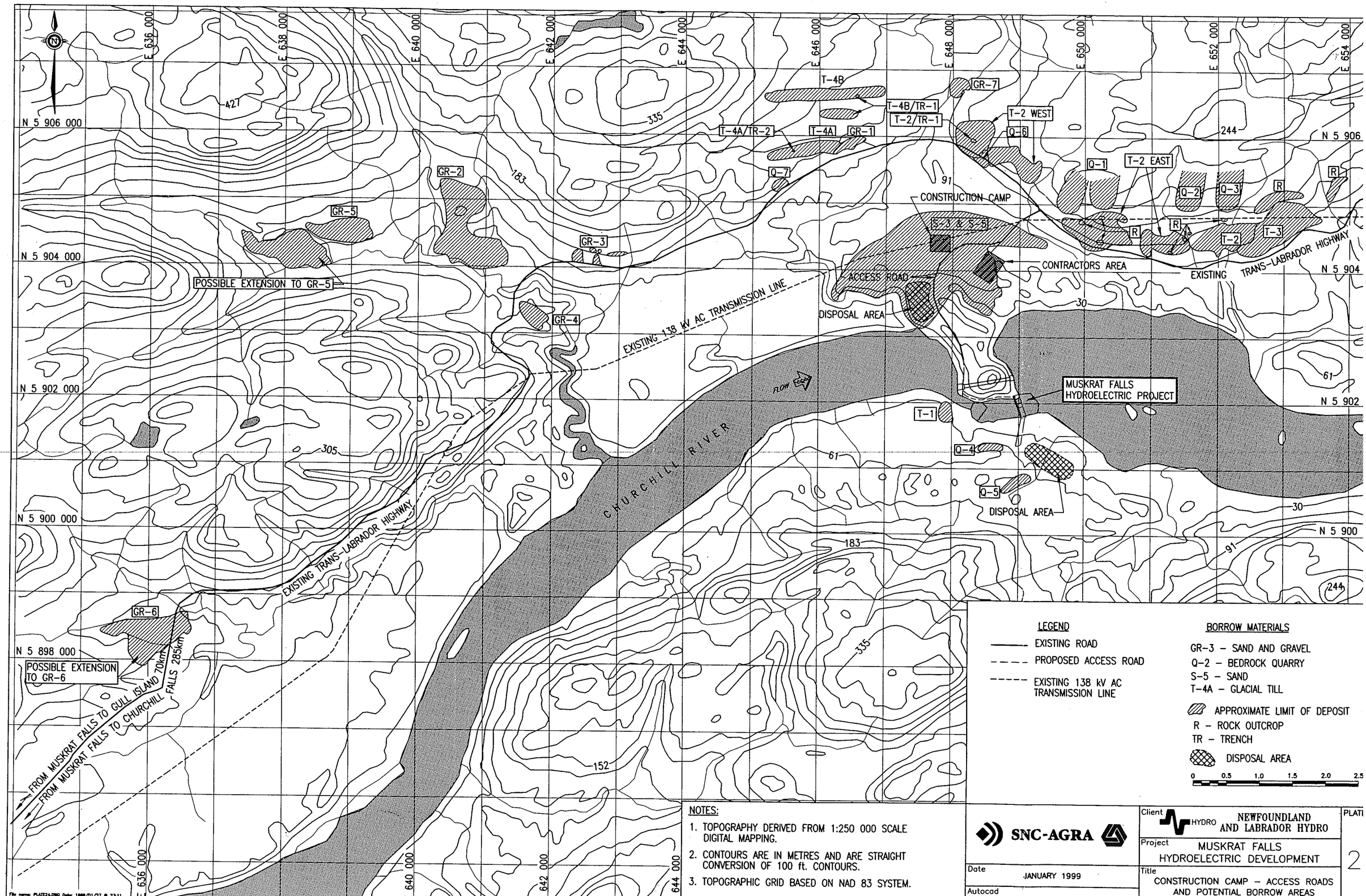
Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Date: JANUARY 1999

Autocad: DC- 721397 - E -08- 04

Title: 230 kV SWITCHYARD GENERAL ARRANGEMENT SHEET 2

PLATE 2



LEGEND

- EXISTING ROAD
- PROPOSED ACCESS ROAD
- EXISTING 138 kV AC TRANSMISSION LINE

BORROW MATERIALS

- GR-3 – SAND AND GRAVEL
- Q-2 – BEDROCK QUARRY
- S-5 – SAND
- T-4A – GLACIAL TILL
- APPROXIMATE LIMIT OF DEPOSIT
- R – ROCK OUTCROP
- TR – TRENCH
- DISPOSAL AREA

0 0.5 1.0 1.5 2.0 2.5

NOTES:

1. TOPOGRAPHY DERIVED FROM 1:250 000 SCALE DIGITAL MAPPING.
2. CONTOURS ARE IN METRES AND ARE STRAIGHT CONVERSION OF 100 ft. CONTOURS.
3. TOPOGRAPHIC GRID BASED ON NAD 83 SYSTEM.

SNC-AGRA

Client: NEWFOUNDLAND AND LABRADOR HYDRO

Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

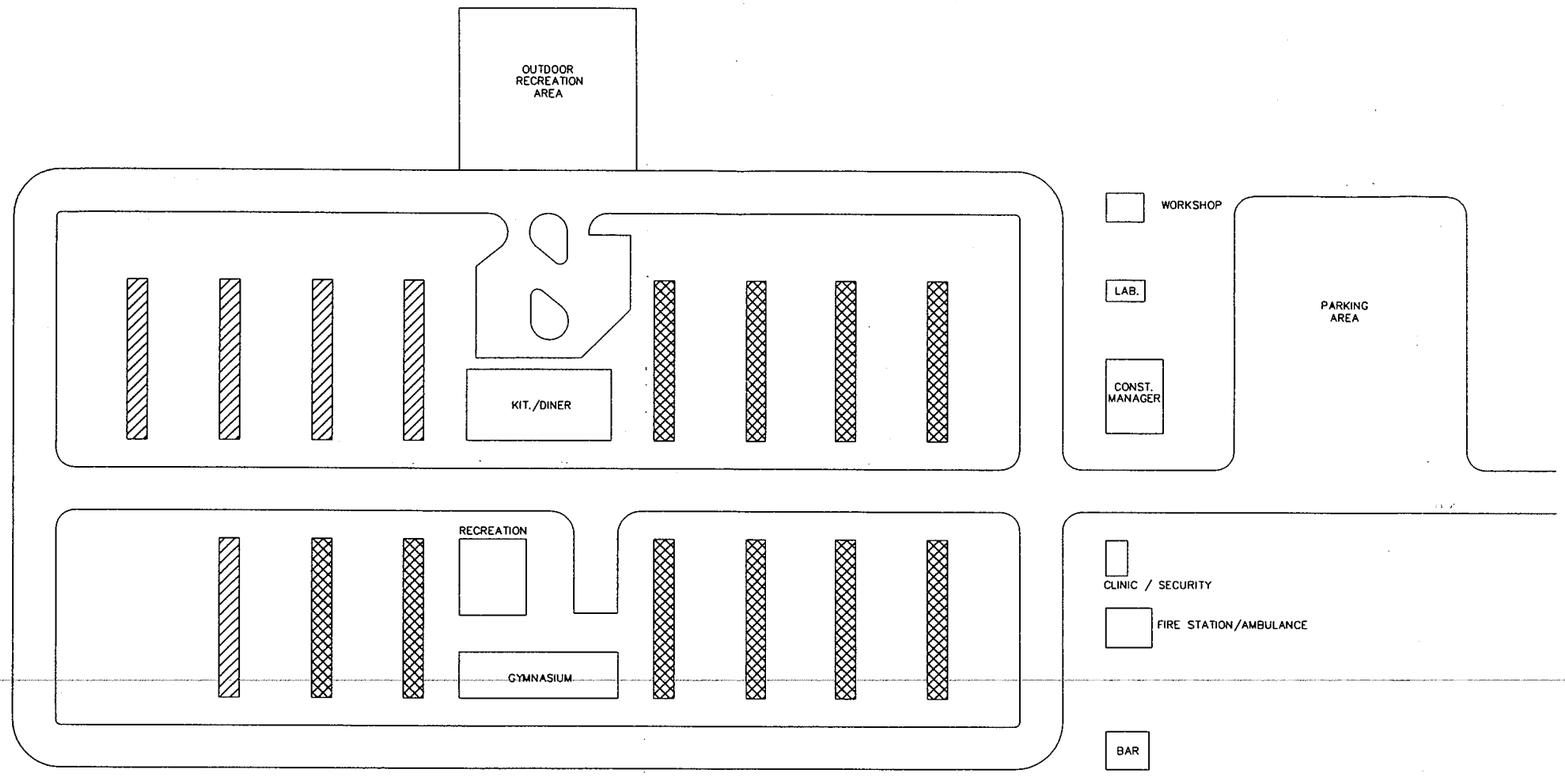
Title: CONSTRUCTION CAMP – ACCESS ROADS AND POTENTIAL BORROW AREAS LOCATION PLAN

Date: JANUARY 1999

Autocad: DC- 721397 - C -99- 17

PLATE 2

File name: PLATE24.DWG Date: 1999/01/27 @ 23:11



PLAN

0 25 50 75 100m

LEGEND

DORMITORY TYPE 1

8.536m X 70.133m X 1 STOREY
43 PERSON COMPLEX
SINGLE ROOMS WITH SHARED WASHROOMS.
5 COMPLEXES @ 43 = 215 ROOMS

DORMITORY TYPE 2

8.536m X 69.490m X 2 STOREYS
50 PERSON PER STOREY = 100 TOTAL
SINGLE ROOMS WITH CENTRAL WASHROOMS.
10 COMPLEXES @ 100 = 1000 ROOMS

SNC-AGRA

Client HYDRO NEWFOUNDLAND AND LABRADOR HYDRO

Project MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

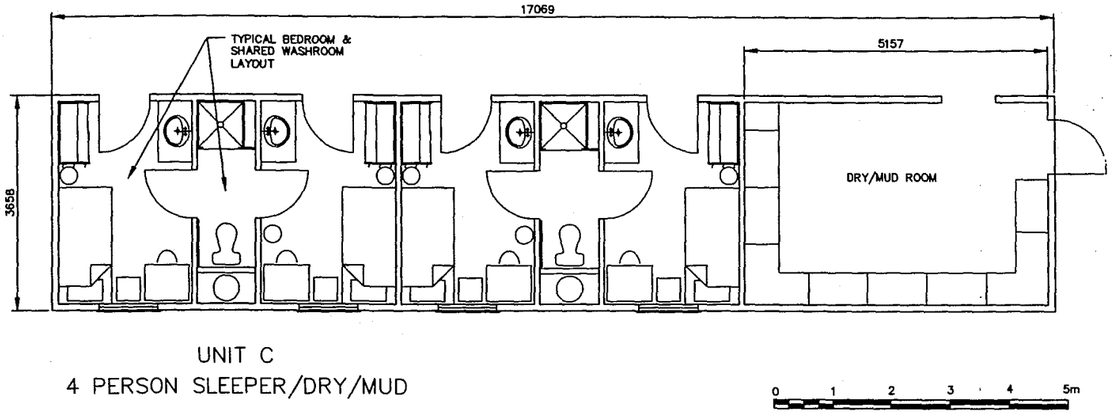
Date JANUARY 1999

Autocad DC- 721397 - C -99- 18

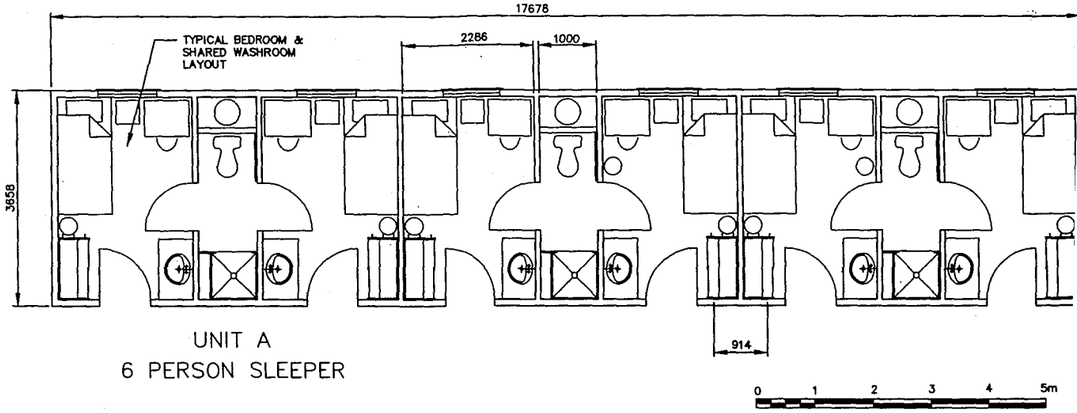
Title CONSTRUCTION CAMP GENERAL ARRANGEMENT

PLATE 25

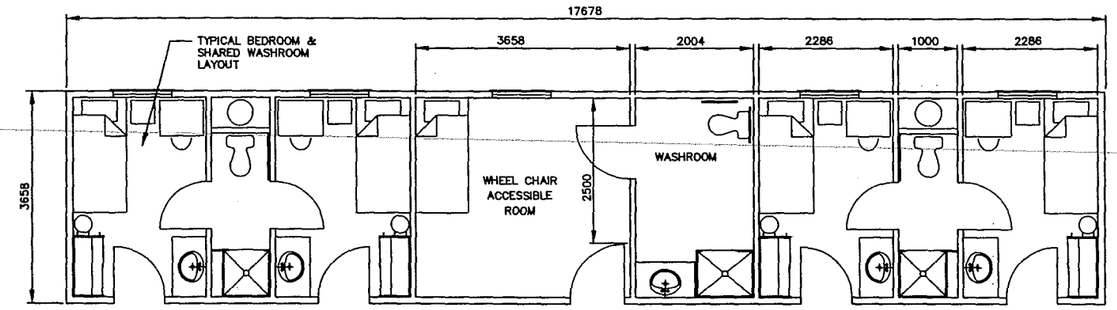
File name PLATE25.DWG Date: 1999/01/27 @ 23:13



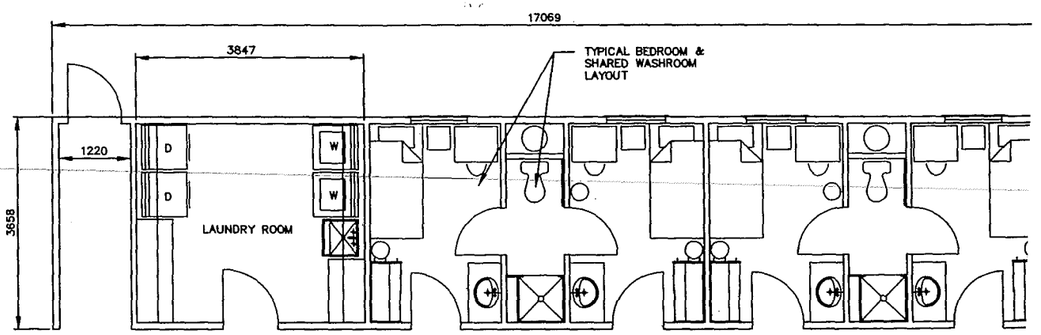
UNIT C
4 PERSON SLEEPER/DRY/MUD



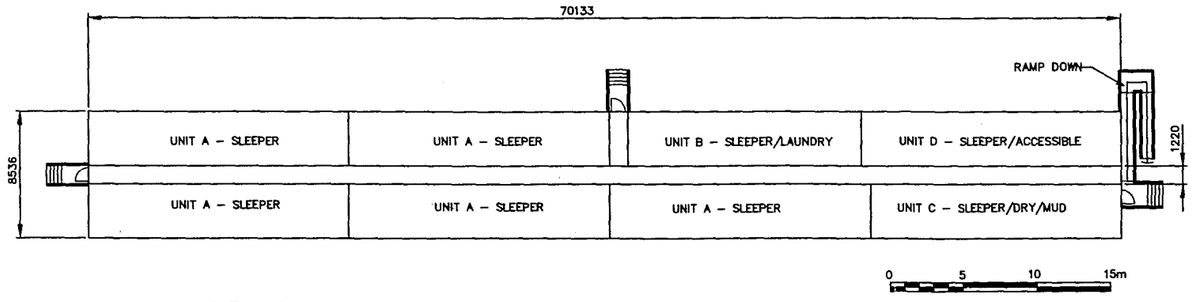
UNIT A
6 PERSON SLEEPER



UNIT D
5 PERSON SLEEPER/ACCESSIBLE

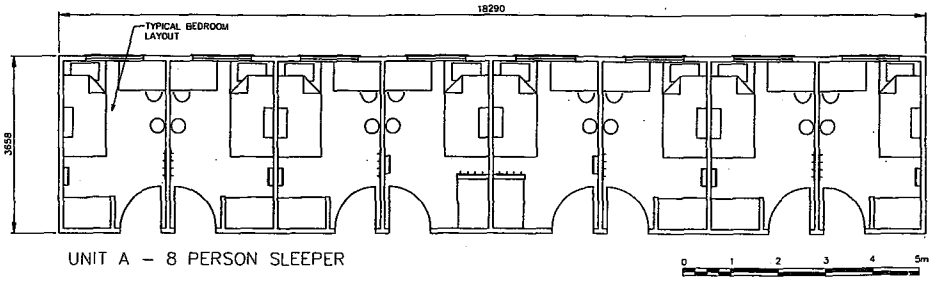


UNIT B
4 PERSON SLEEPER/LAUNDRY

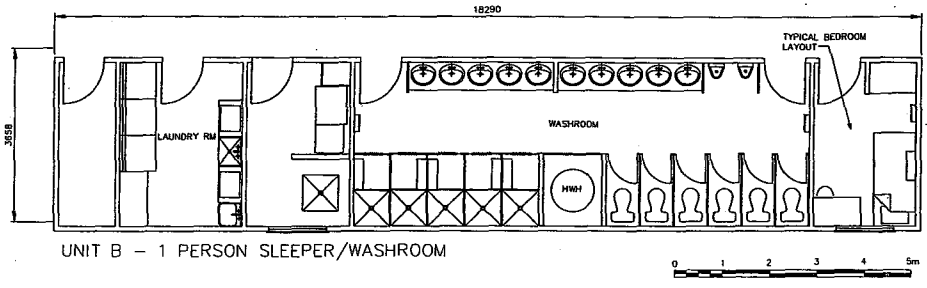


KEY PLAN
TYPICAL 43 PERSON COMPLEX

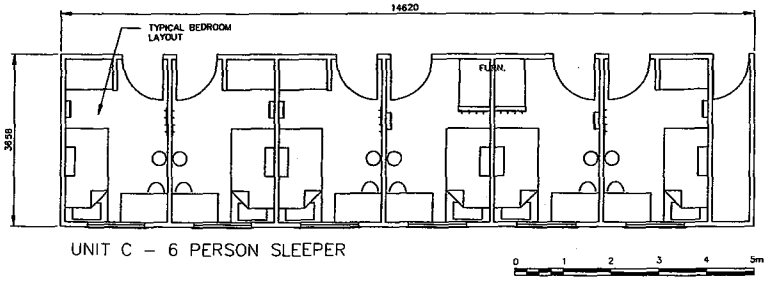
SNC-AGRA	Client: HYDRO NEWFOUNDLAND AND LABRADOR HYDRO
	Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
Date: JANUARY 1999	Title: CONSTRUCTION CAMP SUPERVISORS AND VISITORS ACCOMMODATION COMPLEX
Autocad: DC- 721397 - C -99- 19	



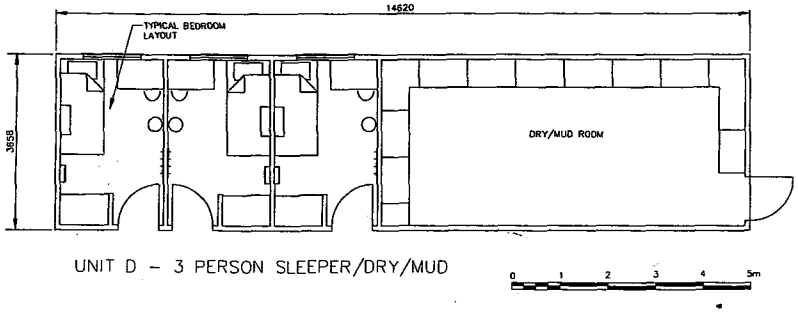
UNIT A – 8 PERSON SLEEPER



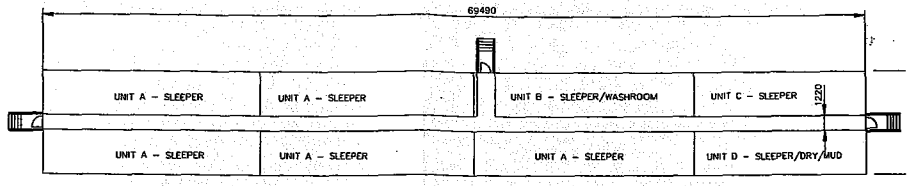
UNIT B – 1 PERSON SLEEPER/WASHROOM



UNIT C – 6 PERSON SLEEPER



UNIT D – 3 PERSON SLEEPER/DRY/MUD



KEY PLAN

TYPICAL 50 PERSON PER FLOOR COMPLEX (2 Floors = 100 PERSONS)

0 5 10 15m

File name: PLATE27.DWG Date: 1999/01/27 @ 23:15

SNC-AGRA	Client: HYDRO NEWFOUNDLAND AND LABRADOR HYDRO	PLATE
	Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT	27
Date: JANUARY 1999	Title: CONSTRUCTION CAMP TRADES ACCOMMODATION COMPLEX	
Autocad: DC- 721397 - C -99- 20		

YEAR −2 | YEAR −1 | YEAR 1 | YEAR 2 | YEAR 3 | YEAR 4 | YEAR 5 | YEAR 6

F M A M J J A S O N D | J F M A M J J A S O N D | J F M A M J J A S O N D | J F M A M J J A S O N D | J F M A M J J A S O N D | J F M A M J J A S O N D | J F M A M J J A S O N D | J F M A M

PROJECT MANAGEMENT

PRELIMINARY WORKS

- Camps and Services
- Construction Power

CAMP OPERATIONS

NORTH SPUR & DIVERSION

- Tunnel Heading
- Tunnel Benching
- Adit
- U/S Cofferdam

MAIN CIVIL WORKS

- North RCC Dam
- Spillway
  - Left Abutment
  - Pier 1
  - Pier 2
  - Right Abutment
- Powerhouse
  - Unit 4
  - Unit 3
  - Unit 2
  - Unit 1
  - Service Bay
- South RCC Dam

TURBINES/GENERATORS

SWITCHYARD

CLEARING

Engineering – Camps & Services
Prepare Tender Preliminary Documents
Works & Camp Contract
E/A
Procure Camp Buildings
Erect Camp Buildings
Phase I Camp
Erect Camp Buildings – Phase II
Phase III Camp
Procure Materials & Equipment
Install Camp Services
Install Camp Services – Phase II
Engineering
Prepare Tender Documents
Tender Transformer Contract
E/A
Procure Materials
Construction Power
Engineering
Prepare Documents
Tender
E/A
Mobilize
Operations, Maintenance and Catering
Preparation & Engineering
Field Investigation & Survey
Engineering & Model Testing
Report
Diversion / Spillway
Engineering
Engineering – North Spur Stabilization
Prepare Tender Documents
Tender Diversion & North Spur Stabilization Contract
E/A
Mobilize
Excavation, Fills & Drainage System
Control of River
Fabricate Gate Guides
U/S Portal Excavation
Complete Portal Exc
Concrete Portal Structure
Gate Guides
Gates & Hoists
Control of River
PROJECT START JUNE 11
PROJECT RELEASE JAN. 1
U/S
Downstream
U/S
U/S
Downstream
Portal To South Tunnel
Complete
Concrete Plug
Grout
Flood
North Bank Access Road
Excavate Downstream Portal
Complete
Flood
Remove U/S & D/S Rock Plugs
Grout
U/S Cofferdam Close Phase II
Flood
D/S Cofferdam
Flood
Flood
Flood
Concrete Plug
Grout
Concrete Plug
Grout
Flood
U/S Cofferdam
Lower
Flood
D/S Cofferdam & Plug Lower
Remove
Engineering – Main Civil Works
Prepare Tender Documents
Tender Main Civil Works Contract
E/A
Mobilize
Overburden Excavation
Finish Prep
RCC
Structural Concrete
Rubber Dam
Fabricate Spillway Gates, Guides & Hoists
Rock Excavation
U/S Wall & Railway
Bridge Gate Guides, Gates & Hoists
D/S Wall & Chute
U/S Pier & Railway
Bridge Gate Guides, Gates & Hoists
D/S Wall & Chute
U/S Pier & Railway
Bridge Gate Guides, Gates & Hoists
D/S Wall & Chute
U/S Wall & Railway
D/S Wall & Chute
Engineering – BOP Mechanical Electrical Contract
Prepare Tender Documents
Tender BOP Mechanical Electrical Contract
E/A
Fabricate Intake Gates, Stop Logs & Trashracks
Rock Excavation
Intake Concrete
Structural Steel
Intake Gate Guides
Intake Gates & Hoists
Draft Tube Concrete
Powerhouse Walls
Structural Steel
Embedded Pieces
Phase II Concrete
Turbine-Generator
Commission
DT Guides
Bulkhead
Remove DT Bulkhead
Intake Concrete
Structural Steel
Intake Gate Guides
Intake Gates & Hoists
Draft Tube Concrete
Powerhouse Walls
Structural Steel
Embedded Pieces
Phase II Concrete
Turbine-Generator
Commission
DT Guides
Bulkhead
Remove DT Bulkhead
Intake Concrete
Structural Steel
Intake Gate Guides
Intake Gates & Hoists
Draft Tube Concrete
Powerhouse Walls
Structural Steel
Embedded Pieces
Phase II Concrete
Turbine-Generator
Commission
DT Guides
Gates
Intake Concrete
Structural Steel
Intake Gate Guides
Intake Gates & Hoists
Draft Tube Concrete
Walls
Structural Steel
Embedded Pieces
Phase II Concrete
Turbine-Generator
Commission
DT Guides
Gates
FIRST POWER 1 OCT YEAR 5
Concrete Gravity Dam
Structural Steel
Erection Bay Concrete
Structural Steel
Powerhouse Crane
Auxiliary Mechanical/Electrical Systems
Transition Gravity Dam Concrete
Overburden Excavation
Finish Prep
RCC
Structural Concrete
Bridge
Engineering
Prepare Tender Documents
Tender Turbine Generator Supply & Erect Contract
E/A
Detail Engineering
Fabricate & Deliver Turbine Generators
Engineering Switchyard
Switchyard Superstructure
Prepare Tender Documents
Tender Switchyard Contract
E/A
Fabricate & Deliver Main Transformers
Erect Switchyard Electrical Equipment
Engineering
Prepare Tender Documents
Tender Reservoir Clearing
E/A
Mobilize
Clear Reservoir Below Construction Flood Level
Complete Reservoir Clearing
Clear Camp Area
Clear North & South RCC Dam

ENGINEERING
ACTIVITY
CRITICAL PATH
CHRISTMAS SHUTDOWN
E/A EVALUATE/AWARD
U/S UPSTREAM
D/S DOWNSTREAM

SNC-AGRA

Client: NEWFOUNDLAND AND LABRADOR HYDRO

Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Date: JANUARY 1999

Autocad: DC- 721397 - C -99-022

Title: PROJECT MASTER SCHEDULE

PLATE 28

File name: NEWPLATE28.DWG Date: 1999/01/27 @ 23:18

APPENDIX A

DETAILED CAPITAL COST ESTIMATE

APPENDIX A

DETAILED CAPITAL COST ESTIMATE

TABLE 1.1
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
SUMMARY OF CAPITAL COST ESTIMATE

Code	Description	June 1998 Canadian Dollars
1000	Permanent Support Facilities	6,171,000 $
2000	Reservoir, Diversion, Dam, Spillway	
	2100 Reservoir	37,600,000 $
	2200 Diversion	45,416,000 $
	2300 RCC Dam and Cofferdams	50,996,000 $
	2400 Spillway	36,548,000 $
	2800 North Spur Stabilisation	18,016,000 $
	Total 2000	188,576,000 $
3000	Powerhouse and Related Facilities	
	3300 Powerhouse/Intake/Tailrace	254,263,000 $
	3400 Turbines and Generators	145,000,000 $
	3500 Electrical Ancillary Equipment	11,076,000 $
	3600 Mechanical Ancillary Equipment	12,016,000 $
	3700 Generator Transformers and HV Switchyard	16,180,000 $
	3800 Spare Parts and Special Tools	7,000,000 $
	Total 3000	445,535,000 $
6000	Communications	
	6500 Communication Systems	1,965,000 $
	Total 6000	1,965,000 $
8000	Construction Facilities and Support	
	8200 Muskrat Falls Site	104,411,000 $
	8300 Happy Valley - Goose Bay	7,320,000 $
	Total 8000	111,731,000 $
	Sub-Total (Codes 1000 to 8000)	753,978,000 $
9000	Management, Engineering and Others	
	9200 Management	79,860,000 $
	9250 Engineering	41,140,000 $
	9300 Field Investigations and Hydraulic Model Studies	2,500,000 $
	9800 Contingencies	87,748,000 $
	Total 9000	211,248,000 $
	GRAND TOTAL	**965,226,000 $**

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
1000		**PERMANENT SUPPORT FACILITIES**					
1112		Site Access Road - Upgrade and New	4.8	km	601,123.00 $	$	2,885,000
1118		Maintain Site Access Road	168	km-mo	3,891.23 $	$	654,000
1120		Permanent Site Accommodation	1	ls	632,400.00 $	$	632,000
1130		Trans Labrador Highway at Reservoir	1	ls	1,000,000.00 $	$	1,000,000
1132		138 Kv Transmission Line At Reservoir	1	ls	1,000,000.00 $	$	1,000,000
		TOTAL - 1000				**$**	**6,171,000**
2000		**RESERVOIR, DIVERSION, DAM & SPILLWAY**					
2100		RESERVOIR					
2110		Clearing	3,760	ha	10,000.00 $	$	37,600,000
		Sub-Total - Division 2100				$	37,600,000
2200		DIVERSION					
2231		Portals (Inlet, Adit & Outlet)					
	2100	Access Road - Construction	4.0	km	143,193.00 $	$	573,000
	2150	Access Road - Maintenance	64	km-mo	3,928.35 $	$	251,000
	2200	Common Excavation	28,000	m3	9.05 $	$	253,000
	2300	Rock Excavation - Mass	234,000	m3	18.23 $	$	4,266,000
	2305	Inlet Plug - Install & Remove	1	ls	402,578.00 $	$	403,000
	2306	Rock Excavation - Outlet Plug	22,000	m3	28.58 $	$	629,000
	2390	Controlled Perimeter Blasting	16,000	m2	67.88 $	$	1,086,000
	2600	Foundation Preparation - Rock	470	m2	50.59 $	$	24,000
	2750	Rock Bolts - Grouted	5,200	m	76.94 $	$	400,000
	2752	Rock Bolts - Ungrouted	2,500	m	60.91 $	$	152,000
	2755	Drain Holes	260	m	18.01 $	$	5,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
	2770	Shotcrete	4,700	m2	68.51 $	$	322,000
	3100	Inlet Structure Concrete	9,460	m3	551.08 $	$	5,213,000
	3800	Reinforcing Steel	400,000	kg	1.80 $	$	720,000
	3900	Winter Protection	1	ls	300,000.00 $	$	300,000
	5510	Inlet Gates & Embedded Metalwork	670,000	kg	8.13 $	$	5,447,000
2233		Tunnel Plugs					
	3500	Concrete Tunnel Plugs	4,100	m3	516.01 $	$	2,116,000
	3800	Reinforcing Steel	40,000	kg	1.80 $	$	72,000
2240		Tunnels					
	2350	Rock Excavation incl. Tunnel Services	310,000	m3	62.62 $	$	19,412,000
	2751	Rock Bolts - Grouted	25,900	m	82.12 $	$	2,127,000
	2755	Drain Holes	2,650	m	63.47 $	$	168,000
	2770	Shotcrete	20,000	m2	68.51 $	$	1,370,000
	3500	Dental Concrete	270	m3	396.79 $	$	107,000
		Sub-Total - Division 2200				$	45,416,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
2300		RCC DAMS & COFFERDAMS					
2310		Site Services					
2312	2800	Dewatering - Dams & Structures	1	ls	3,346,806 $	$	3,347,000
2318	2100	Access Road - Construction	4.0	km	93,170.00 $	$	373,000
	2150	Access Road - Maintenance	120	km-mo	4,455.98 $	$	535,000
2320		Cofferdams - Upstream and Downstream					
2320	2200	Common Excavation	10,000	m3	8.16 $	$	82,000
2322	2450	Dumped Till - Zone 1A	74,000	m3	14.44 $	$	1,069,000
2322	2510	Compacted Till - Zone 1	31,000	m3	18.27 $	$	566,000
2323	2450	Dumped Rock Fill - Zone 4A	87,000	m3	8.10 $	$	705,000
	2455	Dumped Rockfill - Zone 4A - South Side	1	ls	320,237.00 $	$	320,000
	2560	Compacted Rockfill - Zone 4	59,000	m3	9.00 $	$	531,000
2326	2450	Dumped Filter - Zone 2A	6,000	m3	10.89 $	$	65,000
	2530	Compacted Filter - Zone 2	28,000	m3	14.47 $	$	405,000
	2540	Compacted Filter - Zone 2B	22,000	m3	2.55 $	$	56,000
2327	3100	Downstream Cofferdam - Concrete Toe W	60	m3	538.39 $	$	32,000
2328		Cofferdam Removal	44,000	m3	8.38 $	$	369,000
2340		RCC Dam Foundation - North and South					
2342	2200	Common Excavation	88,000	m3	8.42 $	$	741,000
2344	3640	Drill & Pressure Grout	4,000	m	211.15 $	$	845,000
2345	2600	Foundation Preparation - Rock	13,000	m2	55.55 $	$	722,000
	2755	Drain Holes	2,200	m	53.23 $	$	117,000
	3100	Dental Concrete	400	m3	417.05 $	$	167,000
	3640	Slush Grout	400	m3	683.10 $	$	273,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
2350		RCC Dam Construction - North and South					
2352	2510	Roller Compacted Concrete	119,000	m3	141.85	$	16,880,000
2353	2560	Concrete - Upstream Face	5,400	m3	623.35	$	3,366,000
	2565	Concrete - Downstream Face	16,300	m3	307.97	$	5,020,000
	2570	Concrete - Crest - South Dam	5,500	m3	502.99	$	2,766,000
	2575	Concrete - Cap and Retaining Walls	5,000	m3	475.55	$	2,378,000
	2580	Concrete - Drainage Gallery	385	m	2,457.52	$	946,000
	2590	Concrete - Bridge Piers	1,200	m3	711.77	$	854,000
	2600	Instrumentation	1	ls	130,000.00	$	130,000
2354		Rubber Dam Sections	175	m	8,998.61	$	1,575,000
2355		Stoplogs	1	ls	211,000.00	$	211,000
2356		Bridge Structure	1	ls	5,550,353	$	5,550,000
		Sub-Total - Division 2300				$	50,996,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
2400		SPILLWAY					
2430		Spillway Structure					
	1860	Protective Fencing	100	m	137.39 $	$	14,000
	2300	Rock Excavation - Mass	8,500	m3	27.21 $	$	231,000
	2600	Foundation Preparation - Rock	3,600	m2	59.38 $	$	214,000
	2760	Rock Dowels	3,200	m	34.87 $	$	112,000
2431		Piers and End Walls					
	3100	Concrete	16,230	m3	541.90 $	$	8,795,000
2432		Slabs and Rollways					
	2800	Drain Holes	300	m	65.89 $	$	20,000
	3100	Concrete	27,440	m3	409.34 $	$	11,232,000
	3640	Drill and Pressure Grout	500	m	222.11 $	$	111,000
2435	3100	Bulkhead Concrete Structure	7,500	m3	400.43 $	$	3,003,000
2433		Bridge					
	3400	Concrete - Deck Slab	120	m3	529.95 $	$	64,000
	4100	Structural Steel for bridge	65,000	kg	5.50 $	$	358,000
2440	3800	Reinforcing Steel for Complete Spillway	2,060,000	kg	1.65 $	$	3,399,000
	4100	Miscellaneous Steel	51,000	kg	15.00 $	$	765,000
2450		Gates, Guides and Hoists					
	3100	Secondary Concrete - Gate Guides	560	m3	1,032.69 $	$	578,000
	4300	Gates, Hoists and Embedded Parts	803.00	t	7,400.65 $	$	5,943,000
2460	7300	Stop Logs	233.00	t	7,335.61 $	$	1,709,000
		Sub-Total - Division 2400				$	36,548,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
2800		NORTH SPUR STABILIZATION					
2820		Upstream Berm					
	2560	Compacted Rockfill	363,000	m3	2.35 $	$	853,000
	2562	Compacted Tunnel Muck	121,000	m3	3.51 $	$	425,000
	2565	Till Blanket	38,000	m3	16.10 $	$	612,000
	2570	Transition Material - Zone 2	14,000	m3	14.79 $	$	207,000
2840		Downstream Stabilization					
	2200	Common Excavation	209,000	m3	8.21 $	$	1,716,000
	2535	Compacted Granular Fill	132,000	m3	17.34 $	$	2,289,000
	2560	Compacted Rockfill	231,000	m3	3.31 $	$	765,000
	2850	Header Pipe (600 mm dia)	360	m	206.15 $	$	74,000
	2855	Toe drains	60	ea	6,042.00 $	$	363,000
2860		Pumpwells					
2861		New Pumpwells	44	ea	30,300.00 $	$	1,333,000
2862		Refurbish Existing Pumpwells	22	ea	19,380.00 $	$	426,000
2880		Crest Unloading & North End of Spur					
	2200	Common Excavation	800,000	m3	7.08 $	$	5,664,000
	2535	Compacted Granular Fill	198,000	m3	14.85 $	$	2,940,000
	2562	Erosion Protection	11,000	m3	13.80 $	$	152,000
	2710	Drainage Manhole	3	ea	8,242.18 $	$	25,000
	2730	Corrugated Steel Pipe (1750 mm dia)	330	m	521.03 $	$	172,000
		Sub-Total - Division 2800				$	18,016,000
		TOTAL - 2000				**$**	**188,576,000**

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost	Total Cost
3000		**POWERHOUSE AND RELATED FACILITIES**				
3300		POWERH'SE/INTAKE/TAILRACE - CIVIL WORKS				
3310		Excavation & Site Services				
	1550	Parking Area & Landscaping	1	ls	134,230.00 $	$ 134,000
	1820	Fences	100	m	132.87 $	$ 13,000
	2250	Tailrace plug	80,000	m3	76.15 $	$ 6,092,000
	2300	Rock Excavation - Mass	472,500	m3	18.57 $	$ 8,774,000
	2360	Instrumentation	1	ls	150,000.00 $	$ 150,000
	2390	Controlled Perimeter Blasting	11,500	m2	71.60 $	$ 823,000
	2600	Foundation Preparation - Rock	10,500	m2	60.45 $	$ 635,000
	2750	Rock Bolts - Grouted	2,200	m	81.82 $	$ 180,000
	2752	Rock Bolts - Ungrouted	3,000	m	66.67 $	$ 200,000
	2755	Drain Holes	1,080	m	63.92 $	$ 69,000
	2757	Rock Dowels	450	m	33.46 $	$ 15,000
	2760	Wire Mesh	2,500	m2	27.85 $	$ 70,000
	2770	Shotcrete	1,400	m2	82.00 $	$ 115,000
3320		Substructure				
	3800	Reinforcing Steel	11,240,000	kg	1.80 $	$ 20,232,000
	4270	Embedded Steel & Anchors	18,000	kg	7.00 $	$ 126,000
3321	3100	Concrete Intake incl F'mwk	93,600	m3	443.27 $	$ 41,490,000
	3140	Intake Gates, Hoists and Embedded Parts	1,766	t	8,201.42 $	$ 14,484,000
	3150	Intake Stoplogs	511	t	7,763.60 $	$ 3,967,000
	3160	Intake Trashracks, Hoists & Embedded Pa	1,212	t	7,080.00 $	$ 8,581,000
3322	3100	Service and Erection Bay Concrete	20,000	m3	639.26 $	$ 12,785,000
3323	3100	Draft Tube Concrete	124,000	m3	558.55 $	$ 69,260,000
3324	3640	Intake - Drill & Pressure Grout	1,000	m	205.39 $	$ 205,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost	
	3645	Drain Holes	700	m	62.00 $	$	43,000	
3325	3100	Powerhouse 2nd Stage Concrete	37,000	m3	665.83 $	$	24,636,000	
	3150	Powerhouse Secondary Concrete	2,160	m3	778.05 $	$	1,681,000	
3327		Winter Protection	1	ls	940,000.00 $	$	940,000	
3330		Superstructure						
	4100	Structural Steel	5,400,000	kg	3.20 $	$	17,280,000	
	4210	Misc. Steel	515,000	kg	8.00 $	$	4,120,000	
	4500	Cladding and Roofing	15,000	m2	200.00 $	$	3,000,000	
	4610	Masonry	5,500	m2	120.00 $	$	660,000	
	4740	Doors	1	ls	550,000.00 $	$	550,000	
	4900	Architectural Work	1	ls	1,200,000.0 $	$	1,200,000	
3340		Draft Tube Gates and Hoists (2 Sets)	738	t	7,660.00 $	$	5,653,000	
3342		Draft Tube Temporary Plugs (x 4)	400	t	4,000.00 $	$	1,600,000	
3350		Powerhouse Crane and Monorail	1	ls	3,800,000.00 $	$	3,800,000	*
3360		Service Elevator	1	ls	400,000.00 $	$	400,000	
3390	5900	Machine Shop Equipment	1	ls	300,000.00 $	$	300,000	
		Sub-Total - Division 3300				$	254,263,000	

*** Prices have been adjusted to correspond to the total price of 3,800,000$ as agreed by SNC-AGRA / HQ / NLH**

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost	
3400		TURBINES AND GENERATORS						
3410		Generators (Units for Propeller Turbines x	1	ls	49,015,000 $	$	49,015,000	*
3415		Generators (Unit for Kaplan Turbine x 1)	1	ls	16,962,000 $	$	16,962,000	*
3430		Propeller Turbines x 3 Units	1	ls	57,470,000 $	$	57,470,000	*
3435		Kaplan Turbine x 1 Unit	1	ls	21,553,000 $	$	21,553,000	*
		Sub-Total - Division 3400				$	145,000,000	
3500		ELECTRICAL ANCILLARY EQUIPMENT						
3510		Generation Systems						
	7200	Isolated Phase Bus	1	ls	4,133,300.00 $	$	4,133,000	
	7300	Station and Site Distribution Services	1	ls	1,708,100.00 $	$	1,708,000	
	7500	Cables and Supports	1	ls	1,463,600.00 $	$	1,464,000	
	8000	Protection and Control	1	ls	1,554,300.00 $	$	1,554,000	
3540		Lighting and Heating	1	ls	919,800.00 $	$	920,000	
3550		Grounding	1	ls	302,400.00 $	$	302,000	
3560		Local Control	1	ls	221,400.00 $	$	221,000	
3570		Supervisory Control	1	ls	340,700.00 $	$	341,000	
3580		Telephone	1	ls	68,100.00 $	$	68,000	
3590		Fire Detection and Alarm System	1	ls	115,000.00 $	$	115,000	
3595		Emergency generator	1	ls	250,000 $	$	250,000	
		Sub-Total - Division 3500				$	11,076,000	

*** Prices have been adjusted to correspond to the total price of 145,000,000$ as agreed by SNC-AGRA / HQ / NL**

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
3600		MECHANICAL ANCILLARY EQUIPMENT					
3610		Cooling System	1	ls	1,732,600.00 $	$	1,733,000
3620		25 mpa Compressed Air	1	ls	1,172,700.00 $	$	1,173,000
3625		0.7 mpa Compressed Air	1	ls	886,700.00 $	$	887,000
3630		Dewatering	1	ls	2,306,900.00 $	$	2,307,000
3635		Clear Water Drainage	1	ls	531,100.00 $	$	531,000
3640		Fire Protection Water	1	ls	2,300,900.00 $	$	2,301,000
3650		Heating and Ventilation	1	ls	1,697,800.00 $	$	1,698,000
3660		Domestic Water	1	ls	590,000.00 $	$	590,000
3665		Sanitary Drains	1	ls	222,300.00 $	$	222,000
3670		Oil Handling System	1	ls	574,300.00 $	$	574,000
		Sub-Total - Division 3600				$	12,016,000
3700		GENERATOR TRANSFORM.& HV SWITCHYARD					
3710		Generator Transformers	1	ls	8,340,000.00 $	$	8,340,000
3711		Spare Transformer	1	ls	4,000,000.00 $	$	4,000,000
3712		Outdoor HV Switchyard	1	ls	3,840,000.00 $	$	3,840,000
		Sub-Total - Division 3700				$	16,180,000
3800		SPARE PARTS AND SPECIAL TOOLS	1	ls	7,000,000.00 $	$	7,000,000
		TOTAL - 3000				**$**	**445,535,000**

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
6000	**COMMUNICATIONS**						
6500		COMMUNICATIONS SYSTEM					
6510		Microwave system	1	ls	1,185,000 $	$	1,185,000
6520		Fiber optic terminal equipment	1	ls	300,000.00 $	$	300,000
6530		In plant communications	1	ls	480,000.00 $	$	480,000
		TOTAL - 6000				**$**	**1,965,000**

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
8000		**CONSTRUCTION FACILITIES AND SUPPORT**					
8200		MUSKRAT FALLS SITE					
8210		Site and General Services					
8211		Site Preparation - Camp Area	1	ls	2,224,356 $	$	2,224,000
8213		Water Supply/Distribution & Fire Protection	1	ls	1,379,704.00 $	$	1,380,000
8214		Electrical Supply for Camp Distribution	1	ls	550,000.00 $	$	550,000
8215		Sewage Treatment / Mains	1	ls	2,922,995 $	$	2,923,000
8217		Dismantling of camp	1	ls	828,188.00 $	$	828,000
8218		Operate and Maintain Camp					
	0100	Medical Salaries & Supplies	1	ls	1,336,875 $	$	1,337,000
	0180	Ambulance	1	ls	98,500.00 $	$	99,000
	0185	Fire Truck	1	ls	202,500.00 $	$	203,000
	0190	Medical Evacuation Flights	6	trips	8,000.00 $	$	48,000
	0200	Catering	1,150,000	pers-da	34.00 $	$	39,100,000
	0301	Building Repair and Maintenance - Labour	1	ls	5,435,238 $	$	5,435,000
	0302	Building Repair and Maintenance - Material	1	ls	1,250,000 $	$	1,250,000
	0350	Electric Heating of Camp and Offices	45000000	kWh	0.028 $	$	1,260,000
	0360	Garbage Removal and Disposal	1	ls	468,750.00 $	$	469,000
	0370	Snow Removal and Sanding	1	ls	645,057.00 $	$	645,000
	0401	Recreational Supervisor and Assistant	1	ls	1,204,000 $	$	1,204,000
	402	Recreational Budget	1	ls	1,000,000 $	$	1,000,000
	0501	Labour Advancement Program	1	ls	2,500,000 $	$	2,500,000
	0601	Security, Fire Protection Crew Personnel & Vehicles	1	ls	2,415,156 $	$	2,415,000
8220		Camp Facilities					
8221		Cafetaria and Kitchen	2,100	m2	1,569.76 $	$	3,296,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost	Total Cost
8222		Recreation Center/Commisary	960	m2	1,070.31 $	$ 1,027,000
8223		Medical Clinic / Security	180	m2	1,421.75 $	$ 256,000
8224		Bar / Restaurant	430	m2	1,191.38 $	$ 512,000
8225		Gymnasium/ Fitness Center	1,800	m2	1,274.00 $	$ 2,293,000
8226		Garage for Management Vehicles	250	m2	1,274.00 $	$ 319,000
8228		Gate house	1	ls	100,000.00 $	$ 100,000
8230		Construction Camp				
8231		Dormitory (Type 1)	215	person	12,273.48 $	$ 2,639,000
8232		Dormitory (Type 2)	1,000	person	9,692.15 $	$ 9,692,000
8240		Management Site Offices	1,600	m2	1,147.75 $	$ 1,836,000
8241		Laboratory and Equipment	150	m2	3,212.75 $	$ 482,000
8242		Workshops	195	m2	1,222.50 $	$ 238,000
8243		Fire Station & Ambulance Shelter	330	m2	1,252.50 $	$ 413,000
8247		Administration / Information Building	110	m2	1,252.50 $	$ 138,000
8249		Site Preparation - Owner's Warehouse & Laydown Area	1	ls	141,578.00 $	$ 142,000
8250		Owner's Warehouse	1,860	m2	832.58 $	$ 1,549,000
8251		Site Prep. - Contractor's Laydown Area	1	ls	612,926.00 $	$ 613,000
8271		Personnel Vehicles - Site Management	1	ls	3,100,000.00 $	$ 3,100,000
8273		Helicopter	1	ls	500,000.00 $	$ 500,000
8277		Bus Transpo - Goose Bay/Site - Manag.	1	ls	1,078,875.00 $	$ 1,079,000
8291		Emergency Generator c/w tank	1	ls	627,500.00 $	$ 628,000
8292		Fuel for Emergency Generator	300,000	lt	0.47 $	$ 141,000
8293		Power Consumption for Contractors	50,000,000	kWh	0.028 $	$ 1,400,000
8295		138kv - Sw'yard, Transformer and Distribution for Construction Power	1	ls	1,713,750.00 $	$ 1,714,000
8297		Construction Communications	1	ls	5,000,000.00 $	$ 5,000,000

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost		Total Cost
8298		Television	1	ls	433,500.00	$	434,000
		Sub-Total - Division 8200				$	104,411,000
8300		HAPPY VALLEY / GOOSE BAY					
8320		Rental of Housing for Management Staff	3,900	month	800.00	$	3,120,000
8321		Upgrade Existing Housing - CFB Goose Ba	100	units	5,000.00	$	500,000
8322		Maintenance of Rental Units	400	unit-yr	1,000.00	$	400,000
8323		Furnishings	100	unit	16,000.00	$	1,600,000
8324		Contractor's Room & Board - Prior to Cam	10,000	pers-da	100.00	$	1,000,000
8325		Management Room & Board - Prior to Cam	2,000	pers-da	100.00	$	200,000
8326		Community Services/Donations etc	1	ls	500,000.00	$	500,000
		Sub-Total - Division 8300				$	7,320,000
		TOTAL - 8000				**$**	**111,731,000**

CAPITAL COST ESTIMATE
MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
4 - 206 MW Units (824 MW)

Code	Sub-Code	Item	Quantity	Unit	Unit Cost	Total Cost
9000		**MANAGEMENT, ENGINEERING AND OTHERS**				
9200		MANAGEMENT	1	ls	79,860,000 $	$ 79,860,000
9250		ENGINEERING	1	ls	41,140,000 $	$ 41,140,000
9300		FIELD INVESTIGATION AND HYDRAULIC MODEL	1	ls	2,500,000.00 $	$ 2,500,000
9800		CONTINGENCIES	1	ls	87,748,000 $	$ 87,748,000
		TOTAL - 9000				**$ 211,248,000**
ESTIMATE GRAND TOTAL						***$ 965,226,000***

SNC - AGRA
Joint Venture

SNC•LAVALIN
AGRA Monenco

❑ P.O. Box 2035, St. John's, NF A1C 5R6 Phone: 709-722-7023, Fax: 709-722-7353
❑ P.O. Box 487, Mount Pearl, NF A1N 2W4 Phone: 709-368-0118, Fax: 709-368-5410
❑ P.O. Box 9600, St. John's, NF A1A 3C1 Phone: 709-724-1900, Fax: 709-739-5458

January 29, 1999

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

Attention: Mr. Robert D. Barnes

Reference: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
FINAL FEASIBILITY STUDY REPORT
REF. 721397/31C-3-176

Dear Sir:

We are pleased to submit to you, our Final Feasibility Study Report on the above project, as follows:

Volume 1: Engineering Report..... 24 copies
Volume 2: 1998 Geotechnical Investigations ..... 24 copies
Volume 3: Detailed Backup of Capital Cost Estimate..... 24 copies

The report reflects all the comments and suggestions which were made by yourself and by the Newfoundland and Labrador Hydro specialists together with those received from Hydro-Quebec representatives.

While the submission of the report completes the study requirements we remain available to respond to any questions relating to the report or to provide information or support services for the next phase of project development engineering and management.

We wish to extend our appreciation to you, your staff and the representatives of Hydro-Quebec for all the cooperation and support throughout the study and for their valuable comments.

Yours very truly,

SNC-AGRA JOINT VENTURE

Marcel J. Dessureault, P.Eng.

Marcel J. Dessureault, P.Eng.
Study Manager

MJD/ks
Attachment

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
(Average Work Force for Quarter)

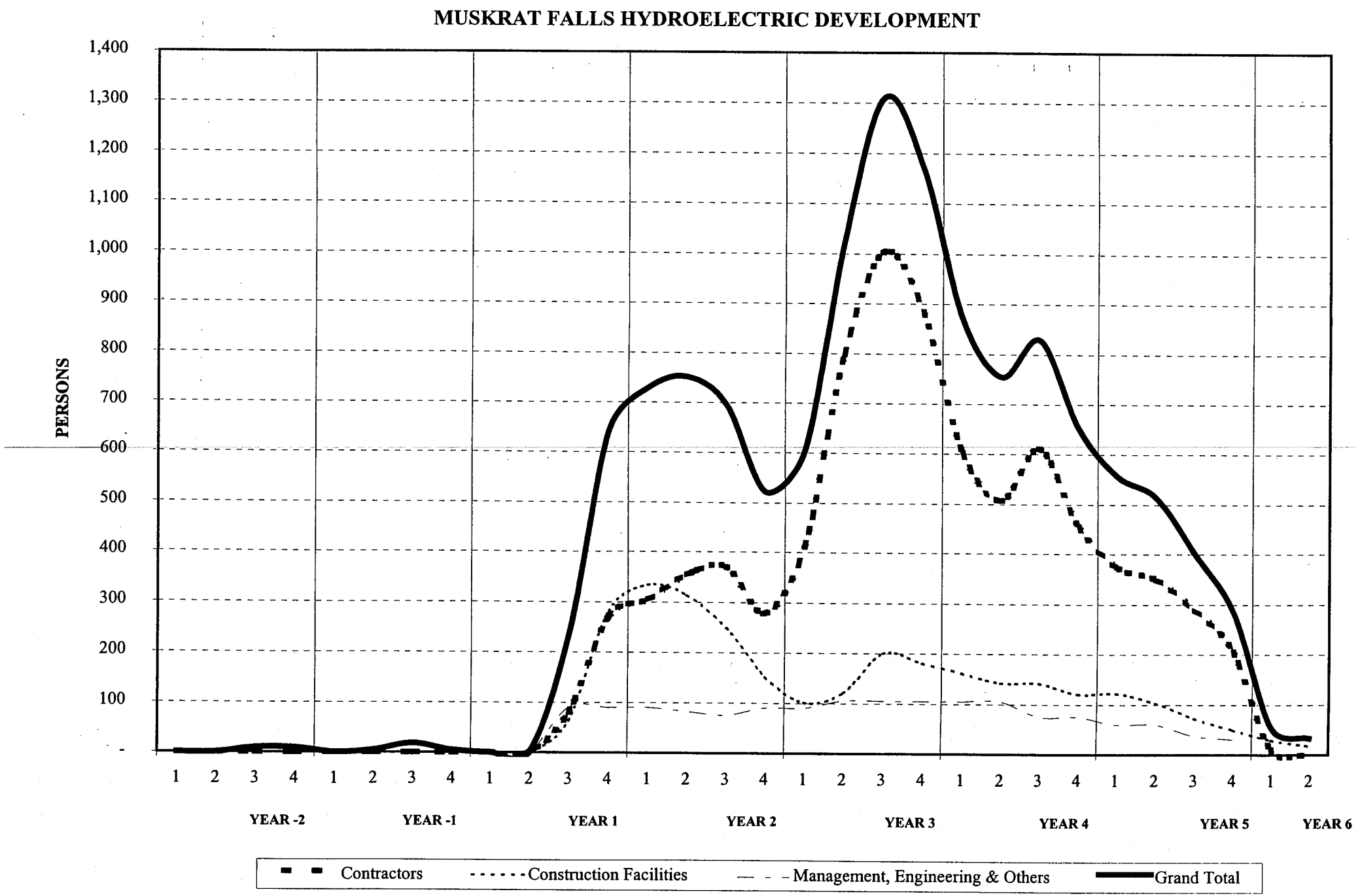
APPENDIX A

CLIENT: NEWFOUNDLAND AND LABRADOR HYDRO	**TITLE: CAPITAL COST ESTIMATE**	**CONTRACT NO.: 721397**
PROJECT: MUSKRAT FALLS POWER DEVELOPMENT	**Description: LABOUR FORCE - SUMMARY**	**DATE OF ESTIMATE: June 1998**
LOCATION: CHURCHILL RIVER, LABRADOR	**PREPARED BY: MG** **VERIFIED BY: LM**	

DESCRIPTION	AMOUNT (,000 $)	Year -2		Year -1				Year 1				Year 2				Year 3				Year 4				Year 5				Year 6	
		3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2
Civil works:																													
Permanent Support Facilities (1000)	1,780									13	12	1	4	4	1	1	1	1	1	1	7	7	1	1	1	7	6	0	0
Reservoir (2100)	20,680									45	114	91	136	170	136	114	136	91	34		23	34	11						
Diversion (2200)	11,207									15	86	136	133	10										34	0				
RCC Dams & Cofferdams (2300)	19,695													87	47	10	156	252	126	10	19	18	21	0	0	0	0		
Spillway (2400)	13,038													2	3			19	80	60	139	110	50						
North Spur Stabilization (2800)	3,964										35	50	50	17															
Powerhouse - Civil Works (3300)	90,958													47	67	249	433	543	558	432	227	240	139	48	48	48	48		
Electrical and mechanical works																													
Turbines and Generators (3400)	27,337																					76	143	220	220	182	105	10	
Electrical Ancillary Equip. (3500)	2,775																	7	5	17	14	14	14	14	14				
Mechanical Ancillary Equip. (3600)	4,206																			29	24	24	24	24	24				
Gen. Transformers & HV Switching (3	1,455																				9	33	11		12				
Spare Parts & Special Tools (3800)																													
Communications (Microwave) (6500)	983																									27	27		
Construction facilities and support																													
Muskrat Falls Site (8200)	47,379									49	251	290	272	219	136	90	108	180	162	144	126	126	108	108	90	63	43	25	16
Happy Valley-Goose Bay (8300)	1,554									9	5	13	14	7	2	2	2	2	2	3	4	3	2	4	4	4	3	1	0
Management, Engineering and Others (9000)																													
Engineering & Management (9200)	71,321									82	82	82	75	69	82	82	96	96	96	96	96	69	69	55	55	34	27	14	14
Field Investigation (9300)	750	8	8		4	16	4																						
Contingencies (9800)	26,324	1	1		0	2	0			21	58	66	68	63	47	55	93	119	106	79	69	75	59	51	47	36	26	5	3
GRAND TOTAL	**345,403**	**9**	**9**		**5**	**18**	**5**			**236**	**642**	**729**	**751**	**694**	**522**	**603**	**1,025**	**1,310**	**1,171**	**870**	**755**	**828**	**652**	**557**	**514**	**400**	**285**	**55**	**33**

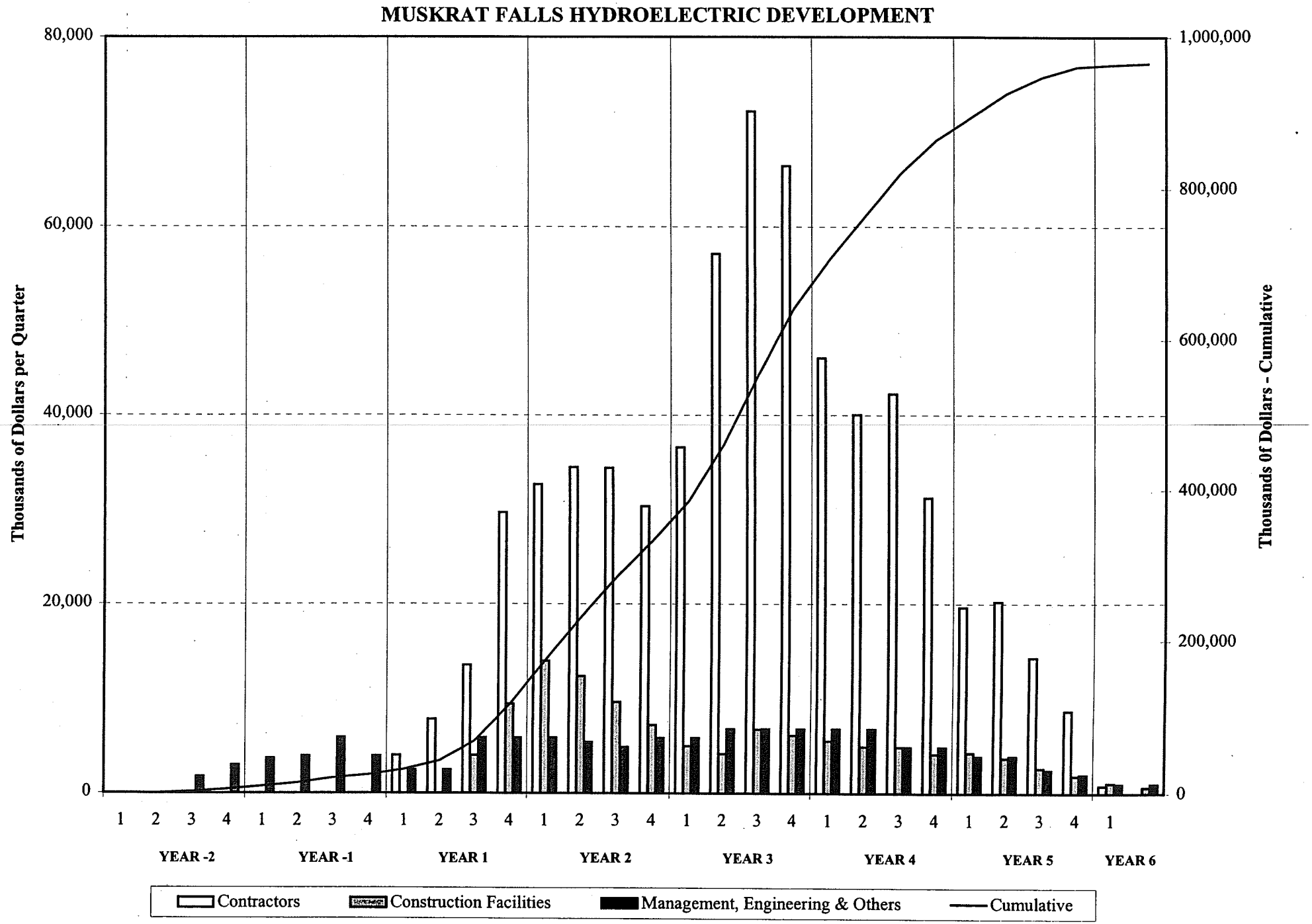
CLIENT: NEWFOUNDLAND AND LABRADOR HYDRO	TITLE: CAPITAL COST ESTIMATE	CONTRACT NO.: 721397
PROJECT: MUSKRAT FALLS POWER DEVELOPMENT	Description: INCURRED COST FLOW - SUMMARY	DATE OF ESTIMATE: June 1998
LOCATION: CHURCHILL RIVER, LABRADOR	PREPARED BY: MG VERIFIED BY: LM	

DESCRIPTION	AMOUNT (,000 $)	Year -2		Year -1				Year 1				Year 2				Year 3				Year 4				Year 5				Year 5	
		3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2
Civil works:																													
Permanent Support Facilities (1000	6,171									1,544	1,659	29	316	328	39	39	39	49	49	49	626	621	39	30	27	343	329	7	7
Reservoir (2100)	37,600									1,504	3,760	3,008	4,512	5,640	4,512	3,760	4,512	3,008	1,128		752	1,128	376						
Diversion (2200)	45,416									2,050	11,377	14,596	14,291	889										2,203	10				
RCC Dams & Cofferdams (2300)	50,996													6,089	2,908	506	8,892	14,933	7,849	511	1,483	2,670	5,003	37	37	37	37		
Spillway (2400)	36,548													245	1,061	735	735	2,066	6,251	4,843	10,321	7,484	2,809						
North Spur Stabilization (2800)	18,016										4,144	5,945	5,945	1,982															
Powerhouse - Civil Works (3300)	254,263												342	10,157	12,748	22,472	33,881	40,973	41,532	33,307	18,302	16,776	9,667	3,351	4,051	3,351	3,351		
Electrical and mechanical works																													
Turbines and Generators (3400)	145,000							3,949	7,786	7,786	7,786	7,786	7,786	7,786	7,786	7,786	7,786	7,786	7,786	3,498	3,498	6,071	8,209	10,782	10,782	7,803	3,989	773	
Electrical Ancillary Equip. (3500)	11,076																	2,067	1,447	1,463	1,195	1,195	1,195	1,279	1,237				
Mechanical Ancillary Equip. (3600)	12,016																			2,403	1,923	1,923	1,923	1,923	1,923				
Gen. Transformers & HV Switching	16,180									600	950	1,295	1,295	1,295	1,295	1,295	1,295	1,295	346		192	2,609	2,000		417				
Spare Parts & Special Tools (3800)	7,000																				1,750	1,750			1,750	1,750			
Communications (Microwave) (650	1,965																									983	983		
Construction facilities and support																													
Muskrat Falls Site (8200)	104,411									2,594	9,127	13,077	11,448	9,152	7,027	4,740	3,923	6,538	5,884	5,230	4,576	4,576	3,923	3,923	3,269	2,288	1,581	935	600
Happy Valley-Goose Bay (8300)	7,320									1,345	306	856	876	464	161	201	201	181	217	281	322	281	181	322	402	322	241	121	40
Management, Engineering and Others (9000)																													
Engineering & Management (9200)	121,000	1,214	2,429	3,643	3,643	4,857	3,643	2,429	2,429	5,803	5,803	5,803	5,319	4,836	5,803	5,803	6,770	6,770	6,770	6,770	6,770	4,836	4,836	3,869	3,869	2,418	1,934	967	967
Field Investigation (9300)	2,500	500	500		250	1,000	250																						
Contingencies (9800)	87,748	171	293	364	389	586	389	638	1,021	2,323	4,491	5,240	5,213	4,886	4,334	4,734	6,803	8,567	7,926	5,836	5,171	5,192	4,016	2,772	2,777	1,930	1,245	280	161
GRAND TOTAL	965,226	1,886	3,222	4,007	4,282	6,443	4,282	7,016	11,236	25,549	49,402	57,635	57,342	53,749	47,674	52,070	74,838	94,232	87,183	64,191	56,880	57,113	44,177	30,490	30,552	21,225	13,690	3,083	1,775



Labour Force

Figure 1.2



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APPENDIX B

DESIGN CRITERIA

APPENDIX B

DESIGN CRITERIA

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
FINAL FEASIBILITY STUDY
PROJECT: 721397

DRAFT
DESIGN CRITERIA

Submitted to:

Newfoundland and Labrador Hydro

SNC-AGRA Joint Venture
January 27, 1999

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1.0 INTRODUCTION

The purposes of the design criteria are:

- To provide an overview of the dimensional parameters for the various components of the project.
- To provide the basis upon which preliminary design, stability analysis, and quantities for the structures have been prepared for the Final Feasibility Study.

The Project Data Summary included herein, in Section 2.0, provides a summary of the important parameters of each of the main structures, including governing elevations, flows and dimensions. The subsequent sections present the design criteria by discipline, by structure and/or by area of a structure.

The Project Design Flood was determined, in accordance with criteria established by the Canadian Dam Safety Association, to be the Probable Maximum Flood (PMF). The value of the PMF was determined, in a separate study by others, to be 22,100 m^3/s.

The project layout was investigated in previous studies dating back to 1979-80. These are reviewed as part of the Final Feasibility Study as described under Chapter 6 of the Report.

The project will comprise an integrated intake/powerhouse and an adjacent spillway located in the river channel, above the lower falls. Closure of the river valley is completed by an RCC overflow dam with rubber dam on the north side, and an RCC overflow dam extending to closure on the south bank.

The total plant capacity was determined, in a separate study by others, to be 824 MW.

The number and size of units were determined in a parallel interim study. The results of that study are described in Appendix E, "Optimization of Number of Generating Units" included in the Final Feasibility Study Report. Based on that study, the project will comprise four generating units, including three propeller turbines and one Kaplan turbine.

The construction flood peak flow, on which the design of the diversion structures is based, was determined in a parallel interim study. The results of that study are described in the report entitled "Construction Flood Study" included as Appendix D in the Final Feasibility Study Report. Based on that study, the project construction flood will be 5,300 m^3/s.

Two diversion tunnels will divert the River around the site during construction. Operable vertical fixed wheel gates at the entrance to the tunnels will regulate the upstream water level to El. 24.0 m for frazil ice control.

2.0 PROJECT DATA SUMMARY

2.0 PROJECT DATA SUMMARY

2.1 DRAINAGE BASIN

• Drainage area (existing)	92,355 km^2
• Romaine and St-Jean River diversions	9,583 km^2
• Project drainage area	101,938 m^3/s
• Long term average flow (existing)	1,841 m^3/s
• Romaine and St-Jean River diversions	185 m^3/s
• Project average flow	2,026 m^3/s
• Maximum plant flow capacity	2,667 m^3/s
• Construction design flood (1:40 year)	5,300 m^3/s
• Project design flood (PMF)	22,100 m^3/s

2.2 GOVERNING ELEVATIONS

• Maximum reservoir flood level (MFL) at PMF	44.0 m
• Maximum operating level	39.0 m
• Minimum operating level	38.5 m
• Maximum tailwater level at PMF	11.0 m
• Maximum tailwater level at 1:100 flood	7.5 m
• Maximum tailwater level at 1:40 flood	6.0 m
• Average tailwater level	3.0 m
• Minimum tailwater level	2.0 m

2.3 RESERVOIR

• Maximum flood level (PMF)	44.0 m
• Maximum operating level	39.0 m
• Minimum operating level	38.5 m
• Maximum water level at diversion design flow (1:40)	30.5 m
• Flooded area at maximum operating level	106.8 km^2
• Reservoir length	60.0 km

2.4 DIVERSION FACILITIES

2.4.1 GENERAL

• Maximum flood level (MFL - Q_{40})	30.5 m
• Normal operating level for ice control	24.0 m
• Maximum flow (Q_{40})	5,300 m^3/s

2.4.2 UPSTREAM COFFERDAM

• Freeboard (on MFL - Q_{40})	1.0 m
• Crest elevation	31.5 m
• Maximum height	23.5 m
• Crest length	320.0 m
• Total volume	306,800 m^3

2.4.3 APPROACH CHANNEL

• Sill invert elevation	12.0 m
• Minimum Width at invert	67.0 m

2.4.4 INLET CONCRETE STRUCTURE

• Rock cover at portal (1.25 x tunnel width, with additional rock support)	
• Number of tunnel inlets	2
• Steel deck elevation	40.0 m
• Top of concrete	31.0 m
• Invert elevation	4.0 m
• Width (each)	20.0 m
• Inlet gates (to be designed to be fully opened or closed, to control flow through tunnels and maintain constant upstream water level).	
– Number	4
– Width	6.6 m
– Height	17.0 m

2.4.5 TUNNELS

• Number	2
• Type	Unlined inverted U
• Design	Pressurized at high flows
• Length	600 m
• Cross Section, finished (W x H)	14.0 m x 17.0 m
• Width of rock pillar	32 m
• Concrete Lining (15 m downstream of inlet)	
– Thickness (minimum)	0.6 m
– Length from inlet	15 m
– Length from outlet	NA
• Shotcrete and Rockbolts	
– Pattern rockbolts with shotcrete for crown.	
– Rockbolts and/or shotcrete, as required elsewhere.	

2.4.6 OUTLET PORTAL

• Rock cover at portal (1.5 x tunnel width)	
• Number of tunnel outlets	2
• Invert elevation	-12.0 m
• Width (each)	14.0 m

2.4.7 OUTLET CHANNEL

• Number of channels	2
• Invert elevation	-4.0 m
• Width at invert (each)	28.0 m

2.5 INTAKE

• Close coupled to powerhouse	
• Deck elevation	45.5 m
• Approach channel invert elevation	1.0 m
• Concrete sill elevation	3.0 m

2.5.1 INTAKE HEAD GATES

• Number (3 per unit)	12
• Gate sill elevation	-1.0 m
• Width	7.33 m
• Height	16.3 m

2.5.2 BULKHEAD GATES

• Number (1 set)	3
• Width	7.33 m
• Height	17.8 m

2.5.3 TRASHRACKS

• Number (4 sets)	12
• Width	7.33 m
• Height	22.0 m
• Clear space between bars	100 mm

2.6 POWER INSTALLATION

2.6.1 POWERHOUSE

• Gross head	37.0 m
• Length	187.5 m
• Total width	68.8 m
• Width of main generator floor	28.0 m
• Maximum height above foundations	78.0 m
• Unit spacing	36.0 m
• Concrete semi-spiral inlet dimensions (W x H)	28.0 m x 14.0 m
• No. of draft tube gates (two sets)	4
• Draft tube gate width	12.5 m
• Draft tube gate height	9.25 m
• Tailrace invert elevation	-10.0 m

2.6.2 TURBINES

• Number	4
• Type	3-Propeller, 1-Kaplan
• Rated output, each	212 MW
• Rated Net head	35.0 m
• Rated discharge, per unit	665 m^3/s
• Runner diameter	9.0 m
• Speed	81.82 rpm
• Runner mass - Propeller	140 t
- Kaplan	230 t
• * Total turbine mass - Propeller	1,600 t
- Kaplan	1,800 t

*Inclusive of
-Draft Tube Cone
-Stay Ring
-Wicket Gates
-Runner
-Head Cover
-Shaft

2.6.3 GENERATORS

• Number	4
• Rated output, each	208 MW
• Rated voltage	15.0 kV
• Power factor	0.95
• Rotor diameter	14 m
• Rotor mass	740 t
• Total generator mass	1,250 t
• Synchronous speed	81.82 rpm

2.6.4 POWERHOUSE CRANES

• Number	2
• Main hook capacity, each	400 t
• Auxiliary hook capacity, each	25 t
• Equalizing beam, mass	50 t

2.6.5 POWER TRANSFORMERS

• Number	2
• Rating	165/165/330 MVA ONAN 220/220/440 MVA ONAF
• Voltage	15/15 - 230 kV
• Shipping mass	250 t

2.7 SPILLWAY

• Width out to out of end piers	57.25 m
• Length to end of apron	72.0 m
• Crest elevation of ogee	18.8 m

• Number of fixed wheel, vertical lift, gates	3
• Width of gate	13.75 m
• Height of gate	20.2 m
• Number of sets of stop logs	1
• Maximum flow capacity (MFL El 44.0)	10,800 m^3/s

2.8 DAMS

2.8.1 UPSTREAM COFFERDAM

- See item 2.4.2, above

2.8.2 DOWNSTREAM COFFERDAM

• Freeboard , Q_{40}	1.0 m
• Crest elevation	7.0 m
• Maximum height	3.5 m
• Crest length	180.0 m
• Total volume	6,200 m^3

2.8.3 NORTH RCC OVERFLOW DAM

• Freeboard	0.5 m
• Rubber dam - crest elevation - inflated	39.5 m
deflated	37.1 m
• Rubber dam - crest length (net)	167.5 m
• Concrete - top elevation	37.0 m
- length (total)	178.5 m
- total volume	72,000 m^3

2.8.4 SOUTH RCC OVERFLOW DAM

• Freeboard	1.0 m
• Crest elevation	40.0 m
• Crest Length	326.0 m
• Total Length	370.0 m
• Overflow crest length (net)	326.0 m
• Total volume	69,000 m^3

2.8.5 FREEBOARD DYKE

Covered under Sections 4.3 to 4.8 (Geotechnical Design Criteria)

3.0 HYDROLOGY/HYDRAULICS

3.0 HYDROLOGY/HYDRAULICS

3.1 DESIGN FLOODS

- Project Design Flood (Probable maximum flood - PMF) — 22,100 m^3/s
- Construction flood (Q_{40}) — 5,300 m^3/s

3.2 GOVERNING ELEVATIONS

3.2.1 Upstream Reservoir

- Maximum Flood Level at PMF — 44.0 m
- Maximum Operating Level — 39.0 m
- Minimum Operating Level — 38.5 m

3.2.2 Downstream

- Tailwater Level (PMF) — 11.0 m
- Tailwater Level for 1:100 year flood — 7.5 m
- Tailwater Level for 1:40 year flood — 6.0 m
- Mean Tailwater Level — 3.0 m
- Minimum Tailwater Level — 2.0 m

3.3 DIVERSION FACILITIES

3.3.1 Upstream Water Levels

- Construction Flood Level, 1:40 year flood — 30.5 m
- Normal Water Level for ice control — 24.0 m
- Average Summer Water Level — 19.0 m

3.3.2 Downstream Water Levels, as in 3.2.2

3.3.3 Hydraulic Design

The following factors were considered in hydraulic design:

- inverts of approach channel sill and inlet structure to be set low enough to capture all flow at time of cofferdam closure with river remaining at approximately normal level;

(Assume Q at closure = 2,000 m^3/s, W.L. at closure ~ 17.5 m)

- tunnel should flow full for large flows and should fill from downstream end to upstream end;
- upstream invert of inlet structure should be set above mean tailwater level, to facilitate construction of tunnel plug in the dry (tunnel hydraulics permitting);
- inlet should be vented to prevent flow instabilities when tunnel is partly full and also to provide access for tunnel plug construction;
- upstream water level control for ice to be done without gate throttling, that is: basic control will be obtained by opening one to four gates to approximately balance inflow and outflow. Fine control will be by daily opening and closing one gate to maintain water level within a range of ± 0.15 m;
- base head loss calculations on the following factors:

k_e at upstream channel entry	=	0.1
k_i inlet	=	0.2
k_x downstream end of tunnel	=	0.7
Manning's "n", approach and outlet channels	=	0.035
Manning's "n", tunnel	=	0.033

- cofferdam closure - preliminary designs to be based on Izbash's equations (Reference 1) final design should be based on model testing.

3.3.4 Tunnel Plug

The greatest of:

- for seepage control $L = 0.08H$
- for resistance to sliding $L = \frac{pD}{4C} + 0.25$ or $L = 0.5D$

where:

L = length of plug (m)
H = mean hydrostatic head (m)
p = mean hydrostatic pressure (kPa)
C = cohesion concrete rock (kPa) (assume 700 kPa)
D = width of tunnel (m)

3.4 INTAKE

- Intake will provide a transition from reservoir to concrete semi-spiral case.
- Intake geometry will take into account the following criteria:

- Velocity at trashracks of 1.5 to 2.0 m/s on net area of trashracks.
- Flow acceleration through intake less than 0.5 m/s per linear m.

- Intake set low enough to avoid creation of vortices in accordance with J.L Gordon's formula (Reference 2): $S = k\,V(d^{0.5})$

  Where:

  S = Submergence below minimum operating level, to soffit at gate section (m).
  V = Velocity at gate (m/s).
  d = Height of gate (m).
  k = 0.54 for symmetrical approach flow.

3.5 SPILL FACILITIES

3.5.1 Design Flood

- Follow Canadian Dam Safety Association Guidelines.

- Consequence Classification:

  - Category High,
  - Some increase in loss of life expected,
  - Substantial increase in social, economic and/or environmental losses.

- Criteria for Design flood:

  - Use probable maximum flood (PMF)
  - Flood routing effect is negligible
  - Spillway design flow (PMF) = 22,100 m^3/s

3.5.2 Flood Handling

a) Normal operation, 1 in 1,000 year flood:

$Q_{1,000}$	=	10,100	m^3/s
Flood Level	=	40.0	m
Spillway Flows:			
• North Dam	=	1,650	m^3/s
• South Dam	=	0	m^3/s
• Gated Spillway	=	8,450	m^3/s
• Total	=	10,100	m^3/s

b) Extreme condition, PMF,

PMF	22,100	m^3/s
Flood Level	44.0	m
Spillway Flows:		
• North Dam	5,800	m^3/s
• South Dam	5,500	m^3/s
• Gated Spillway	10,800	m^3/s
• Total	22,100	m^3/s

3.5.3 Hydraulic Design

The Hydraulic design, of the ogee profiles for the North and South dams and Gated Spillway, was based on profile geometry developed by the U.S. Army Corps of Engineers Waterways Experimental Station (W.E.S.). (References 3 and 4).

Flow capacities at MFL = 44.0 m, were based on the following weir coefficients as derived from W.E.S. design recommendations:

- North Dam (with rubber dam crest) Cw = 2.0
- South Dam Cw = 2.2
- Gated Spillway Cw = 2.1

3.6 REFERENCES

1) Isbash, S.V. and Khaldre, K.Y. 1970, Hydraulics of River Channel Closure, (Translated from Russian by G.L. Cairns) Butterworths - London.

2) Gordon, J.L. April 1970, Vortices at Intakes, Water Power.

3) Chow Ven T. (1959) Open - Channel Hydraulics, McGraw-Hill Book Company, New York.

4) ASCE, (1995), Hydraulic Design of Spillways, New York (as adapted from USACE Manual #12).

4.0 GEOTECHNICAL

4.0 GEOTECHNICAL

4.1 GENERAL

This section presents the design criteria for the cofferdams, embankments, freeboard dykes, impervious and drainage blankets for spur stabilization, and rock support for the diversion tunnels and open cut rock excavations for the structures.

4.2 EMBANKMENT MATERIALS

4.2.1 Impervious Fill (Class 1)

Well graded glacial till that will compact into a homogeneous impervious mass with high shear strength and low permeability to be used for the cofferdams, core of the freeboard dykes and for impervious blankets on the abutments and spur which will be obtained from borrow areas T2 (West), T4A, T4B or other approved sources (Figure 4.1) and falls within the following gradation limits as shown below.

Impervious Fill (Class 1) - Gradation Limits

Sieve Size	Percent Passing (by weight) *
75mm	100
20mm	75-100
4.7mm (No. 4)	60-97
2.0mm (No. 10)	50-90
0.4mm (No. 40)	35-75
0.15mm (No. 100)	25-60
0.074mm (No.200)	20-40

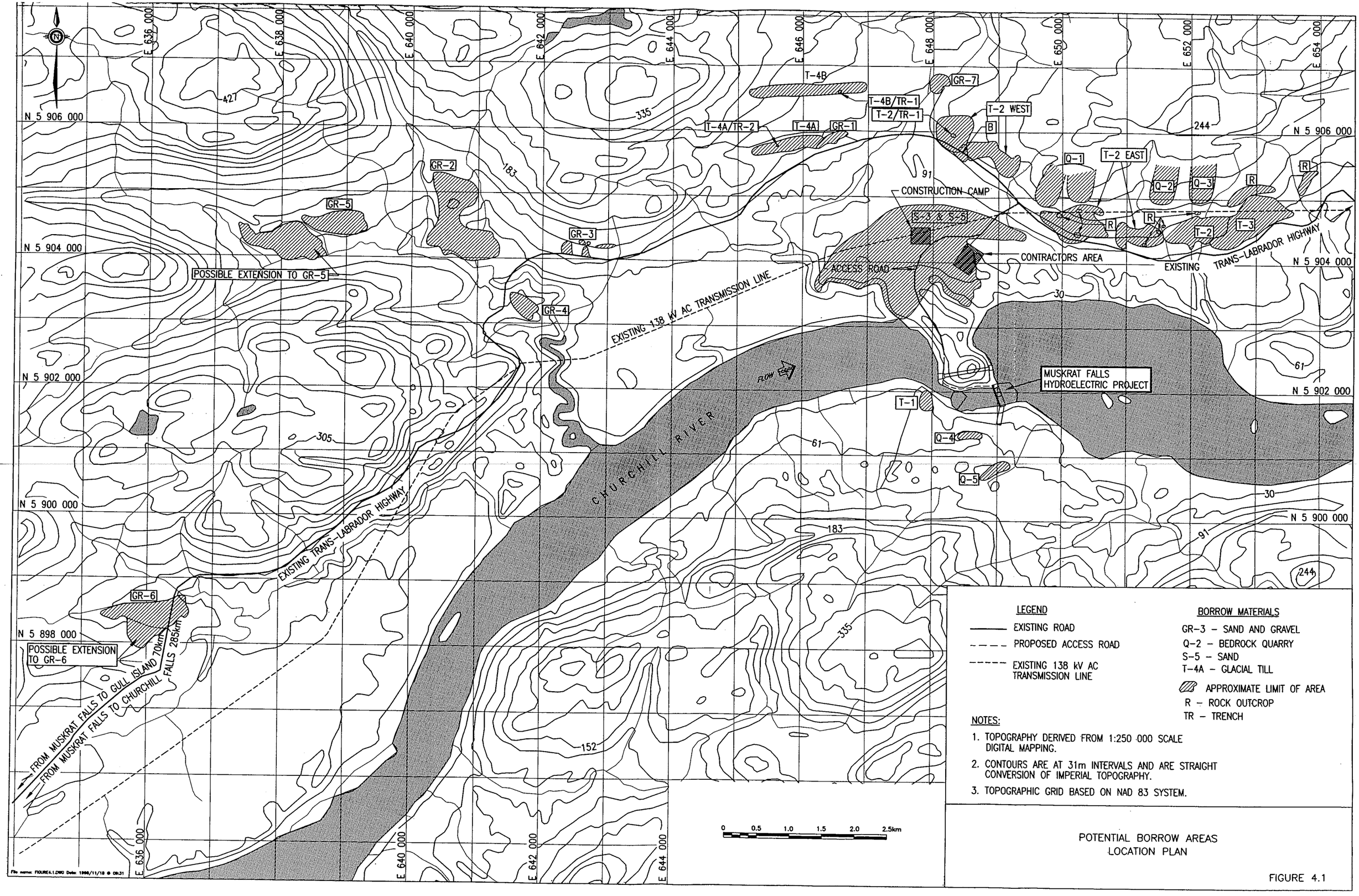
The Class 1 fill will be compacted to an average of 98% Standard Proctor maximum dry density (ASTM D698) within ±2% of the Standard Proctor optimum moisture content.

4.2.2 Dumped Impervious Fill (Class 1A)

Well graded glacial till, as specified in Section 4.2.1 above, will be dumped in the wet to construct impervious blankets for cofferdams and compacted in cofferdams above water level to an average of 95% Standard Proctor maximum dry density (ASTM D698).

4.2.3 Granular Fill (Class 2)

Well graded, gravely sand, free-draining with less than 5% passing the 0.074 mm (No. 200) sieve having a coefficient of uniformity greater than five (5). This material is to be obtained from borrow areas GR-4, 5 or 6 or other approved sources (Figure 4.1) and may require selective excavation or processing. Class 2 fill is used as filter-drains in the cofferdams,



LEGEND

- EXISTING ROAD
- PROPOSED ACCESS ROAD
- EXISTING 138 kV AC TRANSMISSION LINE

BORROW MATERIALS

- GR-3 – SAND AND GRAVEL
- Q-2 – BEDROCK QUARRY
- S-5 – SAND
- T-4A – GLACIAL TILL
- APPROXIMATE LIMIT OF AREA
- R – ROCK OUTCROP
- TR – TRENCH

NOTES:

1. TOPOGRAPHY DERIVED FROM 1:250 000 SCALE DIGITAL MAPPING.
2. CONTOURS ARE AT 31m INTERVALS AND ARE STRAIGHT CONVERSION OF IMPERIAL TOPOGRAPHY.
3. TOPOGRAPHIC GRID BASED ON NAD 83 SYSTEM.

POTENTIAL BORROW AREAS LOCATION PLAN

FIGURE 4.1

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freeboard dykes, and stabilizing and drainage blankets for the spur. Class 2 will be compacted to an average of 95% of the maximum vibrated density by ASTM D2045 and falls within the following gradation limits.

Granular Fill (Class 2) - Gradation Limits	
Sieve Size (mm)	**Percent passing (by weight)**
75mm	100
20mm	95-100
4.7 mm, No. 4	40-90
2.0 mm, No. 10	15-80
0.4 mm, No. 40	0-50
0.15 mm, No. 100	0-30
0.074 mm, No. 200	0-5

4.2.4 Dumped Granular Fill (Class 2A)

Well graded gravely sand, with gradation as specified in 4.2.3 above, which will be dumped in the wet to construct transition fill for cofferdams and compacted in cofferdams above water level to an average of 95% of maximum vibrated density (ASTM D2045).

4.2.5 Random Fill (Class 2B)

Uniform to well graded semipervious sand with less than 10% passing the 0.074 mm (No. 200) sieve and having a coefficient of uniformity greater than 3 for use as granular backfill. This material is to be obtained from surficial sands from required excavations and borrow pits in the project vicinity (Figure 4.1) and compacted to an average of 95% of the maximum vibrated density (ASTM D 2045).

Random Fill (Class 2B) - Gradation Limits	
Sieve Size (mm)	**Percent Passing (by weight)**
75 mm	100
20 mm	90-100
4.7 mm, No. 4	70-100
2.0 mm, No. 10	60-100
0.4 mm, No. 40	25-95
0.15 mm, No. 100	5-40
0.074 mm, No. 200	0-10

4.2.6 Transition (Class 3C)

Crushed rock, well graded from 150 mm to sand sizes from selective borrow area excavation or processing of rock from structural excavations and quarries is to be used for transition material in the cofferdams for erosion protection. Graded as follows, placed in 0.3 m lifts and compacted with four passes of a 10 t vibratory roller or equivalent compactive effort with the tracks of a D-8 dozer.

Transition (Class 3C) - Gradation Limits	
Sieve Size (mm)	**Percent Passing (by weight)**
150mm	100
75mm	80-100
4.7mm (No. 4)	40-90
0.4mm (No. 40)	5-40
0.074 mm (No. 200)	0-5

4.2.7 Rockfill (Class 4)

Quarry run rockfill from structural excavations and quarries to be used in the cofferdams, and for north spur stabilization and elsewhere as required. Graded from 1m to fine rock, containing less than 15% of particles smaller than 25 mm, placed in 1 m lifts and compacted by four passes of a 10 t vibratory roller.

4.2.8 Dumped Rockfill (Class 4A)

Quarry run rockfill from structural excavations and quarries to be used in cofferdam construction, well graded from approximately 1 m or as required to obtain cofferdam closure and for erosion protection elsewhere.

4.2.9 Tunnel Muck

Fine rockfill from diversion tunnel underground excavation, well graded from a maximum size of 150 mm to fines and used as erosion protection and transition between rockfill and bank material in areas where filtration is non-critical.

4.2.10 Riprap (Class 5)

Broken rock material, clean and well graded within the following gradation limits from required structural excavations or quarries, intended for erosion protection, if required, for cofferdams, embankments and freeboard dykes.

Riprap (Class 5) - Gradation Limits	
Nominal Cube Size (mm)	**Percent of Sample Count Finer Than**
850	100
550	45-100
250	10-45
100	0-5

4.2.11 Road Topping (Class RT)

Well graded, sand-gravel or gravel sized crushed rock from borrow areas, structural excavations, or quarries, intended for road surfacing, gradation as shown below.

Road Topping (ClassRT) - Gradation Limits	
Sieve Size	**Percent Passing (by weight)**
25mm	100
20mm	95-100
No. 4	40-70
No. 10	20-55
No. 40	10-35
No. 200	5-15

4.3 EMBANKMENT DESIGN

4.3.1 Embankment Material Properties

The parameters to be used in stability, settlement, seepage and earth pressure calculations are shown in Table 4.1.

Table 4.1
Design Properties of Embankment Fill

Material Class	Unit Weight (kN/m^3)	Permeability k(cm/s)	Effective Strength Parameters Ø' (°)	Effective Strength Parameters c'(kPa)	Pore Pressure Parameter $\bar{B}$
1	21.0	1×10^{-5}	40	0	0.3
1a	18.5	1×10^{-5}	35	0	0.4
2	20.5	1×10^{-2}	35	0	0
2a	21.0	1×10^{-2}	35	0	0
3c	20.5	1×10^{-2}	35	0	0

4.3.2 Filters

Design will be based on the following criteria (Fell *et al*, 1992; Wilson and Marsal, 1979):

- The 15 percent filter size will not be greater than five times the 85% of the base material;
- The 15 percent filter size will be at least five times the 15% size of the base material;
- where the base material contains a large percentage of gravel, the filter will be designed on the basis of the gradation curve finer than 25 mm sieve;
- critical filters will be checked and conform with requirements of the USSCS criteria (Sherard and Dunnigan, 1989);
- the uniformity coefficient of the filter D60/D10 will not exceed 20 to limit segregation potential;
- filters will contain no more than 5% fines passing the 0.074 mm sieve and these fines will be non-plastic.

The gradation curves for the various classes of embankment fill will be specified to provide the following base-filter relationships.

Base Material	Filter Material
Class 1, 1A	Class 2,2A
Class 2, 2A	Class 3C, 4B
Class 3C or 4B	Class 4
Class 4	Class 5*
*D15/D85 < 5 only	

4.3.3 Riprap and Erosion Protection

The design of riprap to protect the upstream slope of embankment dams and dykes will be based on Corps of Engineers (1994) and Saville *et al* (1963) procedures, modified as necessary for the effects of ice action (CEA, 1988). Erosion protection necessary for overburden along approach and downstream channels will be based on Corps of Engineers (1991) procedures. Final design will consider recent studies carried out by Quebec Hydro and the Canadian Electrical Association.

4.4 FOUNDATION MATERIALS

Foundation materials under the embankment structures are:

- alluvium;
- surficial stratified deposits - sands, silty sands, silts and clays;
- glacial till;
- bedrock.

Cofferdams and freeboard dykes will be constructed with a positive cutoff to bedrock or suitable insitu material. Weak, compressible surficial deposits will be stripped as necessary to provide adequate stability. Concrete structures will be founded on bedrock.

4.4.1 Foundation Material Properties

Foundation material design parameters are shown in Table 4.2

Table 4.2
Design Parameters for Foundation Materials

Foundation Material	Unit Weight (kN/m³)	Effective Strength Ø'(°)	Effective Cohesion c'(kPa)	Pore Pressure Parameter B	Undrained Shear Strength Cu(kPa)	Permeability (cm/s)	Comments
Alluvium	21.0	35	0	0	-	1×10^{-2}	
Surficial Stratified Deposits	20.5	25	0	0.4	100	$10^{-2} - 10^{-4}$	
Till	21.0	40	0	0.3	150	1×10^{-5}	
Rock	26.0	35 45	0 700	0 0	- -	$10^{-4} - 10^{-6}$	Parallel Foliation Perpendicular Foliation

4.5 STABILITY

Stability analyses for the cofferdams and embankment dams will be done using effective stress methods utilizing circular or non-circular methods as applicable (Bishop's simplified, Janbu or Morgenstern Price) using manual or computer programs (Stabl, Slope 2) as appropriate with steady state piezometric and uplift pressures calculated from flow nets for the appropriate loading cases.

The factors of safety to be used are:

- End of construction - minimum factor of safety 1.30
- Long term steady seepage..... 1.50
- Rapid drawdown (normal)..... 1.30
- Rapid drawdown (extreme)..... 1.10

- Temporary excavated slopes 1.30

The Muskrat Falls site is located in a zone of low seismicity (NRC Zone 0). Seismic effects on the stability of the structures will be minor, but effects of a 0.03 g acceleration will be evaluated by pseudostatic measures (Seed, 1979).

4.6 SETTLEMENT

Foundation settlement is to be determined using oedometer test data for structures founded on overburden. Internal settlement of compacted embankments founded on rock will be taken as 0.5% of the embankment height for summer compaction. The settlement allowance provided for embankments will equal 1.5 times the anticipated post-construction settlement.

4.7 SEEPAGE

Design of the structures for controlling seepage will:

- minimize seepage quantities and exit gradients;
- limit seepage pressures to ensure stability with respect to shear failure;
- prevent internal erosion or piping of embankment and foundation soils.

Flow nets will be developed manually and by finite element methods. Seepage control measures will include:

- shortest seepage path through the impervious core of dykes and cofferdams and will be not less than half the water pressure head;

- seepage through dam foundations will be controlled with a single line grout curtain in bedrock to a minimum depth of 0.50 times the water pressure head (or 8 m) under concrete gravity structures, and pressure relief drains where the head is > 16 m.

- the length of the shortest seepage path through any pervious zone in an abutment shall be at least ten times the water pressure head. The seepage exit will be filtered with a reverse filter to prevent particle migration. Alternatively, the seepage path may be increased to 15 times the water pressure head by blanketing or other means.

4.8 CREST

Elevation of the crest of freeboard dykes will equal the maximum reservoir level, plus freeboard and settlement allowance. Sufficient granular fill will be provided to prevent frost degradation of the top of the core. Reservoir maximum operating water level will be El. 39.0 m. Freeboard calculations will be based on methods outlined by Saville *et al* (1963) with effective fetch, setup, significant wave height and freeboard to be determined for each structure. Settlement allowances will be determined for each embankment dam section as outlined in Section 4.6.

4.9 EARTH PRESSURE

4.9.1 Active Pressure

Against vertical or nearly vertical surfaces, Coulomb's theory will be used assuming the surface to be frictionless. Rankine's theory will be used wherever the pressure against a vertical plane within the soil is considered. For active conditions to exist the structure must tilt or yield at least 1/1,000 of its height under this pressure.

4.9.2 At-Rest Pressure

Soil pressure at rest will be applied wherever the structure yields or tilts under the load of a compacted fill less than 1/1,000 of its height. It will be taken as either twice the active pressure or based on the coefficient of earth pressure at-rest $K_o = 1 - \sin.\varnothing'$.

4.9.3 Passive Pressure

Passive soil pressure against vertical, or nearly vertical, structural surfaces shall be calculated using Coulomb's theory assuming surfaces to be frictionless. In order to develop full passive pressure the structure must yield or tilt towards the soil at least 1/1,000 of its height.

4.10 SPUR STABILIZATION

Stratigraphic sequence consists of 12-15 m of terrace sands, ice-contact stratified drift consisting of interbedded sand and silt from El. +50 to 0 m, and marine clay from El. 0 to -60 m which is underlain by proglacial or glaciofluvial sand and gravel which infills the preglacial bed of the Churchill River, to as deep as El. -250 m. Mass wasting necessitated installation in 1981-82 of an interim line of pumped wells to help stabilize the spur.

Additional measures to ensure long term stability under operating conditions include:

- an upstream rockfill stabilizing berm to provide wave protection and prevent drawdown failure under operating conditions;
- downstream crest unloading, erosion protection and relief wells to augment stability under operating conditions;
- improve drainage of the kettle lakes north of the spur to reduce ground water recharge.

The hydrogeologic model of the spur will be refined using data on pumping performance and piezometer response since 1982 and stability of the spur refined using both circular and non-circular stability analyses. Long term efficiency of the pumping system will be assessed and the need for extension of the pumping system assessed on the basis of Corps of Engineers relief well theory and flow nets. Design of stabilization measures will be finalized.

4.11 ROCK EXCAVATIONS

Bedrock is, generally fresh, Precambrian granite gneiss with mafic bands and pegmatite stringers. Foliation is well developed, striking east-west and dipping at 30° to 60° to the south. Rock quality is mainly good to excellent but zones of poor quality, fractured rock occur locally due to shear zones and minor faults. These zones will intersect the tunnels and structural excavations, and necessitate treatment with rock bolts, shotcrete and drain holes. Three joint sets exist, with parameters shown in Table 4.3, which in conjunction with shear zones and faults will form wedges and require support. The geologic mapping shows faults and shear zones trending dominantly east-west parallel with the river and dipping either sub-vertically or parallel with foliation.

Table 4.3
Joint Set Characteristics

Set	Strike	Dip	Spacing (m)	Continuity	Properties
J1	N-S	80-90^{0}W	0.2-1.0m	>10m	Planar, smooth
J2	E-W	25-70^{0}S	0.2-1.0m	>10m	Planar, rough
J3	W	65^{0}N	0.2-1.0m	>10m	Planar, rough

A partly re-healed 6.5 m to 7.5 m wide zone striking NE-SW with dip 80^0 N is present on the southeastern flank of the rock knoll. Several 0.2 m wide gouge zones occur in this feature, with rock quality generally ranging from poor to excellent.

4.11.1 Rock Support Design Parameters

The parameters shown in Table 4.4 will be used for rock excavation and support design as representative of the overall rock mass, although local variation will occur in fault, shear and fracture zones.

Table 4.4
Rock Support Parameters

Property		Parameter	Comments
Unit Weight		26.0 kN/m^3	Rock mass
Unconfined Compressive Strength		125 Mpa	
Modulus of Elasticity E		60 GPa	
Shear Strength	- rock mass	Ø' = 45^0 c=700 kPa	Rock mass
	- foliation	Ø' = 35^0 c'=0	Lower in fault/shear zones
	- other joints	Ø' = 40^0 c=0	Lower in fault/fractured zones
Permeability	- Top 10m	3 x 10^{-4} cm/s (30 Lugeons)	Locally higher in fractured zones adjacent to faults
	- Below 10 m depth	1 x 10^{-5} cm/s (1 Lugeon)	
	- Fractured zones	1 x 10^{-3} cm/s (100 Lugeons)	

Potentially unstable rock wedges on the excavation faces will be identified from geologic mapping and their stability assessed by manual or computer stereonet (DIP) and wedge (WEDGE) analysis. Rock support measures (rockbolts, SFR shotcrete, drain holes) will be designed to provide a minimum factor of safety of 1.5 for temporary construction support and 2.0 for permanent support using procedures outlined by Hoek and Bray (1981). Rockbolts necessary for permanent support will be fully grouted for corrosion protection with cement grout. Pre-bolting of the powerhouse draft tube rock noses will be designed as necessary to minimize over break and backfill concrete quantities.

4.11.2 Tunnel Rock Support

Rock quality along the diversion tunnels will be evaluated for drilling and geologic mapping data and rock quality established using the NGI Q system (Barton *et al*, 1974) and checked using the geomechanics (RMR) system (Bienawski, 1974) of rock classification. Rock support requirements will be designed based on rock quality class with provision for modification as necessary during tunnel excavation to suit actual rock quality exposed.

Due to the large span of the tunnels, crown pattern bolting and shotcrete support will be necessary in portions of the tunnels with fair-poor quality rock. Side wall support will range from spot bolts to pattern bolting depending on rock quality. Potentially errodable gouge and shear zones will be treated with shotcrete (EPRI, 1987).

4.11.3 Foundation Grouting and Drainage

Foundation grouting under the embankment dams and gravity structures will be designed based on geologic mapping and water pressure test data using procedures outlined by Houlsby (1982) and from precedent from major hydro developments on the Canadian Shield (James Bay and Nelson River). Grout curtains will be constructed to a depth of 0.33 times the head at maximum water level (minimum 8 m) under embankment dams and to a depth of 0.50 (minimum 8 m) under concrete gravity structures. Grout curtain depth will be deepened locally in highly pervious fractured zones as required. All grout curtains will be closed by split-spacing with primary grouthole spacing of 6 m. Drain curtain depth and spacing will be selected from precedent hydro developments on the Canadian Shield.

4.12 REFERENCES

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Sherard, J.L. and Dunigan L.P. (1989) "Critical filters for impervious soils" ASCE J. Geotech Eng. Vol. 15, No. 7

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U.S.A.C.E. (1991) "Hydraulic design of flood control channels "Engineering Manual EM 110-2-1601.

Wilson, S. D. and Marsal, R. J. (1979) Current Trends In Design and Construction of Embankment Dams, ASCE New York

5.0 CIVIL/STRUCTURAL

5.1 GENERAL

5.1.1 References

- Code for design of Concrete Structures CSA Standard - A23.3 - 94
- United States Department of the Interior - Bureau of Reclamation - Design Standard No. 9
- National Building Code of Canada 1995
- American Concrete Institute ACI Standard 318 - 8.9 (Revised 1992)
- Prestressed Concrete CSA Standard A735

5.1.2 Properties of Materials

5.1.2.1 Unit Weights

• Reinforced concrete	=	23.5	kN/m^3
• Mass concrete	=	22.75	kN/m^3
• Roller compacted concrete	=	23.5	kN/m^3
• Steel	=	77.0	kN/m^3
• Water	=	9.81	kN/m^3

5.1.2.2 Material Stresses

- Concrete in accordance with Canadian Standards Association (CSA)
- Steel in accordance with Canadian Standards Association (CSA)

5.1.2.3 Reinforced Concrete Properties

• Cylinder strength at 28 days	=	25	MPa
• Aggregate size	=	38	mm for 1st stage
	=	19	mm for 2nd stage
• Reinforcing steel yield strength	=	400	MPa
• Reinforcing steel strength			
– above normal operating water levels	=	100	% of yield
– below normal operating water levels	=	75	% of yield
– concrete semi-spiral case	=	50	% of yield

5.1.2.4 Structural Steel Properties

• Design yield strength			
– for main members	=	350	MPa
– for secondary members	=	300	MPa

5.1.2.5 Foundation Bedrock Properties

• Maximum Allowable Compressive Stress	=	5.0	MPa (subject to confirmation by site inspection)
• Unit Shear resistance in rock or rock/concrete	=	700	kPa
• Coefficient of Friction between concrete and rock	=	1.00	
• Angle of shear resistance (sound rock)	=	45	degrees
• Unit weight of bedrock	=	26	kN/m^3

5.1.3 Design Loads

5.1.3.1 Design Load Factors

- Dead Load = 1.25
- Hydrostatic = 1.4
- Hydraulic = 1.4
- Live Loads = 1.5

5.1.3.2 Earth Loadings

- Refer to Section 4.9.
- For structures free to deflect, use the Rankine active pressure coefficient.
- For structures not free to deflect, use the Rankine passive pressure coefficient.
- Where vehicular traffic runs adjacent to face of walls, apply a surcharge loading.

5.1.3.3 Ice Loads

• Longitudinal expansion of ice cover against vertical surfaces	=	150	kN/m
• Lateral expansion of ice cover, between piers	=	180	kN/m
• Longitudinal ice cover impact force (horizontal)	=	3,575	kN
• Transverse ice cover impact force (horizontal)	=	2,700	kN
• Longitudinal expansion of ice against surfaces at 45° to horizontal	=	100	kN/m
• Loading against piers on North RCC Dam (per Canadian Bridge Code)	=	4,300	kN

During final design, ice loads will be reviewed and/or refined to take into account findings of the research on ice loading being done by Fleet Technology Inc. for the Canadian Electrical Association (C.E.A.).

5.1.3.4 *Waterhammer*

- Waterhammer design load 10% Hs (Hs = static head = 36 m).

5.1.3.5 *Structure Loadings*

- Snow Load based on Ground Snow Load (Ss), modified to suit snow build up around equipment and structures. = 4.8 kN/m^2
- Snow load rain component (S_R) = 4.8 kN/m^2
- Wind Load 1:30 years = 0.40 kPa
- Office Areas = 4.8 kN/m^2
- Bridge to carry Highway Load (CS 600) and checked for construction traffic.

5.1.3.6 *Seismic Loads*

- Region of low seismic intensity - (Goose Bay Za = 0, Zv = 0, v = 0.0)

5.1.4 Serviceability Requirements

Under Normal Loads:

- Deflection of bridges less than span/800
- Sway of towers less than height/400

5.1.5 Factors of Safety

5.1.5.1 *Sliding*

- Factor of safety against sliding defined as :

$$FS = \frac{(W-U)f + CA}{H}$$

where:

FS = Factor of safety
W = Gravitational force
U = Uplift
f = Concrete-rock friction coefficient = 1.0
C = Concrete-rock cohesion 700 kPa on sound rock
A = Contact area
H = Total Horizontal Force

- Factor of safety with cohesion, normal condition, FS >= 3.0
- Factor of safety with cohesion, extreme condition, FS >= 1.0

5.1.5.2 *Overturning*

- Factor of safety against overturning, defined as:

$$FS = \frac{\text{Restoring Moment}}{\text{Overturning Moment}}$$

- Factor of safety with resultant within middle third, normal condition, FS >= 1.5
- Factor of safety with resultant within middle half, extreme condition, FS >= 1.1
- In final design, influence of upstream crack would be evaluated per USBR/FERC procedures.

5.2. SIGNIFICANT DATA

5.2.1 Diversion Structures

• Design flow (Q_{40})	5,300	m^3/s
• Approach channel sill invert elevation	12.0	m
• Maximum WL, Q_{40}	30.5	m
• Normal operating level for ice control	24.0	m
• Invert elevation	4.0	m
• Number of inlet gates	4	

5.2.2 Intake/Powerhouse

5.2.2.1 *Intake Substructure*

• Approach channel invert elevation	1.0	m
• Water passage sill elevation	3.0	m
• Number of Intake head gates	12	
• Width of Intake head gate	7.33	m
• Height of Intake head gate	16.30	m
• Number of Bulkhead gates	3	
• Width of Bulkhead gate	7.33	m
• Height of Bulkhead gate	17.5	m
• Number of Trashracks	12	
• Width of Trashracks	7.33	m
• Height of Trashracks	22.0	m

5.2.2.2 *Intake Gate House*

Intake gate house consists of a continuous, below deck, gate hoist enclosure with removable roof hatches for maintenance access.

5.2.2.3 *Powerhouse Substructure*

• Generator floor elevation	14.0	m
• Turbine floor elevation	8.0	m
• Distributor centerline elevation	-4.5	m
• Drain sump elevation	-30.5	m
• Draft tube exit elevation	-28.0	m
• Maximum Tailrace Water Level at 1:100 year flood	7.5	m
• Mean Tailrace Water Level	3.0	m
• Minimum Tailrace Water Level	2.0	m
• Turbine type:	3-Propeller 1-Kaplan	
• Rated Turbine Discharge per unit	665	m^3/s
• Number of generating units	4	
• Unit Spacing	36.0	m
• Gross head	36.0	m
• Rated Net head	35.0	m
• Runner diameter	9.0	m
• Generator rotor diameter	14.0	m
• Powerhouse length	187.5	m
• Powerhouse width	68.8	m
• Width of main generator floor	28.0	m

5.2.2.4 *Powerhouse Superstructure*

• Switchyard loads on Powerhouse roof:	not available.
• Transmission line pull-off loads on Powerhouse roof:	not available.

5.2.3 Spillway

5.2.3.1 *Spillway Substructure*

• Width to outside of end piers	57.25	m
• Length to end of apron	72.0	m
• Number of bays	3	
• Width of bay	13.75	m
• Width of pier	4.0	m
• Normal Reservoir Operating Water Level	39.0	m
• Maximum Flood Level	44.0	m
• Spillway crest Ogee elevation	18.8	m
• Approach channel invert elevation	5.0	m
• Exit channel invert elevation	5.0	m
• Type of spillway gate	Vertical fixed wheel	
• Number of gates	3	
• Width of gate (nominal)	13.75	m

- Height of gate: (sill to maximum operating WL) 20.2 m
- Number of sets of stop logs 1
- Maximum flow capacity 10,800 m^3/s
- Adjacent North Dam Crest Elevation 39.5 m
- Road deck elevation 45.5 m

5.2.3.2 *Spillway Superstructure*

- Spillway superstructure includes: gate support structure, hoist bridge and enclosure.

5.3 LOAD CONDITIONS

5.3.1 Intake/Powerhouse

5.3.1.1 *Intake Substructure*

- Analyze intake/powerhouse structure as a monolith keyed into foundation rock.
- Normal uplift, assuming drain efficiency of = 67 %
- Extreme uplift, assuming drains blocked = 100 %
- Hydrostatic - normal with upstream WL = 39.0 m
- Hydrostatic - extreme with upstream WL = 44.0 m
- Check for flotation with gates closed.
- Load Combinations as follows:

Intake Structure Load Combination

Case	Condition	Gate Positions	Uplift Pressure	Ice	Wind	Seismic Activity
A	normal	12 gates open	drains in operation	static pressure	yes	no
B	normal	9 gates closed 3 gates open	drains in operation	impact	yes	no
C	normal	9 gates closed 1 Set of Stoplogs in place	drains in operation		yes	no
D	extreme	as A	drains blocked	static pressure	yes	no
E	extreme	as B	drains blocked	impact	yes	no
F	extreme	as C	drains blocked		no	no

5.3.1.2 *Intake Deck*

- Deck live load:
  - Highway loads (CS600) and Canadian Bridge Code Loading
  - Mobile crane loading from headgate, headgate hoist and emergency stop log handling.
  - Construction loading - to be spread to meet above criteria.

5.3.1.3 *Powerhouse Substructure*

- Floor Live Loads

– Control Room, Turbine floor:	15	kN/m^2
– Generator room floor:	50	kN/m^2 or MS 250 truck loading
– Repair Bay:	70	kN/m^2 or Maximum Point Load
– Erection Bay:	70	kN/m^2 or uniform load of 25 kN/m^2 plus concentrated blocking loads from a generator rotor of 6000 kN
– Transformer deck:		
– shipping weight on two points		
– live load	25	kN/m^2

- Construction Loads - to be spread to meet above criteria.

5.3.1.4 *Powerhouse Superstructure*

- Two 400 t capacity Powerhouse cranes with 25 t capacity auxiliaries and a 50 t mass equalizing beam - loads will depend on suppliers crane bridge dimensions.
- Switchyard, etc. - loads will depend on suppliers structure and equipment details.
- Horizontal force due to transmission line tension = 670kN (to be confirmed).

5.3.2 Gated Spillway

5.3.2.1 *Spillway Substructure*

- Analyze spillway structure as a monolith comprising pier and adjacent half bay.

• Longitudinal expansion of ice cover	=	150	kN/m
• Lateral expansion of ice cover between piers	=	180	kN/m
• Longitudinal ice cover impact force (horizontal)	=	3,575	kN
• Transverse ice cover impact force (horizontal)	=	2,700	kN
• Normal uplift, assuming drain efficiency of:		67	%
• Extreme uplift, assuming drains blocked:		100	%
• Hydrostatic - normal with upstream WL	=	39.0	m
• Hydrostatic - extreme with upstream WL	=	44.0	m

Load Combinations as follows -:

Spillway Load Combination						
Case	Condition	Sluices	Uplift Pressure	Ice	Wind	Seismic Activity
A	normal	2 gates closed	drains in operation	static pressure	yes	no
B	normal	1 gate closed 1 gate open	drains in operation	impact	yes	no
C	normal	1 gate closed 1 Set of Stoplogs in place	drains in operation		yes	no
D	extreme	as A	drains blocked	static pressure	yes	no
E	extreme	as B	drains blocked	impact	yes	no
F	extreme	as C	drains blocked		no	yes

5.3.2.2 *Spillway Superstructure*

- Hoist Loads:
  - Gates in normal operation
  - Gate blocked at any level and horizontal reactions
  - One gate blocked on both sides

5.4 RCC DAM

Loading Conditions

Normal	-	Hydrostatic	WL = 39.0 m + ice at 0.3 below crest
	-	Hydrostatic	WL = 40.0 m (for Q_{1000})
	-	Uplift	assuming 66.7% drain efficiency
Extreme	-	Hydrostatic	WL = 44.0 m (PMF)
	-	Uplift	assuming all drains inoperative

6.0 MECHANICAL

6.0 MECHANICAL

6.1 DIVERSION FACILITIES

6.1.1 Inlet Structure Gates and Hoists

- The Diversion mechanical equipment must be designed to handle the 40 years recurrence flood and be capable to control the upstream water level to at least elevation 24.0 m in winter during the construction phase.

- Diversion Gates will be required at each tunnel inlet, to control the upstream water level in winter during the construction phase. However, the gate operation shall not require permanent gate throttling.

- The two vertical lift gates upstream of each portal inlet will be equipped with downstream seal, and associated downstream air ventilation shafts.

- The Diversion gates will all be heated and have heated gains to guarantee the required gate operations at any time during the winter seasons of the construction phase. The operation of each gate will be achieved by means of individual hoisting mechanism.

- The gate will close under its own weight and will have sufficient weight (using concrete ballast if necessary) so that the sum of all the forces in a downward direction will exceed the sum of the opposing forces by 25% when closing against the maximum possible upstream water level at closure and with the maximum flow obtainable assuming free discharge.

- Steel liner plates will be provided between the wheel track and side seal frame and at the sides of the water passage downstream of each gate guide to protect the concrete against cavitation and abrasion damage. These plates will be offset from the water passage line at their upstream end and will taper into the water passage line at a 1 in 12 slope.

6.2 INTAKE

- Each unit of the four unit intakes will comprise three water passage openings.

- Each unit intake will have a set of three service (or head) gates and will be equipped with a set of three trashracks.

- A single set of three bulkhead gates will be provided for use as required for dewatering of the four unit intakes.

6.2.1 Service (Head) Gates and Hoists

- Preferred service gate type : vertical fixed wheel with upstream seal.
- Gates to be designed to close under their own weight.
- Service gates will be designed to be lowered against maximum transient flow through the generating unit. The rated unit continuous flow is approximately 665 m^3/s.
- Embedded parts of wheel paths and sealing surfaces will be fully machined. Wheel tracks will be 50 BHN points higher in hardness than the wheel rims. Seal contact strips will be stainless steel.
- Each service gate will be operated by a wire rope hoist.
- The wire rope hoist will be capable of raising, holding or lowering the gate at any position during any operation with maximum head. It will also be capable of cracking the gate open against full reservoir head. After the water passage has been filled, hoisting will be completed under balanced pressure.
- Service gate in the raised position will be stopped approximately 300 mm above the lintel and can be lowered from the full gate open position in less than five minutes in an emergency situation.
- The filling of the water passage shall be done by cracking the gate to about 300 mm.
- Service gate shaft to serve as an air vent during water passage filling and dewatering.
- Normal lowering speed of intake service gate will be 1.2 m/min.
- Emergency lowering speed of intake service gate will be between 2.1 and 3.0 m/min and will be controlled by a fan brake on the hoist.
- Automatic emergency closing of service gate will be initiated on occurrence of one or more of several specific trip settings on the generating units. Manual emergency closing could be initiated from governor cabinet on the turbine floor, the control room or the hoist chamber.
- During winter, the water surface in the service gate shaft will be heated.

6.2.2 Bulkhead Gates

- Specific slots will be provided upstream of each service gate to install the bulkhead gates required to isolate one generating unit for maintenance purposes.
- Bulkhead gate will be designed for installation under hydraulically balanced conditions and to withstand the maximum upstream static pressure.

- Bulkhead gates will be sectional and installed/removed with a mobile crane using a two hook lifting beam.

- Bulkhead gate upper sections will be equipped with filler valves operated by the lifting beam.

6.2.3 Trashracks

- Specific slots will be provided at the upstream face of the intake to install trashracks required for the protection of each generating unit against large debris.

- The trashrack will be designed for 7 m of head differential.

- Maximum trashrack flow velocity will be 1.5 m/s for gross intake opening assuming cleaned reservoir.

- The trashrack bars shall have a clear spacing of 100 mm and shall be designed against vibration.

- Trashrack panels will be installed and removed with a mobile crane.

6.3 POWERHOUSE

6.3.1 Turbines

6.3.1.1 General

- The Plant installed summer capacity will be 824 MW at the high voltage bushing of the main power transformers.

- Rated net head normally determined from the head duration curve taking the 50% occurrence point. For Muskrat Falls the rated net head is selected at 35.0 m.

- For feasibility study design head may be taken equal to the rated head.

- Conical section of the draft tube shall be protected by a ribbed steel lining extending to the point where the water velocity is around 8 m/s.

- Draft tube exit flow velocity will be no more than 3.0 m/s.

6.3.1.2 *Type*

For a unit operating under a head range varying between 10 m to 65 m, the use of Kaplan or Propeller turbine types is recommended.

6.3.1.3 *Number of Units, Rotational Speed and Setting*

- Maximum unit capacity at optimal synchronous rotational speed with the adequate unit setting determines the turbine dimensions and number of units required.

- In order to reduce equipment costs unit capacity and rotational speed have to be maximized taking into account transport and dimensional limitations as well as the civil costs associated with the unit setting.

- Choice of high speed turbines is limited by factor "K", defined as the product of the specific speed times the square root of the rated head.

- Upper limit of "K" for large Propeller turbines = 2,780; large Kaplan turbines = 2,930.

- For maximum reliability, recommended upper limit of "K" for large Propeller turbines = 2,650; large Kaplan turbines = 2,800.

6.3.1.4 *Turbine Design Details*

- The runner blades of the Propeller or Kaplan turbines shall be in stainless steel. For Kaplan runners, or Propeller runners with bolted blades, the runner hubs in cast or fabricated mild steel will be acceptable.

- The surfaces of the bottom ring and head cover adjacent to the wicket gates shall be overlaid with type 18-8 stainless steel to improve abrasion resistance.

- The wicket gate operating mechanism consists of two oil servo-motors acting on the operating ring to which the links and levers of the wicket gates are connected.

- The hydraulic system has a rated pressure of 6,300 kPa. The governor pressure tank shall have a volume equal to 20 times the swept volume of the servo-motors. The pumping system of each unit includes a main pump, whose flow per minute shall be twice the piston stoke of the servo-motors, and an auxiliary pump to compensate continuously for system leaks.

- The guide bearings, runners, shafts, wicket gates, wicket gate levers and links, bushings, seals, and all other parts of similar design are interchangeable to enable substitution or replacement to be made quickly and easily when needed.

6.3.2 Powerhouse Crane

- Required crane capacity shall be determined by the generator rotor weight (approximately 740 tonnes).
- Two electric cranes, each of approximately 400 tonnes capacity, will be provided and operated in tandem for the rotor lifting.
- Crane rail elevation will be set so it is possible to lift the maximum unit component assembly over the generator room floor.
- Auxiliary hook capacity: 25 tonnes.
- An equalizer beam (approximately 50 tonnes mass) will be required for handling assembled rotors.
- Hoists will be controlled from a cab.
- Power will be supplied to crane by insulated conductors.
- Main hook maximum lifting speed: 1.5 m/min.
- Auxiliary hook maximum lifting speed: 4.5 m/min.
- Trolley travel: 15 m/min.
- Bridge or gantry travel: 30 m/min.
- Hoist motors will be of wound rotor type.
- Dynamic, electric braking and mechanical brakes.
- Operator's controls of the deadman, vertical master lever type.
- Controls and motors will be enclosed in dust proof housing.
- Girder deflection shall not exceed 1/1,000 of the span.

6.3.3 Draft Tube Crane

- Required crane capacity sufficient to handle one section of draft tube gate.
- Two hook follower beam for draft tube gates handling.
- Trolley and bridge will be controlled from upstream side by pendant push-button station.
- Follower low speed: 1 m/min, high speed 5 m/min.
- Trolley travel: 15m/min
- Gantry travel: 30 m/min.
- Hoist motors will be of totally enclosed squirrel cage induction type.
- Lowering brakes will be of the disc type.
- Holding brakes will be of the mechanical band type.
- Controls and motors will be enclosed in weather proof housings.

6.3.4 Draft Tube Gates

- Sectional stoplog gates will be of the upstream (turbine side) skin plate and seals type.
- Two sets of gates will be provided to isolate one draft tube.
- Filler valve will be installed on the top of the top gate section.
- Follower beam will be designed to withstand the worst load combinations.
- Steel gate sections will be designed for installation in hydraulically balanced conditions and to withstand maximum tailwater elevation.
- Elastomer type synthetic seals will be used.

- Leaf springs will be provided to assist in seal closure.
- Dogging devices will be provided for securing the gates at deck level.
- Support devices will be provided for securing gates in storage position upstream of the gate slots when not in use.

6.3.5 Powerhouse Auxiliary Systems

6.3.5.1 Raw Water System

- Each unit will be equipped with a raw water intake connection feeding the raw water header. The connection will be located upstream of the intake gate but downstream of the bulkhead gate.

- Each unit connection will be sized so the total powerhouse water needs can be satisfied with one unit out of service.

- The water requirement will be defined from the following:

  - total cooling water needs.
  - total filtered water needs.
  - total fire protection water needs.
  - 67% of the total service water needs.

- Piping material: ASTM A-53, Grade B.

- Maximum water velocity in pipes:

  - normal operation: 2.5 m/s.
  - emergency operation: 3.5 m/s.

- All raw water piping and equipment will be insulated to prevent condensation.

6.3.5.2 Generating Equipment Cooling Water

- Purpose: Cool the generators and the generating unit bearings.

- Cooling water source: Raw water header.

- Each unit will have an independent cooling water supply and the flow control will be achieved through a motorized valve located on the return pipe.

- Cooling water from each unit will be returned to its own tailrace downstream of the unit.

- The maximum head loss for the generator coolers will be 100 kPa.

- The cooling water flow will be calculated using a temperature rise of 8 ^{0}C.

- Piping material:

  - 4 in. & lower: sch 10, stainless steel, ASTM A312m TP 304L.
  - 6 in. & above: sch 40, carbon steel, ASTM A53, Grade B.

- Water velocity in pipes:

  - 4 in. & lower: based on 50 kPa/100 m friction loss.
  - 6 in. & above: 2.5 m/s.

- Insulation for condensation control.

6.3.5.3 Fire Protection Systems

- Water based fire protection systems will be provided as follows:

  - Generating equipment: automatic water fixed systems (deluge)
  - Governors areas & oil storage room: automatic water sprinkler systems.
  - Powerhouse, general: stand pipe and hose stations.

- Fire protection systems to be designed in accordance with the following NFPA Standards:

  - no. 10 : Portable Fire Extinguishers.
  - no. 13 : Standard for the Installation of Sprinkler Systems.
  - no. 14 : Standard for the Installation of Standpipe and Hose Systems.
  - no. 15 : Standard for Water Spray Fixed Systems for Fire Protection.
  - no. 20 : Standard for the Installation of Centrifugal Pumps.
  - no. 851 : Recommended Practice for Fire Protection for Hydroelectric Generating Plants.
  - no. 2001 : Standard on Clean Agent Fire Extinguishing Systems.

- Full reliability of water supply even during plant shutdown.

- Water source: Raw water header.

- Full redundancy of pumping equipment and power supply:

  - Two pumps, each with sufficient capacity to sustain 100% of the water requirements.
  - Fire pumps power supply from two reliable sources, i.e. the electrical system and a diesel generator.

- Fire protection water requirements:

  - Oil filled transformer : 0.37 (L/s)/m^2 of floor surface.
  - Generator : 2.5 (L/s)/ m of rotor diameter.
  - Oil storage room & governor areas : 0.17 (L/s)/ m^2 of floor surface.
  - Maximum instantaneous demand: simultaneous operation of one transformer water spray fixed system and two hoses 65 mm dia. hoses.

- Piping material:

  - Wet pipe: sch. 40 carbon steel pipe, ASTM A53, Grade B.
  - Dry pipe: sch. 10 stainless steel, ASTM A312 TP304L.

- Maximum water velocity in pipes: 3.5 m/s.

- The control, telecommunication and computer rooms will be protected with a clean agent fire extinguishing system.

- Portable fire extinguishers of suitable types will be provided at appropriate locations.

6.3.5.4 Potable Water System

- Purpose: Feeding filtered water to the turbine shaft seals and domestic water to drinking fountains, kitchen sink, service sinks, lavatories, showers and toilets with the last three items also supplied with hot water. Portable emergency eyewash and showers are provided in battery room.

- Particulate removal: 25 microns, through sand filters.

- Bacterial control treatment (chlorine or UV) with storage tank sized for a residence time of 20 min.

- Reliability:

  - Three sand filters, any two capable to feed 100% of the potable water and filtered water needs.

  - Two centrifugal pumps (1 running & 1 stand-by).

- Sand filter capacity criteria: 2.8 L/s of water per m^2 of filtration surface.

- Material of construction: Stainless steel.

- Water requirements:

  - Potable water: 3 L/s.
  - Filtered turbine carbon seal: 1 L/s per generating unit. No bacterial control required.

- Piping Material:

  - Potable water: copper, ASTM B88, type K.
  - Turbine seals : sch. 10 stainless steel, ASTM A312 TP304L.

- Pipe sizing:

  - 6 in. & above : 2.2 m/s.
  - 4 in. & lower : based on 50 kPa/100 m.
  - Plumbing Code.

- Insulation for condensation control.

6.3.5.5 Sewage Treatment System

- Purpose: Drainage, treatment and discharge of sanitary waste in accordance with proper environmental standards.

- Requirement: Septic tank and recirculating filter bed.

- Capacity requirement: 15 employees at a daily water use of 0.13 m^3 per person.

- Reliability: All pumps to have 100% redundancy and provision for occasional pumping of the septic tank to a sanitary truck.

- Piping material: PVC.

- Piping sizing: Plumbing Code.

6.3.5.6 Turbine Dewatering and Filling System

- Purpose: Dewatering of Turbine spiral case, runner and draft tube cone, one unit at a time, with intake gates closed and draft tube gates in place, plus occasional complete dewatering of draft tube.

- System capacity: Dewatering of one unit to below turbine runner level in eight hours using dewatering sump system.

  Occasional dewatering of lower draft tube in eight hours using portable sump pump.

- Reliability: Three pumps provided with one stand-by. These pumps will supplement the drainage pumps in emergency conditions (refer to Section 6.3.5.7). One portable sump pump is provided for complete dewatering of the lower draft tube and for gate leakage discharge.

- Capacity data for portable pump sizing:

  - Draft tube gate leakage: 10 L/s.
  - Intake gate leakage: 15 L/s.
  - Draft tube unwatering 100 L/s.

- Piping material: sch. 80 carbon steel, ASTM A53 Grade B.

6.3.5.7 Drainage System

- Purpose: Collect in a drainage sump and evacuate to tailrace the following:

  - Powerhouse seepage water.
  - Leakage from the turbine seals.
  - Fire protection water.
  - Leakage from intake and draft tube gates when a unit is dewatered.
  - Floor washing water.
  - Backwash water from motorized strainer and sand filters.
  - Cooling water from compressors and HVAC systems.

- Capacity criteria: sum of the following:

  - All of the continuous water sources (seepage estimated at 275 L/hr per 30 m^2 of powerhouse foundation area.
  - The two most important intermittent sources.
  - The maximum occasional source.

- Reliability: three pumps (1 stand-by), each with a capacity of at least 50% of the total pumping requirement. In the remote event of total drainage pump failure and rising water in the sump, the rising water will initiate opening of a connection to the dewatering pump suction and the dewatering pumps will automatically start and evacuate the water, down a low water cut out level.

- Sump sizing: capacity to allow a minimum of 15 min pump running time per cycle.

- Environmental control: oil interceptors will be incorporated in all drain lines from possible sources of oil contamination to retain any oil contained in the drainage water for periodical removal and disposal.

- Piping material:
  - network: PVC.
  - pump discharge: sch. 40 carbon steel, ASTM A53, Grade B.
- Slope for sizing piping network & trenches: 1 %.

6.3.5.8 Oily Water Drainage and Interception System

- Purpose:
  - To intercept and separate any leakage or accidental oil spill from the oil filled transformer bay and from governor and oil storage room areas.
  - Such discharges are piped directly to an oil interceptor tank.
- Reliability:
  - Oil separator is designed to retain the maximum possible volume of oil from each area while allowing rain or fire protection water proper drainage.

6.3.5.9 Lubricating/Hydraulic Oil Storage and Handling System

- Purpose:
  - Storage of the new oil delivered from a tank truck into a "new oil" storage tank (approximately 20,000 litres capacity).
  - Moving oil from the "new oil" storage tank through the portable purifier and either back to the same tank or directly to a generating unit.
  - Transferring purified oil from the "new oil" storage tank to a generating unit via a transfer pump.
  - Periodically transferring oil accumulated in oil separators at major sources of oil spill for treatment or disposal.
  - Transferring used oil from any generating unit to the "used oil" storage tank for eventual disposal to a tank truck by means of the transfer pump.
  - Recirculating oil from a generating unit through the portable purifier.
  - Only one of the above operations is to be performed at a time.

- Capacity:
  - tanks: sufficient to contain the lubricating/hydraulic oil volume of one unit.
  - purifier: 2.3 L/s.
  - pumps: positive displacement type, 2 L/s.
  - retention pit: oil volume of the two tanks.
- Piping material: sch.40 carbon steel, ASTM A53, Grade B.
- Piping sizing: based on maximum flow velocity of 1 m/s.

6.3.5.10 Low Pressure Compressed Air System

- Purpose: to provide compressed air for maintenance and repair using pneumatic tools as well as back-up for the generator unit brakes in emergency.
- Capacity: simultaneous operation of six grinders, two drills and two screwdrivers.
- Rated pressure: 860 kPa.
- Reliability:
  - compressors: two water cooled oil lubricated, screw type compressors each with a capacity of 100% of the maximum demand.
  - air receiver: 2 m^3.
- Piping material: sch. 40 carbon steel, ASTM A53, Grade B, galvanized.
- Pipe sizing: pressure drop to the farthest user point of max. 10 %.

6.3.5.11 High Pressure Compressed Air System

- Purpose: provide compressed air to the governor pressure system operating at a rated pressure of 6.3 MPa and to the generator braking system.
- Reliability: two air cooled, lubricated, piston type compressors at 6.9 MPa, each sized for 100% of the needs.
- Capacity:
  - governor air receiver: sufficient to enable the filling of one governor and to pressurize tank in 2 hours, maximum, with the help of the compressors.

- brake air receiver: sufficient capacity to permit the stopping of all the generating units at once.
- compressors: in the air: sufficient capacity to re-establish in 4 hours the initial pressure receiver after the filling of one pressure tank.

- Piping material: stainless steel, ASTM A312 TP304L.

6.3.6 HVAC

6.3.6.1 Basic Requirements

- Purpose:
  - maintain a healthy atmosphere;
  - maintain acceptable temperature levels in the various areas of the powerhouse;
  - exhaust smoke;
  - pressurize staircases and other escape passages to permit evacuation of operating personnel.

6.3.6.2 Code and Standards

- Systems designed in accordance with the following technical Guides, Codes and Standards :
  - ASHRAE.
  - NFPA 90 A "Standard for the installation of air conditioning and ventilation systems".
  - NFPA 204 M "Guide for smoke and heat venting".
  - Canadian National Building Code.

6.3.6.3 Outside Conditions

- Design temperatures based on Goose Bay operating experience:

Design Temperatures		
Design parameter	**Summer**	**Winter**
Outside air	27 °C	-35 °C
River water	16 °C	1 °C

6.3.6.4 General Principles

- Control air circulation and temperature automatically.
- Exhaust all areas exposed to harmful gases or odors.
- Reliability: full, HVAC systems serving energy production equipment or areas to have standby provisions.
- Outside air intake to be located away from exhaust.
- Outside air supply to be ducted and insulated.

6.3.6.5 Ventilation Rates

- Minimum outside air ventilation rate: one air change per hour.
- Total air circulation rate: 6 air changes per hour, based on the working level volume including outside air to avoid inside air stagnation.

More specifically, the following guide will be followed:

Space	Outside air changes per hour	Conditions when air changes should be applied
Oil storage and transfer	12	Continuous
Toilets	12	Continuous
Maintenance product storage (paint, solvent, etc.)	6	Continuous
Welding room	12 / 6	During welding / off time
Battery room	12	Minimum or sized to maintain hydrogen gas concentration below 1 percent by volume during maximum gas generation conditions.
Generator floor	1	Dependent on heat gain from equipment
Sewage handling room	12	Continuous

6.3.6.6 *Inside Conditions*

The ventilation systems are controlled to maintain the following inside ambient temperatures :

ROOM OR AREA	TEMPERATURE (°C)	
	Maximum	Minimum
Main floor and mezzanines	25	18
Generator floor	25	18
Turbine floor	25	18
Drainage Gallery	25	10
Workshops	25	18
Mechanical service rooms (compressor room, pump room, filter room, oil storage room, HVAC rooms, etc.)	25	18
Battery charger room (UPS)	25	18
Battery room	25	18
Offices	22	20
Control, Computer and Telecommunication rooms	22	20

- Minimum supply air temperature from air handling units: 18°C.
- Minimum powerhouse inside temperature during shutdown: 10°C.

6.3.6.7 *Cooling and Heating Loads*

- Louvers in the generator cooling circuit permit circulation of heat from the generator for powerhouse heating.
- The heat dissipation from internal sources will create local cooling loads that will have to be met at all times by the HVAC systems.
- The building winter heat load will also have to be met and appropriate ventilation provided at all times by the HVAC systems.
- Emergency winter heating capacity will be provided to cover the maximum anticipated plant shutdown period.

6.3.6.8 *HVAC Equipment General Arrangement*

- Due to the size of the powerhouse, multiple local water cooled air handling units to be considered to minimize outside air requirements and to reduce duct sizes.

6.3.6.9 *Battery Room Ventilation*

- Room pressure: negative, continuous.
- Minimum air flow based on the most demanding of the following :

  Q (L/s) = 0.025 x no. of cells x charge current
  Q (L/s) = 12 air changes / hour

- Reliability:

  – two exhaust fans (one stand-by),
  – room air is exhausted outside through a dedicated duct.

6.3.6.10 *Normal Exhaust*

- Each of following locals or areas also require an independent exhaust system. A dedicated exhaust duct is provided for each system.

  – oil storage room
  – lunch room and kitchen
  – toilet/locker room and sewage treatment room
  – service bay (when combustion engine vehicles enter the powerhouse).

6.3.6.11 *Emergency Ventilation Systems*

- Purpose:

  – pressurization of staircases and other escape passages to permit evacuation of the operating personnel and give access to fire fighting personnel;

  – exhaust smoke outside and prevent it from migrate to the areas that are not affected by the fire ; this system should cover the powerhouse main floor.

- System design: consider using fans used for the normal powerhouse ventilation.

6.3.6.12 *Air Conditioning*

- Purpose: one system to serve the control room, computer room and telecommunication room. The offices may be served by the same or by an independent system.

- Reliability: two air conditioning units (one stand-by) and a separate outside air supply.
- Humidity control: to be maintained between 30 and 55 % RH.
- Energy conservation and maintenance: consider using service water instead of refrigerant.

6.3.6.13 Noise Criteria

- In the powerhouse, the noise coming from the turbine/generator units generally covers the one from the HVAC equipment. A maximum level of 85 dbA is expected to be met.
- In other areas, the HVAC systems to be treated acoustically to meet the neighboring local noise criteria.

6.4 SPILLWAY

6.4.1 Gates and Hoists

- Required total development spill capacity is 22,100 m^3/s. The spillway structure will be equipped with three gates 13.75 m wide by 20.2 m high for spill capacity of 10,800 m^3/s at the PMF water level of EL. 44.0 m. The additional spill capacity is provided by overflow discharges at the North and South RCC Dams.
- For cold climate and remote control operation, the preferred gate will be of the fixed wheel, vertical lift type, with upstream seal.
- Ratio of height to width for flat gates 1.2 to 1.7.
- All gates and gate guides will be heated for winter operation.
- Gate hoist of the wire rope type.
- Gate operation from push button station at each hoist and/or from remote control center.
- Gates to be designed to close under their own weight.

6.4.2 Stoplogs

- One set of stoplogs will be provided for maintenance and repair.
- Stoplogs will be designed for installation under hydraulically balanced condition with the use of a mobile crane.

- The top stoplog will be provided with a bypass valve. The valve will be held closed with a spring of non-corrodible material and will be operated by the gate follower to flood the space between the stoplogs and gate.

6.5 RUBBER DAM

- Purpose: As one of the main flood handling structures, to:

  - retain water in the reservoir at the normal operating level of 39.0 m;
  - flush floating debris from the reservoir;
  - supplement the gated spillway in discharging the 1 in 1,000 yr flood;
  - supplement the gated spillway and the south dam in discharging floods in excess of the 1 in 1,000 yr occurrence.

- Crest Levels:

  - inflated, 39.5 m
  - deflated, 37.1 m

- Spill Capacity (deflated):

  - normal operation: 1 in 1,000 yr flood, water level at El. 40.0 m, 1,650 m^3/s;
  - extreme condition: PMF, water level at El. 44.0 m, 5,800 m^3/s;

- Materials:

  - rubber dam: standard type body construction with reinforced EPDM rubber, ozone resistant, minimum thickness 12.4 mm;
  - fixing materials: clamps, anchor bolts, etc. shall be galvanized;
  - piping, 4 in. & lower: minimum sch 10, stainless steel, ASTM A312m TP 304L;

- Design will be based on:

  - a height of 2.4 m;
  - a maximum inflation time of 30 min;
  - fixings shall be designed for a factor of safety ≥ 3.0;
  - automatic, PLC based, inflation/deflation system for multiple spans, with graphic interface and remote communication capability. Deflation trip levels as follows:

Span	Water Level
1	40.0 m
2	40.5 m
3	41.0 m
4	41.5 m
5	42.0 m

- Reliability: system will allow for sufficient redundancy of the major mechanical components such as electric blower units and automatic exhaust valves.

- Stoplogs: unwatering of a stoplog bay will require closure against a head of about 2.5 m. Closure of an individual rubber dam bay for inspection and service of the facilities will be effected by means of either:

  - a pontoon supported closure panel sealed against the pier nose and sill face, or;

  - a system of stoplogs and posts which can be installed from the bridge deck.

It is recommended that both of the above alternatives be evaluated in the final design phase.

7.0 ELECTRICAL

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The design characteristics given herein are for costing purposes only. Final characteristics will be established following system studies at the design stage.

7.1 GENERATOR

7.1.1 Generator

• Type	Vertical/air cooled/salient poles	
• Quantity	4	
• Rating	220	MVA
• Voltage	15	kV p-p
• Phases	3	
• Synchronous speed	81.8	rpm
• Power factor	0.95	
• Frequency	60	Hz
• Temperature rise	ANSI C50.13 8	(Class B)
• Insulation:		
◆ Stator Materials	Class F	
◆ Design	80	°C rise
◆ Rotor Materials	Class F	
◆ Design	80	°C rise
• Inertia Constant (H)	4	approx.
• Rotor Mass	740	tonnes
• Pit Dimensions	Approx. 14 m x 7.5 m	
• Number Poles	88	
• Bus rating	10,000	Amps
• Temperature probes	RTD's for local indication and DCS inputs	

7.1.2 Generator Protection

The main protection for the generator will be by dual multifunction digital generator protection packages, with one for the main protection and the other for backup protection and other plant protection. Power supply will be 125 VDC. The listed protections will be covered by the main multifunction relay and backed up by a separate multifunction relay and separate descrete relay(s) if required to backup all functions listed. Independent dc or dual dc powered relays tripping into dual trip logic will be used powered by the group A and group B battery systems. The generator and overall differential protections will be in separate systems. The protection will consist of the following:

- 100% stator ground fault (Note a)
- 90% stator ground fault (60Hz Voltage) as backup
- Loss of excitation
- Negative phase sequence current (Alarm & Trip stages)

- Reverse power
- Over and under frequency
- Split phase if dual windings used
- Over/under voltage
- Voltage restrained overcurrent and/or distance backup
- Generator circuit breaker breaker fail protection tripping into the transformer protection logic
- Out of Step
- Stator / rotor over temperature (RTD direct and trip logic)
- Generator Differential protection
- Overall Differential protection overlapping the generator circuit breaker
- Transformer over excitation
- Fuse failure detection (duplicate PT's)
- Station service transformer protection (overcurrent)
- Excitation transformer protection (overcurrent)
- Generator ground alarm or time delay trip (generator breaker open)
- Rotor earth fault alarm (2) (Note b)
- 15 kV Generator breaker fail

Notes:

a) The method of providing 100% stator ground fault will depend on the manufacturer and will be either natural (generated) third harmonic, under voltage or differential balance or second harmonic injection and is only feasible with individual generator transformer connection (3 windings)

b) The rotor earth fault detection will probably be part of the excitation system.

7.1.3 Excitation System

• Type	SCR
• Control Type	Digital with LAN interfaces,
• Continuous current	by manufacturer
• Voltage rating	by manufacturer
• Temperature rise	40 °C
• Ceiling Voltage	10 pu.
• Voltage range	20 to 50 %
• Transformer	Dry type, epoxy
• SCR bridge	As required +1 redundant stand-by bridge
• Memory	Programmable parameters
• Cooling	Air with redundant fans
• Protection and Limiters Alarm	overexcitation, rotor earth fault, maximum excitation time-out
• Control set point	15 kV bus or joint VAR control via DCS
• Interface into the protection and DCS systems	Provided at digital level
• Field Breaker	By manufacturer

Some of the function may be built into the exciter software.

7.1.4 Generator Bus

- Type — Isolated phase, Continuous bonded
- Rated Current — 10,000 Amps
- Rated Voltage — 15 kV
- BIL — 110 kV
- Length — 20 m (for costing purpose only)
- Potential Transformers — 3 sets
- Current transformers — 3 sets
- Bus taps — 2,000 Amps
- Surge arrestors — 1 set
- Surge Capacitors — 1 set
- 90 deg. Bends/phase — 2(estimate for costing purposes)

7.1.5 Generator Breaker

- Type — SF_6
- Voltage — 15 kV
- Class — 15-24 kV
- Current — 10,000 Amps.
- Interrupting Duty — 100 kA, 1 sec.
- Control Voltage — 125 Vdc
- BIL — 110 min. kV
- Alarm Interface — As required
- Frequency — 60 Hz
- Disconnect — Telescopic , Air Type
- Auto open logic on SF_6 Fail will open bus disconnect

7.1.6 Governor

- Control Type — Digital with LAN interfaces,
- Control set points — Frequency with option for joint power control via DCS system
- Protection and alarm — Interface into the protection and DCS systems.

7.1.7 Local Control

Local control will be provided on the generator floor adjacent to each generator in a panel lineup that includes a local control panel, the protection and synchronizing panel, the automatic start-stop control sequence panel and the distributed control system interfaces. The AVR and governor digital panels may also be located in the same lineup of panels. These panels will provide:

- Local interfacing into the governor and excitation control logic.

- Local manual step by step control of the start-stop control sequence from the local panel.
- DCS local PLC logic control to automatically start and stop the unit from standstill to speed no load with interface into the group control and DCS control systems.
- Local direct wire control (125 Vdc), synchronizing logic, local indication and alarm unit with repeat and interposing relays to interface with the DCS Data Processing Unit (DPU) and the start-stop automatic control sequence PLC system.
- A digital instrumentation and/or transducer system for the generator measurements with 4-20 mA and/or LAN interface to the local DCS system DPU.
- DCS Data Processing Unit (DPU) for generator ancillary system status, measurements and alarm interfacing to the DCS system.

7.2 TRANSFORMATION AND SWITCHYARD

7.2.1 Power Transformer

The transformer has two 15 kV windings and one 230 kV winding. The transformer has the following characteristics:

• Type	outdoor, oil, air cooled
• Quantity	2
• Rating	165/165/330 MVA ONAN 220/220/440 MVA ONAF
• Generator Voltage	15 kV
• Secondary Voltage	230 kV
• Maximum operating voltage	245 kV
• Frequency	60 Hz
• BIL:	
◆ Low Voltage	110 kV
◆ High Voltage	950 kV
◆ Neutral end	110 kV
◆ High voltage bushings	1050 kV
◆ Neutral bushings	110 kV
• Tap changer off load kV	+/-5% (Note)*
• Connection	Delta/Wye grounded

Note*: subject to the system study results on-load taps may be required to compensate for low voltage when operating on one line.

7.2.2 Transformer Protection and Local Control

The main protection will be a digital differential protection package with a separate digital package for backup protection. The listed protections will be covered by the main multifunction relay and backed up as required by the second multifunction realy. Independent 125 Vdc

powered relays tripping into dual 125 V dc trip logic will be used powered by the group A and group B battery systems. The mechanical trips (Buchholz, WTT etc.) and differential protections will be in separate systems. The protection will consist of the following:

- A digital transformer differential relay overlapping the generator circuit breaker, and the transformer HV bushing CT's.
- Digital overcurrent relays for high side phase and neutral overcurrent protection fed from high side phase and neutral bushing CT's.
- Alarm interface and trip logic for transformer mechanical protections; gas, surge (Buchholz), winding temperature, pressure relief etc.
- Dual trip logic operating into the appropriate high side circuit breakers and to the generator trip logic. The transformer high side disconnect will be opened automatically for all "lockout" type faults.
- Local direct wire control (125 Vdc), synchronizing logic, local indication and alarm unit with repeat and interposing relays to interface with the DCS DPU.
- A digital instrumentation and/or transducer system for the transformer bay with 4-20 mA and/or LAN interface to the local DCS system DPU.
- DCS Data Processing Unit (DPU) for breaker and disconnect status, measurements and alarm interfacing to the DCS system (Bay Box).

7.2.3 Station Bus Protection and Local Control

A bus protection system will be provided for each section of bus overlapping the line and section circuit breaker. The interface to the generator transformer will be from bushing CT's and will overlap the transformer 230 kV disconnects. It will require one CT core per circuit and be connected to provide phase by phase protection, CT open circuit alarms and internal check and block logic. The other facilities will be:

- Alarm interface and trip logic to trip all circuit breakers connected to that section of bus.
- Trip logic to receive circuit breaker fail input to trip all circuit breakers connected to that section of bus.
- Trip logic to interface with the transformer (or line) disconnect trip circuits for isolation purposes.
- Digital overcurrent relays for the bus tie circuit breaker(s) phase and neutral overcurrent protection.
- Digital circuit breaker fail protection for the bus tie circuit breaker(s) tripping into the bus protection logic.

- Local direct wire control (125 Vdc), synchronizing logic, local indication and alarm unit with repeat and interposing relays to interface with the DCS DPU.

- A digital instrumentation and/or transducer system for the tie breaker and bus voltages with 4-20 mA and/or LAN interface to the local DCS system DPU.

- DCS Data Processing Unit (DPU) for breaker and disconnect status, measure and alarm interfacing to the DCS system (Bay Box).

7.2.4 Transmission Line Protection and Local Control

Dual line protection will be provided together with protection signalling to the remote station, plus local and remote backup. The protection at both ends of each line to be the same.

A first main protection consisting of a digital optic fibre differential protection using multiplexed digital channels or two directly connected fibers in the OPGW dependent on the optical power output available. The following will be provided:

- independent phase to phase differential protection,
- standby direct transfer trip facilities for CB fail etc.,
- facilities for single pole tripping and auto-reclose interfaces,
- end to end data transmission delay compensation, and
- channel monitoring and fault data storage.

A second main protection consisting of digital distance protection fed from line CVT's and separate CT cores with the following facilities:

- tele-protection logic for permissive under or over reach and ground fault direction comparison logic,

- three zone distance, phase and ground elements with single pole trip logic for ground faults,

- ground directional elements operating in a directional comparison mode for three pole tripping only,

- time delayed non-comparison ground directional backup,

- distance to fault measurement, PT monitoring and data storage,

- a minimum of three zones of measurement, and

- facilities for single pole tripping and selectable single and/or three pole automatic re-close interfaces.

Protection logic as follows:

- Auto-reclose logic with the following facilities:
  - single pole high speed reclose (500-1,000 ms),
  - three pole delayed reclose with synchrocheck (5-15 s)
- synchronizing logic with selection for dead-line/live bus, live bus/dead-line, and/or synchronize close,
- synchrocheck and or automatic (system) synchronizing relays,
- under and over voltage protection (frequency protection if required),
- digital overcurrent relays for phase and neutral overcurrent protection,
- alarm interface and trip logic,
- digital circuit breaker fail protection tripping into the bus protection logic,
- local direct wire control (125 Vdc), synchronizing logic, local indication and alarm unit with repeat and interposing relays to interface with the DCS DPU,
- a digital instrumentation and/or transducer system for the line bay with 4-20 mA and/or LAN interface to the local DCS system DPU, and
- DCS Data Processing Unit (DPU) for breaker and disconnect status, measurement and alarm interfacing to the DCS system (Bay Box).

7.2.5 Teleprotection System

- First main teleprotection signaling with four channels using multiplexed data channel (56-64 kb) on the optic fiber as data medium as follows:
  - Channel 1 Direct transfer trip for CB fail etc.
  - Channel 2 Permissive transfer trip for distance protection
  - Channel 3 Directional comparison signal for directional ground
  - Channel 4 Generation Shed

- Second main teleprotection signaling with four channels using data channels (56-64 kb) on the microwave multiplex system as follows:
  - Channel 1 Direct transfer trip for CB fail etc.,
  - Channel 2 Permissive transfer trip for distance protection,
  - Channel 3 Directional comparison signal for directional ground,
  - Channel 4 Generation Shed.

7.2.6 Circuit Breaker (switchyard)

• Type	SF_6
• Voltage	230 kV
• Voltage Class	245 kV
• Current	2,500 Amps
• Interrupting Duty	40 kA
• Control Voltage	125 VDC
• BIL.	950 kV
• Temperature range	-50 ^{0}C to 40 ^{0}C

- Reclosing — Single pole and three pole
- Alarms: — Low SF_6 closing status Pole discrepancy trip circuit fail
- Fail Safe Circuits — Breakers blocking Low SF_6

7.2.7 Disconnect Switch Bus Side

- Type — Vertical or Horizontal Break
- Voltage — 230 kV
- Voltage Class — 245 kV
- Current — 2,000 Amps
- Operation — Motor or Manual fully interlocked.
- Motor Supply — 125 Vdc

7.2.8 Line Disconnect Switches

- Type — Vertical or Horizontal Break
- Voltage — 230 kV
- Voltage Class — 245 kV
- Current — 3,000 Amps
- Operation — Motor and manual, fully interlocked
- Motor Drive — 125 Vdc
- Ground Switch — 3 pole 40 kA (make & carry) manual, mechanically interlocked with main drive

7.2.9 Transformer Disconnect Switches

- Type — Vertical or Horizontal Break
- Voltage — 230 kV
- Voltage Class — 245 kV
- BIL — 950 kV
- Current — 2000 Amps
- Operation — Motor and manual, fully interlocked
- Motor Drive — 125 Vdc
- Ground Switch — 3 pole 40 kA (make & carry) manual, mechanically interlocked

7.2.10 Current Transformers

- Type — Outdoor SF_6, Oil or Epoxy insulated
- Cores — Five
- Accuracy — 1 Metering and 4 protection
- Ratio
  - Bus Tie — 2,500-5
  - Line Bays — 2,500-5
- Secondary — 5 Amps

7.2.11 Potential Transformers

- Type — Outdoor / Capacitor Voltage Transformer
- Cores — Three
- Accuracy — 1 Metering and 2 protection
- Voltage — 230/√3 kV -115/115/√3:115/√3 volts
- Ratio — 2,000/1
- BIL — 950 kV

7.2.12 Lightning Arrestors

- Type — Station /Metal Oxide
- Voltage — 230 kV class.
- MCOV — Per detailed design study
- Class — 40 kA

7.3 PLANT AUXILIARY SYSTEMS

7.3.1 Station Service Transformer

- Type — Dry type
- Capacity
  - Station/Unit Transformers — 2500 kVA
  - Unit Transformers — 1000 kVA
- Primary Voltage — 15 kV
- Secondary Voltage — 600/347 V
- Frequency — 60 Hz
- Connection — Delta/wye
- Neutral Connection — Solidly grounded
- Quantity — 2 Station/Unit Transformers
  2 Unit Transformers

7.3.2 AC Distribution

- Voltage — 600 VAC, 3 phase
- Station Service Board — 600 VAC, 3 phase
  Normal supply to each section from station/unit transformer
  Interconnection between section
- Essential Service Board — 600 VAC, 3 phase
  Double section (physically separated)
  Normal supply to each section from station service board
  Interconnection between section
  Emergency diesel power to either board via automatic transfer switch

- Emergency Power — One 600 VAC, 3 phase, 500 kW diesel generator, automated

7.3.3 DC Battery and Charger

Two batteries, A battery and B battery are provided for protection and control. A third battery is provided for field flashing. The characteristics of these batteries are as follows:

- Voltage — 125 VDC
- Number Cells — 60
- Capacity — load 8 hrs.
- Minimum Capacity — 250 Ah
- Sizing — Per IEEE 485
- Quantity — 3
- Type — Sealed Lead Acid
- Mounting — Rack
- Chargers — 6 (two per battery)
- Input Voltage — 208/120 or 600

7.3.4 UPS System

Continuous output power ratings for UPS's will be determined during the detail design. Two are provided. The characteristics of these are as follows:

	Power & Lighting UPS	Protection & Control UPS
• Minimum Load	30% of rating	30% of rating
• Input Voltage	600/347	208
• Phase	3	1
• Frequency	60 Hz	60 Hz
• Load Pickup	¼ Hz	¼ Hz
• Number	1	2
• Bypass auto/manual	yes	yes
• Transfer Switch	By-pass to station service	By-pass to station service
• DC Supply	own battery	own battery
• Capacity	Load 2 hrs.	Load 2 hrs.

7.3.5 Grounding

- Design Base — ANSI/IEEE 80
- Conductor Size — Main grid 500 MCM
Interconnections 250 MCM
- Connections — 2 per major equipment (all EHV)

7.3.6 Motor Control Centres

- Voltage — 600V, 3 phase, 60 Hz
- Enclosure — EEMAC 12
- Bus — Copper
- Motor Starters — Drawout type, microprocessor controlled
- Control — Local and remote with DCS interface

7.3.7 Diesel Generator

- Capacity — 500 kW
- Voltage — 600/347
- Speed — 1,200 rpm
- Temperature — -10 ^{0}C to +40 ^{0}C
- Fuel — #2 diesel
- Generator Insulation — Class F
- Neutral Grounding — Solidly Grounded
- Power Factor — 0.80
- Day Tank — Minimum 1000 litres
- Starting — Air or Battery (individual storage)
- Control Panel — Standard with local direct and digital control for automatic start stop and interfacing to the DCS local DPU
- Operational Testing — Required at full load
- Metering — kWh, Amps, Volts, kW, PF with 4-20 mA or digital LAN interface to DPU
- Synchronizer — Yes
- Breaker — 600 V static trip with remote control

7.4 CONTROL AND TELECOMMUNICATIONS

7.4.1 Station Control System

- The station will be controlled from the station control room using a fully Distributed Control System (DCS) with customized micro computer systems distributed throughout the plant. Data Processing Units (DPU's) for data input and Programmable Logic Controller (PLC) functions for station control logic (main generator and diesel generator start-stop sequence control, headgate control, MCC transfer logic etc.) will be provided close to each controlled device. Generator control interface will be at the generator local control panels on the generator floor. The PLC functions will be provided in locked program sequences fixed and guaranteed by the supplier. User customizing, but not user re-programmable will be available. The facilities provided will be:

- Local control interfaces into the local generator control panels.

- Local control interfaces into the local standby switchgear controls.

- Local control interfaces into the auxiliary systems and diesel generator.
- Local control interfaces into the gate and headworks control systems.
- Station server(s) for data archiving and historical data retrieval.
- Duplicate MMI display (twin screen) and control consoles for operations from the station control room.
- Station logging printers (2).
- Station archive data transfer unit (CD ROM writer).
- Dual LAN controllers with dual LAN to all outstation DPU's and PLC's using dual Ethernet or dual Token Ring LAN's.
- Data interface to the Regional Control Center at Happy Valley-Goose Bay via dual data channel one over the Microwave MUX the other over the Fiber Optic MUX. Facilities to extend the system to a remote terminal will also be provided. This will either be a full RTU local emulation or a data interface into a Communications controller, compatible with the Regional Control Centre systems.
- The system will be open using standard hardware and operating systems (Unix and X Windows or Windows NT).

7.4.2 Network

- The fiber optic cables will be terminated in Optic Fiber Distribution Frames (ODF's) and routed from there to the protection, control and communications multiplexer systems as appropriate. The trunk cables to the overhead lines will be 24 fiber cables to match the overhead line fiber design in heavy duty sheaths (kevlar or equivalent) and individual plastic conduits up to the junction boxes on the transmission towers.
- Local fibres will be 4 fiber pairs in heavy duty sheaths and installed in plastic conduits throughout the station to the various local panels.
- Mechanical splices will be used in the station while fusion splices will be used in the overhead trasmission line junction boxes.
- While fiber optic LAN connections are preferred within the powerhouse and switchyard short lengths of suitably protected twisted pair may be used to interconnect local systems.
- An independent dial up network will be provided to pick up engineering data from the protective relays and from the digital metering systems. This will be by modem links as the data will be required by the Engineering office and not the control rooms.

7.4.3 Telephone Communications

- Fixed telephones with priority dialing to the Regional Control Centre, Churchill Falls, and Hydro Quebec will be provided in the station control room. Fixed telephone instruments will also be installed at each auxiliary system panel and adjacent to each local generator control panel and in the switchyard control and relay panel area.

7.4.4 Plant Internal Communication

- Consideration will be given to microcell type private cell phone transmitters in the station to allow for operator roaming in a normally unmanned station.

7.4.5 Microwave System

- Microwave system to act as a fully redundant teleprotection and supervisory communication channel

- System Design Perimeters:

Frequency:	7 GHz band.
Diversity:	Frequency (space only if required for path reliability).
Link Reliability:	99.9999%
Format:	Synchronize Optical Network (SONET) - OC3 Data Rate.

- For redundant teleprotection to the Montagnais Terminal Station. The existing Churchill Falls to Moire Analog Microwave Link must be upgraded to meet the system design perimeters.

APPENDIX C

NORTH SPUR STABILIZATION

APPENDIX C

NORTH SPUR STABILIZATION

3.0 FEASIBILITY LAYOUT OF STABILIZATION MEASURES

3.1 STABILITY ANALYSIS

The investigations showed that high piezometric levels in the Intermediate Sand unit in the spur was a major factor in the landslides which had occurred on the downstream spur slope, other factors include bank erosion and river level staging downstream due to winter ice loading which results in up to 16m of rapid drawdown on the spur downstream slopes. The design methodology adopted was to reduce these factors significantly to ensure an adequate long-term factor of safety against sliding by assessing the effects of:

- lowering the piezometric levels in the Intermediate Sand layer by a line of pumpwells;
- unloading the top of the downstream slope;
- stabilizing rockfill berms, rockfill erosion protection and downstream inclined relief drains.

Stability analyses were conducted using circular slip surfaces and strength parameters of $\varnothing' = 30^0$ and 34^0 and c'=0 corresponding to the lowest and average strength from laboratory testing for existing conditions in 1979. The increase in factor of safety was assessed for groundwater levels in the spur lowered to El. 15 and 3.5 m by well pumping, along with effects of slope unloading and providing stabilizing rockfill berms at the toe.

The results of the stability analyses for the downstream slopes are shown in Figure 7. Stability of the recent slide section and the intact downstream slope between the northern old slide and the kettle lakes outlet were analyzed. The analyses show that lowering the water table in the spur with pumpwells is the most effective stabilizing approach, followed by crest unloading and stabilizing berms. However, to provide an adequate long-term factor of safety with the reservoir impounded, a combination of stabilizing measures is necessary. It was concluded that lowering the groundwater table in the spur to at least El. 15 m and crest unloading in key areas would provide an adequate long-term factor of safety, in conjunction with rockfill erosion protection along the downstream toe and inclined relief drains to eliminate any buildup of groundwater behind the downstream slide debris which bypassed the pumpwells.

The upstream slope of the spur is relatively flat for the first 400 m north of the spur (4H:1V) and has an adequate factor of safety against sliding under reservoir drawdown loading, although erosion protection against wave action will be required. North of this section, the upstream bank of the spur steepens to about 1.5H:1V. Stability analyses (Figure 8) show that:

- a slight increase in stability occurs with reservoir impounding;

- a 20 m wide rockfill berm with crest at El. 40 m provides essentially the same factor of safety against sliding under 13 m of drawdown as existed under conditions in 1979;

- the rockfill berm will increase the factor of safety against sliding at full reservoir level by about 40%.

3.2 PUMPWELLS

The stability analyses showed that lowering the groundwater table in the spur would be the most effective method of stabilizing the downstream slope of the spur. A line of pumpwells at 6m spacing from the rock knoll to the outlet of the Kettle Lakes was selected as the most cost-effective option. The pumpwells would extend to the bottom of the sandy Intermediate Sand at about El. - 10 m and would lower the water table in the spur under operating conditions to below El. 15m. The wells would be screened full depth through sandy stratae below El. 30 m, with a buried header pipe. Automatic water level monitoring in each well which would be transmitted to a control building on the spur, as shown in Figure 9.

3.3 DOWNSTREAM SLOPE

As shown in Figures 9 and 10 secondary lines of defense would be provided which include:

- local unloading of the top of the downstream slope above oversteepened slide scarps by excavation of a 20 m wide berm at El. 45 m, and excavation of the backslope above El. 45 m to 2H:1V to minimize surface runoff erosion and promote natural revegetation;

- installation of a row of inclined 50 m long relief drains along the downstream toe of the spur with 5 cm slotted PVC pipe at 6 m spacing to drain seepage which may bypass the pumpwell screen upstream;

- installation of rockfill erosion protection along the downstream toe of the slope to prevent regression of the shoreline due to wave and current action;

- grading of the valley of the creek downstream of the lower kettle lake and installation of a 1.75 m CSP in gravel backfill to improve drainage in this area and lower the groundwater table beyond the north end of the pumpwell line.

3.4 UPSTREAM SLOPE

Protection of the upstream slopes against the effects of drawdown and wave action consists of placement of a 3 m thick rockfill blanket underlain by granular filter for the first 400 m upstream of the rock knoll for erosion protection, and 20 m wide rockfill stabilizing berm to El. 42 m against the steeper slopes further upstream.

4.0 INTERIM PUMPWELL SYSTEM

4.1 DESCRIPTION

The major slide in 1978 and minor slides in 1980-81 led to a decision in 1981 to install an interim dewatering system to protect the spur from further slides which could result in loss of the site. The system (Figure 11) consists of 22 pumpwells spaced at 30 m with an average depth of 63 m. The pumpwells are 30 cm in diameter with 15 cm diameter PVC riser pipe and slotted PVC screens with filter sand in the drillhole annulus to prevent piping of fine-grained soils into the well. Pumps are installed at about El. 0 and are Berkeley Model 4BL-21, 1.5 HP with 60 l/min capacity. Limit switches were installed for wellpump operation between El. 5 and 10 m. Pumping rates for Wells W-1 to W-7 are 56 l/min but Wells W-8 to W-22 had pumping rates decreased to 10 to 12 l/min by discharge line valve adjustment to prevent frequent starting and stopping of the pumps in these low capacity wells. The header pipe is insulated 30 cm diameter PVC buried for frost protection[2].

Seventeen Telemac vibrating wire piezometers were installed to monitor groundwater levels in the spur. A complete telemetering system was installed for transmitting pumpwell and piezometer data by radio link to Happy Valley.

The pumpwell system was installed as a temporary stabilization measure, with well spacing at 30 m instead of the 6 m planned for the permanent system. However, it was considered that the system would lower groundwater levels in the spur sufficiently to prevent further slides under present conditions. Performance of the pumpwell and its efficiency in lowering groundwater levels in the spur also provides firm data on which to optimize the design of the permanent pumpwell system.

4.2 PERFORMANCE

The pumpwell system began operation in September 1981 and operation has continued to the present. Early in 1984 the power line to Muskrat Falls was struck by lightning and the resulting spike destroyed all the vibrating wire piezometers. As a result, the last piezometer data was obtained in December 1983. Subsequent records covered pump function only until an additional 12 Casagrande piezometers were installed in September 1997.

Hydro staff carried out maintenance inspections in 1994 and 1995 in which the pumps were removed, serviced and reinstalled. In August 1996 the pumpwell system was inspected and rehabilitated[5] during which:

- pumps were removed, inspected, cleaned and reinstalled;
- sediment buildup was flushed from the bottom of the wells;
- wells were inspected with a TV video camera before and after flushing;
- water samples taken and chemical analyses conducted; and
- performance of the system was assessed.

The 1996 inspection showed the pumpwell system in reasonably good condition. Most of the wells have a low yield and most (96%) of the water is pumped from five wells. Table 1 shows daily pump operation for selected days from 1991 to 1996. The total daily hours of pump operation for the system range from 26.72 hours on 7/1/92 with seven operating pumps, to 16.22 hours on 1/5/93 with 14 operating pumps. Wells W-4, 9, 14, 15 and 19 pump most of the water, with about 80% of the water pumped from W-4 near the south end of the pumpwell line. Piezometer response to the initial two years of pumpwell operation is shown in Table 2. Response of the piezometers located about 30 m downstream of the pumpwell line in the southern part were high, in the order of 15 m. Response was in the order of 5 m in the central part of the pumpwell line and about 0.4 m in the north part of the pumpwell line.

TV Video inspection of the wells in 1996 showed them to be in generally good condition. Iron staining in the wells was generally slight to moderate, iron encrustation which may reduce efficiency was noted in 8 of the 22 wells (W-3, 4 5, 6, 17, 18, 20 and 21). Sediment buildup in the bottom part of 8 of the 22 wells (W-1, 2, 3, 5, 7, 17, 18 and 20) reflects minor migration of fines through the filter sand and well slots.

4.3 ASSESSMENT

An initial assessment of the pumpwell system in March 1982 [4] shows the pumpwell system is highly effective in lowering the groundwater table in the south portion of the pumpwell line, as shown in Figure 11. Drawdown cones from 69 hours of pumping on W-3 and 6 days of pumping from W-1 to W-4 inclusive show an asymmetric drawdown cone extending several hundred meters upstream-downstream but less than 50 m along the axis of the spur. Logging of the pumpwell drill holes provides a detailed picture of stratigraphy along the axis of the spur as shown in Figure 12.

A 1997 review [5] of the hydrogeologic performance of the spur and pumpwell operation and efficiency since 1981 is summarized in Table 3. The pumpwell system has effectively lowered piezometric levels in the narrow south and central portions of the spur and no further landsliding has occurred at the site since 1981. The drawdown at the north end of the pumpwell line has been minor (0.4 m) but it must be remembered that spacing of the interim wells is 30 m - compared to the 6 m spacing planned in the 1980 feasibility report. Pumpwell efficiency in this area with closer spacing will be higher, as discussed below.

In 1997, 12 Casagrande piezometers were installed in 7 boreholes drilled from the top of spur at locations shown in Figure 13. The 1997-98 piezometric levels indicate somewhat better drainage in the north section of the pumpwell line than discussed in the 1997 review[5].

5.0 OPTIMIZATION OF DESIGN

As discussed above, the additional information now available from pumpwell installation and operation allows refinement of the design of the stabilization measures necessary for spur stabilization after reservoir impounding. This section describes the optimized layout. Further work will be necessary in final design for refinement of details.

5.1 PUMPWELLS

The spacing of pumpwells for a required drawdown can be estimated from well formula or use of Sichart's equation [6] and graphical construction, as shown in Figure 14 for illustrative purposes. From Sichart's equation

$$R=c(H-h_w)k^{1/2}$$

where R = radius of drawdown (ft.), $H-h_w$ = drawdown (ft.) at the pumpwell, c = 1.5 from performance data for a single line of wellpoints, and k is the permeability in 10^{-4} cm/sec units, the radius of drawdown and efficiency of various well spacings can be evaluated. For effective dewatering, well spacing must be less than R to obtain cumulative drawdown effects from overlapping drawdown cones.

Figure 14 shows the drawdown cones for individual wells and cumulative drawdown for the south, central and north sections of the pumpwell line for 25 m of drawdown at the well for the existing 30 m well spacing. In the south section of the pumpwell line where drilling data indicates thick and relatively clean sand, an equivalent permeability of 1 x 10^{-3} cm/sec appears reasonable. Along the line of pumpwells the water table is drawn down to about El. 15 m halfway between the wells, resulting in a groundwater level of about El. 13 m downstream of the wells. The efficient dewatering results from overlapping of three drawdown cones at the midpoint between the wells for a radius of drawdown R of 90 m. Closing the well spacing to 15 m would result in only minor improvement in groundwater level downstream (El. 12 m) after reservoir impounding but would provide backup for loss in well efficiency with time due to iron encrustation plugging the slots and silt buildup inside the wells. The performance of W-1 to W-7 shows that spacing wells much closer than 15 m is not warranted.

The central portion of the pumpwell line (W-8 to W-17) shows less sand and more silty sand and lower equivalent permeability (1 x 10^{-4} cm/sec) is used to approximate well performance in this area. Radius of drawdown R is about 30 m but a steeper drawdown cone would develop than for the previous case, resulting in a groundwater level of about El. 31 midway between the wells and El. 28 m downstream of the line of wells.

The north portion of the pumpwell line (W-18 to W-22) shows mainly silty clay and silty sand. Taking an equivalent permeability of 1 x 10^{-5} cm/sec, resulting radius of drawdown of each well is 1 m. The predicted drawdown half way between the wells is nil and the drawdown downstream of the wells is less than 0.1 m.

The actual radius of drawdown for the three sections of the pumpwell line differ somewhat from the approximations in Figure 14 but can be estimated from response of the piezometers installed in 1979, which are located 30 m downstream of the pumpwell line. Drawdown of about 20 m in the southern zone confirms that pumpwell spacing of 15 m is conservative. In the central portion of the spur, drawdown of about 4 m shows pumpwell spacing at about 10 m will be necessary to lower water levels downstream of the pumpwell line to about El. 15 m.

The 0.5 m response in 1980-81 of the piezometers downstream of the northern section of the pumpwell line indicates a radius of drawdown of about 30 m, but the drawdown cones are very steep in this low permeability material. Hence, relatively close pumpwell spacing (6 m) will be necessary for drainage to El. 15 m downstream of the pumpwell line. The current standpipe piezometer levels in this area indicate this spacing will be adequate to achieve drawdown to El. 15 m.

The final pumpwell spacing must provide sufficient pumping capacity in the more pervious sections of the spur to provide local overloading after reservoir impounding if some of the pumpwells are temporarily out of service for maintenance. Installation of an impervious blanket from El. 19 to 42 m along the upstream face of the spur between W-1 and W-7 will limit seepage increase with reservoir impounding in the pervious south portion of the pumpwell line.

For service conditions after reservoir impounding, pumpwell spacings based on prototype performance and available field data are shown below.

Zone	Present Pumpwell Spacing	Spacing Required (m)	Reasons	No. Additional Pumpwells
South W1-W7	30m	15m	Backup and dewater to El. 13 m	6
Central W8-W17	30m	10m	To dewater to El. 15 m	18
North W18-W22	30m	6m	To dewater to El. 15 m	20
			Total	**44**

The existing pumpwell system ends about 30 m south of the ravine draining the Kettle Lakes on the north end and about 200 m north of the rock knoll on the south end of the spur. Treatment of these areas is discussed below.

5.2 NORTH AND SOUTH ENDS OF SPUR

5.2.1 North End

Appendix A contains the logs of Wells W-18 to W-22 at the north end of the pumpwell line. W-18, W-19 and W-21 penetrate zones of silty sand and in which closely spaced (6 m) pumpwells would be expected to provide dewatering. W-22 contains silty sand and sand above El. 27 m which closely spaced pumpwells should be effective. W-19 contains sand above El. 35 m. However, extension of the drainage curtain further north is not considered cost-effective in view of the limited occurrence of sand and silty sand below El. 30 m in the drillholes at the north end of the pumpwell curtain, as other stabilization measures will be more economical.

The bed of the creek draining the kettle lakes is at about El. 22 m at the extension of the pumpwell axis. Seepage from the pervious granular zones overlying low permeability sediments in W-22 (El. 31.5 m) will discharge into the creek valley upstream of the pumpwell axis. Extension of the 1980 drainage layout planned for the creek valley by:

- backhoe excavation of a drainage trench in the creekbed from the El. 15 m contour to the projection of the pumpwell axis and infill with free-draining filter sand;
- backhoe excavation of a 4m deep drainage trench infilled with free-draining filter sand on the axis of the pumpwell line

will increase drainage to the ravine and prevent significant recharge downstream. Long term stability will be augmented by additional downstream crest unloading.

5.2.2 South End

Between W-1 at the south end of the pumpwell line and the rock knoll, two drillholes exist (D2 and D3) at about 100 m spacing (Figure 11). This section of the spur narrows to its minimum width (70 m). Slopes above El. 40 m are inclined at about 1.5 H;1V. Downstream, a 100 m wide plateau at El. 40 to 35 m may be an old terrace, but the possibility of it being an old slump block cannot be eliminated at this time. Sliding at this section would cut access to the project, and could result in breach of the spur.

Available drillhole data from north to south below El. 39 m is summarized below:

Drillhole	Elevation	Description
W-1	39 - 33, 8	silt and clay
	33.8 - 24.8	fine grained sand, trace silt
	24.8 - 9.8	silty sand
	9.8 to -0.2	sandy silt
	-0.2 to -4.2	silt, trace clay
D2	39 - 32.6	alt. layers of fine silty sand and silty clay,
	32.6 - 9.1	sensitive silty clay of low to medium plasticity with

		occasional sand seams
	9.1 to -14.1	slightly sensitive brown silty clay of medium plasticity
	-14.1 to -15.5	glacial till, well graded sand, gravel and silt with occasional cobbles
	-15.6 to -17.7	bedrock, granite gneiss
D3	39. - 32.3	sensitive silty clay of low to medium plasticity with occasional sand seams
	32.3 - 18.2	granite gneiss bedrock

Logs and piezometric data for drillhole D2 are included in Appendix A. The available data shows that, between the south end of the existing pumpwell line (W-1) and drillhole D2, the Intermediate Sand Unit pinches out. Sensitive marine clay with thin fine silty sand layers extends from reservoir level (El. 39 m) to the top of the Lower Clay in D2 (El. 9 m) and bedrock in D3 (El. 32 m). Piezometric level in the silty clay is about El. 26 m from piezometer D2 1980-81 readings.

The south 200 m of the spur is composed of sensitive silty clay with occasional thin silty sand seams with index properties similar to the sensitive marine clays of the NBR complex which were intensely studied by SEBJ (Ref. 8) through the 1970's and 1980's[8]. Properties of these clays which are similar to the Muskrat Falls upper clay are:

- laboratory shear strength parameters ranging from $\varnothing' = 30°$ to 34° and $c' = 3$ to 7 kPa;
- back analysis of stable natural slopes indicating operative strength parameters of $\varnothing' = 32°$ and $c' = 7$ to 8 kPa;
- mass permeability in the order of 1×10^{-7} cm/s;
- spacing of 0.30 m sand drains in the order of 2.5 m necessary for effective excess pore pressure dissipation under dikes.

From SEBJ experience with the similar NBR clays in several instrumental testfills, it appears that 0.3 m diameter pumpwells at about 2.5 m spacing would be necessary to achieve much drainage of the south 200 m of the spur. Efficiency of the system would, however, be fairly low as the pumpwells would only partially depressurize the sand seams and a series of perched watertables downstream of the pumpwell line would result in saturation of the downstream slope, as illustrated in Figure B-1 (Appendix B).

A stability check (Figure B-2, Appendix B) shows that without drainage, the rise in piezometric level downstream of about 10 m which would follow reservoir impounding will decrease the overall sliding stability of the slope by about 20%. Local toe stability of the downstream slope from El. 26 to 10 m would decrease to about 1.08, and retrogressive failure of the toe is likely. Potential would exist for a relatively small toe failure to develop into a much longer slide due to the sensitive nature of the Upper Clay.

Flownet analysis (Figure B-3, Appendix B) shows that a drainage trench to El. 17 m infilled with free-draining granular fill will adequately drain the downstream slopes after reservoir impounding.

The considerations above resulted in the layout of stabilization measures for the south 200 m of the spur as shown in Figures 15 to 18. These include the following measures:

- Topcutting the spur to El. 45 m for a 200 m length immediately south of pumpwell W-1 to eliminate potential for local sliding in the top 10 to 15 m of the spur effecting powerhouse access, and to improve overall sliding stability;

- Excavation of a drainage trench to El. 20 m with 10 m bottom width, with a 2.5 m wide trench to El. 17 m excavated by backhoe, and infilled with free-draining granular fill from Borrow Areas GR-4 or GR-5 (Appendix C);

- Construction of a 10 m wide rockfill berm with crest at El. 10 m along the downstream toe of the north spur and placement of compacted free-draining granular fill between the rockfill berm and the downstream slope of the spur at El. 20 m, to prevent toe erosion, buttress the lower part of the downstream slope, and eliminate any potential in this area for future retrogressive slope failure.

5.3 LAYOUT OF STABILIZATION MEASURES

Figures 15 and 16 show the optimized layout for the spur stabilization. Components of the optimized layout are discussed below.

Top of Spur:

Along 75% of the length of the spur, additional pumpwells will be installed at spacing from 6 to 15 m between W-1 and W-22 to provide pumped drainage to El. 15 m. These pumpwells will utilize the existing header pipe and power supply system.

Upstream Face of the Spur:

Rockfill erosion protection and a stabilizing rockfill berm would be constructed to El. 42.0 m as shown in Sections C-C, D-D, and E-E with some refinement of the 1980 feasibility scheme. Use of tunnel muck would be substituted for the processed granular filter between the rockfill and bank below El. 35 m, as drawdown will not be a common service condition. In the section between El. 35 and 42 m, well-graded granular fill would be placed as a 4 m wide transition between the rockfill and bank material to provide effective filtration in the zone of reservoir fluctuation and wave action after impounding.

Downstream Erosion Protection and Stability Fill:

The location of the rockfill toe berm has been revised from the 1980 location to the shoreline for the first 300 m north of the rock knoll to prevent erosion of the existing toe of slumped debris. The rockfill berm forms a solid base founded on intact material for a stabilizing berm of compacted granular fill which slopes gently up from the berm to the downstream slope of the spur at El. 20 m, as shown in Sections C-C and D-D.

Toe Relief Drains:

Toe relief drains will extend for about 600 m along the downstream toe of the spur from pumpwell W1 to the north end of the spur as shown in Figure 16. Spacing of the relief drains ranges from 10 m at the south end of the relief drain line to 6 m at the north, based on efficiency of the existing pumpwell line discussed above. The orientation of the relief drains will be vertical to facilitate installation. Three of the relief drains will be extended to a depth of 80 m to penetrate the top of the Lower Aquifer and function as standpipe piezometers. In the unlikely event of a connection between the reservoir and lower aquifer, these relief drains would provide additional pressure relief to supplement the camp pumpwell until additional relief wells are installed.

Downstream Slope Unloading:

Sections of the downstream slope above high, oversteepened slopes will be unloaded by excavation of a 20 m wide berm at El. 45 m, as shown in Figure 16 and Sections D-D and H-H

on Figure 17.

South end of the Spur: Treatment of the south end of the spur will consist of:

- topcutting full width for a length of 200 m to El. 45 m, as shown in Figures 15 and 16 (Section C-C);
- excavation of a drainage trench (Figure 15) and backfill with compacted free-draining granular fill;
- placement of downstream stabilizing granular fill to El. 20 m to buttress the downstream toe of this section of the spur as discussed above.

5.4 COSTS AND SCHEDULE

Quantities for the layout shown on Figures 16 and 17 and unit costs from the 1980 feasibility report are used below to tabulate costs for the stabilization measures, for comparison with the cost of the 1980 layout.

Item / Description		Quantity	Unit Price ($1980)	Subtotal ($1980)
1. Upstream:	Rockfill Blanket	30,000 m^3	$0.75/$m^3$	$ 22,000.
	Rockfill Berm	300,000 m^3	3.51/m^3	1,052,000.
	Tunnel Muck	110,000 m^3	3.51/m^3	386,000.
	Impervious Blanket	34,800 m^3	7.99/m^3	272,000.
				1,733,000.
2. Top of Spur:	Pumpwells	44 nos.	$29,975/ea	1,319,000.
3. Downstream:	Rockfill Toe Berm	200,000 m^3	$3.51/$m^3$	702,000.
	El. 25 Berm	8,000 m^3	3.51/m^3	28,000.
	Granular Fill	120,000 m^3	11.11/m^3	1,333,000.
	Overburden Excavation	190,000 m^3	5.64/m^3	72,000.
	CSP Header 0.6 m dia.	360 m	200/m^3	72,000
	Relief Drains	100 nos.	6,780.00	678,000.
				3,885,000.
4. South end of Spur:	Excavation El. 60-45 m	560,000 m^3	$5.64/$m^3$	84,000.
	Excavation El. 45-20 m	181,00 m^3	7.00/m^3	1,267,000.
	Excavation El. 20-17 m.	1,320 m^3	12.00/m^3	16,000.
	Granular Fill Class 2	177,000 m^3	11.11/m^3	1,966,000.
	Erosion Protection Class 4B	5,000 m^3	5.00/m^3	25,000.
	Manhole/Offtake Pipe	L.S		15,000.
				6,447,000.
5. North end of Spur:	Overburden Excavation	7,000 m^3	$12.00/$m^3$	84,000.
	Granular Fill Class 2	28,000 m^3	11.11/m^3	311,000.
	1.7 m dia. CSP	330 m	399/m	132,000.
	Erosion Protection Class 4B	5,000 m^3	5.00/m^3	25,000.
				522,000.
			TOTAL	**$13,936,000.**
			TOTAL 1980 REPORT	**$13,721,500.**
			INCREASE	**$ 214,500.**

Figure 18 showing layout, quantities and escalated costs in 1998 dollars for the earlier studies on stabilization of the spur is of interest in evaluating the layout presented above. The initial 1965 study of the site by Acres Canadian Bechtel established the presence of sensitive marine clays but, with only limited drilling and testing data from one drillhole on the spur, extremely weak marine clay with sensitivities up to 400 were assumed. A conservative (and costly) layout resulted, based on topcutting to El. 50 m of the entire spur, stabilizing fill downstream, and an impervious blanket and stabilizing granular berm upstream resulting in a cost in the order of $110 million - due mainly to the large volume of granular fill downstream necessitated by the deep riverbed scour hole. Cost of stabilization of the spur would have been in the order of 11% of the project direct costs.

The 1978 Acres layout allowed a significant reduction in stabilization cost following additional field examination of slopes along the river, testpits, and additional testing which showed the marine clays be appreciatively stronger than assumed in 1965, with a cementation in the order of 1500 psf, which allowed a significant reduction in volume of downstream stabilizing fill. Cost of the stabilization works reduced to about 4% of the project direct costs.

The 1980 SNC-Lavalin layout utilizing pumped drainage resulted in a further reduction in stabilization costs to about 2% of the project direct costs, reflecting a better understanding of properties of the soils forming the spur and groundwater conditions.

The cost of the 1998 layout described above is, for practical purposes, the same as the 1980 estimate. Some changes in final design can be expected with refinement of the design, but it is anticipated that the final cost will be within 10% of the 1980 estimate.

No effect on schedule is anticipated as the spur stabilization measures are not on the critical path.

No effect on construction access is anticipated as staged excavation of the spur to El. 45 m between the south end of the existing pumpwell line and the rock knoll is possible.

6.0 FUTURE WORK

6.1 INVESTIGATIONS

Additional field investigations will be required to provide more detailed data on soil properties and groundwater conditions at the north and south ends of the existing pumpwell line for final design of seepage control measures at the ends of the pumpwell line. Nine drillholes will be required with continuous Shelby tube sampling, falling-head permeability tests at 1.6 m, spacing or changes in stratigraphy, and geophysical logging to:

- better define soil conditions, insitu permeability and groundwater conditions at the north and south ends of the existing pumpwell line where drillhole data is presently sparse;

- provide information on stability of the El. 40-35 m plateau immediately downstream of the south 200 m of the spur.

Casagrande piezometers will be installed in these drillholes after completion and falling-head tests conducted to define the permeability and continuity of pervious and semi-pervious members as well as piezometric levels. These piezometers would be used for long-term monitoring of the performance of the pumpwell system.

In two of the drillholes, vane shear testing and piston sampling of the silty clay in the south 200 m of the spur will be necessary to allow laboratory testing on undisturbed samples. Pneumatic piezometers will be installed to provide more detailed information on piezometric levels in the Upper Clay for excavation design and more refined analysis, of long-term stability. The piezometers would be used for long-term performance monitoring.

6.2 LABORATORY TESTING

As discussed above, the index properties of the Muskrat Falls Upper Clay are similar to the NBR marine clays which have been closely studied by SEBJ. The test data on these clays indicates the 1979 strength testing at Muskrat Falls was affected by sample disturbance to some extent. Strength testing on undisturbed (block or piston samples) will be necessary to refine strength parameters for final design.

7.0 REFERENCES

(1) SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kV Transmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

(2) SNC Lavalin Newfoundland Ltd. (1982) Muskrat Falls Dewatering System Construction Report Operation and Maintenance Information, March Report No. 1199.18.

(3) Acres International Ltd. (1998) Muskrat Falls Standpipe Piezometer Installation Program, Report, February, P11759.02

(4) SNC Lavalin Newfoundland Ltd. (1982) Muskrat Falls Dewatering System Engineering Assessment March, Report, Report No. 1199.18

(5) Acres International Ltd. (1997) Muskrat Falls Hydroelectric Project Dewatering System. Assessment and Rehabilitation. Final Report, Feb. P11759.01

(6) Mansur, C.I. and Kaufman, R.I. (1962) "Dewatering" in Foundation Engineering ed. Leonards, G.A.. Mc Graw Hill, New York, pp 241-350.

(7) Fell, R., MacGregor, P., and Stapleton, P. (1992) Geotechnical Engineering of Embankment Dams, Balkema, Rotterdam, p.267.

(8) SEBJ (1983) - Final Report - Committee of Specialists on the Sensitive Clays of the NBR Complex, August, 162 pp.

TABLES

Table 1

Duration of Pumping From Wells

Well	Daily Pump Operation (Hours)					
	Jul 20/91	Jan 7/92	Dec 17/92	May 1/93	Jun 27/94	Jul 15/96
W-1	No Pump					
W-2	Pump Disconnected					
W-3	-	-	-	-	-	-
W-4	20.07	23.47	23.33	14.35	17.22	16.00
W-5	-	-	-	-	-	-
W-6	-	-	-	0.02	-	-
W-7	0.05	0.45	0.18	-	-	-
W-8	0.02	-	-	0.01	-	0.02
W-9	1.12	1.00	0.49	0.38	0.38	0.87
W-10	-	-	0.01	-	-	0.22
W-11	0.10	-	0.03	0.03	-	0.02
W-12	0.17	0.12	0.06	0.06	0.06	0.08
W-13	0.10	0.12	0.05	0.06	0.07	0.23
W-14	0.77	0.90	0.43	0.47	0.44	0.18
W-15	0.48	0.48	0.27	0.28	0.28	0.08
W-16	0.20	-	0.10	0.10	0.11	0.25
W-17	-	-	0.03	0.04	0.04	0.07
W-18	0.07	0.18	0.05	0.05	0.04	0.10
W-19	1.37	-	0.54	0.26	0.55	0.40
W-20	-	-	-	-	-	0.02
W-21	0.07	-	0.04	0.04	0.04	0.08
W-22	0.13	-	0.07	0.07	0.07	0.17

• Dates selected at random.

Reference: Table 7/Acres International Ltd. (1997) Muskrat Falls Hydroelectric Project Dewatering System. Assessment and Rehabilitation. Final Report, Feb. P11759.01

Table 2

Summary of Water Level Measurements in Piezometers
(m)

Piezo-meter	Tip Elevation	Water Elevation			Drawdown	
		Sep 16, 1981, Prior to Pumping	End of Oct 1982	End of Nov 1983	After 13 Months	After 27 Months
P-1	-10.35	20.10	19.80	18.90	0.30	1.20
P-2	-0.93	24.95	12.50	11.10	12.45	13.85
P-3	0.07	30.84	19.65	18.40	11.19	12.44
P-4	1.47	28.91	* 11.00	-	** 17.91	-
P-5	1.03	28.17	11.30	9.20	16.87	18.97
P-6	-2.64	26.57	22.65	21.80	3.92	4.77
P-7	-2.70	22.80	20.00	19.45	2.80	3.35
P-8	-0.94	22.56	17.65	17.00	4.91	5.56
P-9	-3.26	14.15	13.50	13.75	0.65	0.40
P-10	-4.61	11.39	11.25	11.00	0.14	0.39
P-11	-0.15	10.69	10.50	10.20	0.19	0.49
P-12	-6.27	9.40	9.65	9.70	-0.25	-0.30
P-13	15.27	18.93	17.60	17.55	1.33	1.38
P-14	25.30	26.40	26.30	26.10	0.10	0.30
P-15	-0.34	28.59	9.45	8.90	19.14	19.69
P-16	14.37	28.65	20.55	19.85	8.10	8.80
P-17	24.75	29.03	21.80	20.95	7.23	8.08

* June 1982

** 9 months

Reference: Table 6/ Acres International Ltd. (1997) Muskrat Falls Hydroelectric Project Dewatering System. Assessment and Rehabilitation. Final Report, Feb. P11759.01

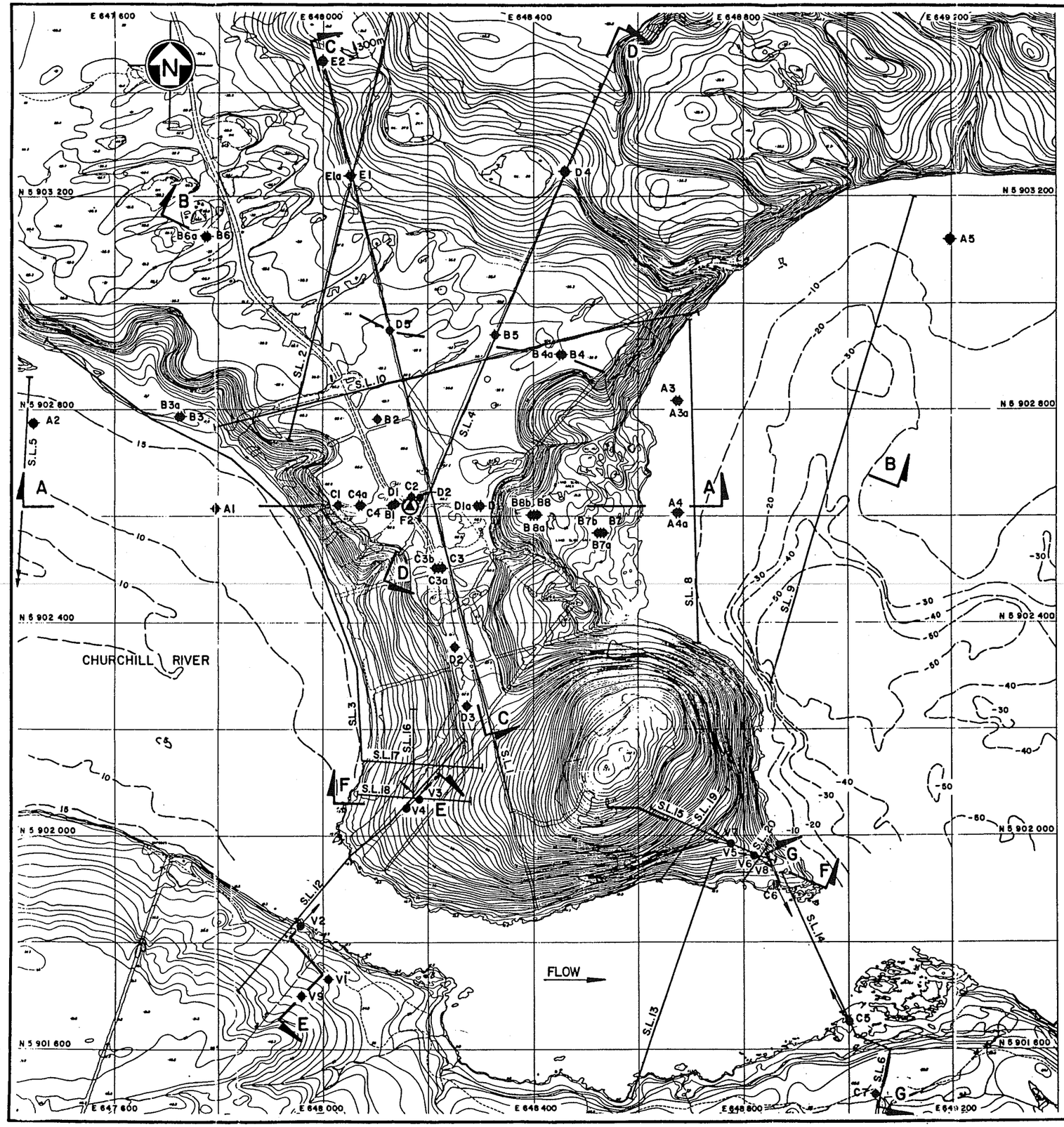
Table 3

Summary of Hydrogeological Observations

Observations	Southern Zone	Central Zone	Northern Zone
Pump Wells in Zone	**W-1 to W-7**	**W-8 to W-17**	**W-18 to W-22**
Proportion of pervious sand and sandy silt layers in stratified drift unit to el 10 m	55%	20%, except for Wells W-4 and W-14 which are similar to southern zone.	5%
Relative bulk hydraulic conductivity (m/s)	1×10^{-5}	1×10^{-7} to 1×10^{-8}	1×10^{-8}
Groundwater lowering in 2 years after start of pumping (m)	High of about 15	Low <5.5	Very low <0.5
Daily operation of pumps	Long, due to steady inflow from pervious layers; pump in W-4 operates 19 hours	Short, generally 1 hour; pumps W-9 and W-14 work longer due to greater pervious thickness	Short, maximum of 1 hour; pumps operate daily
Recovery in water level after 10 days of pumps shut down in 1996	Very small <2 m	High, about 35 m in Well W-13 equivalent to prepumping level. Steady state reached in about a week	Moderate to about 10 m as in Wells W-21 and W-22 to el 20, equivalent to invert of nearby stream from Kettle Lakes
Filling of well casing using 2000 gal of water for flushing	Difficult, unable to cause sediments in Wells W-3, W-4, W-5 and W-7 to rise above top of casing	Easy in all wells except W-9	Easy in all wells
Benefit of continued pumping in groundwater lowering	High	Moderate	Negligible

Reference: Table 9/ Acres International Ltd. (1997) Muskrat Falls Hydroelectric Project Dewatering System. Assessment and Rehabilitation. Final Report, Feb. P11759.01

FIGURES



LEGEND:

- B4a VERTICAL BOREHOLE (B4a)
- C6 INCLINED BOREHOLE (C6)
- F2 PUMPING WELL (F2)
- S.L. 2 SEISMIC LINE (Nº 2)

NOTES:

1) FOR SECTIONS "A-A" & "B-B" SEE PLATE 8
2) FOR SECTIONS "C-C" & "D-D" SEE PLATE 9
3) FOR SECTIONS "E-E","F-F" &"G-G" SEE PLATE 10
4) FOR RESULTS OF SEISMIC SURVEYS SEE VOLUME II APPENDIX 3

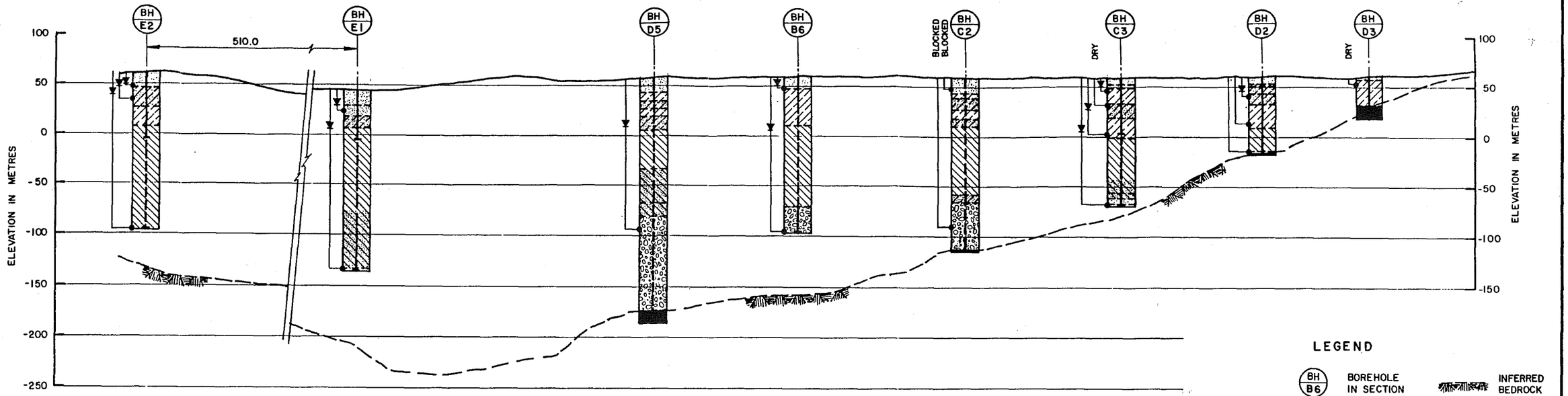
0 100 200 m

Reference: Volume 1, Plate 7/SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kVTransmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

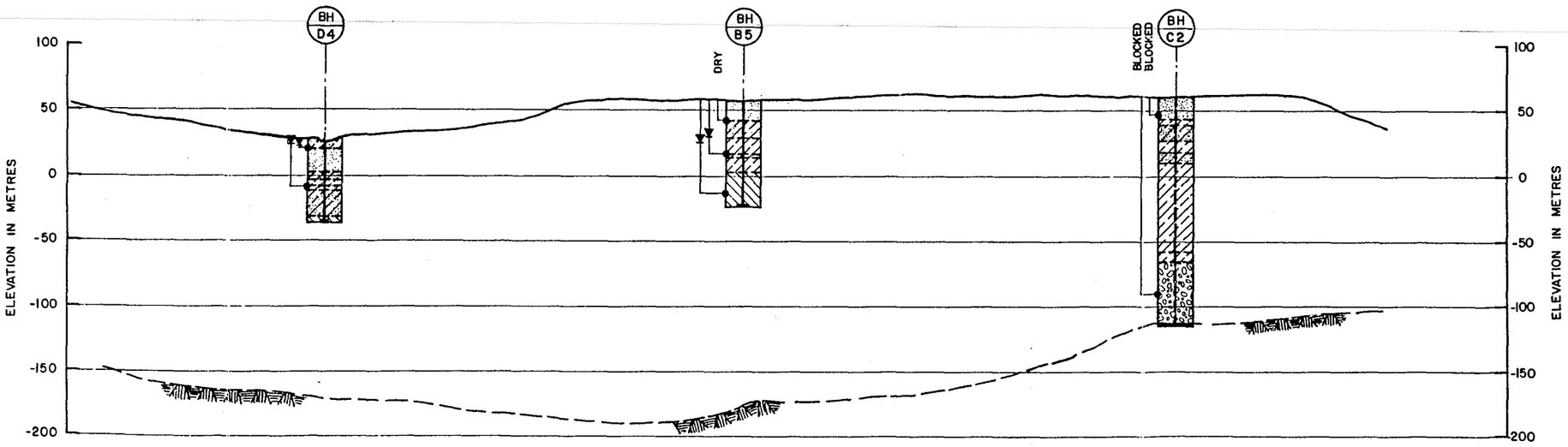
NOTE
ABOVE WATER CONTOURS ARE BASED ON GSC DATUM PRE 1978
TO OBTAIN CORRECTED GEODETIC DATUM (MSL) SUBTRACT 0.97m
BATHYMETRIC CONTOURS ARE ADJUSTED TO THE 1979 CORRECTED DATUM (MSL).

FIELD EXPLORATION
SUMMER 1979
LOCATION PLAN

Figure 1



SECTION C-C



SECTION D-D

LEGEND

BH B6 BOREHOLE IN SECTION

INFERRED BEDROCK

OBSERVED STRATIGRAPHY

INFERRED STRATIGRAPHY

PIEZOMETER LOCATION AND WL ON NOV. 1/79

SOIL STRATIGRAPHY

UPPER SAND
COMPACT TO VERY DENSE GREYISH BROWN FINE TO MEDIUM SAND

UPPER CLAY
SENSITIVE FIRM TO VERY STIFF BROWN SILTY CLAY OF LOW TO MEDIUM PLASTICITY WITH OCCASIONAL SAND SEAMS

INTERMEDIATE SAND
VERY DENSE GREY FINE SILTY SAND

LOWER CLAY
SLIGHTLY SENSITIVE STIFF TO VERY STIFF SILTY CLAY OF MEDIUM PLASTICITY CHANGING PROGRESSIVELY TO CLAYEY SILT, SILT AND SANDY SILT WITH DEPTH

LOWER AQUIFER
SILTY SAND/SAND WITH SOME GRAVEL AND OCCASIONAL COBBLES AND BOULDERS (VARIABLE)

GRANITE / GNEISS BEDROCK
GRANITE GNEISS

0 20 60 100m

Reference: Volume 1, Plate 9/ SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kVTransmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

NOTES :

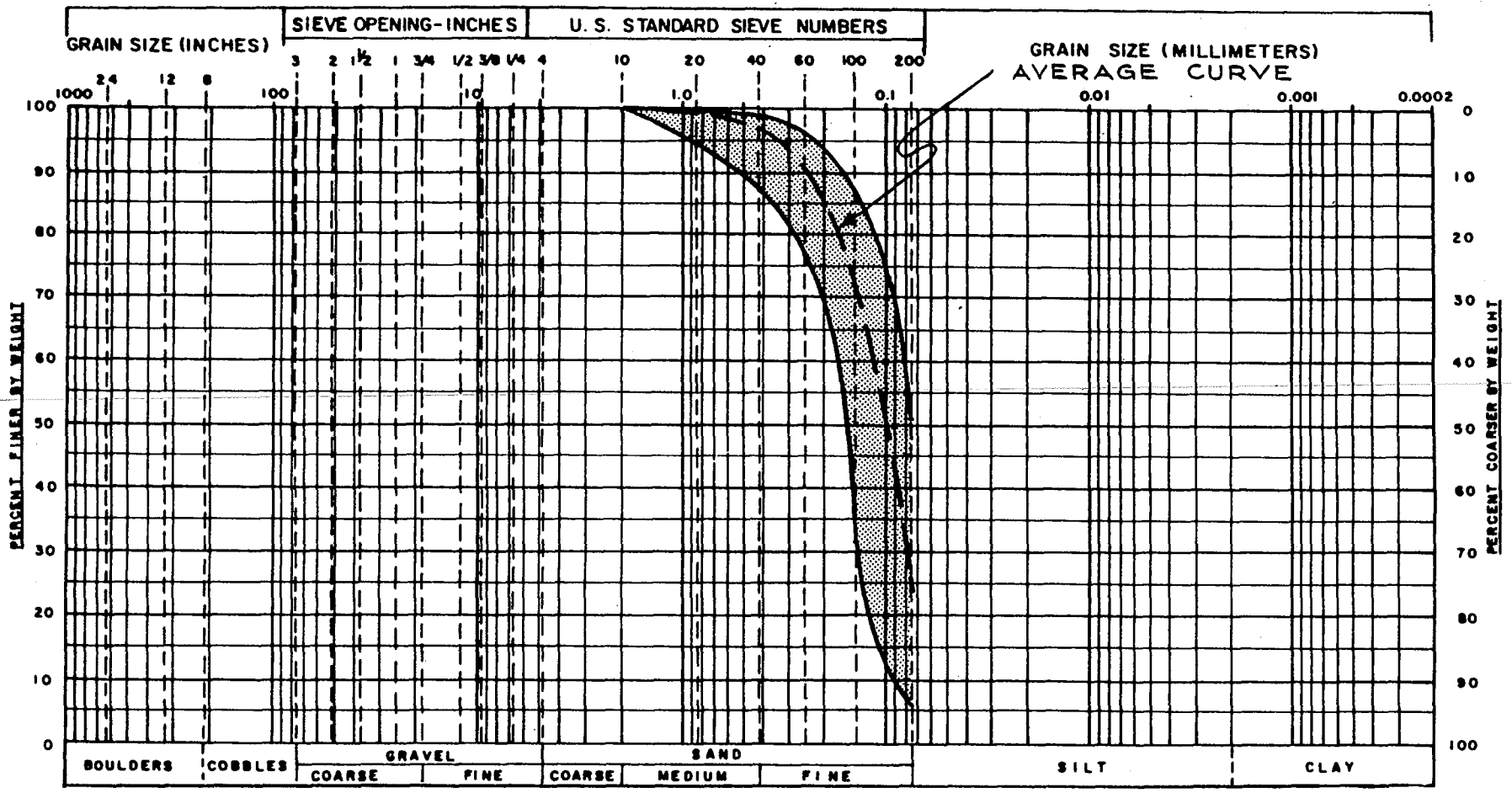
1 FOR LOCATION OF SECTIONS C-C AND D-D SEE PLATE NO. 7
2 GROUND SURFACE PROFILE BASED ON CORRECTED GEODETIC DATUM (MSL) 1979
3 APPROXIMATE BEDROCK PROFILE DERIVED FROM SEISMIC LINE RESULTS
4 FOR DETAILED SOIL STRATIGRAPHY SEE INDIVIDUAL TEST HOLE RECORDS IN VOLUME III APPENDIX A

SOIL STRATIGRAPHY SECTIONS C.C. & D.D.

Figure 2

GRAIN – SIZE DISTRIBUTION

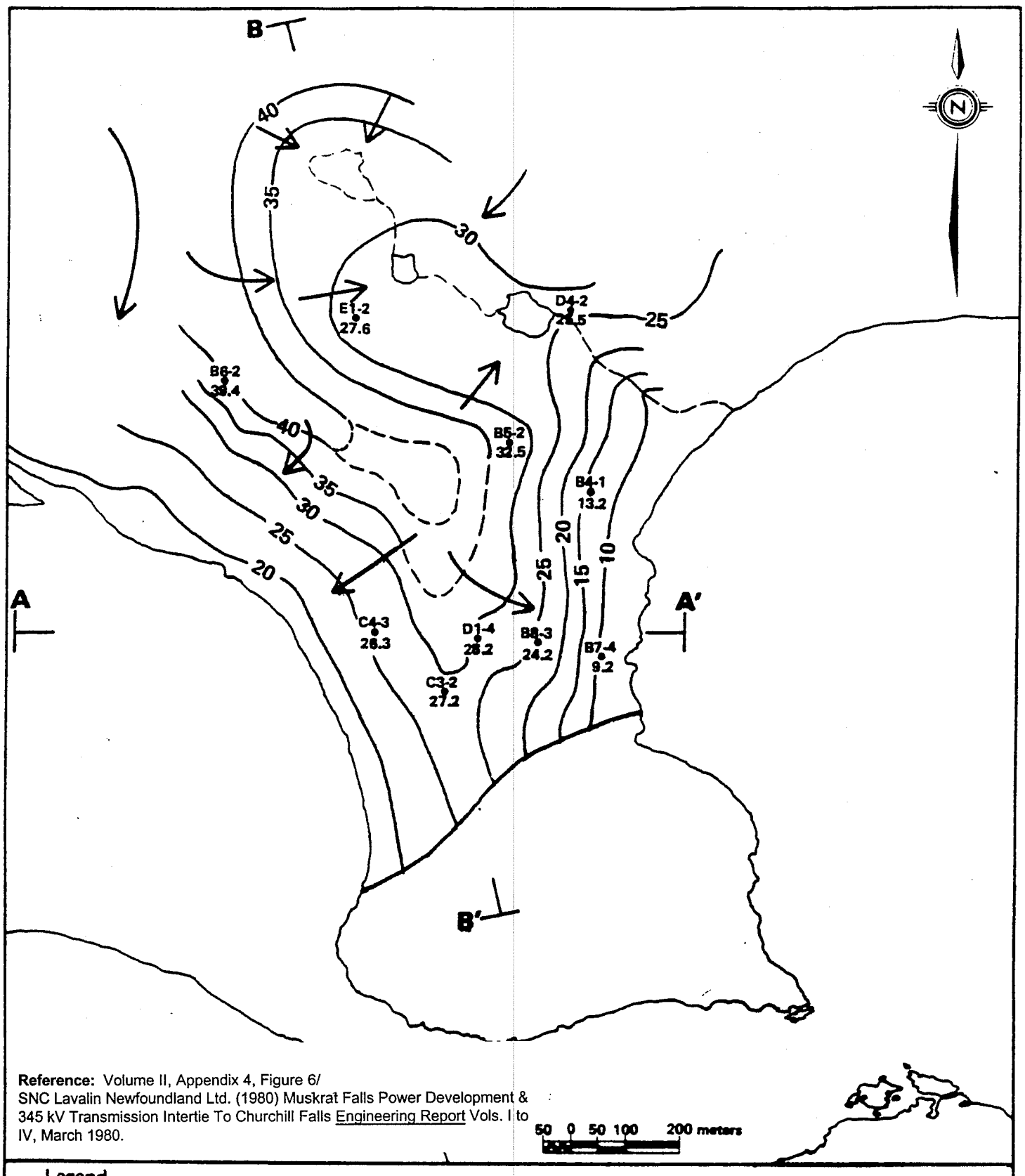
MUSKRAT FALLS



INTERMEDIATE SAND LAYER

Reference: Volume II, Appendix 2, Figure 2.2/ SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kVTransmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

Figure 3



Reference: Volume II, Appendix 4, Figure 6/
SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kV Transmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

Legend

—10— Piezometric contours and elevations

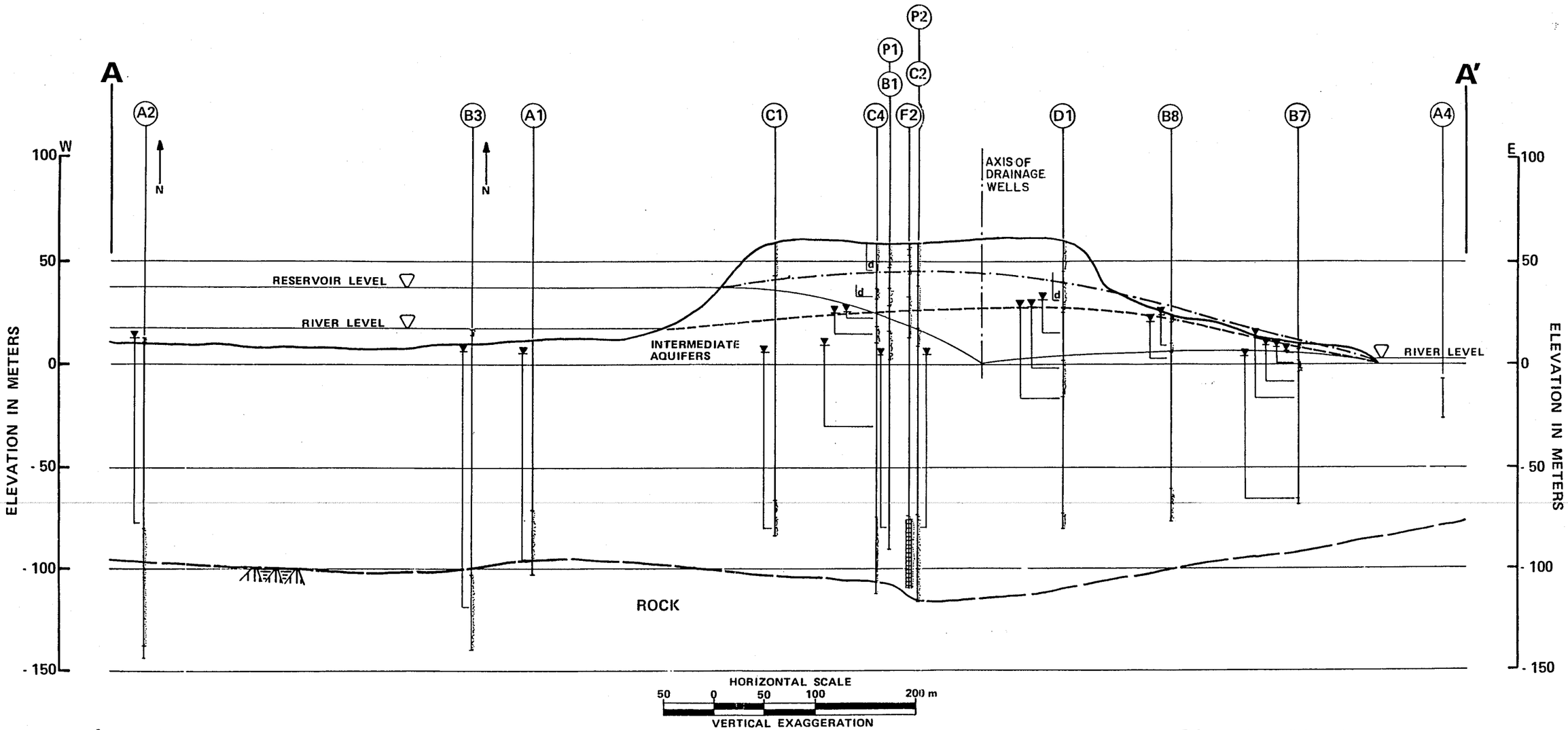
C4-3 26.3 Location point of piezometers and wells, with identification number / Piezometric levels (elevations in meters above sea level)

A — A' Location of cross sections

Direction of groundwater flow

1979 EXPLORATION PROGRAM
PIEZOMETRIC SURFACE
OF THE INTERMEDIATE AQUIFER

Figure 4



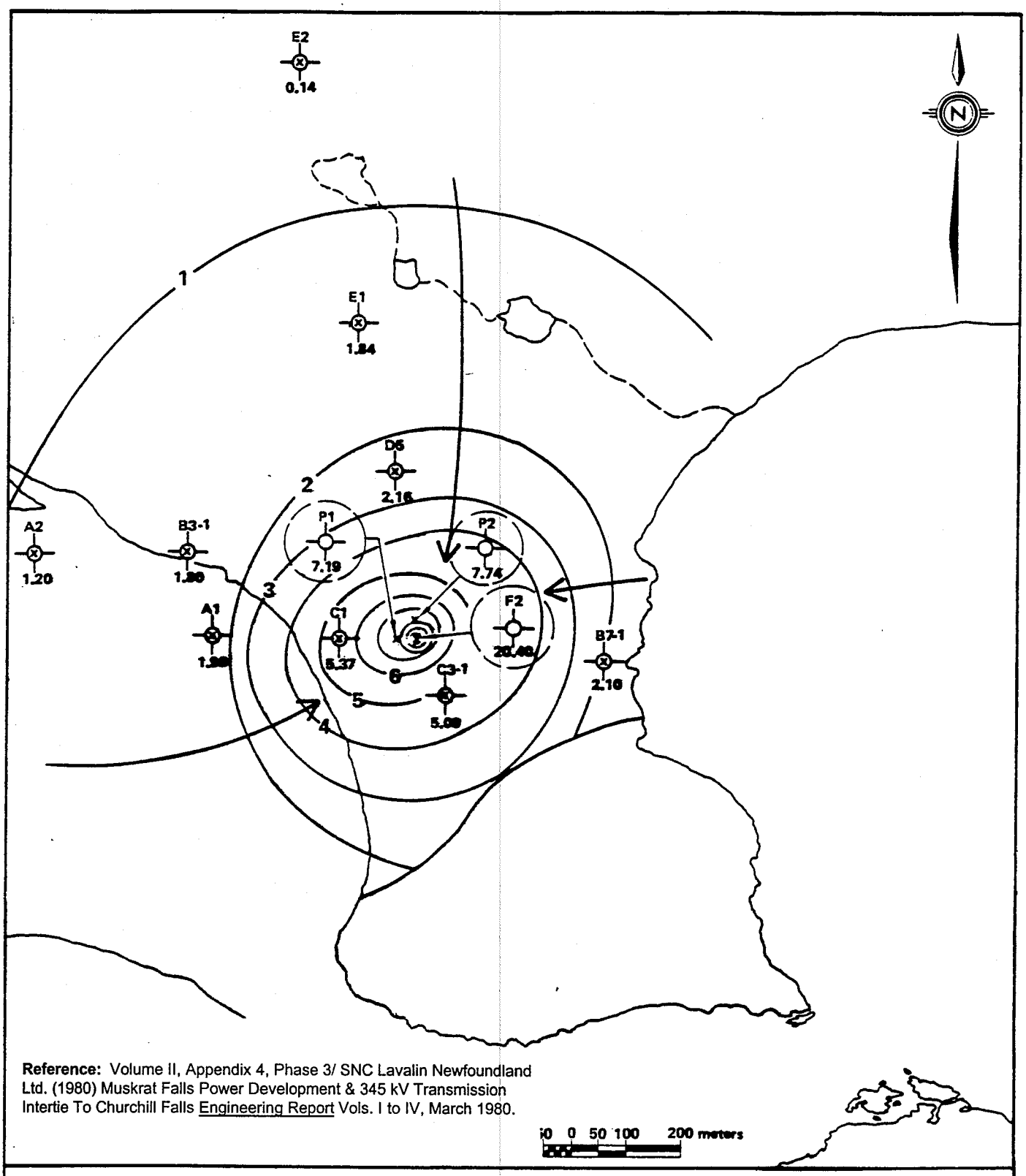
Legend

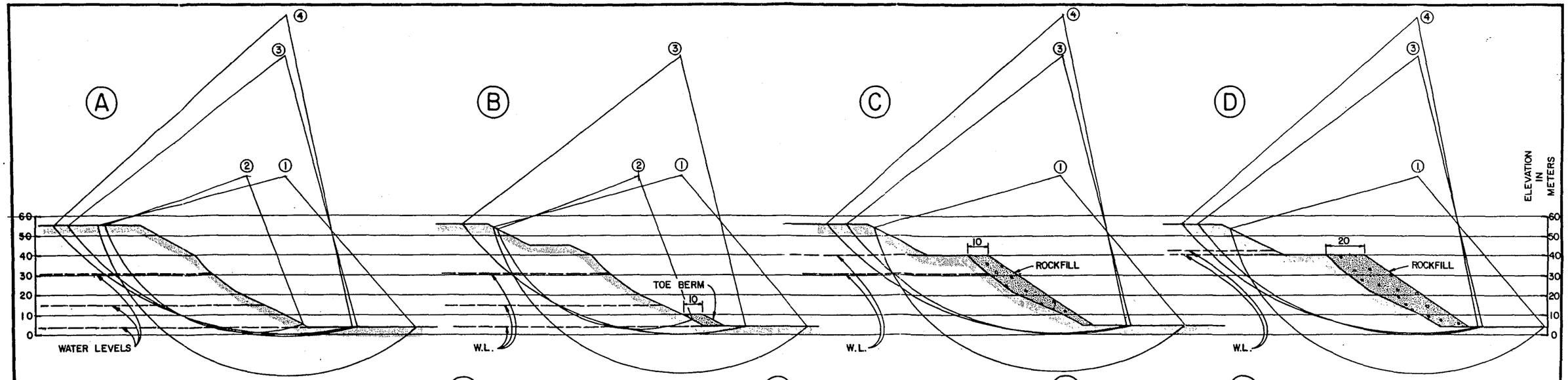
- C1 — Location and identification of piezometers
- Piezometric level (d) if dry
- Depth of piezometer
- Well screen
- Aquifer zone
- Approximate geological contact
- Water levels used were taken the 23 and 24 october 1979
- Piezometric surface of the Intermediate Aquifers under present conditions
- Expected piezometric surface of the Intermediate Aquifers after 20 m rise of the reservoir (without drainage)
- Probable shape of the cone of depression after extensive drainage

Reference: Volume II, Appendix 4, Figure 7/ SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kV Transmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

HYDROGEOLOGICAL CROSS SECTION AA INTERMEDIATE AQUIFERS

Figure 5


E2
0.14
N
1
E1
1.84
D6
2.16
2
P1
7.19
P2
7.74
A2
1.20
B3-1
1.80
3
A1
1.88
C1
5.37
F2
20.48
B7-1
2.16
6
C3-1
5.09
5
4
Reference: Volume II, Appendix 4, Phase 3/ SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kV Transmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.
0 50 100 200 meters
Legend
2
Drawdown contours in meters
B3-1
1.80
Location and identification of the piezometers and of the pumping well F2
Drawdown after 5,000 minutes of pumping (oct. 10–16 , 1979)
Direction of groundwater flow
1979 EXPLORATION PROGRAM
DRAWDOWN CONTOURS FOR
THE LOWER AQUIFER PUMP TEST
Figure 6



A

STRENGTH PARAMETERS		ACTUAL PROFILE WL AT 31M SLIP SURFACES				ACTUAL PROFILE WL AT 15 SLIP SURFACES			ACTUAL PROFILE WL AT 3.5 SLIP SURFACES		
C'	ϕ'	①	②	③	④	①	②	③	①	②	③
0	30	1.15	0.93	0.96	1.05	1.35	1.22	1.22	1.55	1.47	1.41
0	34	1.34	1.09	1.12	1.22	1.58	1.42	1.42	1.81	1.71	1.65

B

STRENGTH PARAMETERS		FINAL PROFILE WL AT 31M SLIP SURFACES			FINAL PROFILE WL AT 15M SLIP SURFACES			FINAL PROFILE WL AT 3.5 SLIP SURFACES		
C'	ϕ'	①	②	③	①	②	③	①	②	③
0	30	1.27	1.14	1.04	1.50	1.47	1.33	1.72	1.76	1.54
0	34	1.48	1.30	1.22	1.75	1.69	1.55	2.02	2.03	1.80

C

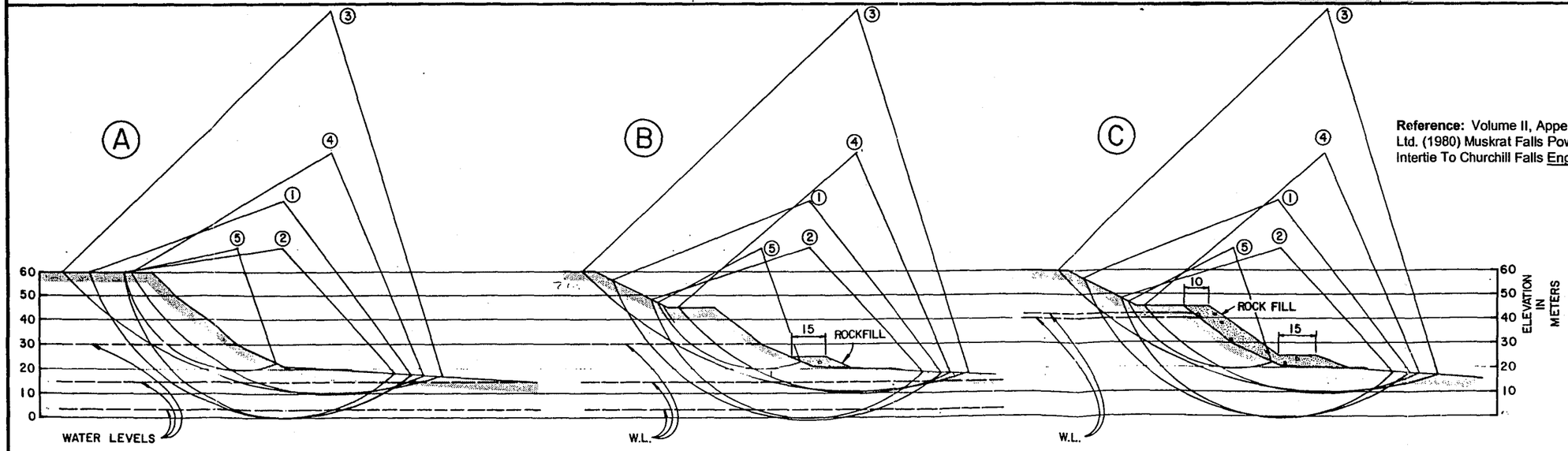
STRENGTH PARAMETERS		PROPOSED SECTION ,10M ROCKFILL WL AT 31M SLIP SURFACES				PROPOSED SECTION ,10M ROCKFILL WL AT 40M SLIP SURFACES			
C'	ϕ'	①		③	④	①		③	④
0	30	1.37		1.23	1.28	1.27		1.08	1.12
0	34	1.61		1.43	1.50	1.48		1.26	1.31

D

STRENGTH PARAMETERS		PROPOSED SECTION ,20M ROCKFILL WL AT 40M SLIP SURFACES				PROPOSED SECTION ,20M ROCKFILL WL AT 42M SLIP SURFACES			
C'	ϕ'	①		③	④	①		③	④
0	30	1.46		1.37	1.39	1.45		1.35	1.37
0	34	1.71		1.60	1.63	1.69		1.58	1.60

COMPARATIVE FACTORS OF SAFETY

SECTION NORTH OF THE OLD SLIDE



A

STRENGTH PARAMETERS		ACTUAL SECTION WL AT 30M SLIP SURFACES					ACTUAL SECTION WL AT 15 SLIP SURFACES					ACTUAL SECTION WL AT 3.5 SLIP SURFACES				
C'	ϕ'	①	②	③	④	⑤	①	②	③	④	⑤	①	②	③	④	⑤
0	30°	1.3	1.3	1.28	1.12	1.02	1.68	1.72	1.64	1.58	1.28	2.00	2.09	1.76	1.75	1.28
0	34°	1.53	1.53	1.50	1.31	1.20	1.96	2.01	1.92	1.85	1.49	2.33	2.44	2.06	2.04	1.49

B

STRENGTH PARAMETERS		FINAL SECTION WL AT 30° SLIP SURFACES					FINAL SECTION WL AT 15 SLIP SURFACES					FINAL SECTION WL AT 3.5 SLIP SURFACES				
C'	ϕ'	①	②	③	④	⑤	①	②	③	④	⑤	①	②	③	④	⑤
0	30°	1.60	1.66	1.46	1.40	1.39	2.18	2.32	2.00	2.16	1.88	2.61	2.82	2.15	2.38	1.88
0	34°	1.87	1.94	1.70	1.64	1.62	2.55	2.71	2.33	2.52	2.18	3.05	3.30	2.52	2.73	2.18

C

STRENGTH PARAMETERS		PROPOSED SECTION WL AT 40M SLIP SURFACES					PROPOSED SECTION WL AT 42M SLIP SURFACES				
C'	ϕ'	①	②	③	④	⑤	①	②	③	④	⑤
0	30°	1.61	1.67	1.44	1.38	1.89	1.57	1.63	1.40	1.35	1.81
0	34°	1.88	1.95	1.69	1.62	2.18	1.84	1.91	1.64	1.58	2.08

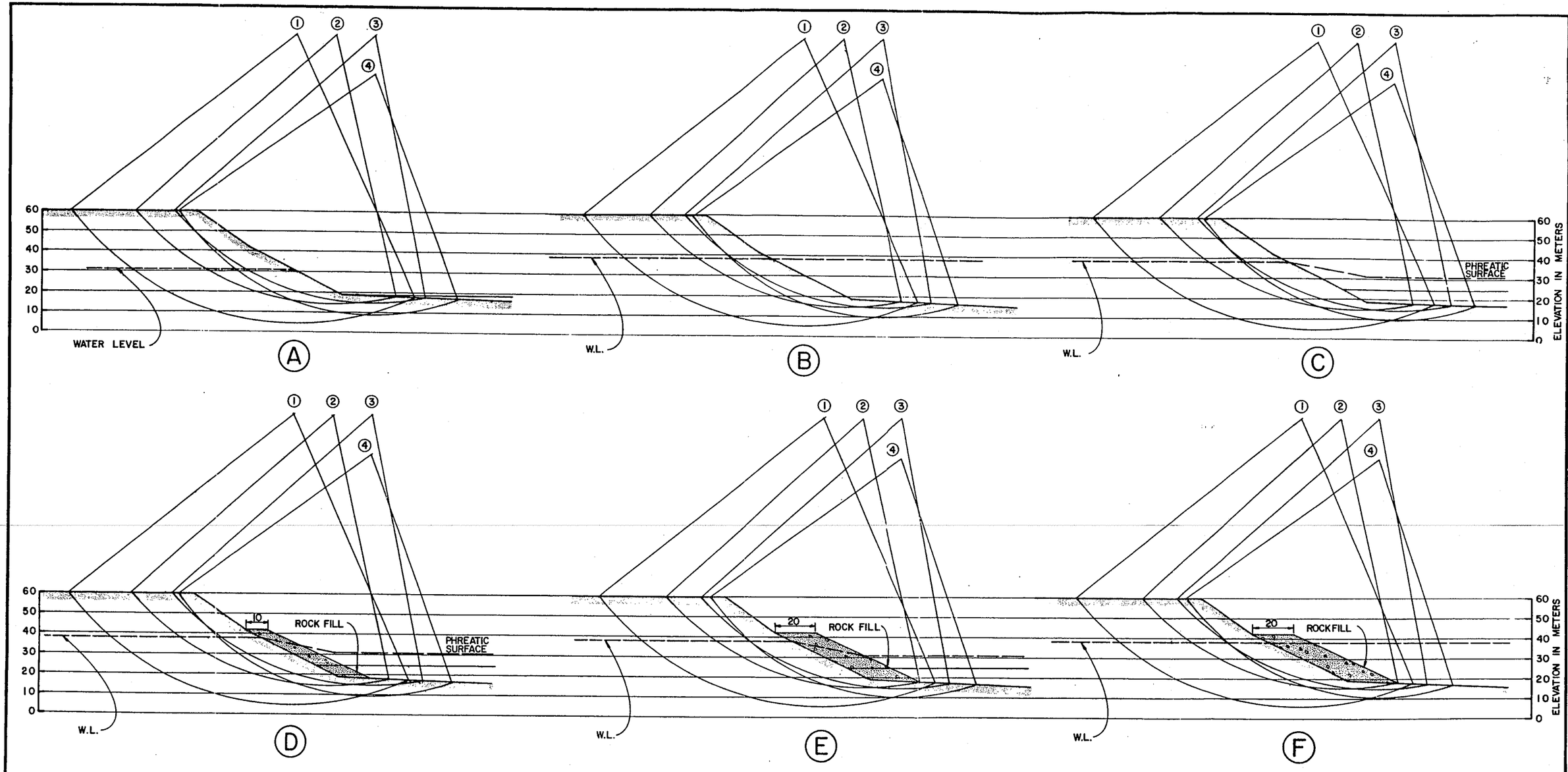
COMPARATIVE FACTORS OF SAFETY

SECTION IN THE RECENT SLIDE

Reference: Volume II, Appendix II, Plate 19/ SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kV Transmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

DOWNSTREAM SLOPE OF NORTH SPUR STABILIZATION ANALYSIS

Figure 7



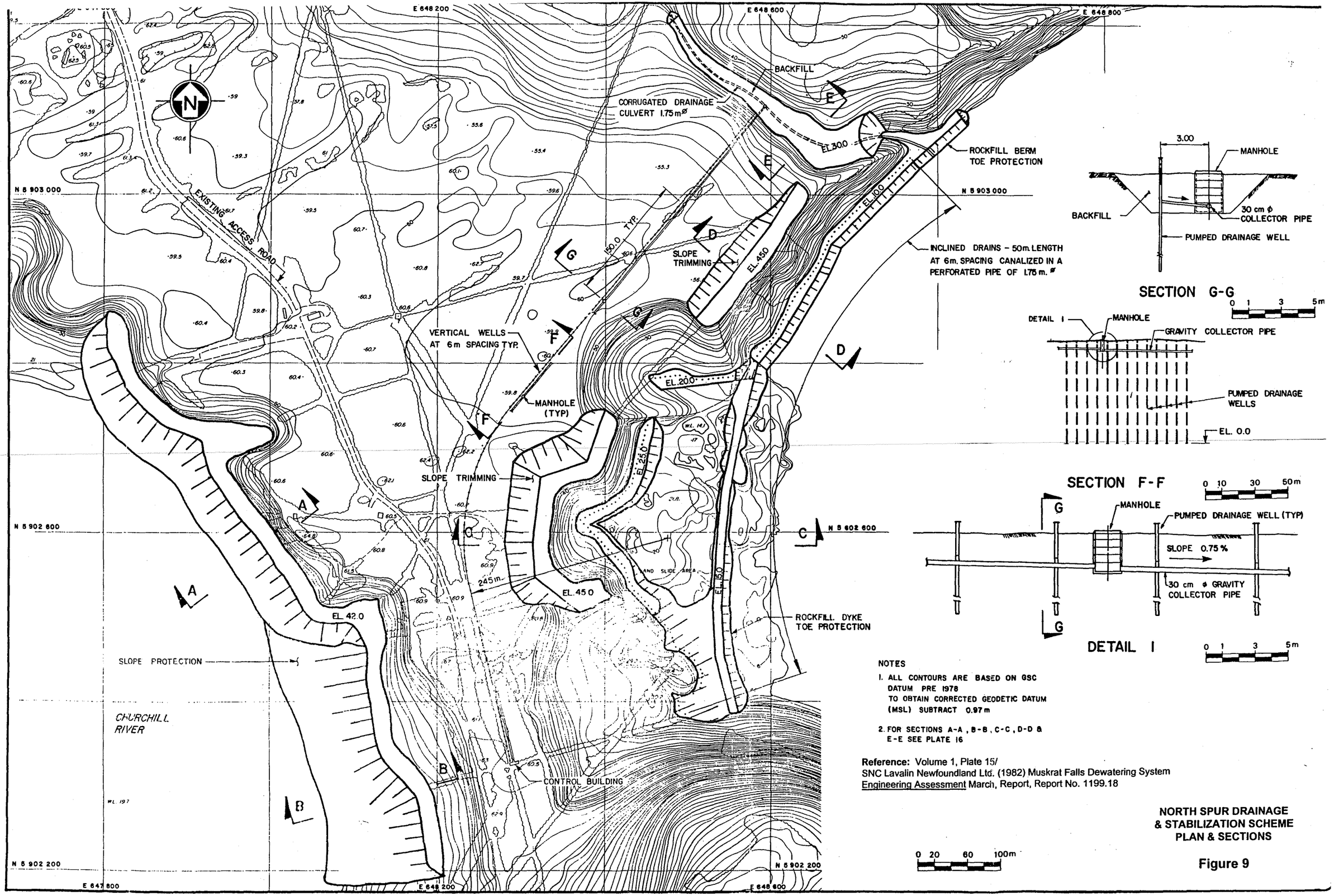
STRENGTH PARAMETERS		A: UPSTREAM SLOPE ACTUAL PROFILE W.L. AT EL. 31 M				B: UPSTREAM SLOPE WL AT 3B NO ROCKFILL				C: RAPID DRAWDOWN (FROM 3B TO 25M) NO ROCKFILL				D: RAPID DRAWDOWN 10M ROCKFILL				E: RAPID DRAWDOWN 20M ROCKFILL				F: UPSTREAM SLOPE WL AT 3E 20M ROCKFILL			
		SLIP SURFACES				SLIP SURFACES				SLIP SURFACES				SLIP SURFACES				SLIP SURFACES				SLIP SURFACES			
C'	ϕ'	1	2	3	4	①	②	③	④	①	②	③	④	①	②	③	④	①	②	③	④	①	②	③	④
0	30°	1.59	1.20	1.04	1.16	1.78	1.31	1.19	1.44	1.88	0.94	0.63	0.46	1.57	1.16	0.89	0.81	1.80	1.41	1.16	1.12	2.15	1.73	1.55	1.65
0	34	1.86	1.40	1.21	1.36	2.09	1.53	1.39	1.68	1.61	1.10	0.73	0.53	1.83	1.36	1.04	0.95	2.10	1.65	1.35	1.31	2.51	2.03	1.81	1.92

COMPARATIVE FACTORS OF SAFETY

Reference: Volume II, Plate 18/ SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kV Transmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

UPSTREAM SLOPE OF NORTH SPUR STABILIZATION ANALYSIS

Figure 8



SECTION G-G

0 1 3 5m

SECTION F-F

0 10 30 50m

DETAIL I

0 1 3 5m

NOTES

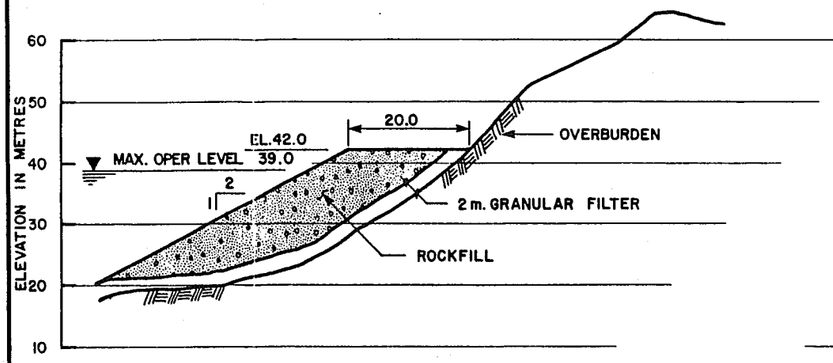
1. ALL CONTOURS ARE BASED ON GSC DATUM PRE 1978
   TO OBTAIN CORRECTED GEODETIC DATUM (MSL) SUBTRACT 0.97 m
2. FOR SECTIONS A-A, B-B, C-C, D-D & E-E SEE PLATE 16

Reference: Volume 1, Plate 15/
SNC Lavalin Newfoundland Ltd. (1982) Muskrat Falls Dewatering System
Engineering Assessment March, Report, Report No. 1199.18

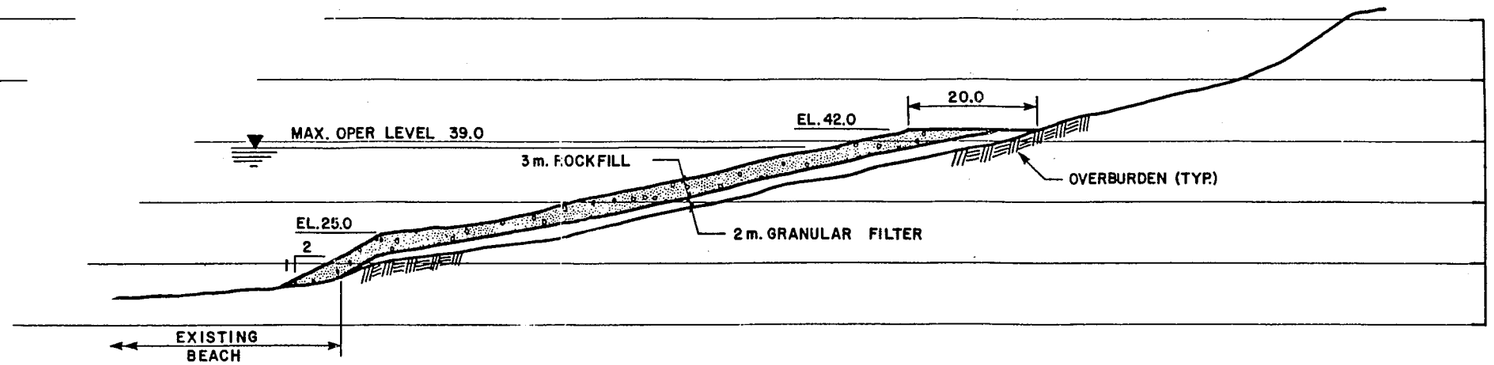
0 20 60 100m

NORTH SPUR DRAINAGE & STABILIZATION SCHEME PLAN & SECTIONS

Figure 9

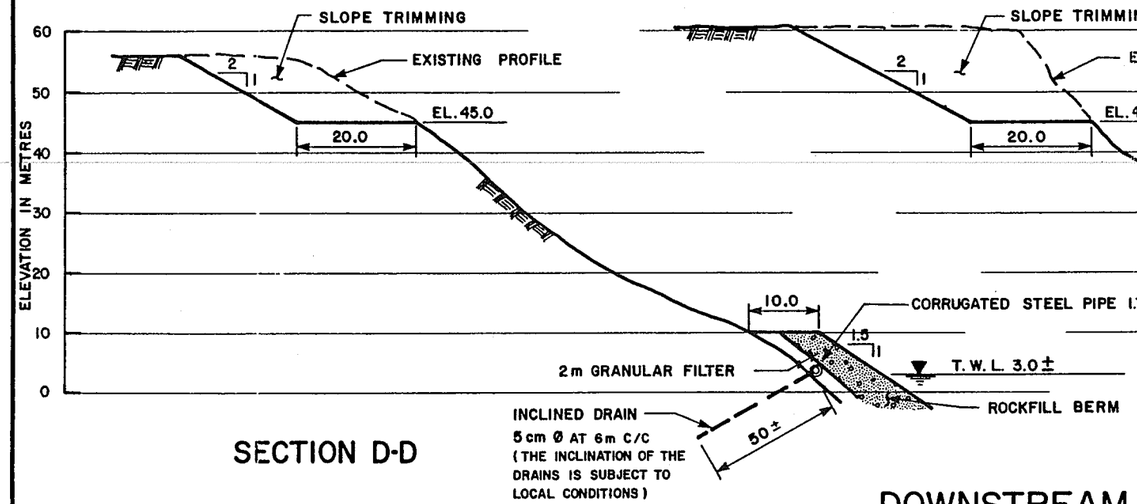


SECTION A-A

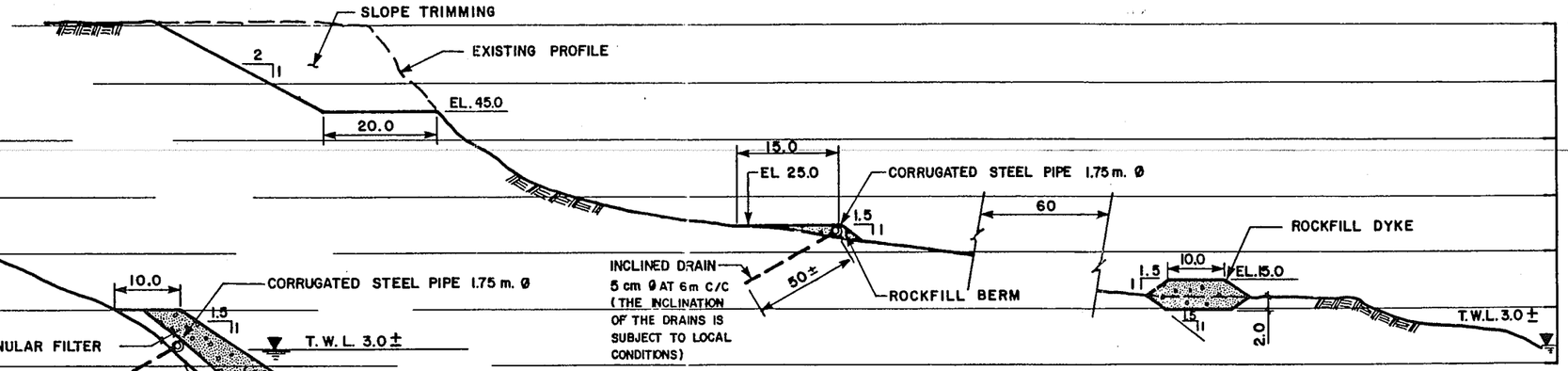


SECTION B-B

UPSTREAM SECTIONS

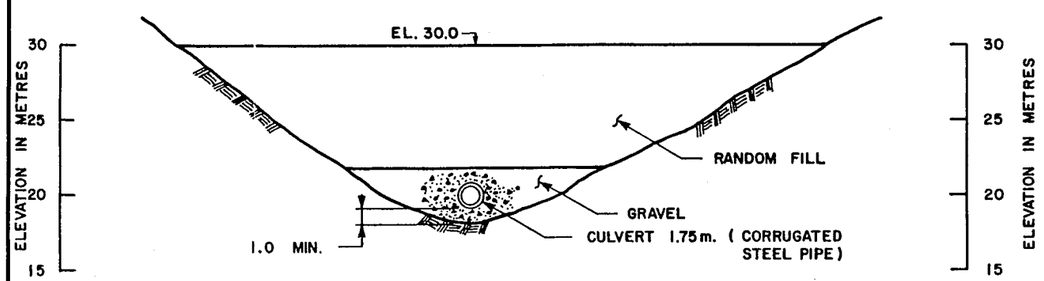


SECTION D-D



SECTION C-C

DOWNSTREAM SECTIONS



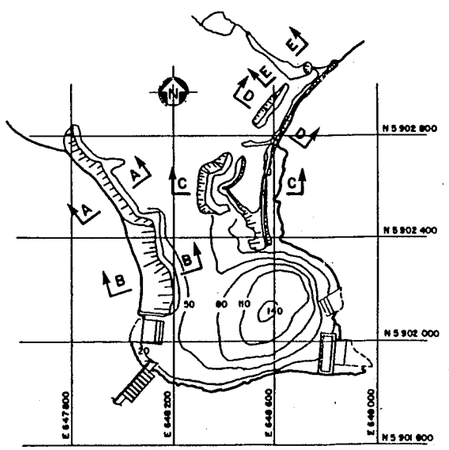
SECTION E-E

VALLEY SECTION

0 5 10 m

NOTE
ALL ELEVATIONS ARE CORRECTED
GEODETIC DATUM (MSL) 1979

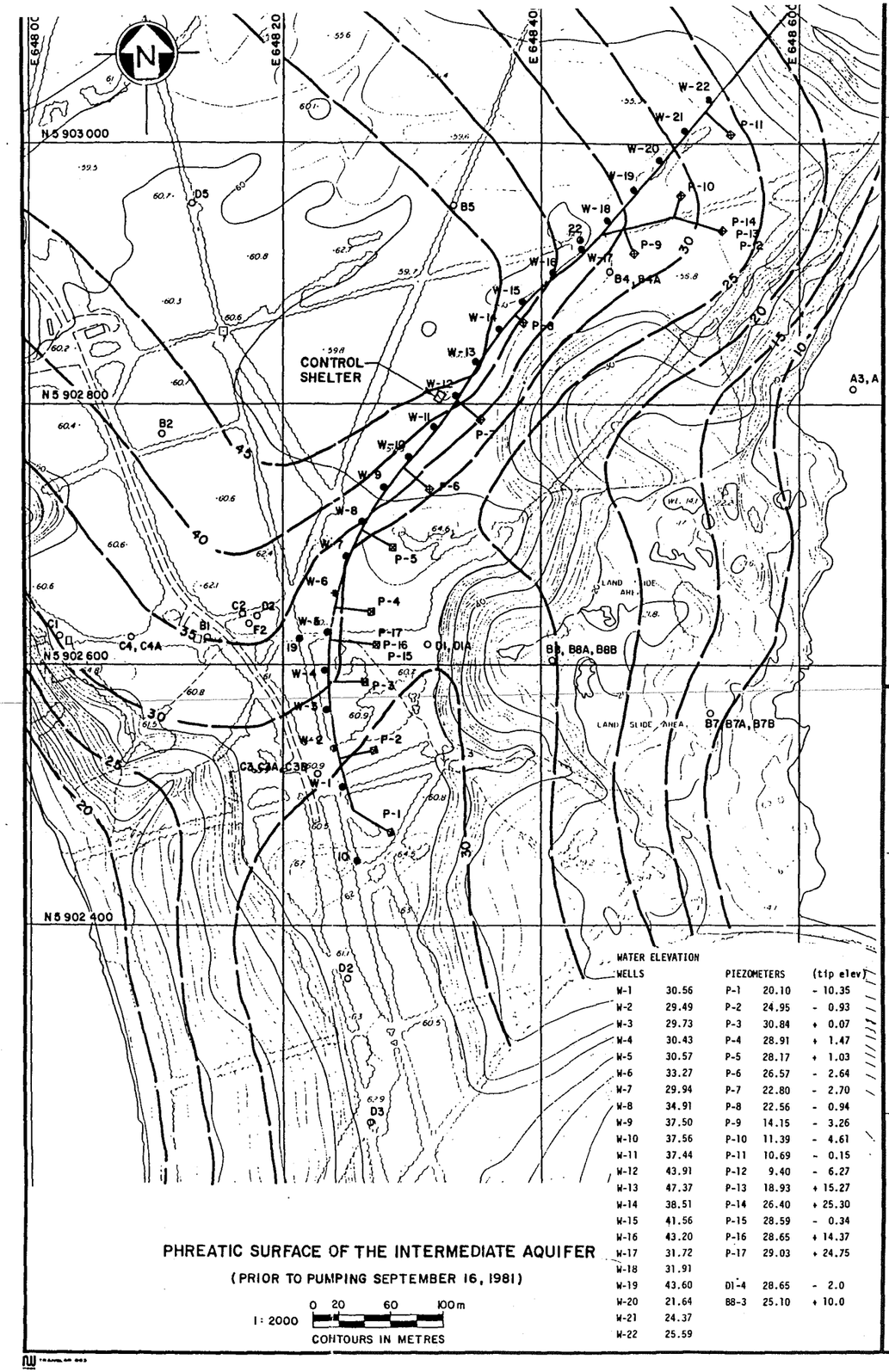
0 10 20 30 m



Reference: Volume I, Plate 16/ SNC Lavalin Newfoundland Ltd. (1980) Muskrat Falls Power Development & 345 kVTransmission Intertie To Churchill Falls Engineering Report Vols. I to IV, March 1980.

NORTH SPUR SURFACE TREATMENT SECTIONS

Figure 10



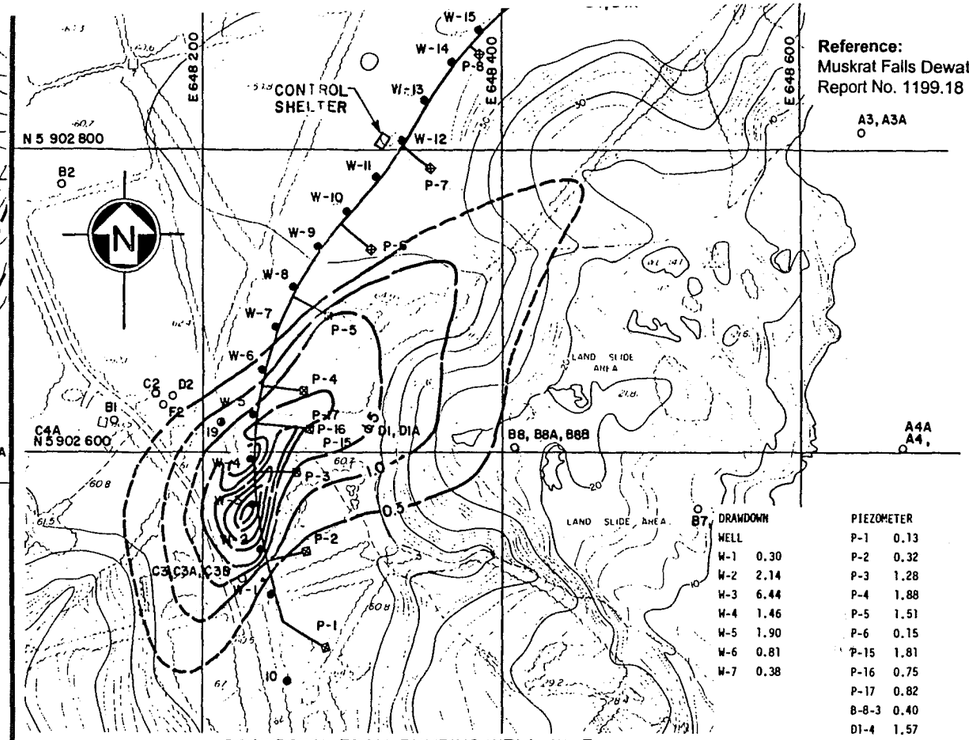
PHREATIC SURFACE OF THE INTERMEDIATE AQUIFER

(PRIOR TO PUMPING SEPTEMBER 16, 1981)

1 : 2000 0 20 60 100m

CONTOURS IN METRES

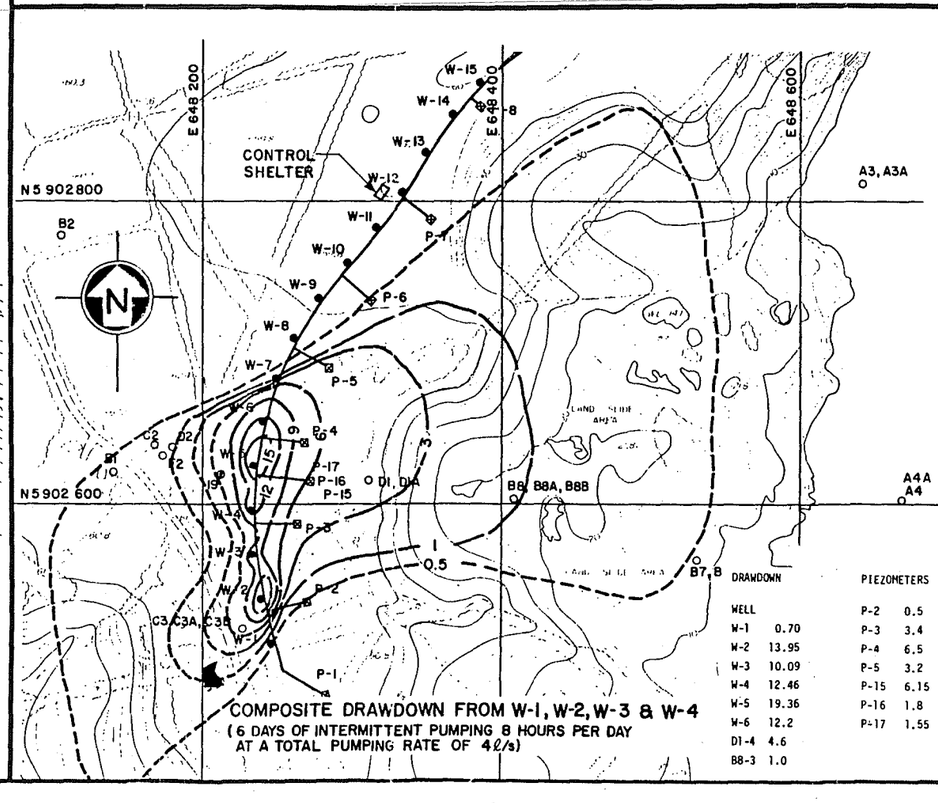
WATER ELEVATION				
WELLS		PIEZOMETERS		(tip elev)
W-1	30.56	P-1	20.10	- 10.35
W-2	29.49	P-2	24.95	- 0.93
W-3	29.73	P-3	30.84	+ 0.07
W-4	30.43	P-4	28.91	+ 1.47
W-5	30.57	P-5	28.17	+ 1.03
W-6	33.27	P-6	26.57	- 2.64
W-7	29.94	P-7	22.80	- 2.70
W-8	34.91	P-8	22.56	- 0.94
W-9	37.50	P-9	14.15	- 3.26
W-10	37.56	P-10	11.39	- 4.61
W-11	37.44	P-11	10.69	- 0.15
W-12	43.91	P-12	9.40	- 6.27
W-13	47.37	P-13	18.93	+ 15.27
W-14	38.51	P-14	26.40	+ 25.30
W-15	41.56	P-15	28.59	- 0.34
W-16	43.20	P-16	28.65	+ 14.37
W-17	31.72	P-17	29.03	+ 24.75
W-18	31.91			
W-19	43.60	D1-4	28.65	- 2.0
W-20	21.64	B8-3	25.10	+ 10.0
W-21	24.37			
W-22	25.59			



DRAWDOWN FROM PUMPING WELL W-3

(AFTER 69 HOURS OF PUMPING AT A RATE OF 1.25 ℓ/s)

DRAWDOWN		PIEZOMETER	
WELL		P-1	0.13
W-1	0.30	P-2	0.32
W-2	2.14	P-3	1.28
W-3	6.44	P-4	1.88
W-4	1.46	P-5	1.51
W-5	1.90	P-6	0.15
W-6	0.81	P-15	1.81
W-7	0.38	P-16	0.75
		P-17	0.82
		B-8-3	0.40
		D1-4	1.57



COMPOSITE DRAWDOWN FROM W-1, W-2, W-3 & W-4

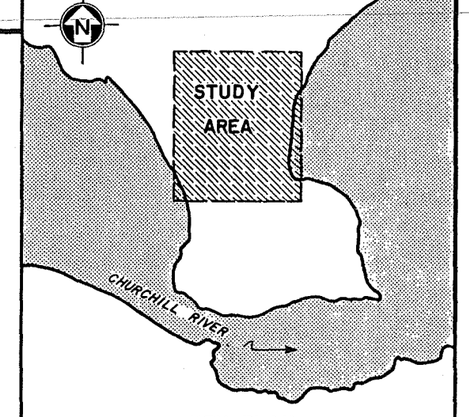
(6 DAYS OF INTERMITTENT PUMPING 8 HOURS PER DAY AT A TOTAL PUMPING RATE OF 4ℓ/s)

DRAWDOWN		PIEZOMETERS	
WELL		P-2	0.5
W-1	0.70	P-3	3.4
W-2	13.95	P-4	6.5
W-3	10.09	P-5	3.2
W-4	12.46	P-15	6.15
W-5	19.36	P-16	1.8
W-6	12.2	P-17	1.55
D1-4	4.6		
B8-3	1.0		

Reference: Plate 4/SNC Lavalin Newfoundland Ltd. (1982) Muskrat Falls Dewatering System Engineering Assessment March, Report, Report No. 1199.18

LEGEND

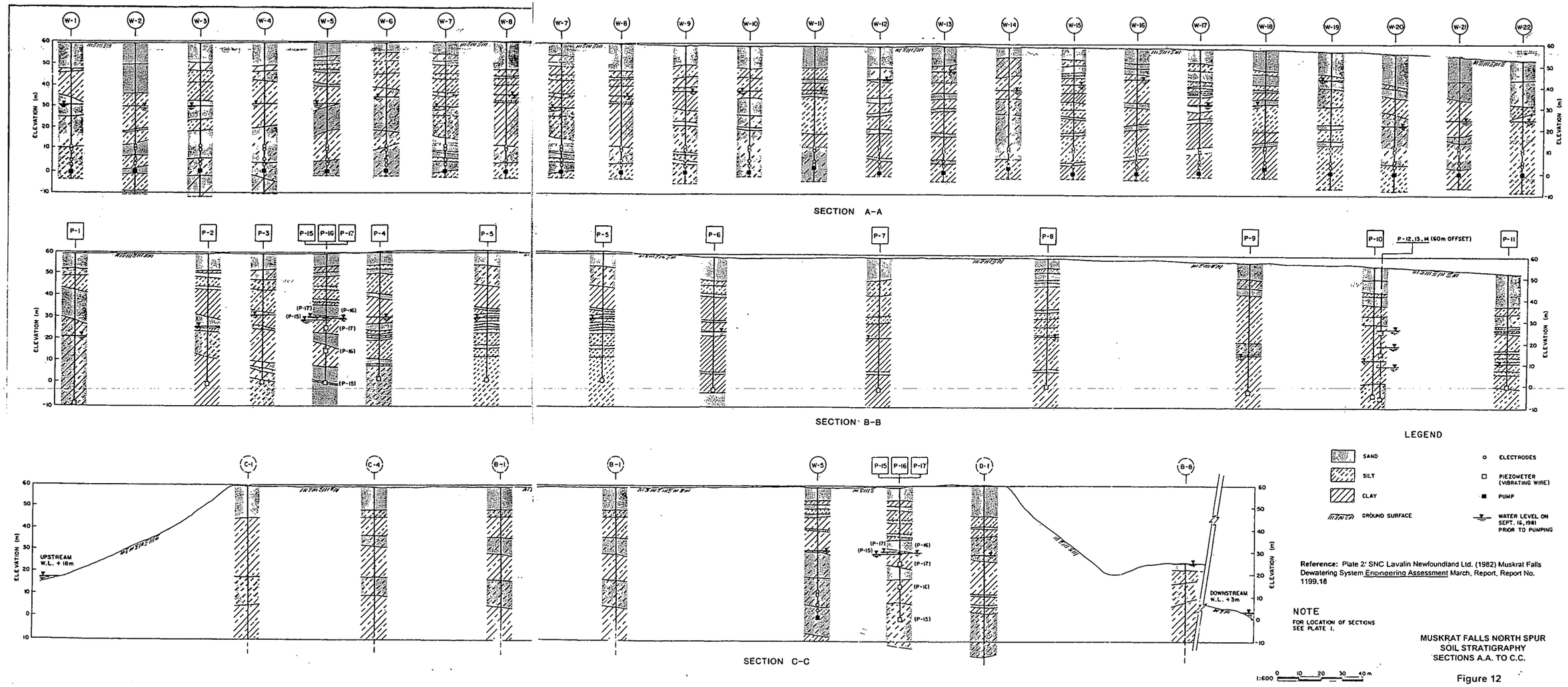
- ○ BOREHOLE (1979)
- ● W-1 PUMPING WELL (1981)
- □ P-1 PIEZOMETER (1981)
- ⊙ SURVEY STATION
- —10— PIEZOMETRIC LEVEL OR DRAWDOWN



KEY PLAN

PIEZOMETERIC LEVELS IN NORTH SPUR

Figure 11

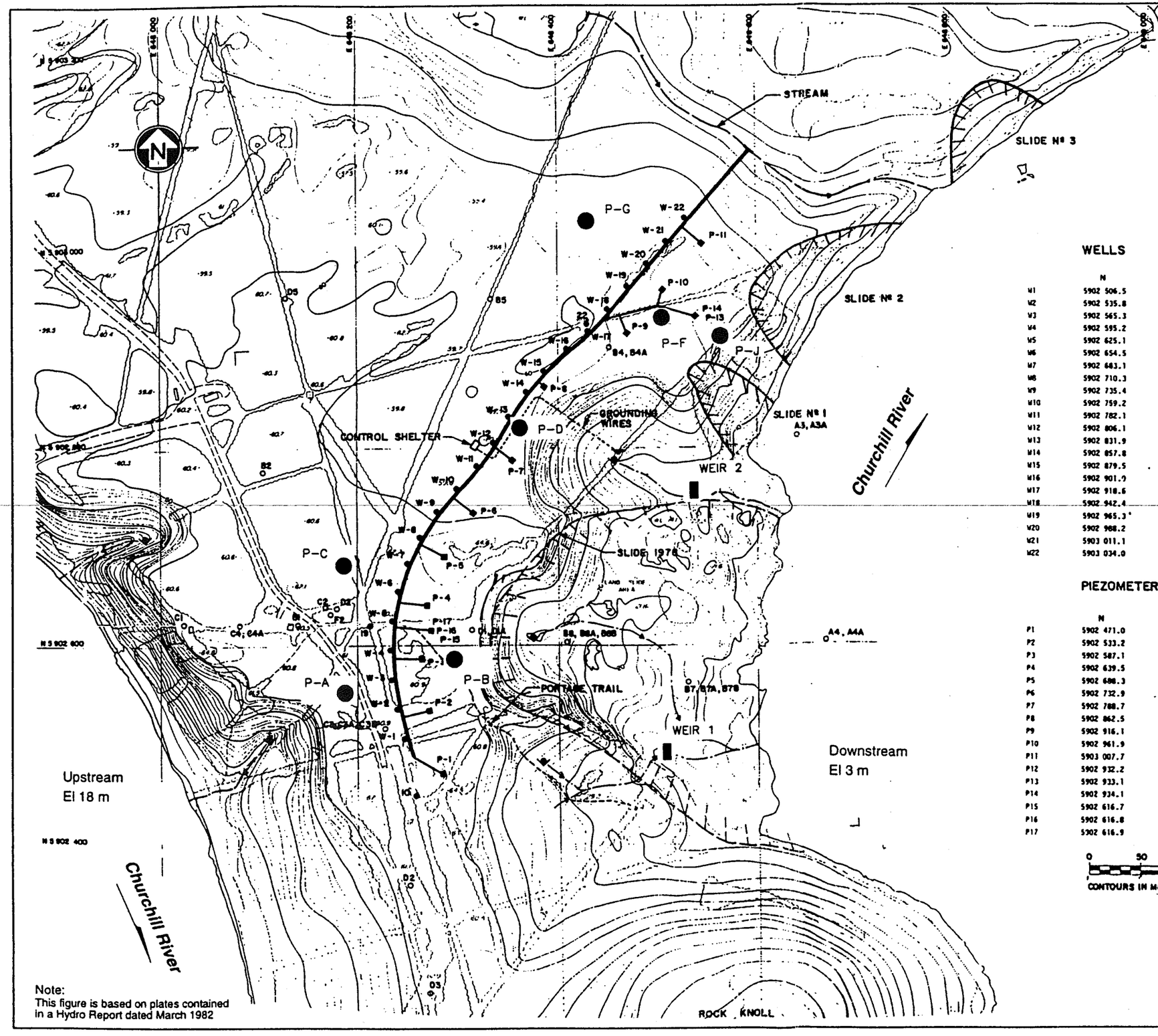


Reference: Plate 2' SNC Lavalin Newfoundland Ltd. (1982) Muskrat Falls Dewatering System Engineering Assessment March, Report, Report No. 1199.18

NOTE
FOR LOCATION OF SECTIONS SEE PLATE 1.

MUSKRAT FALLS NORTH SPUR
SOIL STRATIGRAPHY
SECTIONS A.A. TO C.C.

Figure 12



Note:
This figure is based on plates contained in a Hydro Report dated March 1982

Reference: ACRES (1998) Muskrat Falls Piezometer Installation Progress Report, February, Figure 1

LEGEND

- BOREHOLE (1979)
- W-1 PUMPING WELL (1981)
- P-1 PIEZOMETER (1981)
- 22 SURVEY STATION
- TRENCH FOR COLLECTOR PIPE AND CABLES
- SPRING
- RECENT LANDSLIDES (1980-1981)
- LANDSLIDE (1978)
- P-G BOREHOLE/PIEZOMETER INSTALLATION (1997)
- WEIR 2 V-NOTCH WEIR INSTALLATION (1997)

WELLS

	N	E
W1	5902 506.5	648 246.8
W2	5902 535.8	648 239.7
W3	5902 565.3	648 234.3
W4	5902 595.2	648 232.7
W5	5902 625.1	648 234.6
W6	5902 654.5	648 240.3
W7	5902 683.1	648 249.4
W8	5902 710.3	648 262.0
W9	5902 735.4	648 277.8
W10	5902 759.2	648 296.5
W11	5902 782.1	648 315.9
W12	5902 806.1	648 333.3
W13	5902 831.9	648 349.1
W14	5902 857.8	648 367.2
W15	5902 879.5	648 385.8
W16	5902 901.9	648 408.5
W17	5902 918.6	648 431.0
W18	5902 942.4	648 451.6
W19	5902 965.3	648 471.0
W20	5902 988.2	648 490.3
W21	5903 011.1	648 509.7
W22	5903 034.0	648 529.1

PIEZOMETERS

	N	E
P1	5902 471.0	648 283.8
P2	5902 533.2	648 271.2
P3	5902 587.1	648 264.1
P4	5902 639.5	648 269.3
P5	5902 688.3	648 285.5
P6	5902 732.9	648 313.5
P7	5902 788.7	648 352.3
P8	5902 862.5	648 384.0
P9	5902 916.1	648 468.6
P10	5902 961.9	648 507.4
P11	5903 007.7	648 546.2
P12	5902 932.2	648 542.2
P13	5902 933.1	648 541.1
P14	5902 934.1	648 539.8
P15	5902 616.7	648 274.9
P16	5902 616.8	648 273.8
P17	5902 616.9	648 272.2

0 50 100 m
CONTOURS IN METRES

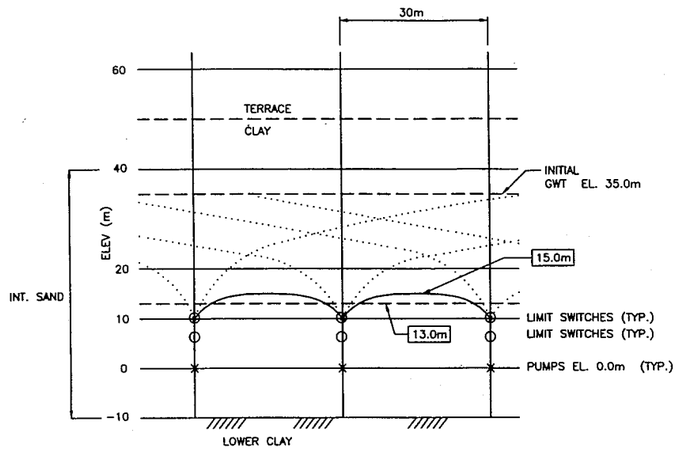
MUSKRAT FALLS STANDPIPE PIEZOMETERS

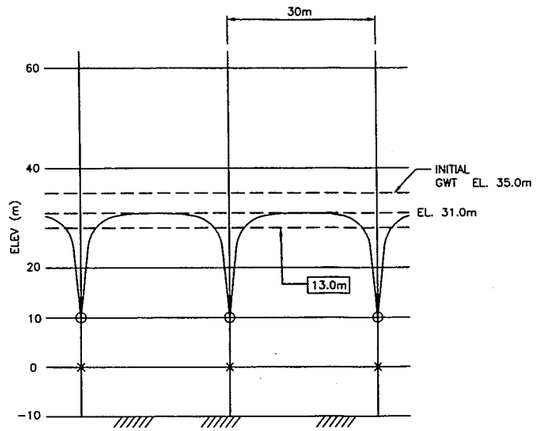
Piezometer Number	El. Tip	Piezometer Elevation (m) 22-May-98	09-Jul-98	07-Aug-98	10-Sep-98
P-A1	6.5	10.60	11.72	10.69	???
P-A2	24.5	DRY	DRY	DRY	DRY
P-B1	1.3	9.24	9.47	9.27	9.17
P-B2	13.7	22.15	22.69	22.66	???
P-C	15.2	34.89	34.82	34.74	34.74
P-D1	12.6	23.41	22.76	22.58	22.55
P-D2	25.7	30.92	30.90	30.83	30.88
P-F1	0.8	12.22	12.25	12.13	12.10
P-F2	12.0	DRY	DRY	DRY	DRY
P-G	4.0	17.52	17.62	17.37	16.84
P-J1	6.1	10.15	10.19	9.96	9.94
P-J2	19.6	11.44	11.57	11.01	10.85

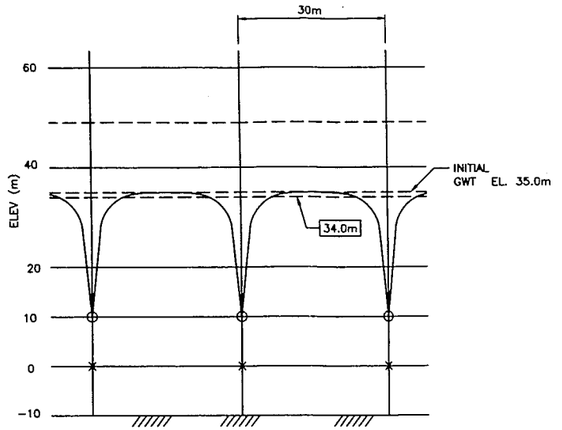
NORTH SPUR CURRENT PIEZOMETER DATA

Figure 13

FOR 30m PRESENT WELL SPACING







LEGEND

DRAWDOWN CONE INDIVIDUAL WELLS (R=90m)

GWT @ ℄ PUMPWELLS (CUMULATIVE)

MEDIAN GWT DOWNSTREAM OF PUMPWELL LINE = D/S WL

SOUTHERN ZONE

TAKE K = 1 x 10^{-3} cm/s AVERAGE EL. −10 TO +35

R = c(H−hw)$\sqrt{K}$ = 1.5 x 60^{1} x $\sqrt{10}$ = 270^{1} = 90m

CENTRAL ZONE

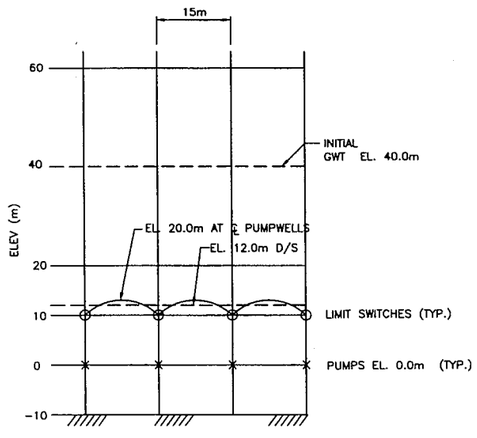
FOR K = 1 x 10^{-4} cm/s R=30m

NORTHERN ZONE

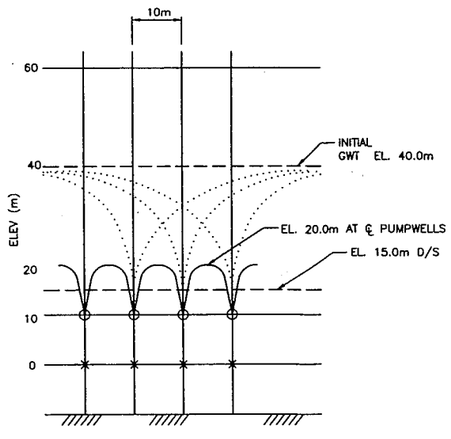
FOR K = 1 x 10^{-5} cm/s R = 1m

FOR FINAL PUMPWELL LAYOUT

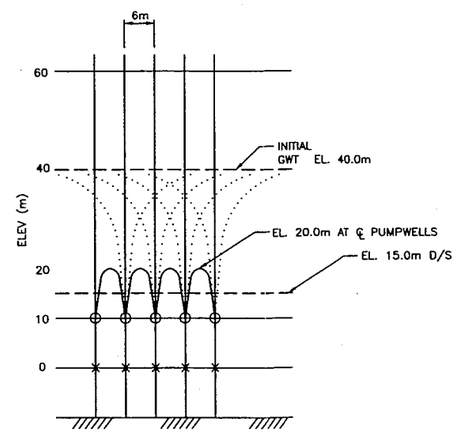
TAKE K = 1 x 10^{-3} cm/s R = 90m FOR 15m SPACING



TAKE K = 1 x 10^{-4} cm/s R = 30m WITH 10m SPACING

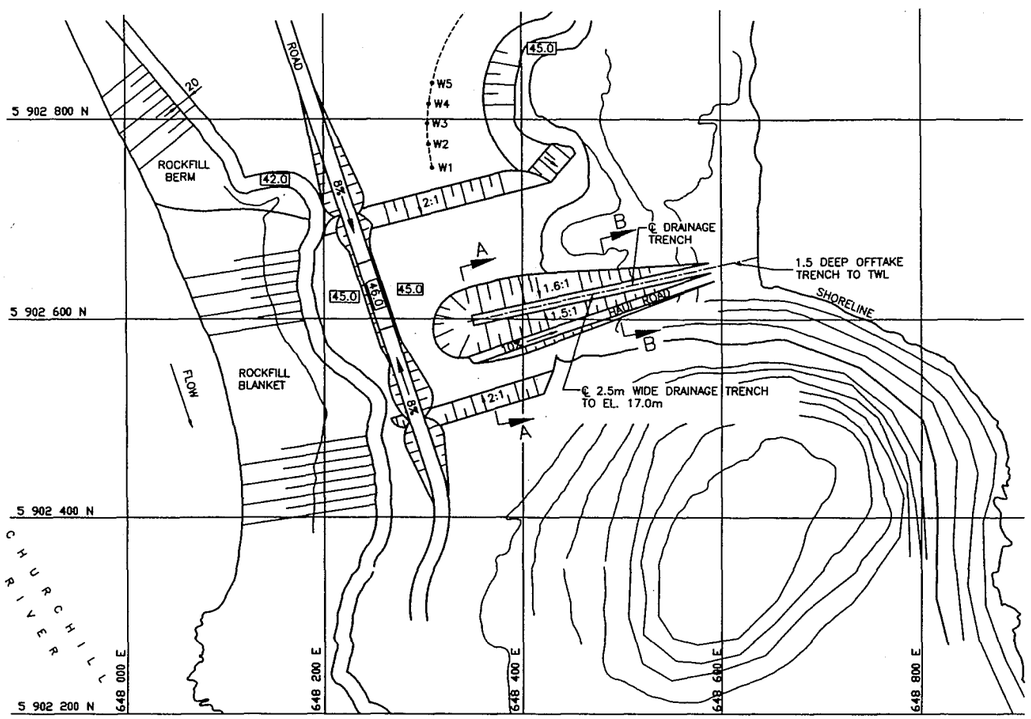


TAKE R = 15m FROM PIEZOMETER RESPONSE WITH STEEP DRAWDOWN CURVES FOR 6m SPACING



MUSKRAT FALLS PUMPWELL DRAWDOWN

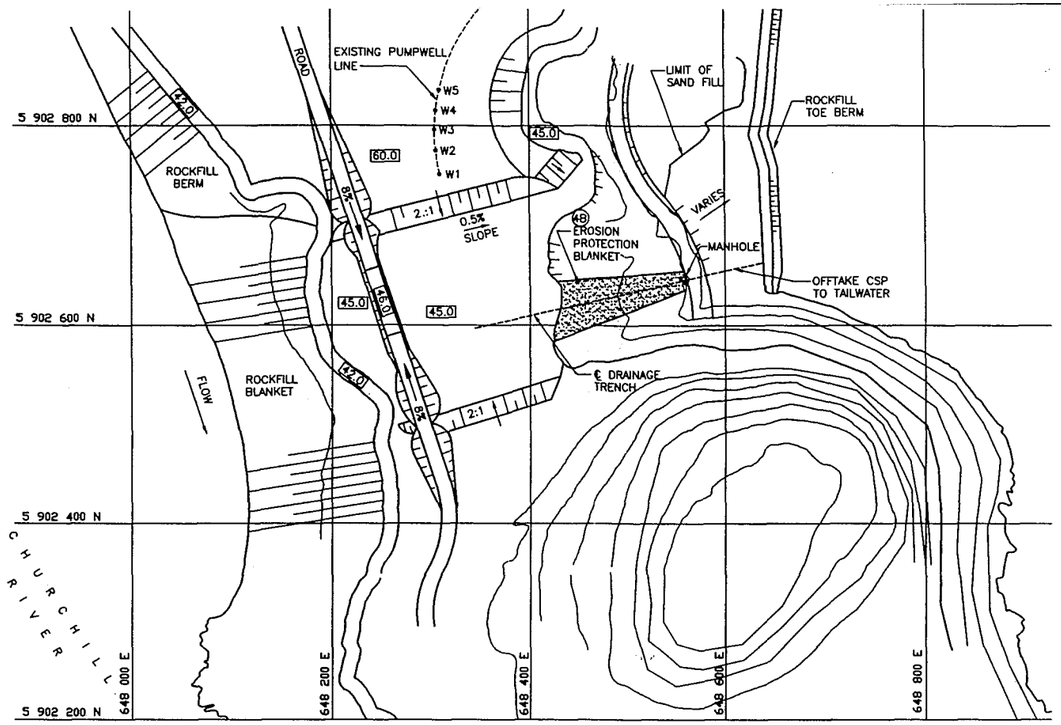
Figure 14



PLAN SHOWING EXCAVATIONS

0 50 0 100m

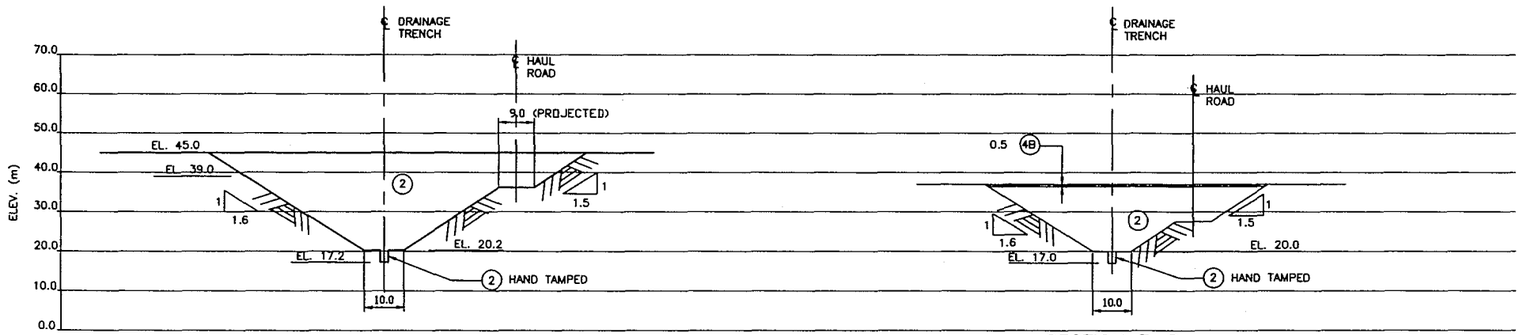
SCALE



PLAN SHOWING FINAL GRADING

0 50 0 100m

SCALE



SECTION A–A

0 50m

SCALE

SECTION B–B

0 50m

SCALE

NOTE:

1. DRAIN TRENCH EL. 20 TO 17m TO BE EXCAVATED AND BACKFILLED WITH HAND TAMPED CLASS 2 GRANULAR FILL.

2. CLASS 2 GRANULAR FILL SELECT SAND-GRAVEL FROM GR-4 OR GR-5 WITH <3% PASSING 0.074mm (NO. 200) SIEVE, COMPACTED TO 95% MAX VIBRATED DENSITY (ASTM 2049).

LEGEND

(2) COMPACTED GRANULAR FILL

(4B) TUNNEL MUCK

COMPARISON PUMPWELL & DRAINAGE TRENCH TREATMENT COSTS (1980 DOLLARS)

ITEM	APPROX. QTY.	UNIT COST	SUB TOTAL
1. EXCAVATION EL. 60–45	560,000m^3	$5.64/m	$3,158,000
2. EXCAVATION EL. 45–20	181,000m^3	$7.00/m	$1,267,000
3. TRENCH EL. 20–17	1,320m^3	$20.00/m	$26,000
4. FILL CLASS 2	177,000m^3	$11.11/m	$1,966,000
5. EROSION PROTECTION CLASS 4B	5,000m^3	$5.00/m	$25,000
6. MANHOLE / PIPE		L. S.	$15,000
TOTAL			$6,457,000

DEDUCT PUMPWELLS AT 2.5m c/c
SAY 80 @ $29,975 = $2,400,000 ($2,400,000)

INCREASE $4,057,000

MUSKRAT FALLS ALTERNATE
SOUTH END STABILIZATION

FIGURE 15

N 5,903,400

N 5,903,200

N 5,903,000

N 5,902,800

N 5,902,600

N 5,902,400

E 647 800

E 648 000

E 648 200

E 648 400

E 648 600

GRANULAR BACKFILL

TRENCH TO EL. 15m AND SAND BACKFILL

CORRUGATED STEEL CULVERT 1.7m ø

EL. 15.0

SAND FILLED DRAINAGE TRENCH

PUMPWELLS AT 6m SPACING

EL. 10.0

EL 45.0

RELIEF DRAINS AT 6m SPACING

PUMP WELLS AT 10m SPACING TYP.

MANHOLE (TYP)

EL. 25.0

RELIEF DRAINS AT 10m SPACING

PUMP WELLS AT 15m SPACING

ROCK BERM

LIMITS ① BLANKET

ROAD

180.0

LIMIT GRANULAR BLANKET

SLOPE

EL. 45.0

VARIES

EL. 10.0

ROCKFILL. TOE BERM

EL. 42.0

MANHOLE

EL. 20.0

OFFTAKE DRAIN AND PIPE

ROCKFILL BLANKET

CHURCHILL RIVER

0.5% SLOPE

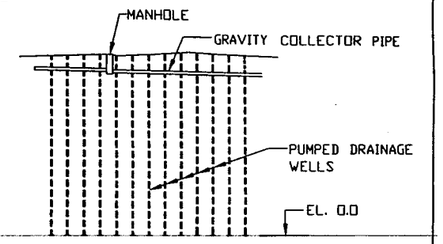
℄ DRAINAGE TRENCH TO EL. 17.0

FLOW

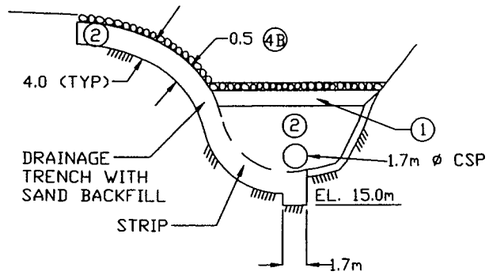
Cut Line

NOTES:

1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.



SECTION A-A



SECTION B-B

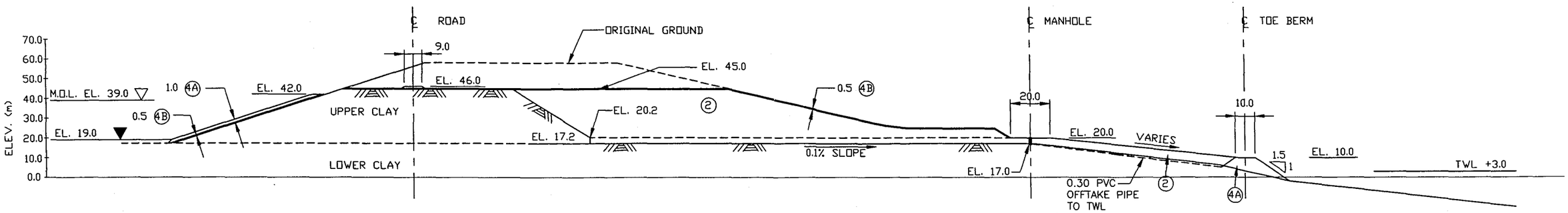
N.T.S.

0 100 200

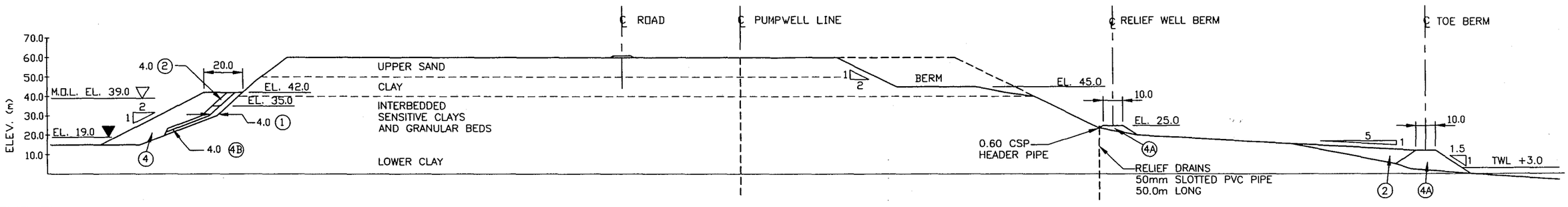
SCALE

NORTH SPUR STABILIZATION MEASURES PLAN & SECTIONS

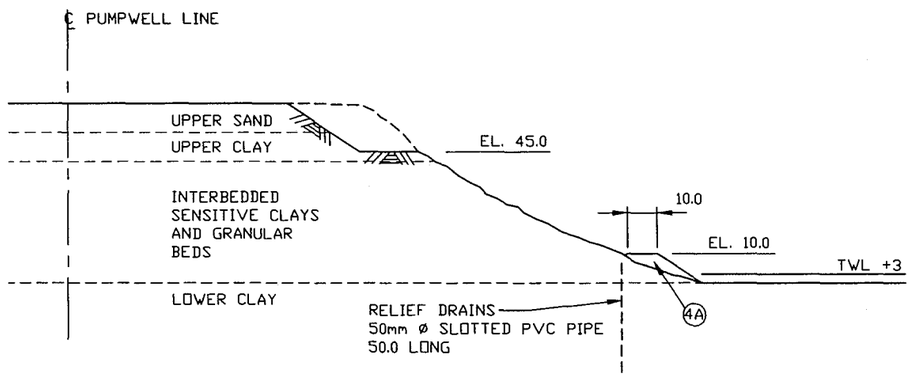
FIGURE 16



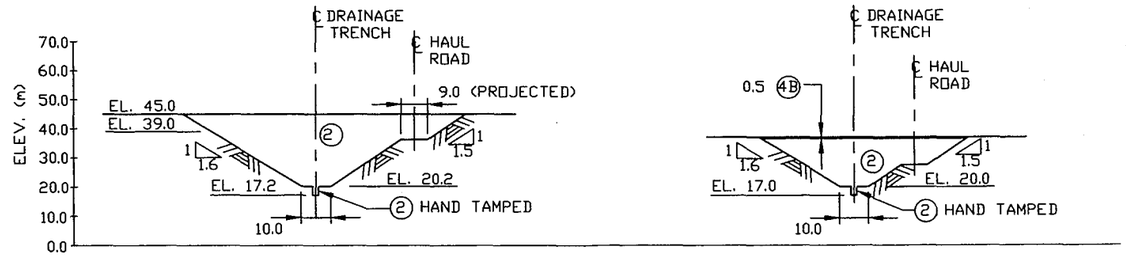
SECTION C–C



SECTION D–D



SECTION H–H

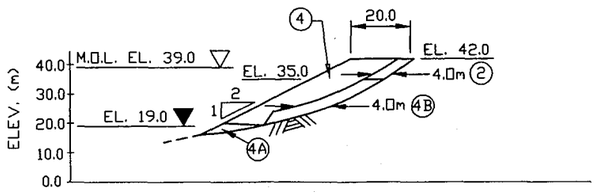


SECTION F–F

SECTION G–G

LEGEND:

M.O.L. ▽	MAX. OPERATING LEVEL
▼	WATER LEVEL AT TIME OF CONSTRUCTION
(1)	COMPACTED TILL
(2)	COMPACTED GRANULAR FILL
(4)	COMPACTED ROCKFILL
(4A)	DUMPED ROCKFILL
(4B)	TUNNEL MUCK

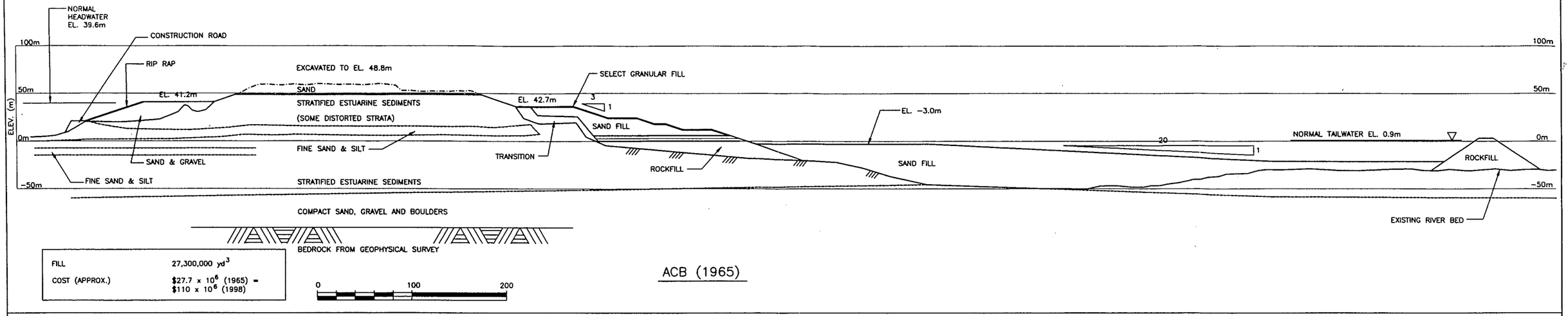


SECTION E–E

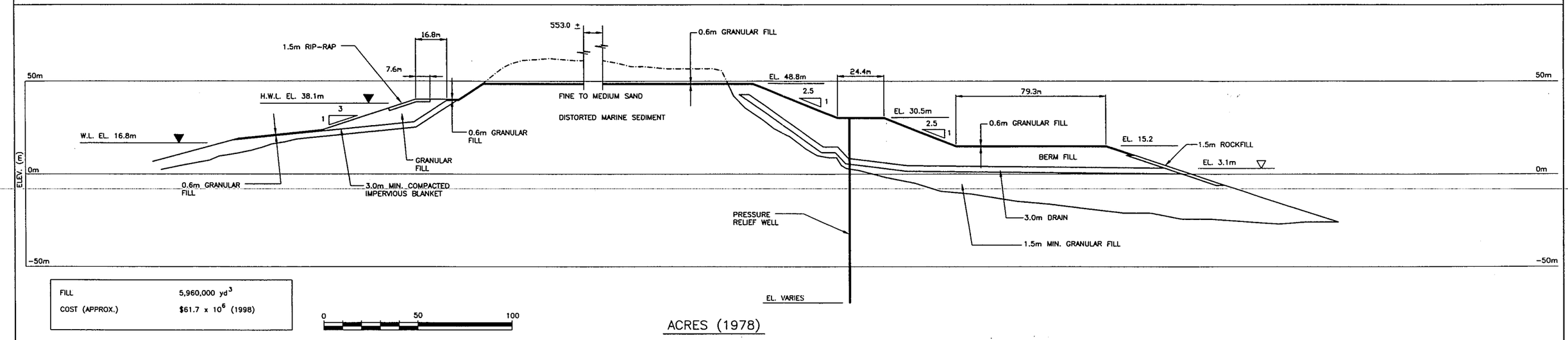
0 50 100

NORTH SPUR STABILIZATION MEASURES SECTIONS & DETAILS

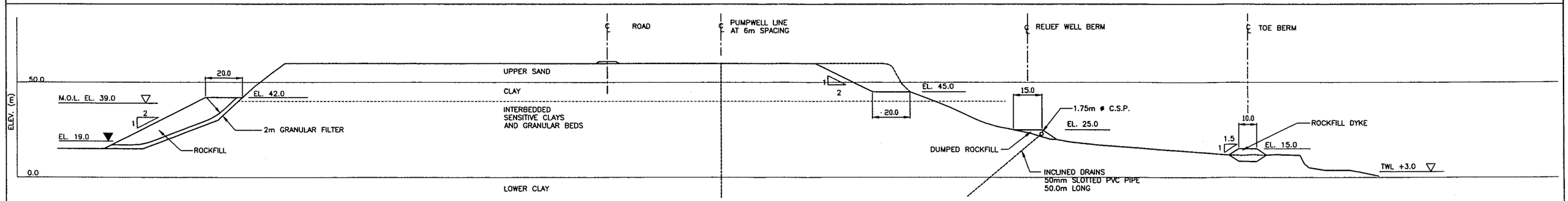
FIGURE 17



ACB (1965)



ACRES (1978)



SNC−LAVALIN (1980)

NORTH SPUR
SCOPE & COST STABILIZATION MEASURES
(1965−1980)

FIGURE 18

APPENDIX "I"

-LA, . NEW. , ...

MUSKRAT FALLS NORTH SPUR

PROJECT: TEMPORARY STABILIZATION MEASURES

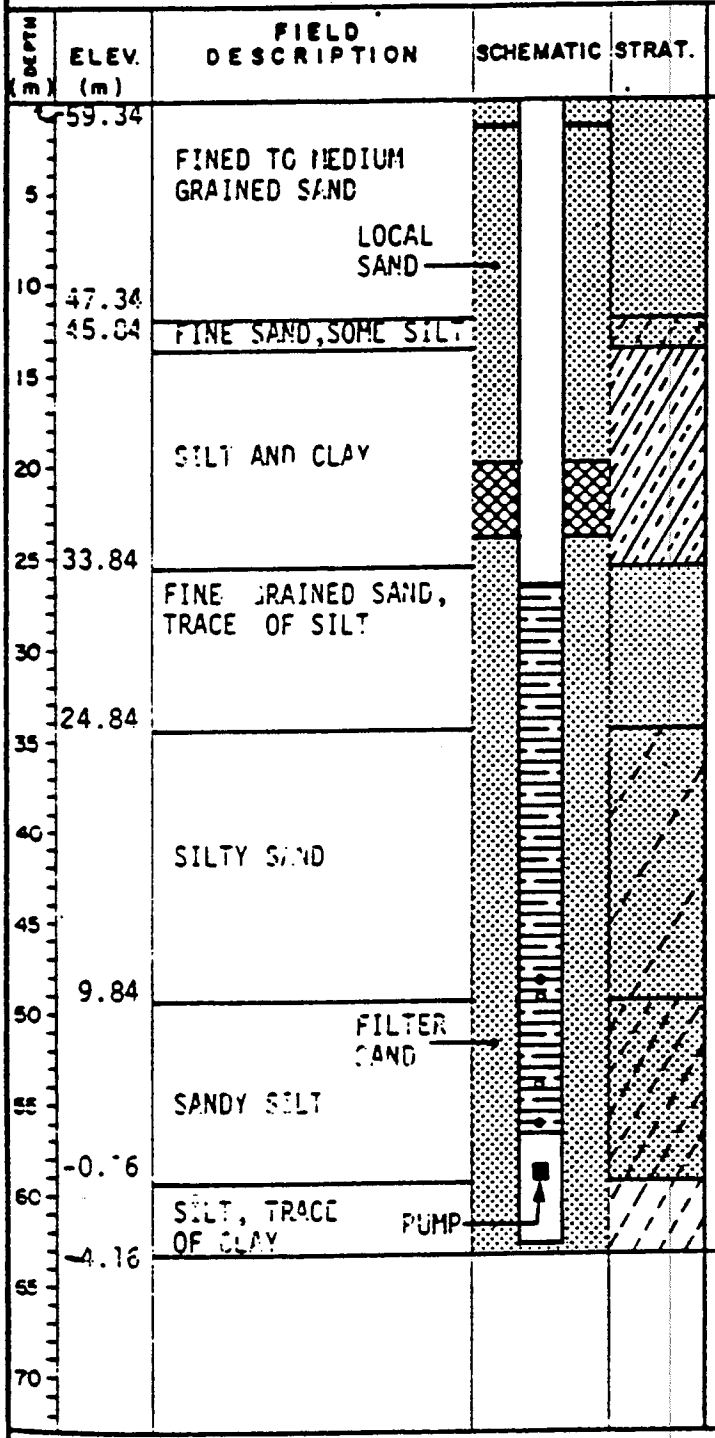
BOREHOLE No M1

LOCATION: 5902 506.5N 648,246.8E

DATE: JULY 27, 1981

ELEVATION DATUM: GEODETIC

FIGURE No _____



GEOPHYSICAL INTERPRETATION

TEST DESCRIPTION

COMMENTS: SUBMERSIBLE PUMP INSTALLED AT ELEVATION 0 m

ELECTRODES AT ELEVATION + 11, + 10, + 5, + 3 m

GEOPHYSICAL LOGGING NOT CARRIED OUT ON THIS HOLE

PUMP SERIAL NO. 8116589 MOTOR NO. H 10330 N

SNC/LAVALIN
Newfoundland Ltd

TEST HOLE RECORD

FIGURE NO. _____

PROJECT MUSKRAT FALLS STUDY NO. _____ HOLE NO. d2

LOCATION 5 902 358 N - 643 234 E SHEET 1 OF 2

ELEVATION DATUM GEODETIC CASING SIZE HW DATE 10-22-79 / 10-28-79

SAMPLE TYPES
SS SPLIT SPOON, 51 mm O.D.
TW THIN-WALL SAMPLER, ____ mm ____ O/D
PS FIXED PISTON SAMPLER, ____ mm ____ O/D
DC DIAMOND CORE, HQ SIZE
WS WASH SAMPLE
AS AUGER SAMPLE
NS NILCON SAMPLER, 50 mm
GW WATER SAMPLE

GROUND WATER
▼ WATER LEVEL

SAMPLE CONDITIONS
ROCK | FAIR | DISTURBED | LOST

TESTS
N S.P.T. TEST VALUE (blows/0.3 m)
K PERMEABILITY (cm/s)
V IN-SITU VANE (kPa)
V_R VANE ON REMOLDED SOIL (kPa)
V_L LABORATORY VANE (kPa)
T FALL CONE ON INTACT SOIL (kPa)
T_R FALL CONE ON REMOLDED SOIL (kPa)
P PRESSIOMETER TEST

TESTS
GSA GRAIN-SIZE ANALYSIS
γ TOTAL UNIT WEIGHT (kN/m³)
U UNCONFINED COMPRESSION (kPa)
Q UNDRAINED TRIAXIAL (kPa)
(%) STRAIN AT FAILURE
C CONSOLIDATION
RQD ROCK QUALITY DESIGNATION
P' PERCENT PASSING 200

DEPTH (m)	ELEV. (m)	DESCRIPTION	STRAT.
0	60.1	GROUND SURFACE	
		DENSE TO VERY DENSE GREY BROWN FINE TO MEDIUM SAND	
	52.9		
		BROWN SILTY CLAY WITH OCCASIONAL SAND SEAMS	
	50.9		
		VERY DENSE GREY BROWN FINE SAND	
		WITH TRACES OF SILT/ FINE SILTY SAND	
	43.5		
		ALTERNATING LAYERS OF FINE SILTY SAND AND SILTY CLAY	
		MAJOR SAND SECTION ENCOUNTERED BETWEEN ±20.4 - 21.3m AND POSSIBLY BETWEEN ±22.9 - 24.4 & 25.9 - 27.4m DEPTH	
	32.6		
		SENSITIVE BROWN SILTY CLAY OF LOW TO MEDIUM PLASTICITY	
		WITH OCCASIONAL SAND SEAMS	
	9.1		

GROUND WATER

SAMPLES COND	NO. & TYPE	REC (%)	DYNAMIC PENETRATION RESISTANCE (blows/0.3 m) 20 40 60 80 100	SHEAR STRENGTH (kPa) 50 100	TESTS	W WATER CONTENT (%) W_P PLASTIC LIMIT W_L LIQUID LIMIT 10 20 30 40
	SS 1	33			N = 38 GSA	
	2	66			N = 55 GSA	
	3	100			N = 15	
	DC 4	74				
	5	47			GSA	
	6	0				
	7	14			GSA	
	SS 8	22			N = 73 GSA	
	DC 9	28			GSA	
	SS 10	100			N = 53 GSA P' = 79	
	DC 11	76				
	12	76				
	13	78				
	14	86				
	15	0				
	16	100				
	17	0				
	18	100				
	19	100				
	20	100				
	21	100				
	22	100				
	23	90				
	24	100				
	25	95				
	26	95				
	27	93				
	28	100				
	29	42				
	30	100				
	31	64				
	32	84				
	33	39				
	34	75				

8-1-4300/007/0

SNC-LAVALIN
Newfoundland Ltd.

TEST HOLE RECORD

FIGURE NO. _____

PROJECT MUSKRAT FALLS STUDY NO. _____

HOLE NO. d 2

SHEET 2 OF 2

DEPTH (m)	ELEV. (m)	PROFILE: DESCRIPTION	STRAT.	GROUND WATER	SAMPLES: COND.	NO. & TYPE	REC. (%)	DYNAMIC PENETRATION RESISTANCE (blows/0.3 m) 20 40 60 80 100 / SHEAR STRENGTH (kPa) 50 100	TESTS	W WATER CONTENT (%) Wp PLASTIC LIMIT WL LIQUID LIMIT 10 20 30 40
52		GROUND SURFACE								
		SLIGHTLY SENSITIVE BROWN SILTY CLAY OF MEDIUM PLASTICITY			DC	34	73			
						35	69			
						36	54			
						37	49			
						38	62			
50						39	52			
						40	84			
						41	68			
						42	60			
						43	78		GSA	
						44	88			
						45	76			
						46	92			
		WELL GRADED SAND AND GRAVEL AND SILT WITH OCCASIONAL COBBLES - GLACIAL TILL				47	76			
	14.1					48	80		GSA	
	15.5					49	59		RQD = 59%	
		GRANITE GNEISS BEDROCK				50	98		RQD = 78%	
	17.7	END OF HOLE								
48										
46										
44										
42										
40										
38										
36										
34										
32										

S-L 9300/008/0

SNC-LAVALIN Newfoundland Ltd

MUSKRAT FALLS NORTH SPUR

PROJECT: TEMPORARY STABILIZATION MEASURES

LOCATION: 5,902,942.4N 648,451.6E

ELEVATION DATUM: GEODETIC

BOREHOLE RECORD

BOREHOLE No W 18

DATE: AUGUST 15, 1981

FIGURE No

DEPTH (m)	ELEV. (m)	FIELD DESCRIPTION	SCHEMATIC STRAT.	GEOPHYSICAL INTERPRETATION	TEST DESCRIPTION
0	57.57	FINE TO MEDIUM GRAINED SAND		SAND	R RES
10	47.77				
	45.97	SILTY CLAY SOME SAND		SILT	
15	40.97	FINE GRAINED SAND TRACE OF SILT		SAND	
	39.37	SAND & SILT SOME CLAY		SILT	
20	38.27	FINE GRAINED SAND		SAND	
	34.17	SILTY CLAY		CLAY	
25	32.27			SAND	
30		FINE GRAINED SAND SOME SILT		SILT	
40	19.57			CLAY	
	13.57	FINE GRAINED			
45	12.57	SILTY SAND		SAND	
50				CLAY	
55	3.57	CLAYEY SILT, TRACE OF SAND			
60	-2.43			END OF BOREHOLE W 18 AT 60.0 m DEPTH.	
65					
70					

COMMENTS: PUMP AT + 2.87 m ELEV.
ELECTRODES AT +11, +10, +5, + 3m ELEV.
PUMP SERIAL NO. 8116537, MOTOR NO. M 10337N

SNC-LAVALIN Newfoundland Ltd

BOREHOLE RECORD

MUSKRAT FALLS NORTH SPUR

PROJECT: TEMPORARY STABILIZATION MEASURES

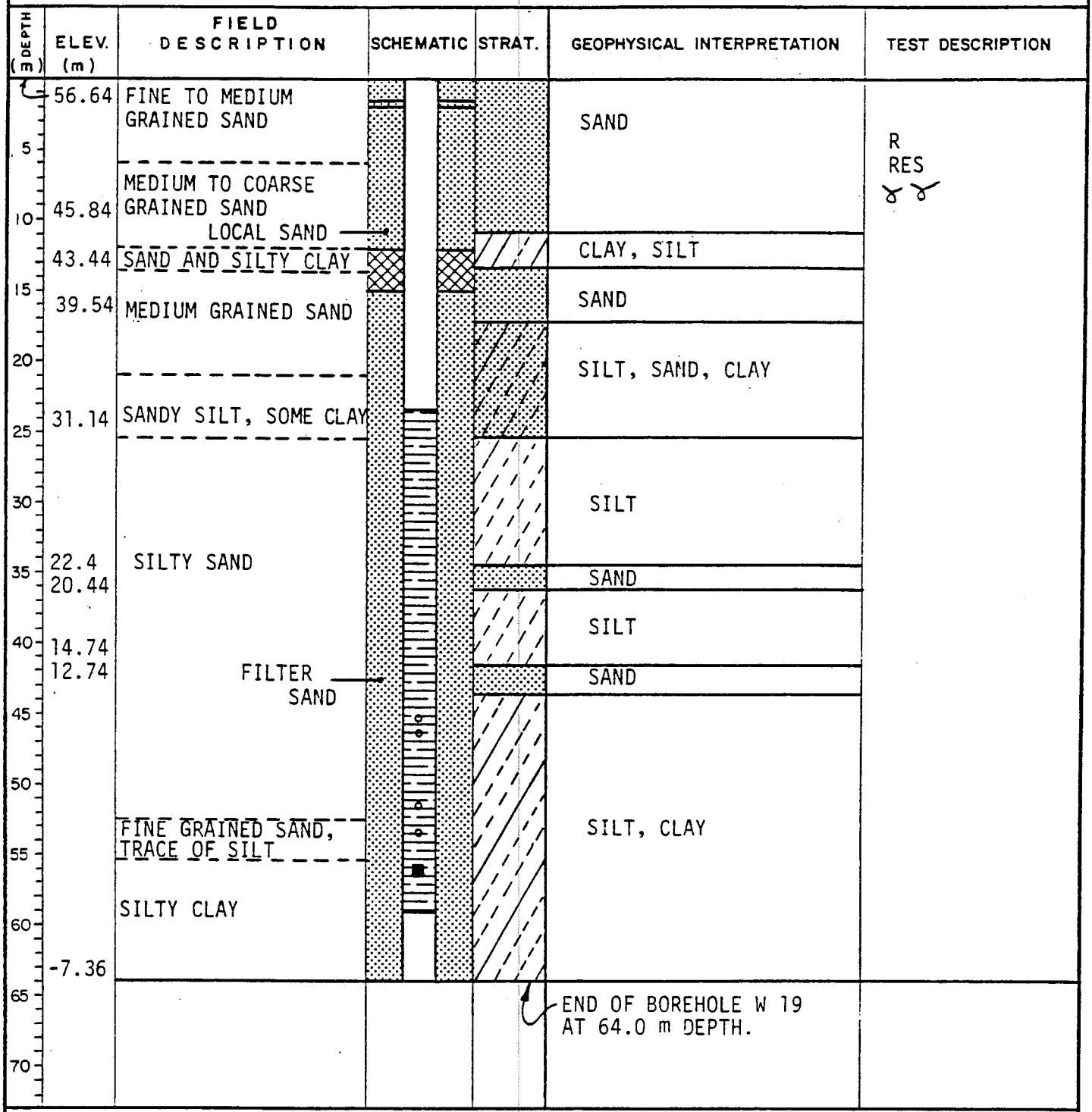
BOREHOLE No W 19

LOCATION: 5,902,965.3N 648,471.0E

DATE:

ELEVATION DATUM: GEODETIC

FIGURE No



COMMENTS: PUMP AT 0 m ELEV.
ELECTRODES AT +11, +10, +5, +3 m ELEV.
PUMP SERIAL NO. 8116542, MOTOR NO. M 10333 N

SNC-LAVALIN Newfoundland Ltd

BOREHOLE RECORD

MUSKRAT FALLS NORTH SPUR

PROJECT: TEMPORARY STABILIZATION MEASURES

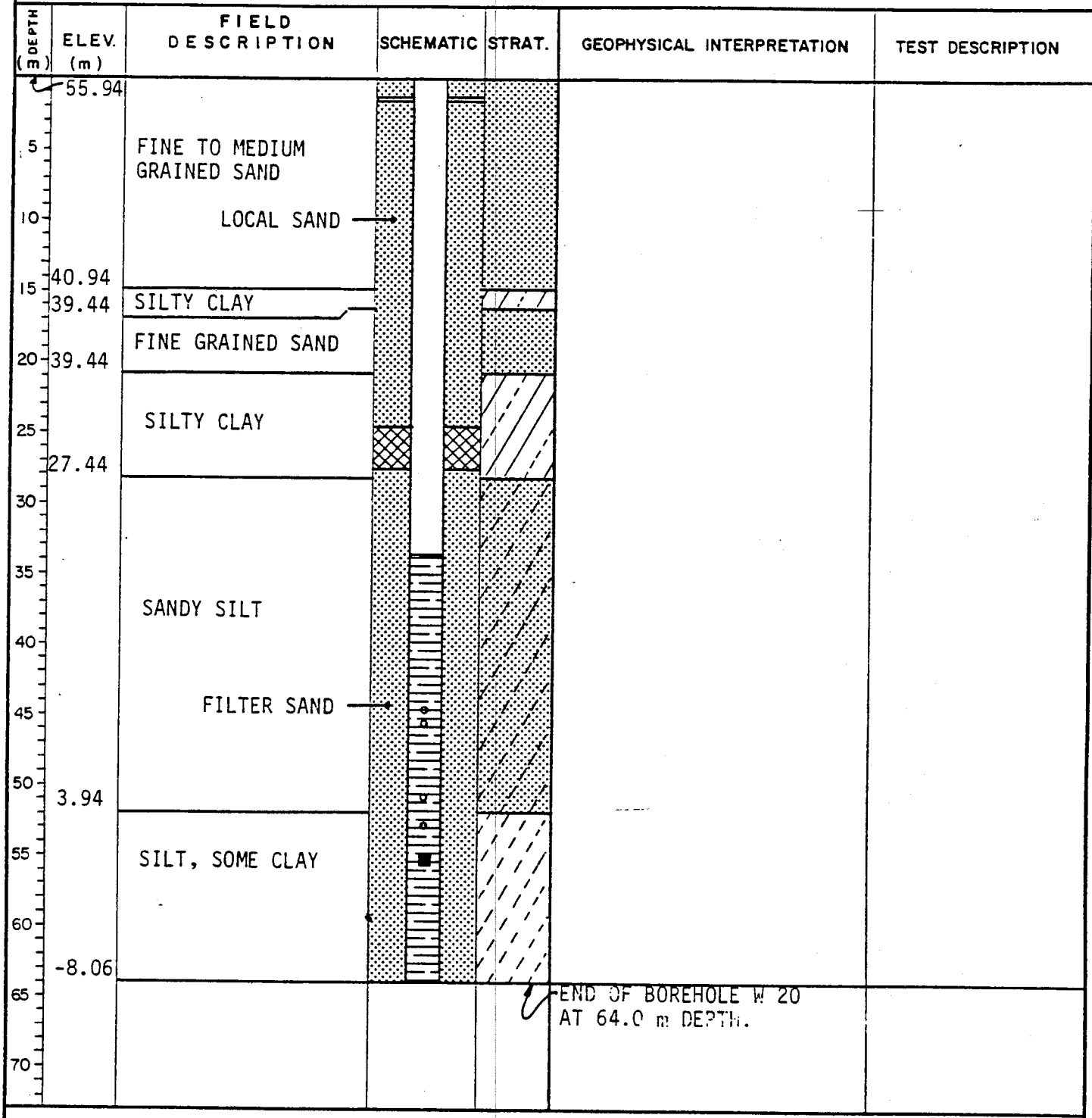
BOREHOLE No W 20

LOCATION: 5,902,988.2N 648,490.3E

DATE: AUGUST 11, 1981

ELEVATION DATUM: GEODETIC

FIGURE No



COMMENTS: PUMP AT 0 m ELEV.
ELECTRODES AT +11, +10, +5, +3 m ELEV.
GEOPHYSICAL LOGGING NOT PERFORMED ON THIS BOREHOLE
PUMP SERIAL NO. 8109559, MOTOR NO. 10262

SNC-LAVALIN Newfoundland Ltd

MUSKRAT FALLS NORTH SPUR

PROJECT: TEMPORARY STABILIZATION MEASURES

LOCATION: 5,903,011.1N 648,509.7E

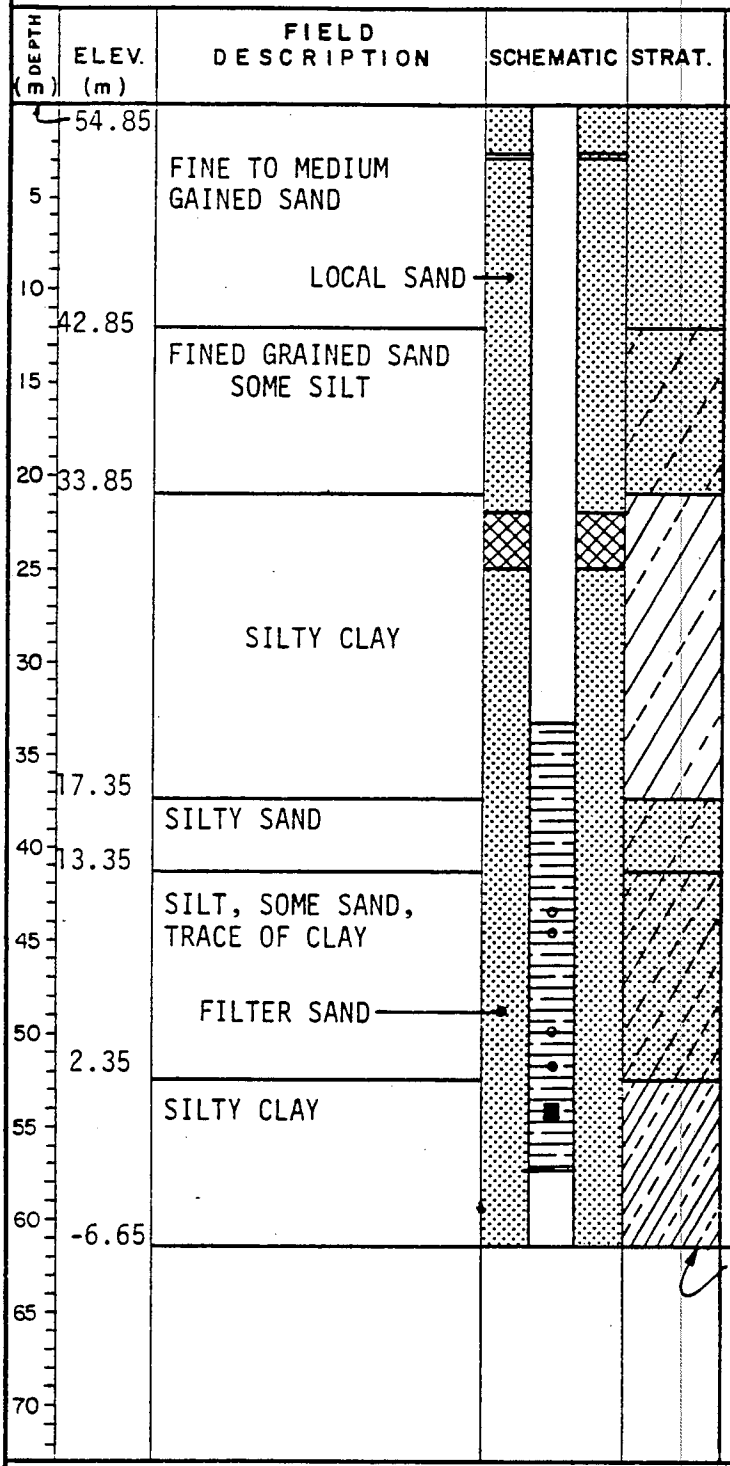
ELEVATION DATUM: GEODETIC

BOREHOLE RECORD

BOREHOLE No W 21

DATE: AUGUST 10, 1981

FIGURE No



COMMENTS: PUMP AT 0 m ELEV.
ELECTRODES AT +11, +10, +5, + 3 m ELEV.
GEOPHYSICAL LOGGING NOT PREFORMED ON THIS BOREHOLE
PUMP SERIAL NO. 8116579, MOTOR NO. M 10339 N

SNC-LAVALIN Newfoundland Ltd

BOREHOLE RECORD

MUSKRAT FALLS NORTH SPUR

PROJECT: TEMPORARY STABILIZATION MEASURES

BOREHOLE No W 22

LOCATION: 5,903,034.0N 648,529.1E

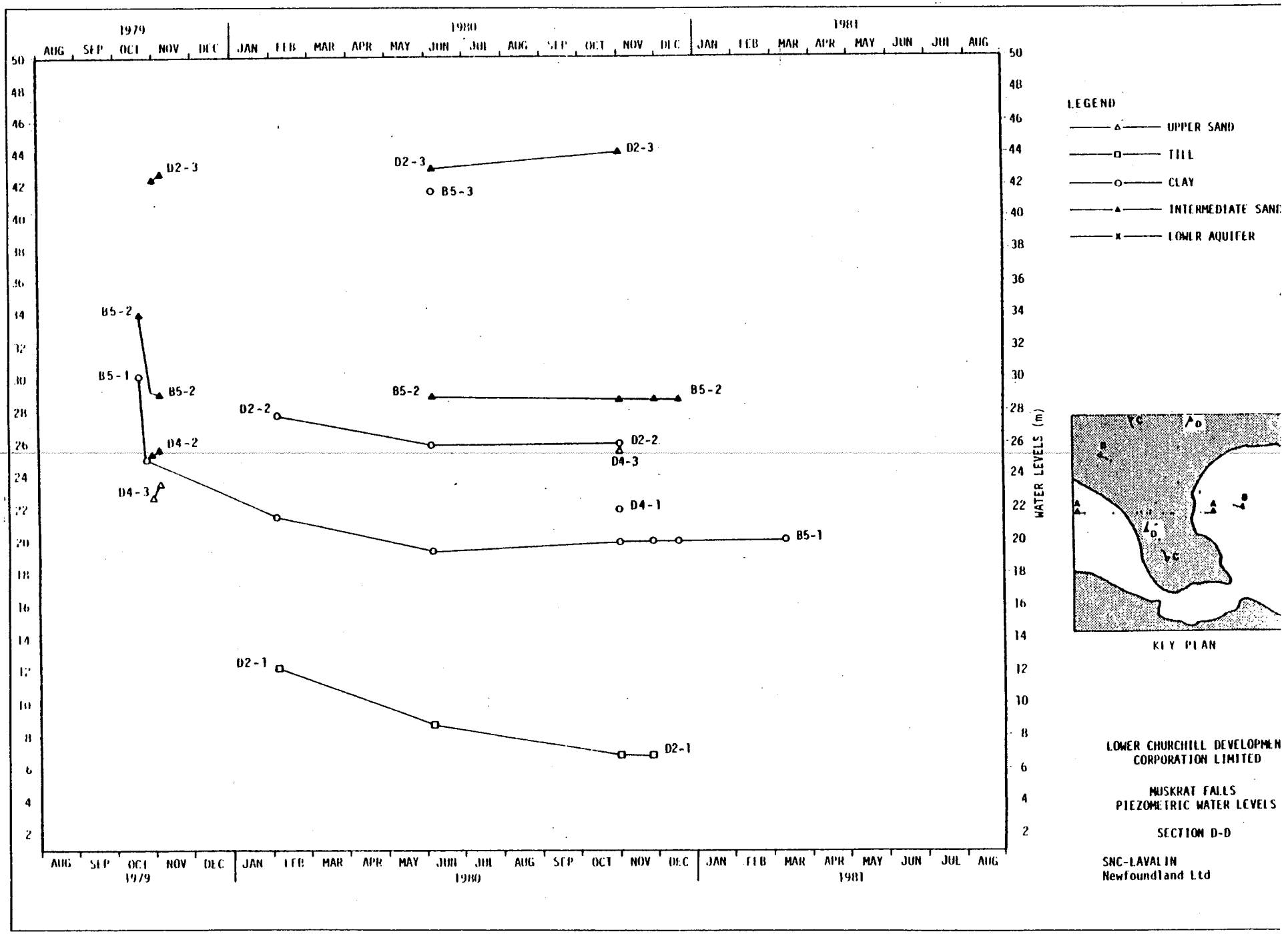
DATE: AUGUST 8, 1981

ELEVATION DATUM: GEODETIC

FIGURE No

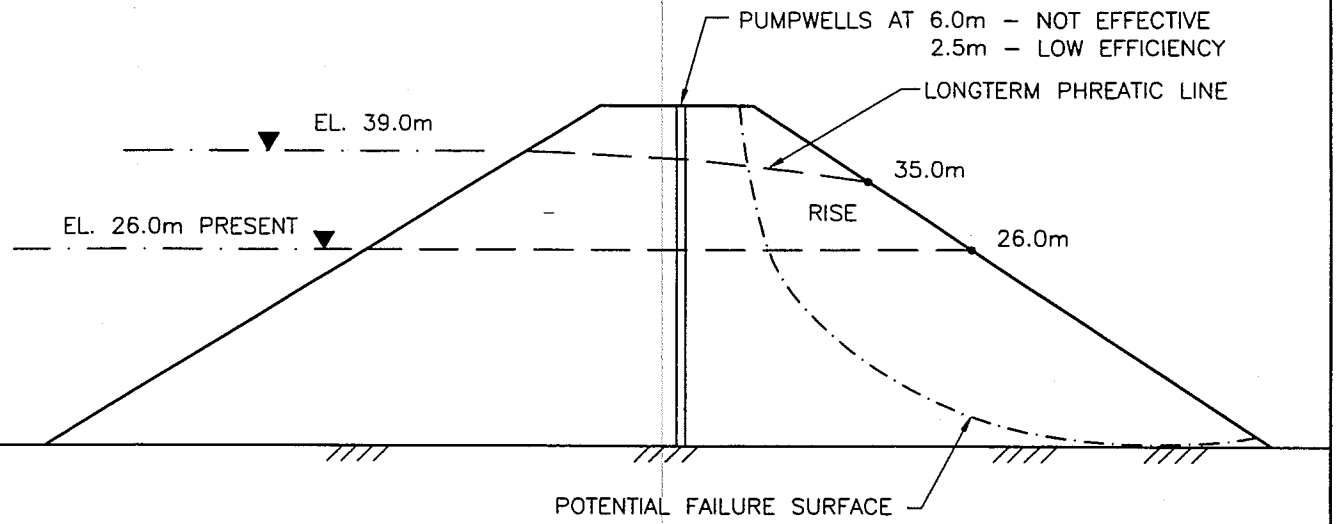
DEPTH (m)	ELEV. (m)	FIELD DESCRIPTION	SCHEMATIC	STRAT.	GEOPHYSICAL INTERPRETATION	TEST DESCRIPTION
0	52.54	MEDIUM TO COARSE GRAINED SAND				
5						
	43.54					
10		FINE GRAINED SAND, TRACE OF SILT.				
15		LOCAL SAND →				
20	31.54					
		SILTY SAND TRACE OF CLAY				
25	27.04					
30		FILTER SAND →				
35						
40		SILT, TRACE OF SAND AND CLAY				
45						
50						
55						
60	-8.46				END OF BOREHOLE W 22 AT 61.0 m DEPTH.	
65						
70						

COMMENTS: PUMP AT 0 m ELEV.
ELECTRODES AT +11, +10, +5, + 3 m ELEV.
GEOPHYSICAL LOGGING NOT PERFORMED ON THIS BOREHOLE
PUMP SERIAL NO. 8125769, MOTOR NO. M 10401 N


1979
1980
1981
AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG
WATER LEVELS (m)
D2-3
B5-3
B5-2
B5-1
D4-2
D4-3
D2-2
D4-1
D2-1
LEGEND
UPPER SAND
TILL
CLAY
INTERMEDIATE SAND
LOWER AQUIFER
KEY PLAN
LOWER CHURCHILL DEVELOPMENT CORPORATION LIMITED
MUSKRAT FALLS
PIEZOMETRIC WATER LEVELS
SECTION D-D
SNC-LAVALIN
Newfoundland Ltd

APPENDIX "II"

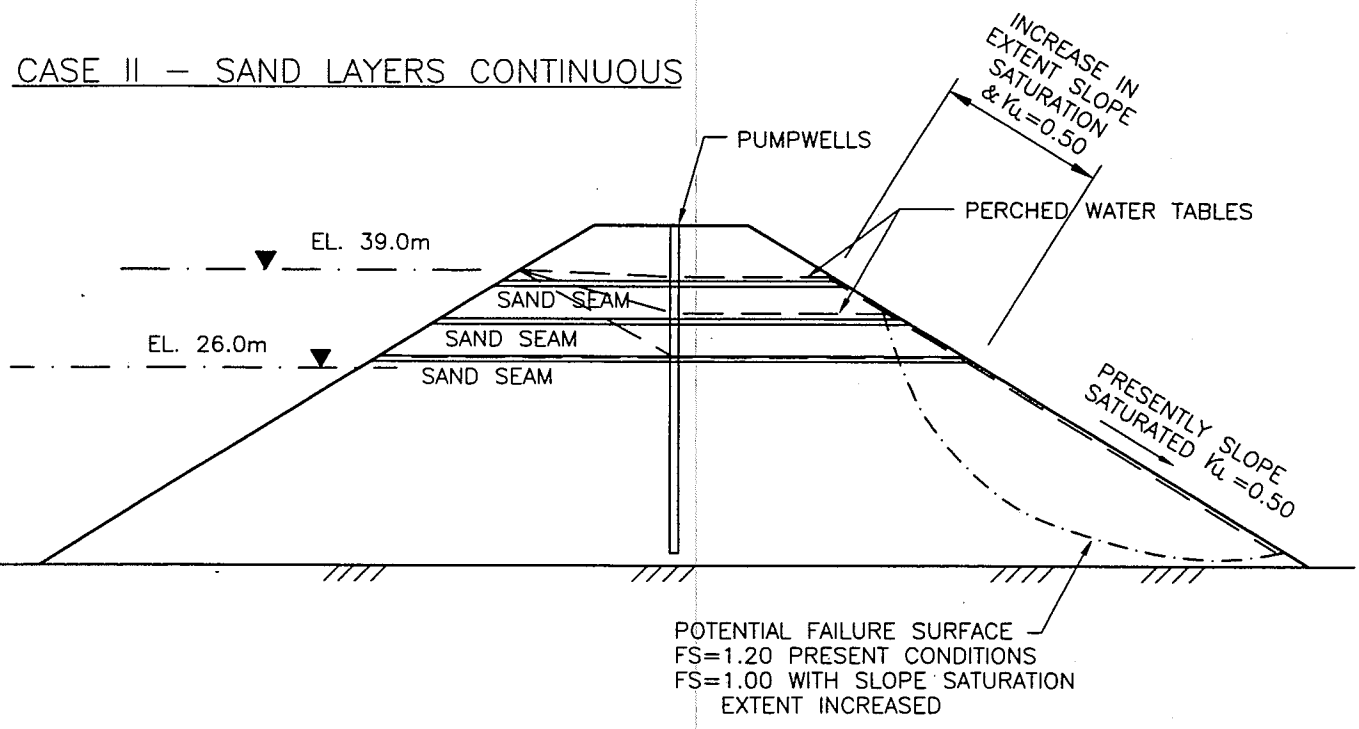
CASE I – HOMOGENEOUS UPPER CLAY $K=1\times10^{-7}$ cm/s



RISE IN GWT OF ABOUT 5.0m DECREASES STABILITY DOWNSTREAM SLOPE

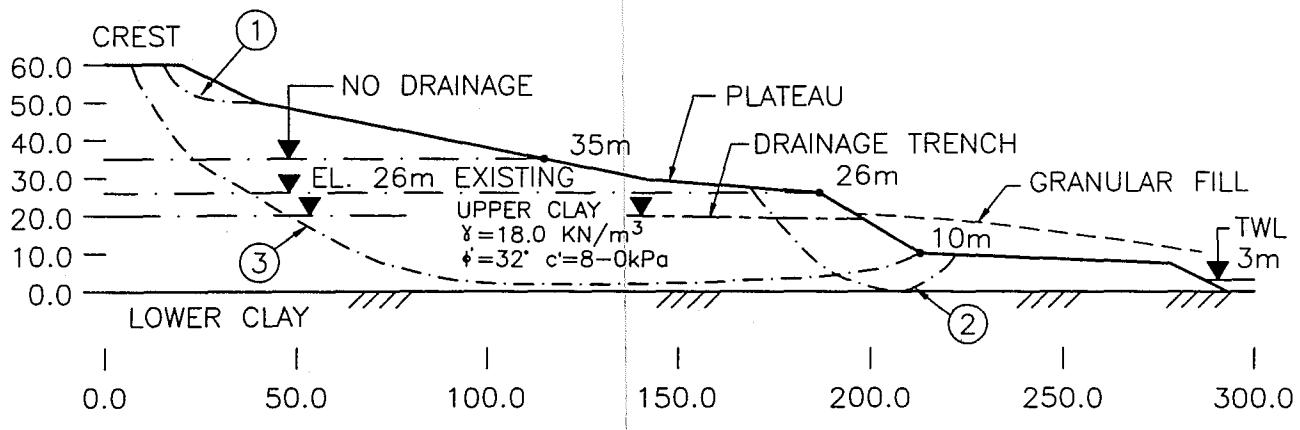
SECTION

CASE II – SAND LAYERS CONTINUOUS



SECTION

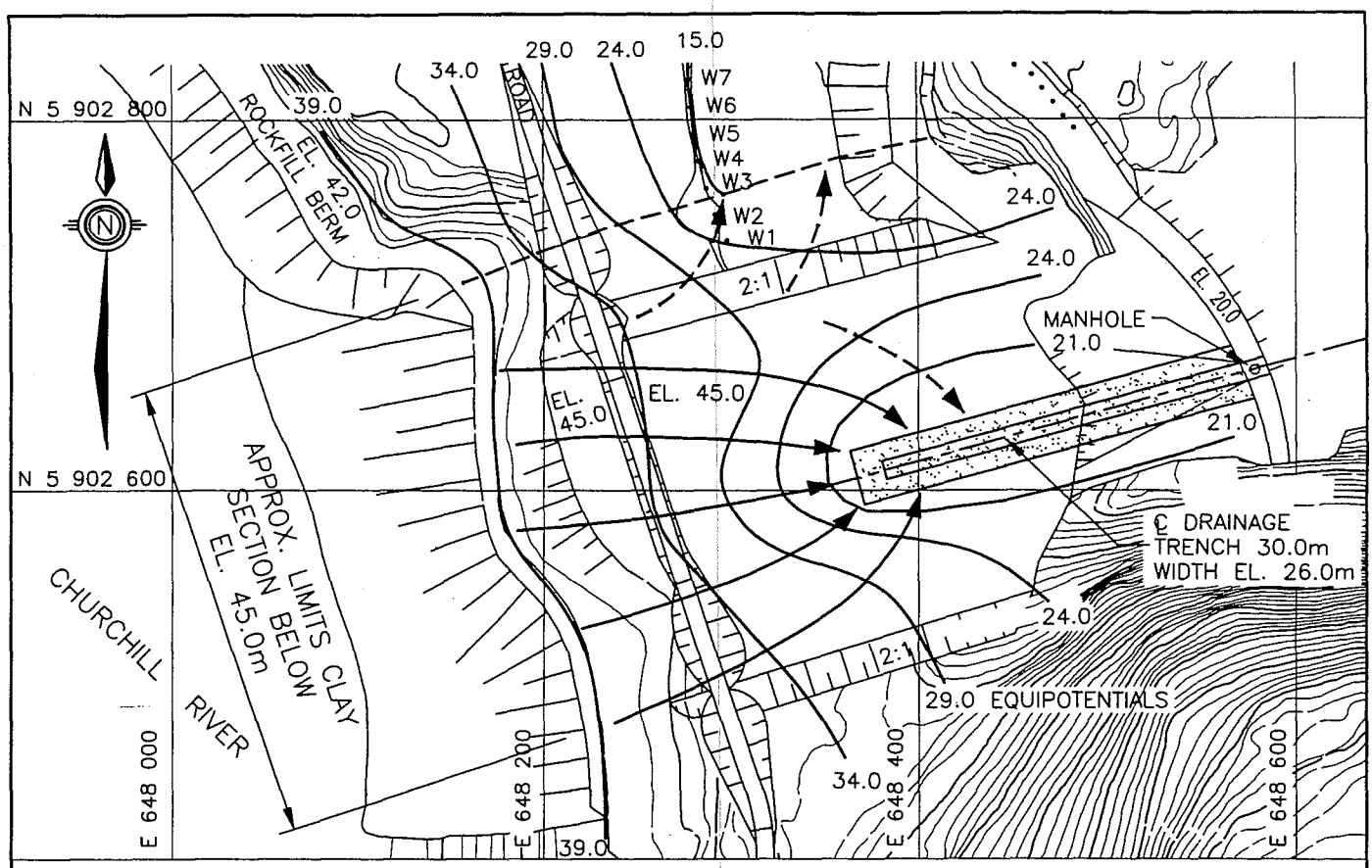
FIGURE B-1
RESULTS PROBABLE WITH PUMPWELLS IN SOUTH 200m OF SPUR



SECTION

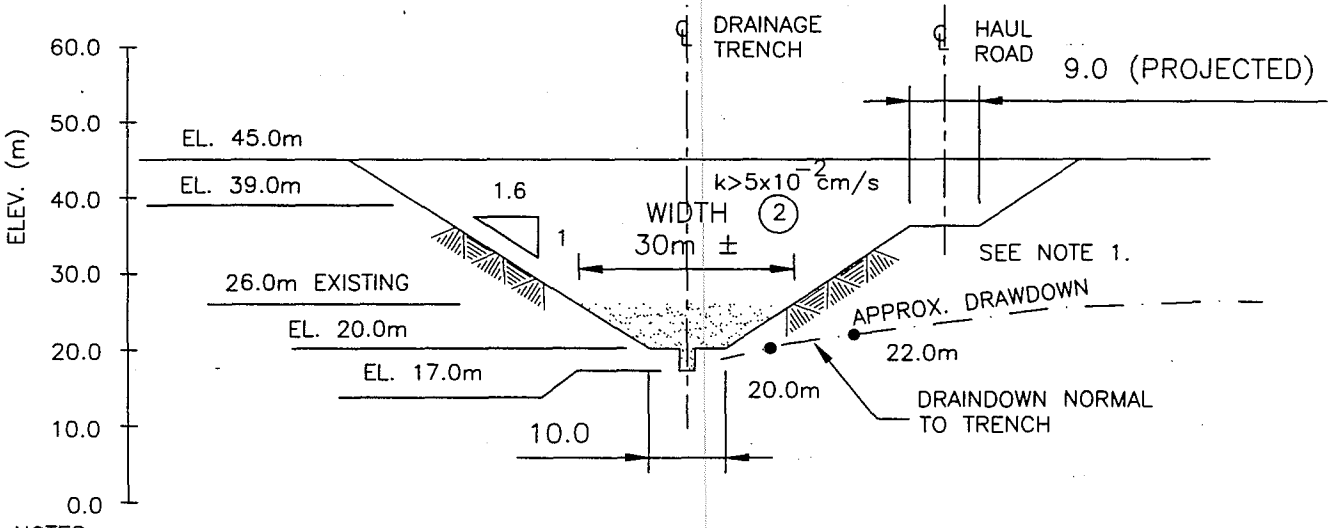
CASE	PARAMETERS φ'	c'	r_u	F.S.	COMMENTS
(1) LOCAL CREST STABILITY	32°	0	0.0	1.27	WHEN SATURATED (RAIN) LOCAL
	32°	0	0.5	0.47	STABILITY MARGINAL, MUST BE
	32°	8kPa	0.5	1.14	TOPCUT AS FLATTENING NOT VIABLE
(2) LOCAL TOE STABILITY	32°	0	0.5	0.48	COHESION (I.E. CEMENTATION) AND
		8kPa	0.5	1.08	SLIGHTLY LOWER GWT EXPLAIN PRESENT STABILITY BUT TOE EROSION WOULD TRIGGER RETROGRESSIVE SLIDE
(2) LOCAL TOE STABILITY • WITH TRENCH DRAINAGE TO EL. 20m	32°	0	0.27	1.19	10% INCREASE BUT STILL LOW
• WITH GRANULAR FILL	32°	0	0.33	1.94	OK
(3) OVERALL SLOPE	32°	0	0.29	1.80	FOR EXISTING GWT AT EL. 26m, BUT
	32°	8kPa	0.29	1.97	DOES NOT REPRESENT CRITICAL CASE WHICH IS RETROGRESSIVE TOE FAILURE.
	32°	0	0.42	1.54	INADEQUATE FOR A SLOPE IN
	32°	8kPa	0.42	1.68	SENSITIVE MARINE CLAY
(3) OVERALL SLOPE WITH TRENCH DRAINAGE TO EL. 20m	32°	0	0.31	2.01	ADEQUATE
• WITH GRANULAR FILL				>2.2	ADEQUATE

FIGURE B-2
STABILITY CHECK



FLOWNET IN PLAN (Kn=Kv)

0 100 200 300m



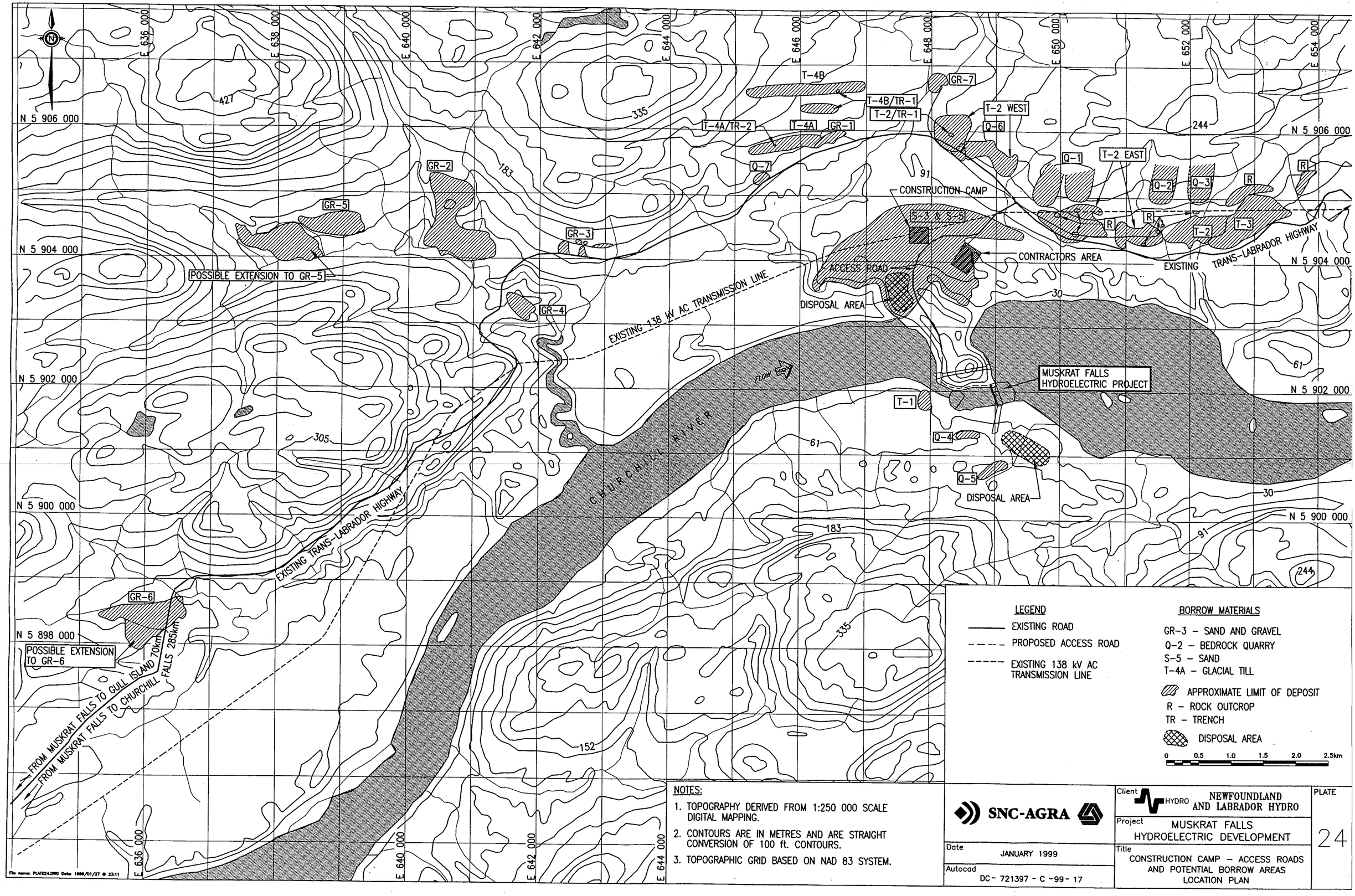
NOTES:

1. R=4x5 TIMES EXCAVATION WIDTH FROM NBR
   SAND-DRAIN SPACING SAY 5x30=150m

SECTION THROUGH DRAINAGE TRENCH

FIGURE B-3
DRAWDOWN EFFECTS OF DRAINAGE TRENCH - SECTION AND PLAN

APPENDIX "III"



LEGEND

- EXISTING ROAD
- PROPOSED ACCESS ROAD
- EXISTING 138 kV AC TRANSMISSION LINE

BORROW MATERIALS

- GR-3 – SAND AND GRAVEL
- Q-2 – BEDROCK QUARRY
- S-5 – SAND
- T-4A – GLACIAL TILL
- APPROXIMATE LIMIT OF DEPOSIT
- R – ROCK OUTCROP
- TR – TRENCH
- DISPOSAL AREA

0 0.5 1.0 1.5 2.0 2.5km

NOTES:

1. TOPOGRAPHY DERIVED FROM 1:250 000 SCALE DIGITAL MAPPING.
2. CONTOURS ARE IN METRES AND ARE STRAIGHT CONVERSION OF 100 ft. CONTOURS.
3. TOPOGRAPHIC GRID BASED ON NAD 83 SYSTEM.

SNC-AGRA

Client: HYDRO NEWFOUNDLAND AND LABRADOR HYDRO

Project: MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Date: JANUARY 1999

Autocad: DC- 721397 - C -99- 17

Title: CONSTRUCTION CAMP – ACCESS ROADS AND POTENTIAL BORROW AREAS LOCATION PLAN

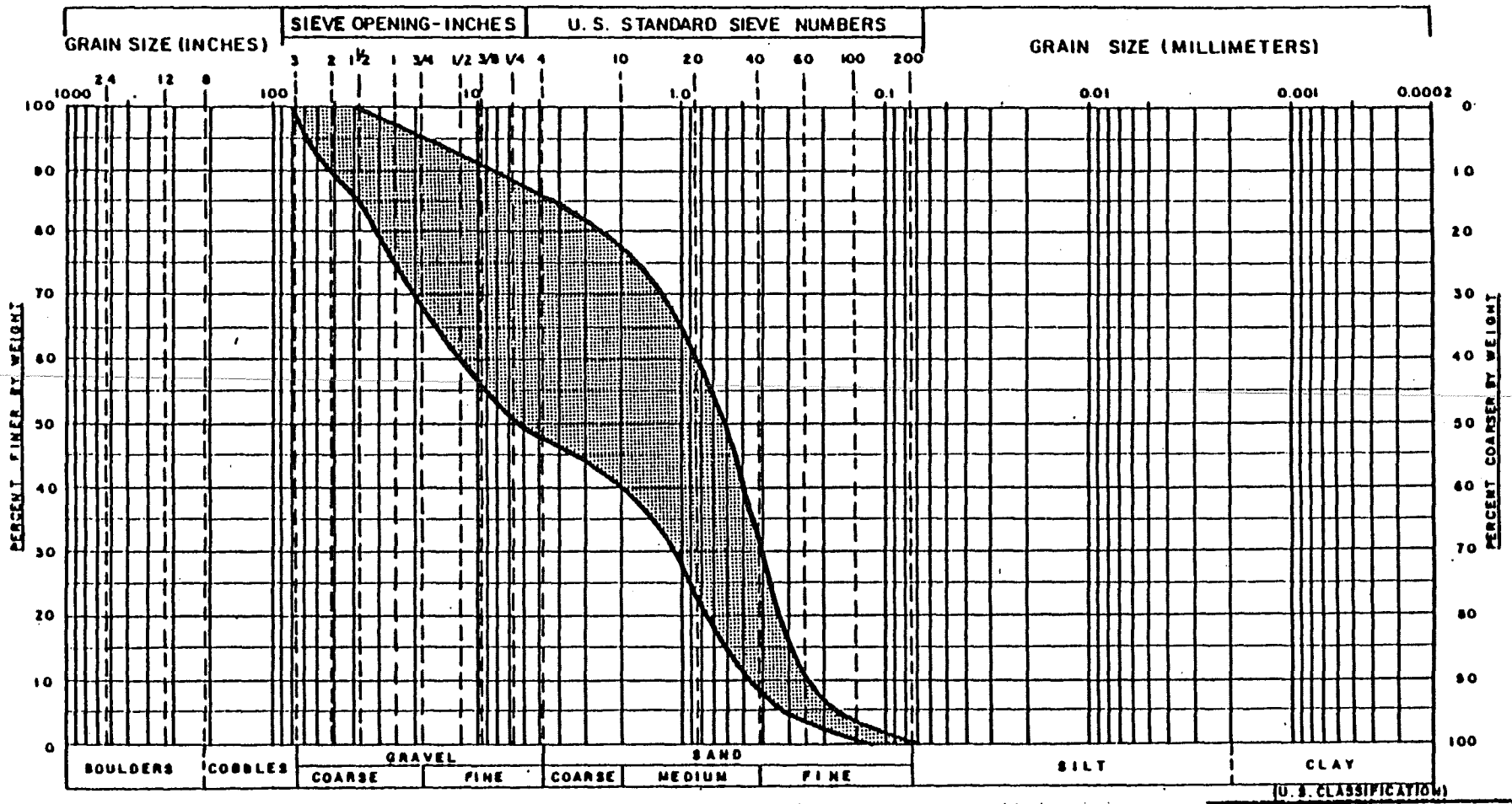
PLATE 24

File name: PLATE24.DWG Date: 1999/01/27 @ 23:11

SNC-LAVALIN
Newfoundland Ltd.
LOWER CHURCHILL
DEVELOPMENT

GRAIN-SIZE DISTRIBUTION

MUSKRAT FALLS



GRAIN SIZE GRADATION BOUNDARIES FOR SAMPLES TESTED FROM TEST PITS 1 THROUGH 10

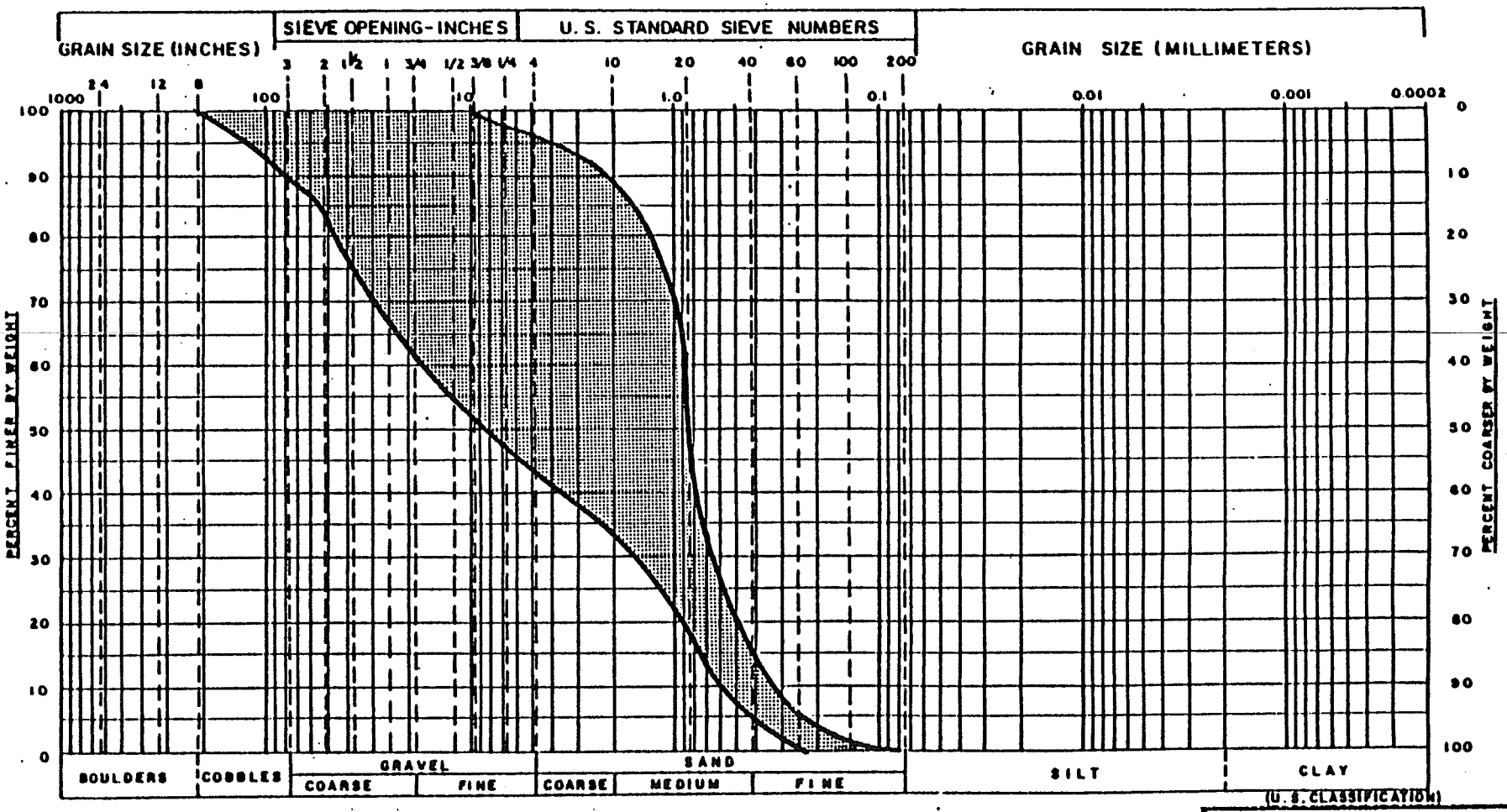
DEPOSIT GR 4

FIGURE IV-175

SNC-LAVALIN
Newfoundland Ltd.
LOWER CHURCHILL
DEVELOPMENT

GRAIN – SIZE DISTRIBUTION

MUSKRAT FALLS



GRAIN SIZE GRADATION BOUNDARIES FOR 8 SAMPLES TESTED FROM TEST PITS 1 THROUGH 8

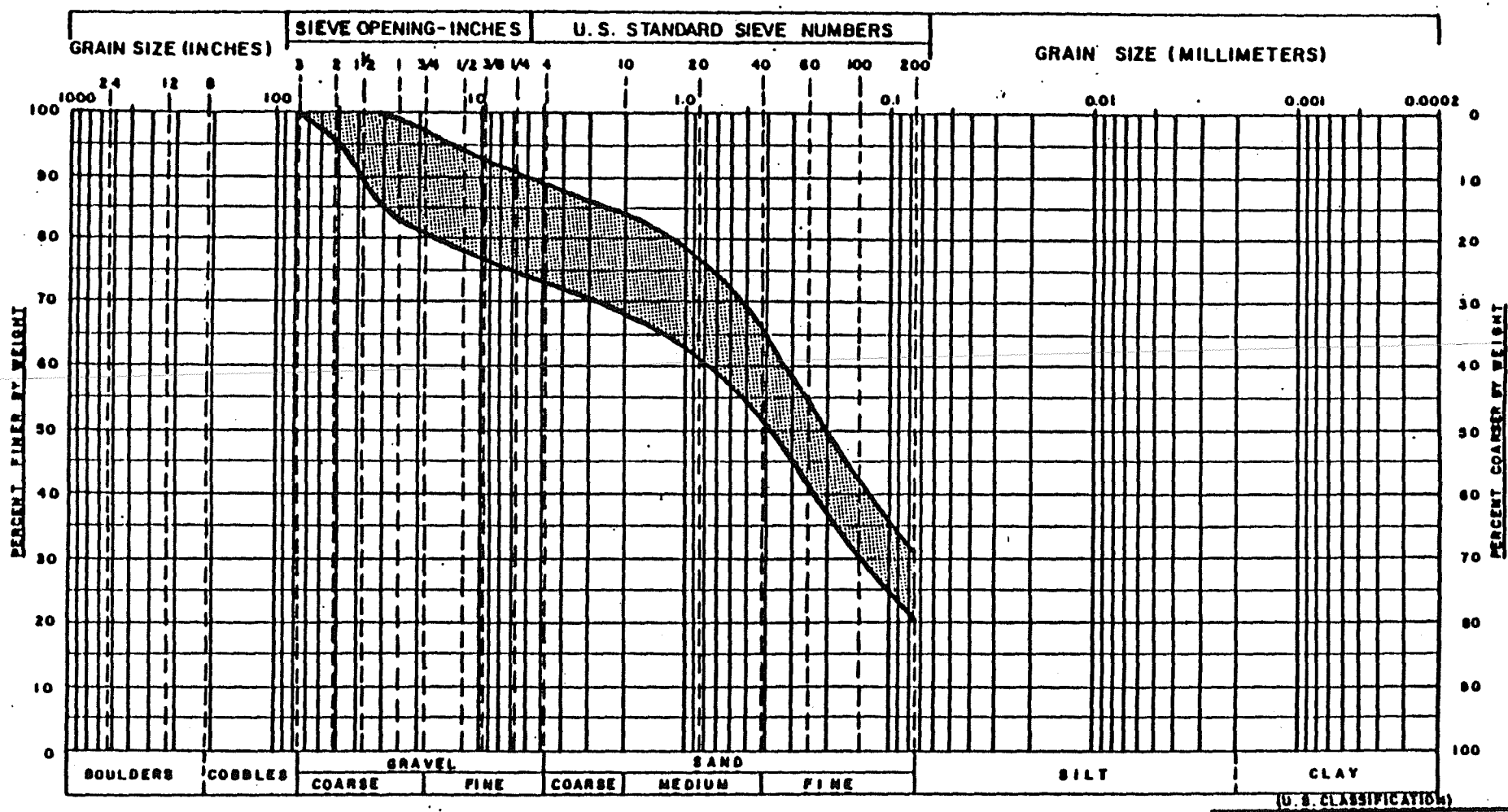
DEPOSIT GR 5

FIGURE IV-187

SNC-LAVALIN
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LOWER CHURCHILL
DEVELOPMENT

GRAIN-SIZE DISTRIBUTION

MUSKRAT FALLS



GRAIN SIZE GRADATION BOUNDARIES FOR SAMPLES TESTED FROM TRENCH 2 AND TEST PITS 1 THROUGH 9

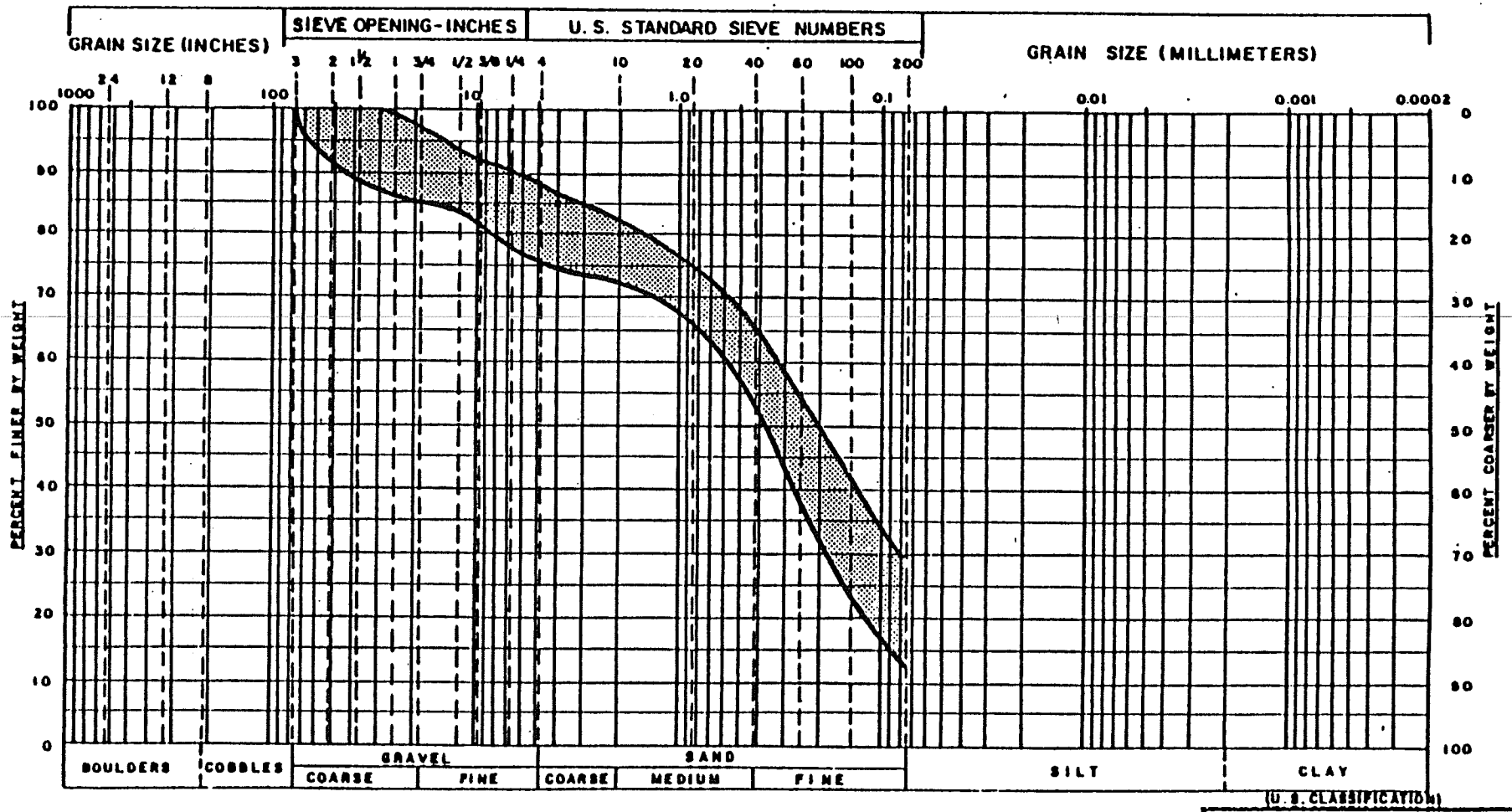
DEPOSIT T4A

FIGURE IV-20

SNC-LAVALIN
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LOWER CHURCHILL
DEVELOPMENT

GRAIN - SIZE DISTRIBUTION

MUSKRAT FALLS



GRAIN SIZE GRADATION BOUNDARIES FOR SAMPLES TESTED FROM TRENCH 1 AND TEST PITS 13 THROUGH 27.

DEPOSIT T4 B

FIGURE IV-54

APPENDIX D

CONSTRUCTION FLOOD STUDY

1.0 CONSTRUCTION FLOOD STUDY

1.1 DIVERSION FLOODS

1.1.1 Introduction

The existing large Ossokmanuan and Smallwood Reservoirs of the Churchill Falls Hydroelectric Project permit significant control of flows discharged through that project which is located upstream of Muskrat Falls. By reducing releases from Churchill Falls during peak flow periods on the Lower Churchill, it is possible to reduce the peak flows for construction floods at Muskrat Falls.

The objectives of this study were:

- to estimate construction flood magnitudes for design of river diversion facilities at Muskrat Falls;
- to design reservoir operating procedures to reduce the peak flows at Muskrat Falls, while avoiding/minimizing water (and energy losses) at Churchill Falls; and
- to test these procedures to demonstrate that the safety of Churchill Falls Dams would not be jeopardized.

This section describes the methodology used and the results obtained for the estimation of the diversion flood at Muskrat Falls.

Based on preliminary construction schedules, it was established that the 40-year return period flood was required for design of diversion facilities at Muskrat Falls.

Diversion into the Upper Churchill Basin of headwater flows from the Romaine and Saint-Jean Rivers was assumed.

1.1.2 The Churchill System

It may be useful to review the configuration of the Churchill system in order to clearly understand the methodology used in this study. The system is illustrated in Figure 1 while Table 1 presents some of the salient data.

The Ossokmanuan reservoir controls the water coming from the southern part of the Churchill River watershed, including Lake Joseph. It includes two control structures:

- Ossokmanuan control structure which is used as a spillway to spill excess water downstream of Churchill Falls powerhouse.
- Gabbro control structure which is used to transfer water from Ossokmanuan to Smallwood reservoir.

The water supply for Churchill Falls extension (CF2) will consist of diverted flows from Saint-Jean and Romaine rivers (located in the Province of Quebec) into Ossokmanuan reservoir.

The Smallwood reservoir collects water from the northern and western parts of the watershed. It also receives the flow releases from Ossokmanuan. The reservoir has one control structure: the Lobstick control structure.

Lobstick controls flow into the West Forebay which includes two other flow control structures:

- Jacopie spillway, which is the system's main spillway; and
- Whitefish control structure which controls flow into the East Forebay.

The East Forebay constitutes the forebay of Churchill Falls' plant. It also includes a spillway which would be used only in the case of a rare emergency situation involving a prolonged outage at Churchill Falls.

Table 1
The Churchill System

Basin	Sub-Basin/River	Local Drainage Area (km^2)	Cumulative Drainage Area (km^2)	
			Without Diversion	With Diversion
Diversion from the Province of Quebec	Saint-Jean River above diversion	1164	0	1164
	Romaine River above diversion	8419	0	9583
Upper Churchill	Lake Joseph	7070	7070	16653
	Ossokmanuan Reservoir	15318	22388	31971
	Smallwood Reservoir	45170	67558	77141
	West Forebay	1098	68656	78239
	East Forebay	611	69267	78850
Lower Churchill	Gull Island	19832	89099	98682
	Muskrat Falls	3256	92355	101938

1.1.3 Basic Principles of the Study

It is necessary to obtain estimates of the 20- and 40-year return period floods at Gull Island and Muskrat Falls. A simplistic method would be to carry out a statistical study of the flow measurements at Muskrat Falls and extrapolate to obtain the 20- and 40-year floods. This, however, would neglect the mode of operation of the Churchill Falls' power station which may call for spill during the flood season and add to the natural flows in the Lower Churchill River. A more realistic approach consists of estimating the floods separately on all the sub-basins, then routing these floods through Ossokmanuan and Smallwood reservoirs, based on the current flood handling procedures, and add the resulting outflow (at Ossokmanuan and Churchill Falls) to the Gull Island or Muskrat Falls local inflow. The latter approach was therefore adopted.

The application of the above methodology requires statistical analyses of flood volumes and flood peaks for:

- inflows into Ossokmanuan reservoir;
- inflows into Smallwood reservoir; and
- local inflows at Gull Island and Muskrat Falls.

The local inflow at Muskrat Falls are the flows generated in the drainage basin between Churchill Falls and Muskrat Falls. This inflow can be obtained by subtracting the flow measured at Churchill Falls from that observed at Muskrat Falls.

Due to the large storage capacities of Ossokmanuan and Smallwood reservoirs, it is expected that the flood volume will be much more important than the flood peak for Ossokmanuan and Smallwood. On the other hand, at Muskrat Falls, it is the peak flow that will dictate the size of the diversion structures.

1.1.4 Available Hydrometric Data

Table 2 summarizes the hydrometric data that was used for the study. The outflow estimations for Ossokmanuan and Smallwood reservoirs are based on the respective reservoir levels.

In Acres' 1989 Flood Handling Study, hydrographs were generated for the 100-yr, 1000-yr, 10 000-yr floods as well as for the PMF on Upper Churchill and corresponding flood volumes were estimated. By plotting (on log-log scales) the flood volumes versus the return period, it was possible to extend the graph and get an estimate of the 40-yr flood volume. The 40-yr flood hydrographs for Ossokmanuan and Smallwood were then derived, based on the shape of the 100-yr flood and on the relative areas of the two watersheds. The two flood hydrographs are shown on Figures 5 and 6.

Table 2
Available data

Source of data	Ossokmanuan	Smallwood	Churchill Falls	Muskrat Falls
Acres St. John's	Reconstituted daily inflows for the period 1976 to 1988	Reconstituted daily inflows for the period 1976 to 1988		
	Daily outflows from 87 to 98	Daily outflows from 87 to 98	Total daily outflows from 1987 to 1998	
	Daily reservoir levels from 87 to 98	Daily reservoir levels from 87 to 98		
	Hydrographs for the 100-yr, 1000-yr, 10000-yr floods and the PMF for Upper Churchill			
HYDAT			Total daily outflows from 72 to 95	Daily flows from 54 to 95

A statistical flood study was carried out for Lower Churchill. The local inflows were obtained by subtracting the Churchill Falls outflows (obtained from Acres and HYDAT) from the Muskrat Falls measurements (HYDAT). The largest measured discharge at Muskrat Falls was recorded on June 9, 1972. It is not known, however, what was the contribution of Churchill Falls in the peak flow. Table 3A presents some characteristics of these floods. The flood volumes were all computed for the period April 15 to August 15, which cover 92 days.

The Lower Churchill local flood peaks and flood volumes statistical study was carried out with data from 1975 to 1995 (19 years). The statistical study provided flood peaks and flood volumes for various return periods. Figures 2 and 3 show the results for peak discharge and flood volumes respectively. The GEV distribution was adopted for the peak discharges and the logPearson III distribution for the flood volumes.

Representative hydrographs for the 20-yr and 40-yr floods were then selected and adjusted to the computed flood peaks and volumes. In order to combine the worst realistic conditions for peak flow at Muskrat Falls, the date of the peak discharge was selected as June 9, which is the date of the maximum discharge ever recorded at Muskrat Falls (1972 spring flood).

Components of the 40-yr flood hydrograph for Muskrat Falls is shown in Figure 7. Table 3B summarizes the results for all sites. The Lower Churchill hydrographs shown in Figure 5 includes the local inflow from the West and East Forebays as these inflows are also unregulated.

Table 3A
Characteristics of Floods on Lower Churchill

Year	Q_{max} (m^3/s)	Date	Volume (hm^3)
1975	4270	June 5	10 900
1976	3490	May 19	10 000
1977	3260	June 5	11 600
1978	4210	May 21	11 400
1979	3050	May 4	10 200
1980	3610	May 23	11 700
1981	3910	June 2	12 500
1982	3690	June 6	10 000
1983	4000	May 2	10 200
1984	3220	May 28	11 300
1985	3520	June 5	9 500
1986	1970	May 21	8 500
1987	1620	May 20	7 700
1988	2600	May 25	8 200
1991	1920	May 29	7 600
1992	2560	June 3	8 000
1993	3010	May 23	8 000
1994	3080	May 22	9 300
1995	2680	May 17	7 500

Table 3B
Statistical sub-basin flood estimates

Sub-basin	Flood peak (m^3/s)	Flood volume (hm^3)
	40-yr	40-yr
Ossokmanuan	3080	11130
Smallwood	6380	22270
Gull Island	3790	10990
Muskrat Falls	4410	12800

1.1.5 Current Flood Handling Procedures

Flood handling procedures for Churchill Falls have been developed by Acres in 1989. These procedures take into account the amount of reservoir inflow that can be expected in the coming freshet. The forecasting of freshet volume is based on snow course measurements carried out at periodically at representative locations in the Churchill Falls watershed.

Based on predicted inflow, a desired May 1 reservoir level is calculated which will provide adequate storage for the expected inflow volume. The current reservoir level and expected

power flow are then used to determine the necessary forced production or pre-spill volume (if any) and time to initiate such operations.

Figure 4 shows the rule curve for both Ossokmanuan and Smallwood.

For the Ossokmanuan reservoir, the target level on May 1 is 475.03[1] m, and it applies to all expected floods. This reservoir level must be maintained between May 1 up to June 13. This is achieved initially by transfering water from Ossokmanuan to Smallwood reservoir through the Gabbro control structure. If the natural inflows to Smallwood become too large, Gabbro must be closed. The reservoir level is then controlled by discharging water into the Churchill River through the Ossokmanuan control structure.

For the Smallwood reservoir, the target level on May 1 is variable. It has been set to 469.68 m for the PMF and to 472.26 m for the mean annual flood. For all the other floods, the reservoir level on May 1 is established between these two limits. The reservoir level is maintained at the target level, by discharging a flow equal to the natural inflow plus the Gabbro discharge, up to June 15 for large expected floods and to June 10 for smaller floods. After these dates, the reservoir level is gradually raised to the full supply level (FSL = 472.74 m) which is reached around July 10 for large floods and July 1 for smaller floods.

Figures 5, 6 and 7 show the result of the present water management at Ossokmanuan, Smallwood and Muskrat Falls when the 40-yr flood occurs in both the Upper and Lower Churchill watersheds. With the current procedures, the peak discharge at Muskrat Falls reaches 8 800 m^3/s. The main reason for this high peak discharge is the obligation to release water from Ossokmanuan and Jacopie spillways to control reservoir levels at Ossokmanuan and Smallwood up to mid-June.

1.1.6 Actual Reservoir Management from 1987 to 1998

The procedures defined above have generally been applied at Ossokmanuan reservoir. For the last 12 years (1987 to 1998), the average level on May 1 was 474.64 m. Only twice has it exceeded the 475.03 m target: 475.21 in 1995 and 475.16 in 1998.

Reservoir management at Smallwood has, however, been very different from the established flood handling procedures for the last 12 years. The observed reservoir level on May 1 has always been lower than the target level for the PMF. The average value for these years was 465.68 m and the highest observed level on May 1 was 467.41 m in 1993. This situation is exceptional and is attributed, mainly, to lower than average annual inflows over that period.

Simulations have shown that, if the above situation still exists during the critical construction period, no changes will be necessary at Churchill Falls for flood handling except for a reduction in turbined flow during the peak period of the flood on Lower Churchill.

[1] Customary water levels are cited in Appendix D, for consistency with current operating practice at Churchill Falls; to obtain geodetric levels, 1.85 m must be subtracted from the levels cited.

1.1.7 Proposed Procedures for the Diversion Floods

Figures 5 to 7 show that with the 40-yr flood in Upper and Lower Churchill and with the current procedures, the peak discharge at Muskrat Falls reaches 8 800 m^3/s. Of this total, nearly 4 000 m^3/s is due to flow releases from Ossokmanuan and Lobstick control structures. Adjustments to the operating rules of the Upper Churchill have, therefore, been investigated with the purpose of avoiding spill at Upper Churchill during medium to high floods, while:

- maintaining full safety against extreme floods; and
- guaranteeing that no water is spilled that otherwise could be turbined.

The May 1 snow accumulation would be used as an indicator of the magnitude of the incoming freshet. If the amount of snow accumulated on the ground shows that a very large to extreme flood is likely to occur (1 in 10 000 years to PMF), then spill would be authorized at Upper Churchill in accordance with the current flood handling procedures. Otherwise, the modified procedures would apply.

Preliminary estimates show that the 1 in 10 000 year flood could be created by snow accumulation between 400 and 520 mm. The PMF could be created by snow accumulation in excess of 600 mm.

The proposed modifications, which are presented below, would be applied only if the snowpack water equivalent (estimated from snow course measurements) in mid-April is less than 500 mm. When the snowpack water equivalent exceeds this amount, the current flood handling procedures (as of Acres, 1989) would be applied because the risk of occurrence of the PMF would be significant and all means should be applied to maintain the safety of the Upper Churchill system. This could result in submergence of the cofferdams at Muskrat Falls, but this is an acceptable risk for temporary diversion facilities; because the risk of this situation occurring is remote, much rarer than 1 in 40 years.

For snow accumulation with less than 500 mm water equivalent, simulations show that it is possible to pass the incoming flood without spilling at Upper Churchill, while reducing flow releases from Churchill Falls during critical peak flow periods on the Lower Churchill.

The proposed modified flood handling procedures are as follows (see also Table 4):

Ossokmanuan:

- if the snow accumulation in Upper Churchill exceeds 300 mm water equivalent, no water transfer from Saint-Jean and Romaine rivers is allowed during the flood season, in the diversion period at Muskrat Falls (note: this value is to be further refined, as explained in Section 1.1.10);
- maintain the water level in Ossokmanuan reservoir at or below the current rule curve by continuously transfering water to Smallwood through the Gabbro control structure;
- do not use the Ossokmanuan control structure to release water into Churchill River; and

- allow the Ossokmanuan reservoir level to rise up to the maximum flood surcharge level of 480.40 m on Ossokmanuan Lake for the construction period.

Table 4
Modified Flood Handling Procedures

Applicable only if the amount of snow on ground on May 1 is less than 500 mm water equivalent

Ossokmanuan Reservoir				
Inflow from Romaine and Saint-Jean Rivers	**Rule Curve**	**Outflow Through Gabbro Control Structure**	**Outflow Through Ossokmanuan Control Structure**	**Maximum Allowable Reservoir Level**
Yes if snowpack is less than 300 mm No if snowpack is more than 300 mm	Unchanged, see Figure 2	Yes	No	480.40 m
Smallwood Reservoir				
Inflow from Gabbro	**Rule Curve**	**Outflow through Lobstick**		**Maximum Allowable**
		Turbined (CF-2)	Spilled (Jacopie)	**Reservoir Level**
Yes	Modified, see Figure 2	2 300 m^3/s (except during peak flow on Lower Churchill)	No	473.23 m

Smallwood

- lower the target reservoir level on May 1 if snow accumulation in Upper Churchill exceeds 300 mm water equivalent (preliminary evaluation shows that a starting reservoir level set at 468.50 m is adequate to pass Upper Churchill floods up to the 1/10 000 year flood without spilling, but further investigation might show that the same objective could be achieved with

a higher starting level (forced production [2] or pre-spilling[3] could be necessary to reach the target level on May 1);

- allow continuous water transfer from Ossokmanuan to Smallwood through the Gabbro control structure for the duration of the flood;
- only the flow requested for power generation will be released from Lobstick control structure;
- the turbined flow for the entire flood season will be 2 300 m^3/s (this is about 95% of the maximum turbinable discharge at Churchill Falls after Churchill Falls extension has been completed);
- the turbined discharge at Churchill Falls will be reduced stepwise, based on the water levels observed at Muskrat Falls, to skim the flood peak at these locations. Maximum generation at Churchill Falls will resume after flood recession has started; and
- allow the Smallwood reservoir level to rise up to the maximum flood surcharge level of 473.23 m during the construction period.

1.1.8. Application Of The Proposed Procedures

Figures 8 to 11 show the results of proposed flood management at Ossokmanuan, Smallwood, Gull Island and Mukrat Falls in case the 40-yr flood is observed on both the Upper and Lower Churchill watersheds. It can be seen that no spill is needed either at Ossokmanuan or at Jacopie spillways. All the outflow from Lobstick is turbined. By controlling the turbined discharge at Churchill Falls during the peak of the flood on the Lower Churchill, it is possible to limit the flow to 5 300 m^3/s at Muskrat Falls. The starting reservoir level on May 1 was set at 468.50 m at Smallwood. The maximum computed water levels in the Ossokmanuan and Smallwood reservoirs are respectively 479.15 m and 471.61 m.

Even though the above results show that the proposed adjustments to the flood handling procedures allow the passage of the 40-yr flood without extra spilling, it is necessary to verify that these procedures could handle larger than expected floods in Upper Churchill. Figures 12 to 17 show the results of the simulations if the 100-yr and 10 000-yr floods are observed on Upper Churchill (the combination of the 10 000-yr on Upper Churchill and 40-yr flood on Lower Churchill could be possible if the snowpack on both basins, at the beginning of the flood, is significant and if a severe rainfall occurs on Upper Churchill only). The starting water level at Smallwood was set at 468.50 m for both these cases. It is seen that it is still possible to limit the peak discharge at Muskrat Falls at 5 300 m^3/s without losing any water by spillage. The maximum computed reservoir levels for the 10 000-yr flood are respectively 480.00 m at Ossokmanuan and 473.10 m at Smallwood.

[2] "Forced Production" means operating Churchill Falls at the maximum plant output so as to lower the water level of Smallwood Reservoir as quickly as possible.

[3] "Prespilling" means opening Jacopie Spillway to release flow in a controlled manner to lower the water level of Smallwood Reservoir, should forced production be inadequate to provide the desired reservoir volume adjustment.

For all the cases described above, no spillage is needed from the Upper Churchill reservoirs and, therefore, no loss of production is observed at Churchill Falls. The maximum water levels computed are also below the maximum allowed at both Ossokmanuan and Smallwood reservoirs.

Whether forced production (before May 1) will be needed or not will depend on the total precipitation over the watershed during mid-fall to mid-spring. Table 5 presents the water balance from November 1 to April 30 in the Upper Churchill reservoirs for two hydrological conditions:

Table 5
Water Balance in Upper Churchill Reservoirs
Between November 1 and April 30

Case	Storage on Oct 31	From Nov 1 to Apr 30		Storage on April 30	Reservoir Elevation on Apr 30	
		Total Inflow	Total Outflow		Ossokmanuan	Smallwood
Average Flood	31 810 hm^3	9 150 hm^3	36 170 hm^3	4 790 hm^3	475.03 m	467.63 m
40-yr Flood	31 810 hm^3	10 750 hm^3	36 170 hm^3	6 390 hm^3	475.03 m	468.02 m

Notes:

- The first column describes the flood case.
- The second column gives the total storage in Ossokmanuan and Smallwood reservoirs on October 31, assuming full supply level (FSL).
- The third column shows the total inflow from Nov 1 to April 30, and it includes the flow diverted from Saint-Jean and Romaine rivers. For the average flood, it assumes an average precipitation of 425 mm, of which 394 mm is snow. This is based on the average precipitation recorded at the Churchill Falls A meteorological station. Twenty-five percent of this precipitation is assumed to run off during the winter.
- The fourth column shows the total outflow from Nov 1 to April 30 from the Churchill Falls power plant, asuuming a contant turbine discharge of 2 300 m^3/s. It is also assumed that Churchill Falls extension (CF-2) is in operation.
- The fifth column is the water balance in the reservoirs: col (5) = col (2) + col (3) - col (4).
- Columns 6 and 7 show the resulting reservoir levels in Ossokmanuan and Smallwood reservoirs on May 1.

Table 5 confirms that it is possible to bring the Ossokmanuan and Smallwood reservoirs to their desired levels for May 1 without spilling, even for the 40-yr flood on Upper Churchill.

Since the Churchill Falls power plant is connected to Hydro-Quebec's grid, the recommended decrease in turbined flow at Churchill Falls during the peak period of the Lower Churchill flood

(about three weeks) would be offset by higher generation from Hydro-Quebec's other plants. The reverse would apply for the fall and winter season preceding the spring flood season: Churchill Falls would then generate at near maximum capacity while other Hydro-Quebec reservoirs would accumulate water.

A further verification of the modified flood handling procedures has been made, which shows that it is even possible to pass the 10 000 year return period flood on the Upper Churchill with the target reservoir level on Smallwood at 469.68 m on May 1, without exceeding 5 300 m^3/s at Muskrat Falls. This results in slightly higher maximum water levels on both Ossokmanuan (480.38 m) and Smallwood (473.18 m), but these levels never exceed the maximum allowed at both places.

The simulations, therefore, show that with proper management of the control structures, it is possible to limit the peak 40-yr flood discharge at Gull Island and Muskrat Falls. Diversion facilities should be designed for a maximum flow of 5 300 m^3/s at Muskrat Falls.

The results for Muskrat Falls apply whether Gull Island has been built or not because there is no significant routing effect at Gull Island during large floods.

1.1.9 Conclusions

A statistical flood study was conducted to estimate the peak discharges and flood volumes for Ossokmanuan and Smallwood reservoirs as well as for the local inflows at Gull Island and Muskrat Falls for different return periods. These floods were then routed throughout the reservoirs following prescribed rules and flood characteristics at Muskrat Falls were determined.

The current flood handling procedures were analyzed and a strict application of these procedures leads to a peak discharge at Muskrat Falls of about 8 800 m^3/s for the 40-yr flood on Upper Churchill. Flood routing simulations based on modified procedures showed that it was possible to lower the peak flow on the Lower Churchill during construction by appropriately managing the outflows from Churchill Falls without encroaching on the safety of the existing Churchill Falls dykes.

The main change proposed to the current flood handling procedures is to lower the target reservoir level at Smallwood on May 1 to elevation 468.50 m, when snow accumulation on the ground is in excess of 300 mm. Another proposed change is that no flow diversion be permitted from Saint-Jean and Romaine rivers during construction of Gull Island and Muskrat Falls if the snowpack water equivalent at the beginning of the flood exceeds 300 mm. It is also proposed that the Ossokmanuan control structure not be used, and to allow higher maximum reservoir levels in Ossokmanuan and Smallwood, during the passage of the floods. With these new rules, it appears possible to simultaneously pass all floods, up to the 10 000 year return period, on Upper Churchill and the 40-yr flood on Lower Churchill with a maximum discharge of 5 300 m^3/s at Muskrat Falls.

If the snowpack water equivalent in mid-April exceeds 500 mm, the current flood handling procedures would be applied and, therefore, the diversion capacity at Gull Island and Muskrat Falls would be exceeded. However, this would be an event greatly in excess of 1 in 40 years.

These studies have also shown that no flood routing effect is possible at Gull Island for large floods before or after its construction.

1.1.10 Recommendations

The examples described above show that, with the presently available hydrometric information, it is possible to control the peak discharge at Muskrat Falls during construction. It is recommended that the analysis be refined at a later stage when more and better hydrologic information becomes available.

It will be important, for example, to analyze in detail the evolution of the snowpack through the winter and especially in the few weeks preceding snowmelt. The work presently done by Hydro-Quebec in reconstituting the inflows into Ossokmanuan and Smallwood reservoirs should be used later to re-evaluate the flood peaks and flood volumes for all sub-basins for different return periods.

For the present final feasibility studies, it is recommended to design the diversion structures for peak discharges of 5 300 m^3/s at Muskrat Falls.

In the event that Churchill Falls Extension (CF2) is not constructed prior to Muskrat Falls, the recommended diversion design flood would remain unchanged. However, without CF2, the forced production flow capacity would be reduced and the required water level adjustments for Smallwood Reservoir may require some prespilling and consequent loss in energy production at Churchill Falls. Assessment of this issue should be undertaken during the final design stage.

N

BIRCH LAKE

OVERFLOW LAKE

MICHIKAMAU LAKE

SAIL DIKES

SAIL LAKE

KASHESHIBAW LAKE

ORMA LAKE

MACKENZIE LAKE

ORMA DIKES

LOBSTICK LAKE

LOBSTICK CONTROL STRUCTURE

SANDGIRT LAKE

SANDGIRT-LOBSTICK DIKES

TO ESKER

195 KILOMETRES FROM POWER INSTALLATIONS AREA TO ESKER

GABBRO CONTROL STRUCTURE

WEST FOREBAY

FLOUR LAKE

CHURCHILL RIVER

JACOPIE LAKE

WHITEFISH FALLS CONTROL STRUCTURE

CHURCHILL FALLS ROAD

JACOPIE SPILLWAY

TWIN FALLS AIR STRIP

EAST FOREBAY

ORMA ACCESS ROAD

GABBRO LAKE

CHURCHILL FALLS

AIRPORT

FOREBAY SPILLWAY

TRANS LABRADOR HIGHWAY

OSSOKMANUAN CONTROL STRUCTURE

UNKNOWN RIVER

TWIN FALLS POWER DEVELOPMENT

CHURCHILL RIVER

OSSOKMANUAN LAKE

CHURCHILL FALLS POWER INSTALLATIONS

230 KV TRANSMISSION LINES TO WABUSH AND LABRADOR CITY

735 KV TRANSMISSION LINES

FOR CLARITY OF PRESENTATION, ONLY MAJOR LAKES AND RIVERS ARE SHOWN

LEGEND

- SMALLWOOD RESERVOIR MAXIMUM LEVEL EL. 472.7
- OSSOKMANUAN RESERVOIR MAXIMUM LEVEL EL. 479.1
- WEST FOREBAY MAXIMUM LEVEL EL. 452.9
- EAST FOREBAY MAXIMUM LEVEL EL. 448.5
- CONTROL STRUCTURES
- DIKES

0 10 20 30
KILOMETRES

NOTE:
Elevations shown are based on local (Churchill Falls) datum. For Geodetic datum subtract 1.85m from these elevations.

Reference: Acres Consulting Services LTD. Churchill River Power and Energy Studies 1978 Report

CHURCHILL FALLS DEVELOPMENT PLAN OF DEVELOPMENT

FIGURE 1

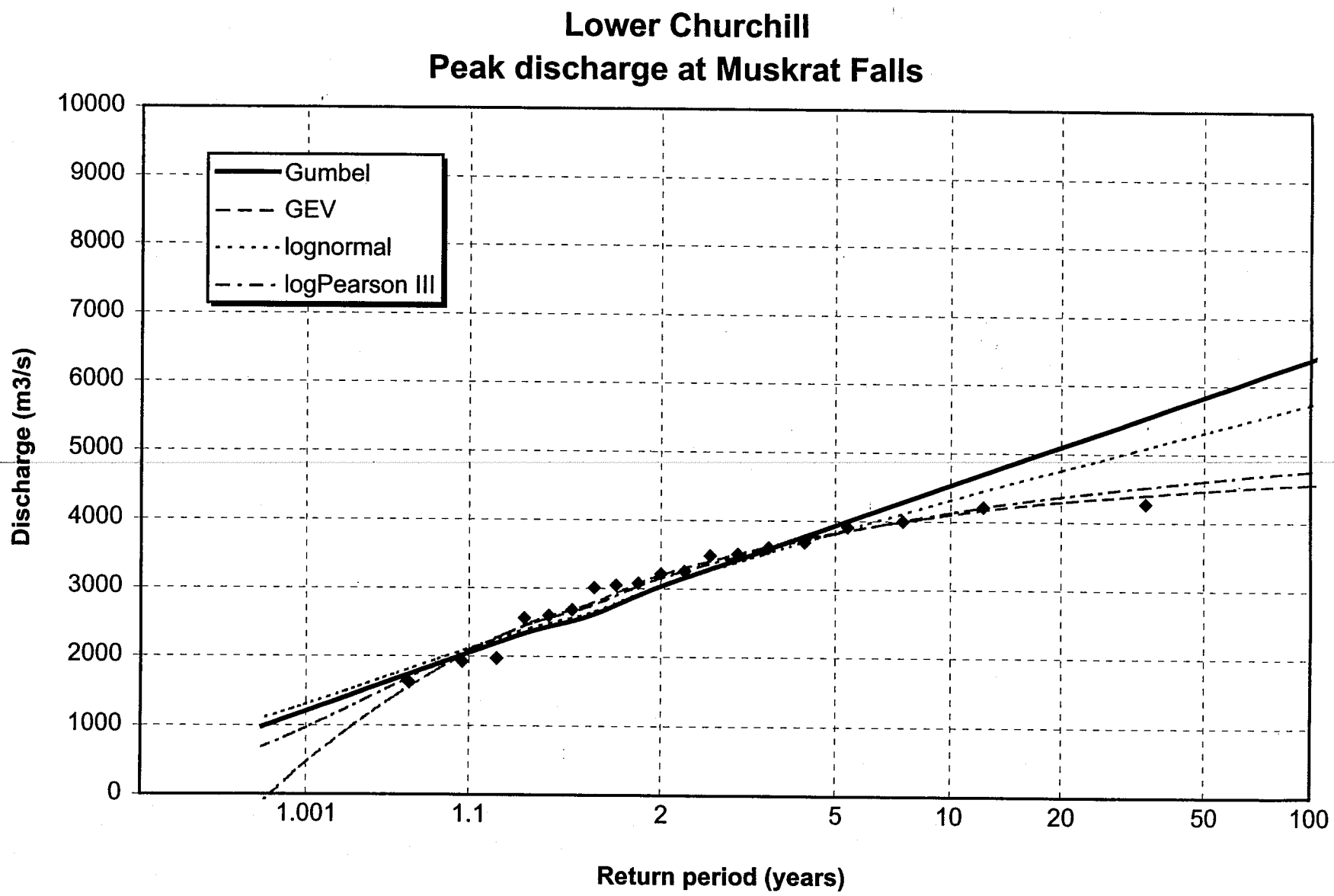

Lower Churchill
Peak discharge at Muskrat Falls
Gumbel
GEV
lognormal
logPearson III
Discharge (m3/s)
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
1.001
1.1
2
5
10
20
50
100
Return period (years)

Figure 2

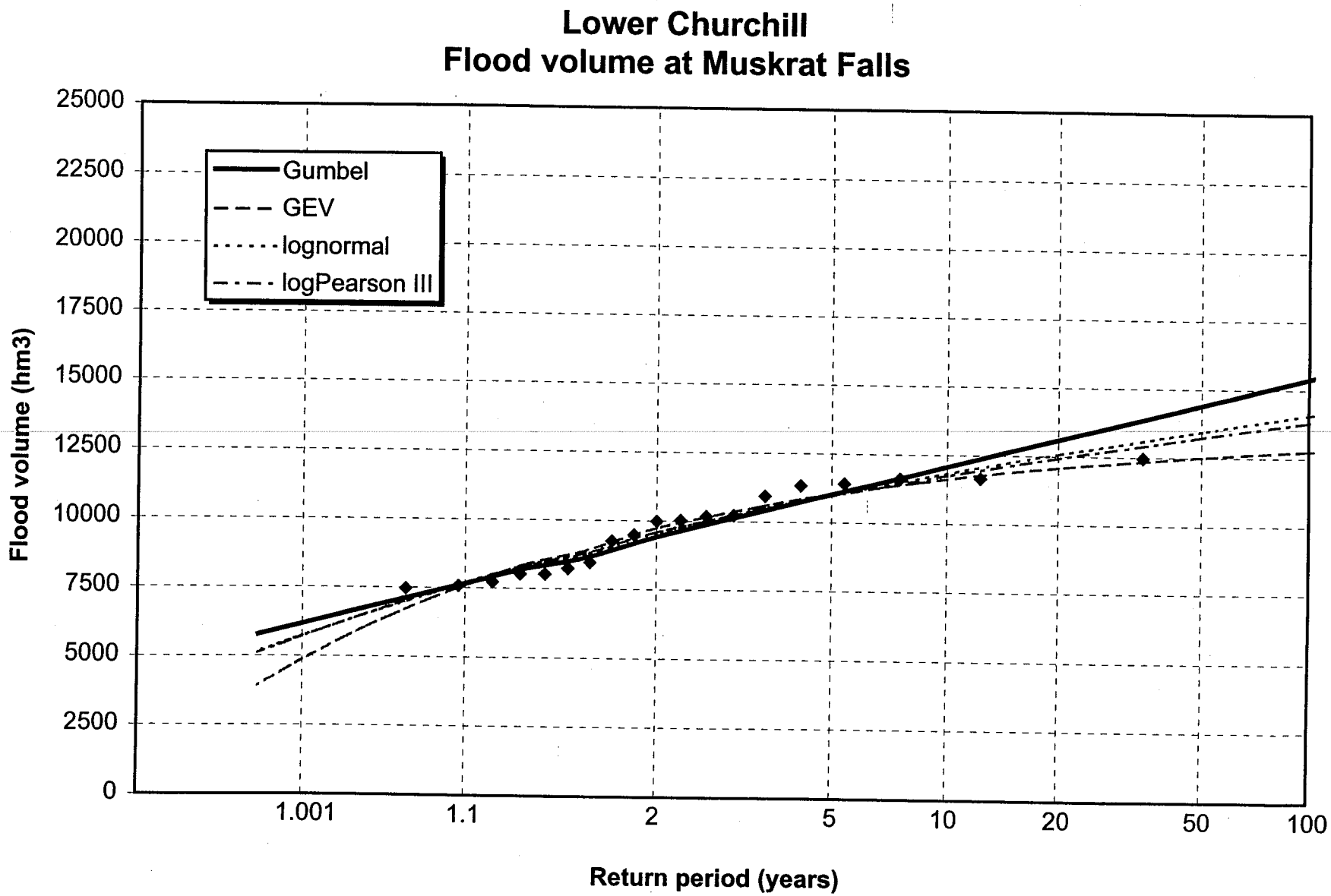
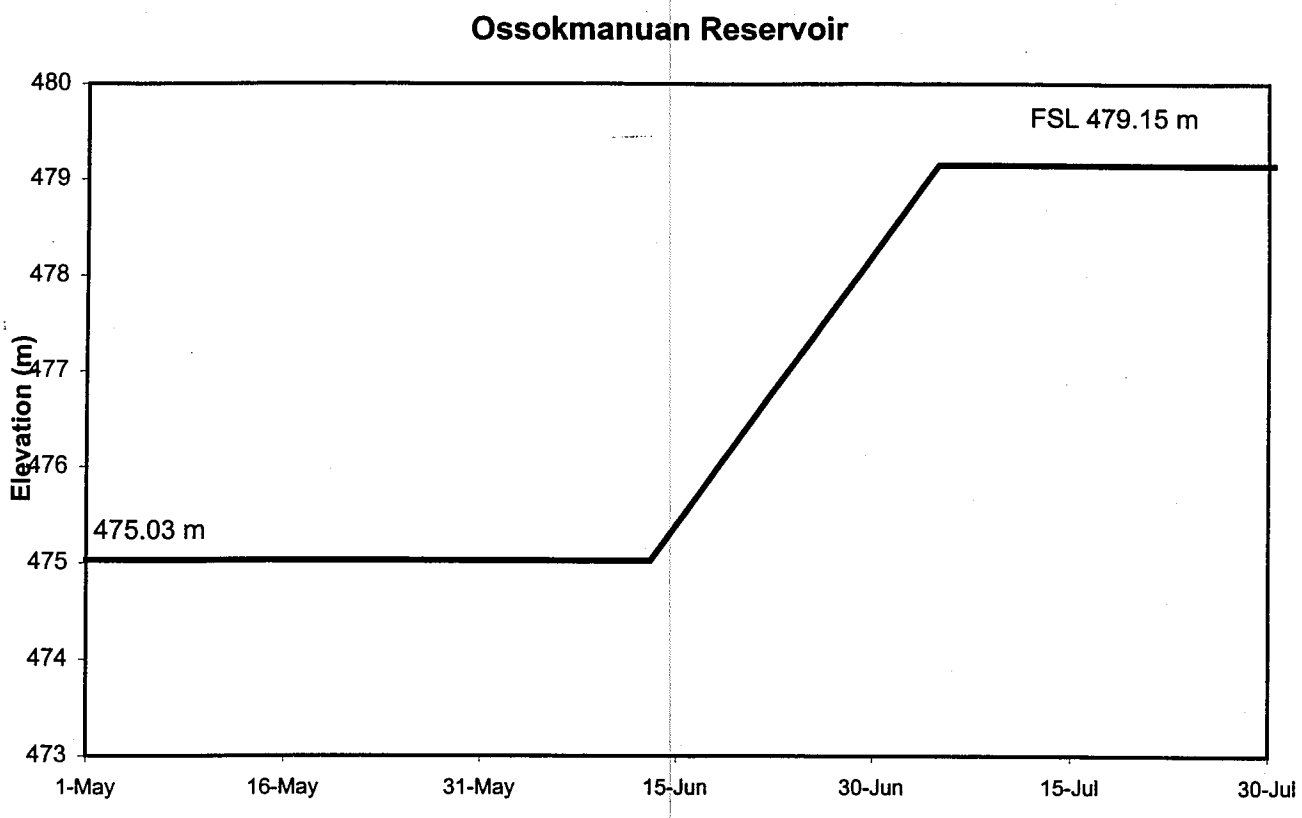
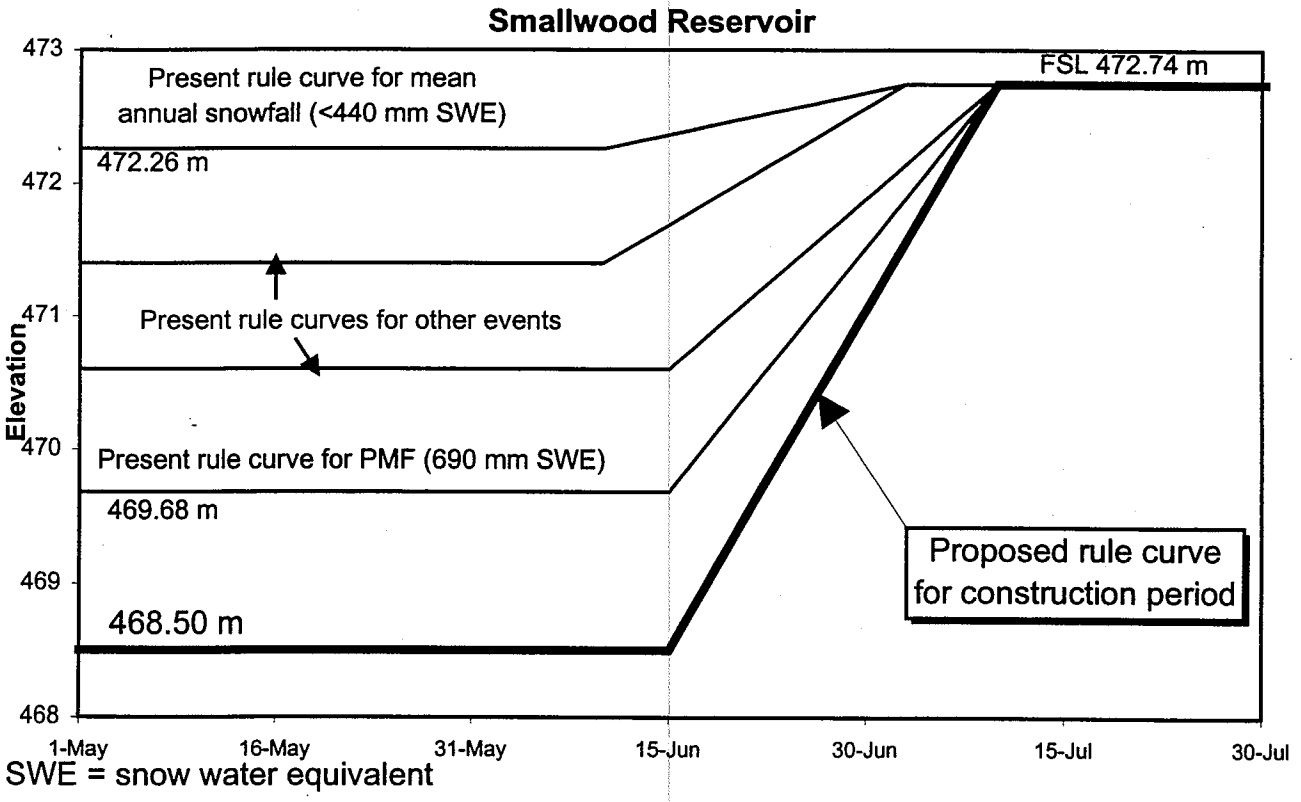

Lower Churchill
Flood volume at Muskrat Falls
Gumbel
GEV
lognormal
logPearson III
Flood volume (hm3)
25000
22500
20000
17500
15000
12500
10000
7500
5000
2500
0
1.001
1.1
2
5
10
20
50
100
Return period (years)

Figure 3





SWE = snow water equivalent

Figure 4

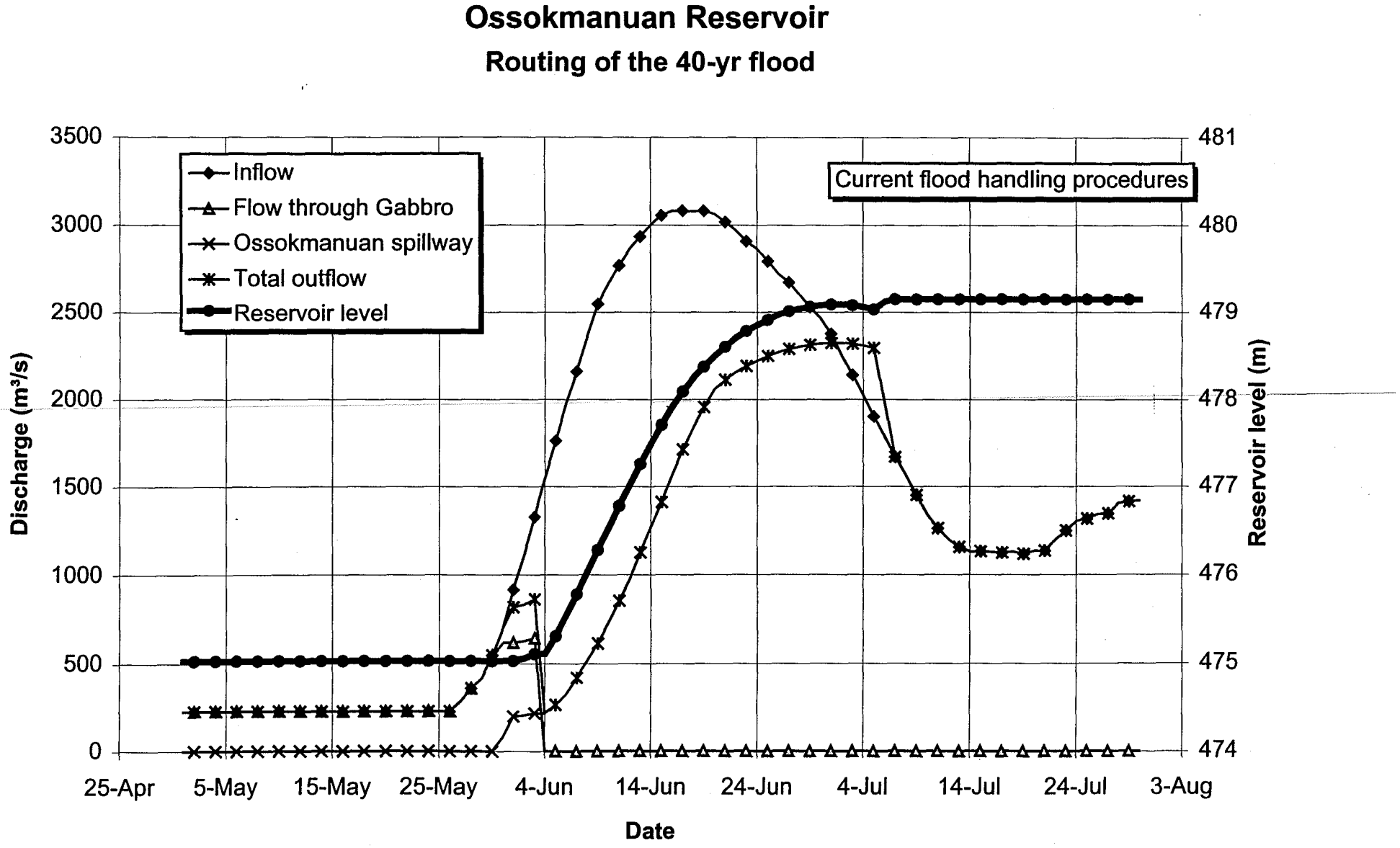

Ossokmanuan Reservoir
Routing of the 40-yr flood
Inflow
Flow through Gabbro
Ossokmanuan spillway
Total outflow
Reservoir level
Current flood handling procedures
Discharge (m³/s)
Reservoir level (m)
Date
3500
3000
2500
2000
1500
1000
500
0
481
480
479
478
477
476
475
474
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug

Figure 5

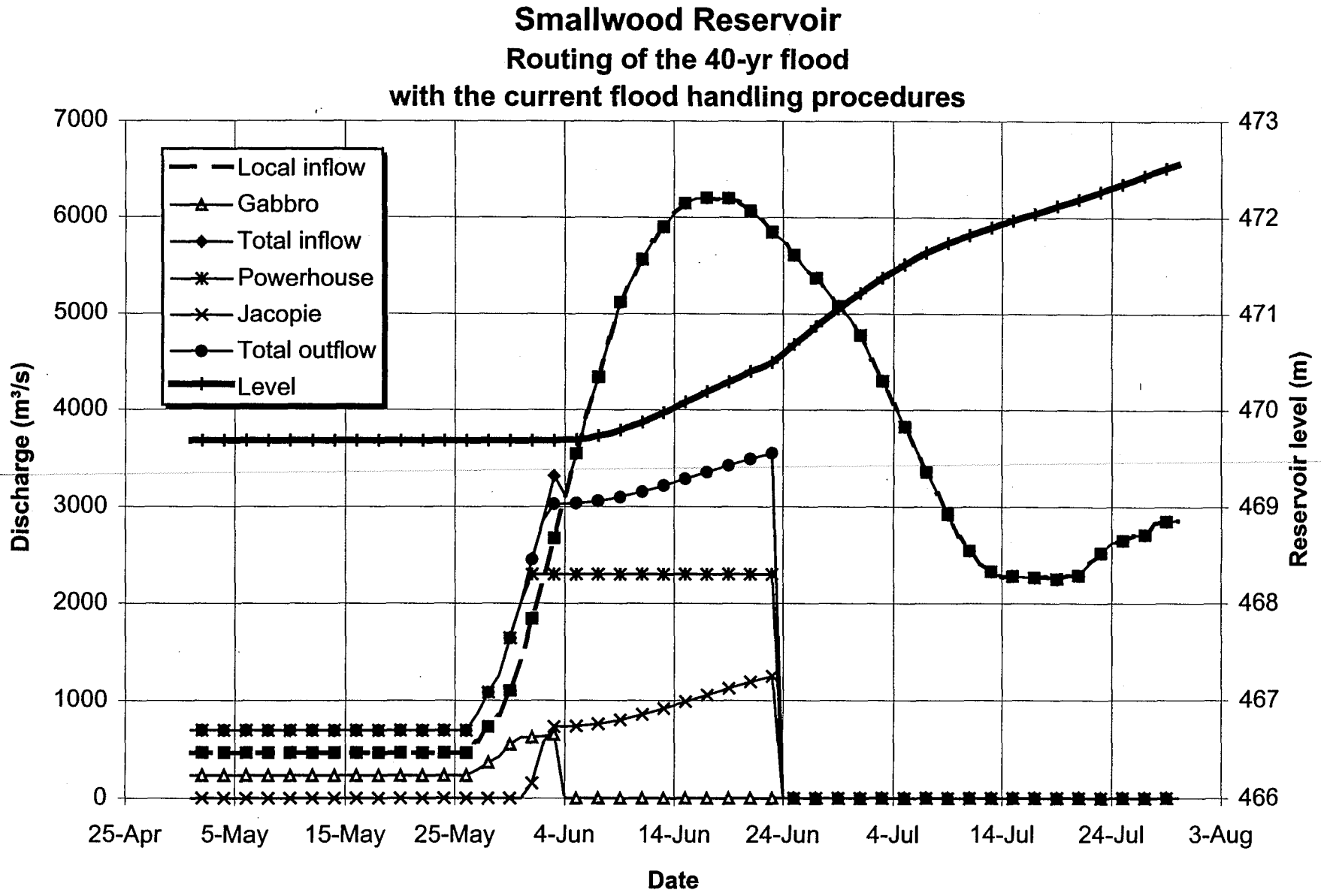

Smallwood Reservoir
Routing of the 40-yr flood
with the current flood handling procedures
Local inflow
Gabbro
Total inflow
Powerhouse
Jacopie
Total outflow
Level
Discharge (m³/s)
Reservoir level (m)
7000
6000
5000
4000
3000
2000
1000
0
473
472
471
470
469
468
467
466
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug
Date

Figure 6

Muskrat Falls

40 yr flood on Upper and Lower Churchill with the current flood handling procedures

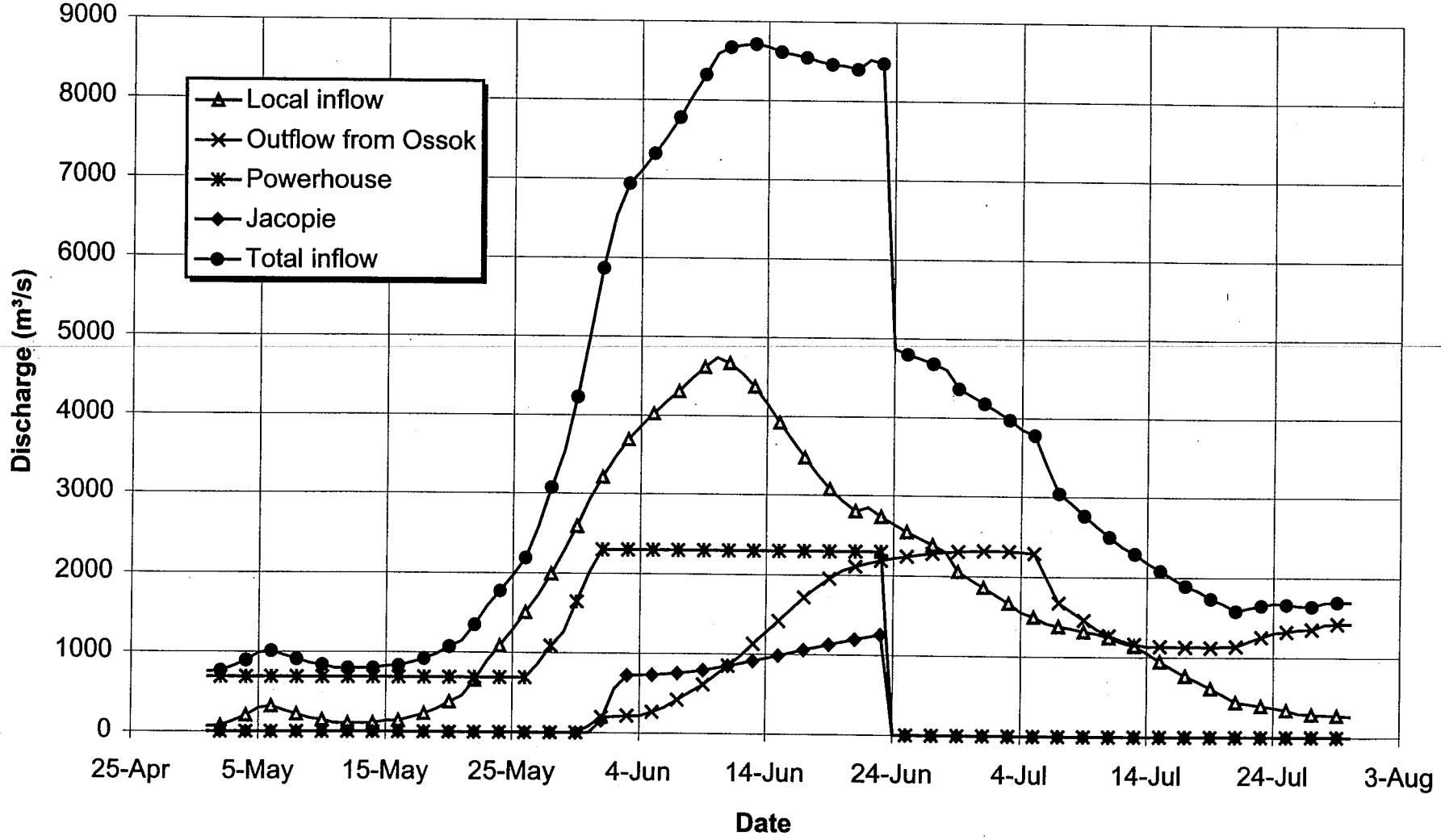


Figure 7

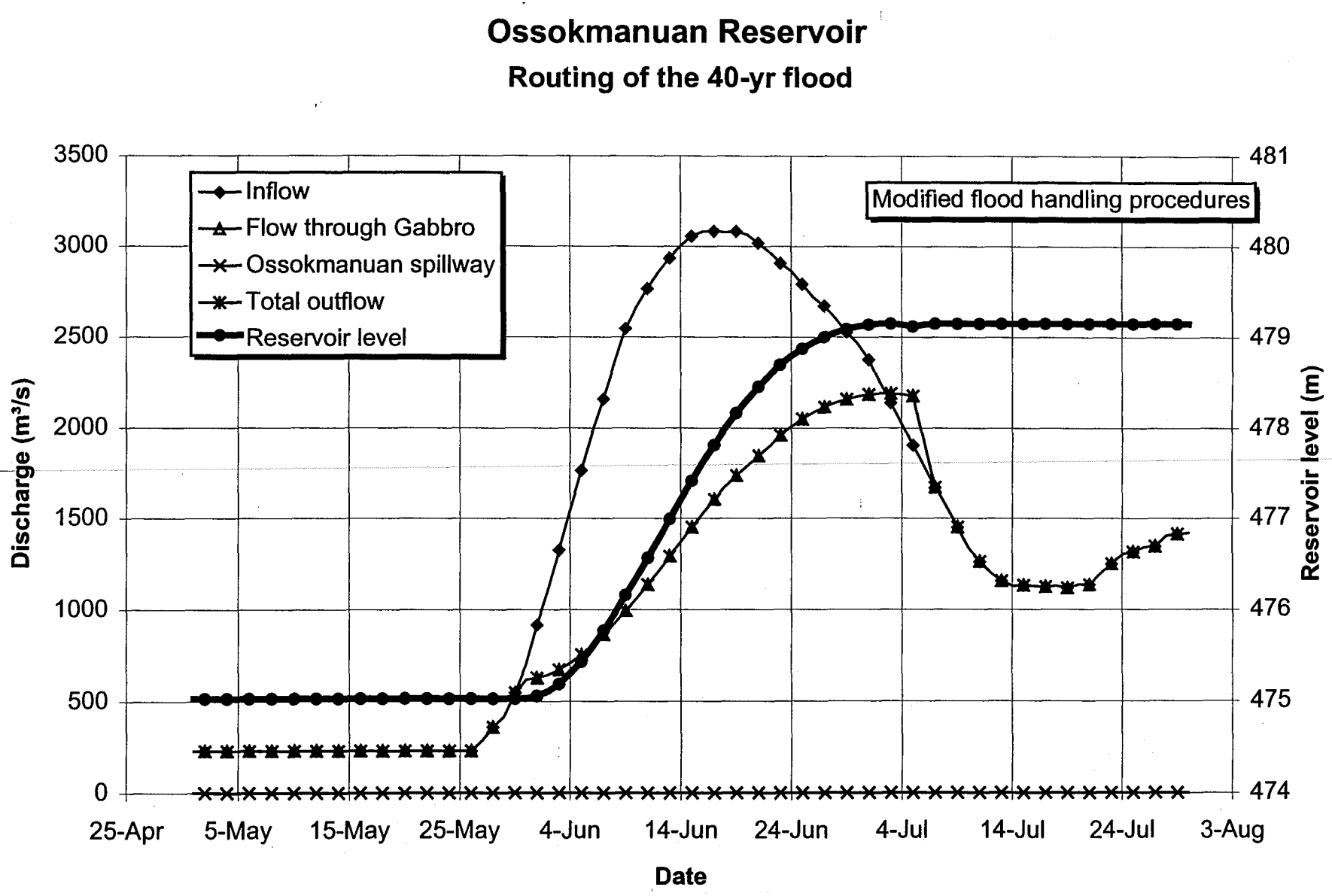

Ossokmanuan Reservoir
Routing of the 40-yr flood
Inflow
Flow through Gabbro
Ossokmanuan spillway
Total outflow
Reservoir level
Modified flood handling procedures
Discharge (m³/s)
Reservoir level (m)
Date
3500
3000
2500
2000
1500
1000
500
0
481
480
479
478
477
476
475
474
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug

Figure 8

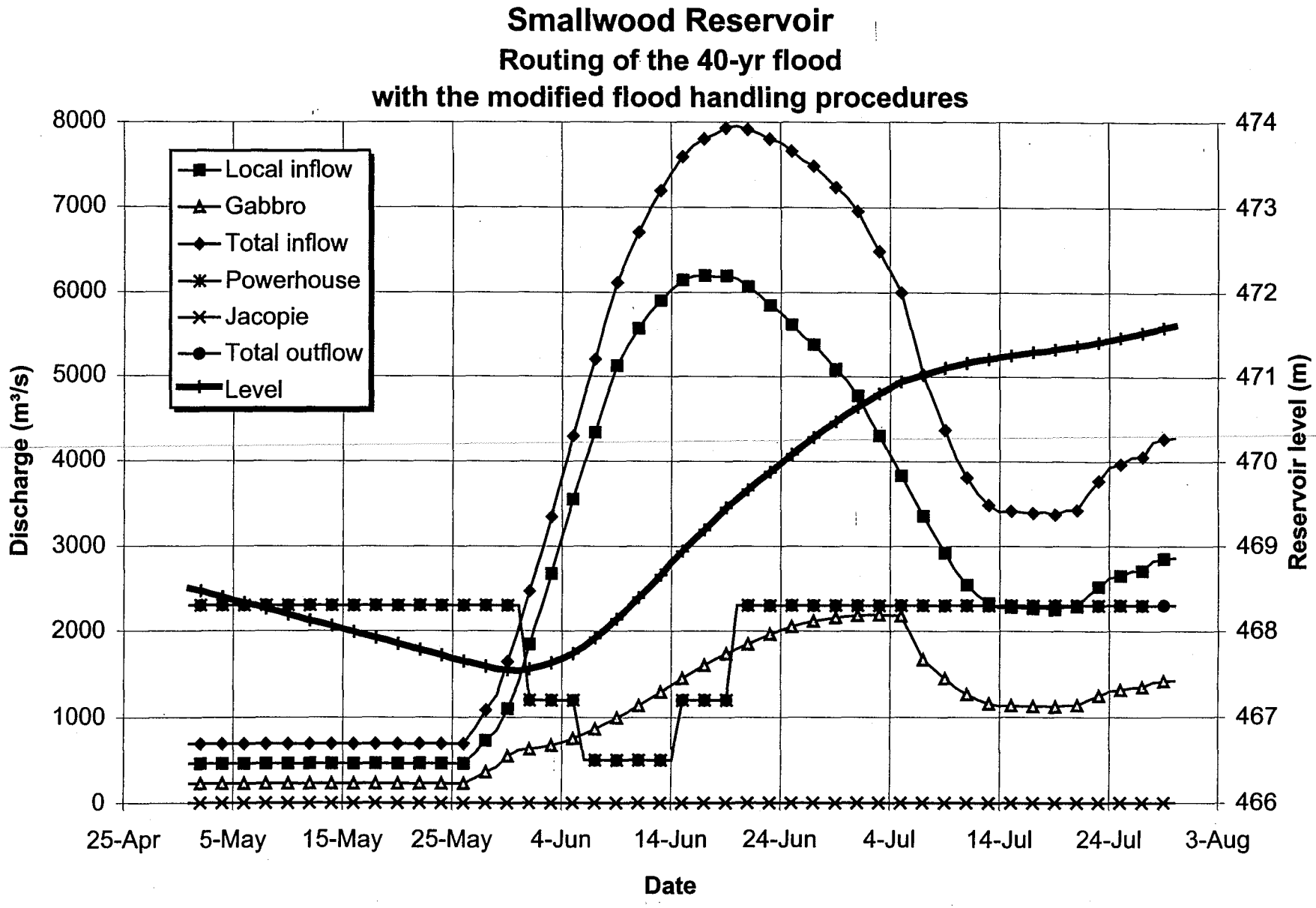
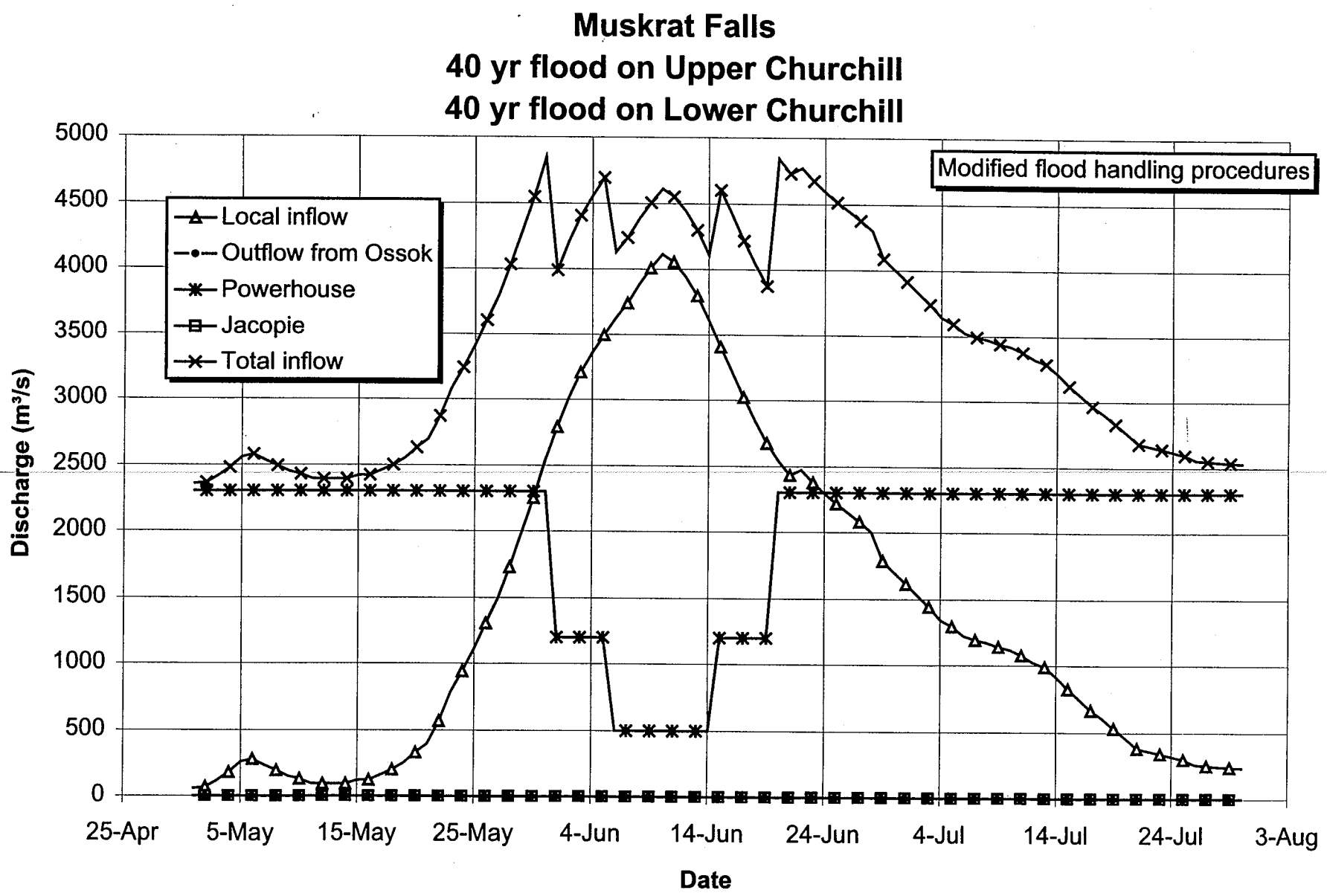

Smallwood Reservoir
Routing of the 40-yr flood
with the modified flood handling procedures
Local inflow
Gabbro
Total inflow
Powerhouse
Jacopie
Total outflow
Level
Discharge (m³/s)
Reservoir level (m)
8000
7000
6000
5000
4000
3000
2000
1000
0
474
473
472
471
470
469
468
467
466
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug
Date

Figure 9

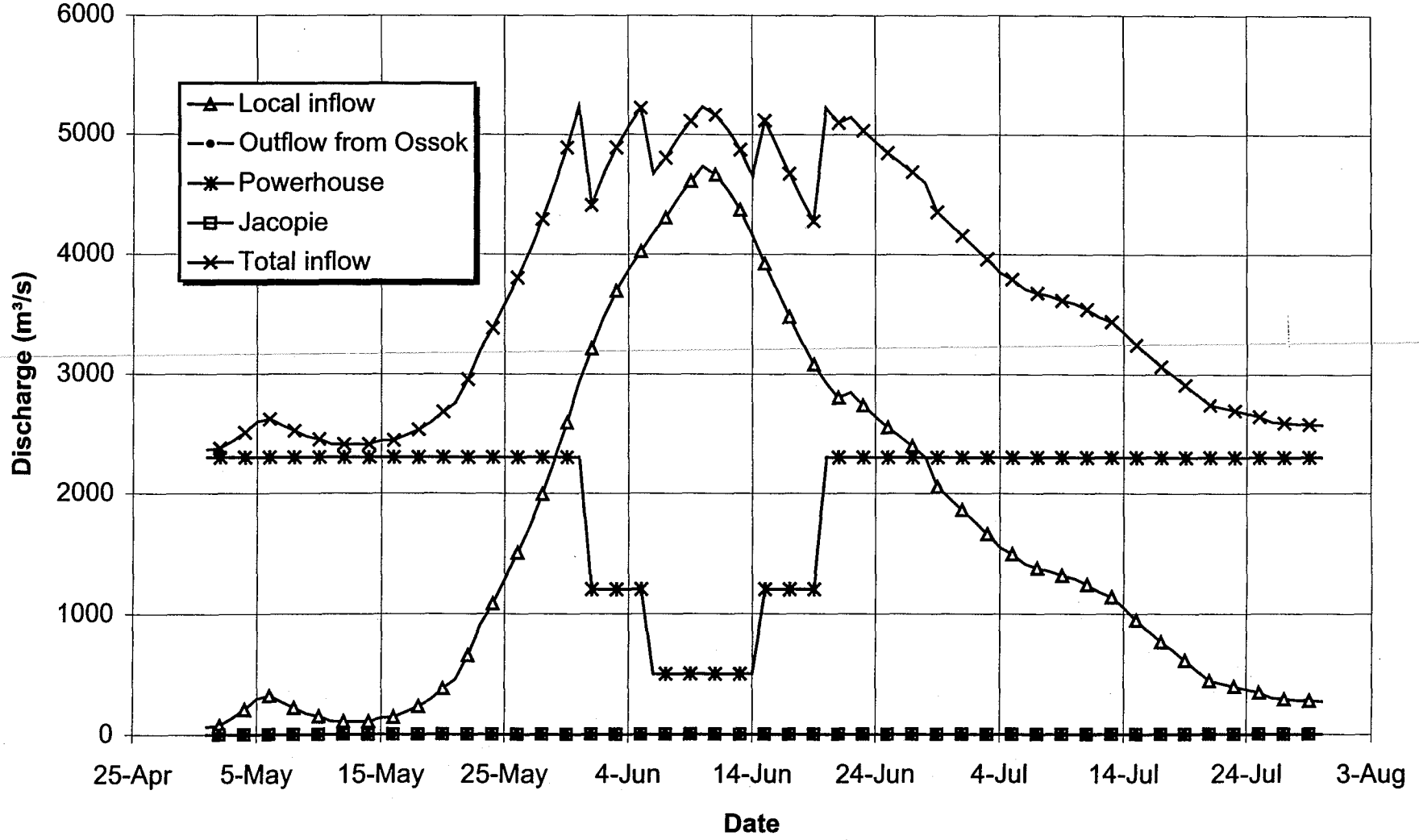


SNC-AGRA
Joint Venture

Figure 10

Muskrat Falls

40 yr flood on Upper and Lower Churchill with the modified flood handling procedures



SNC-AGRA
Joint Venture

Figure 11

Ossokmanuan Reservoir

Routing of the 100-yr flood

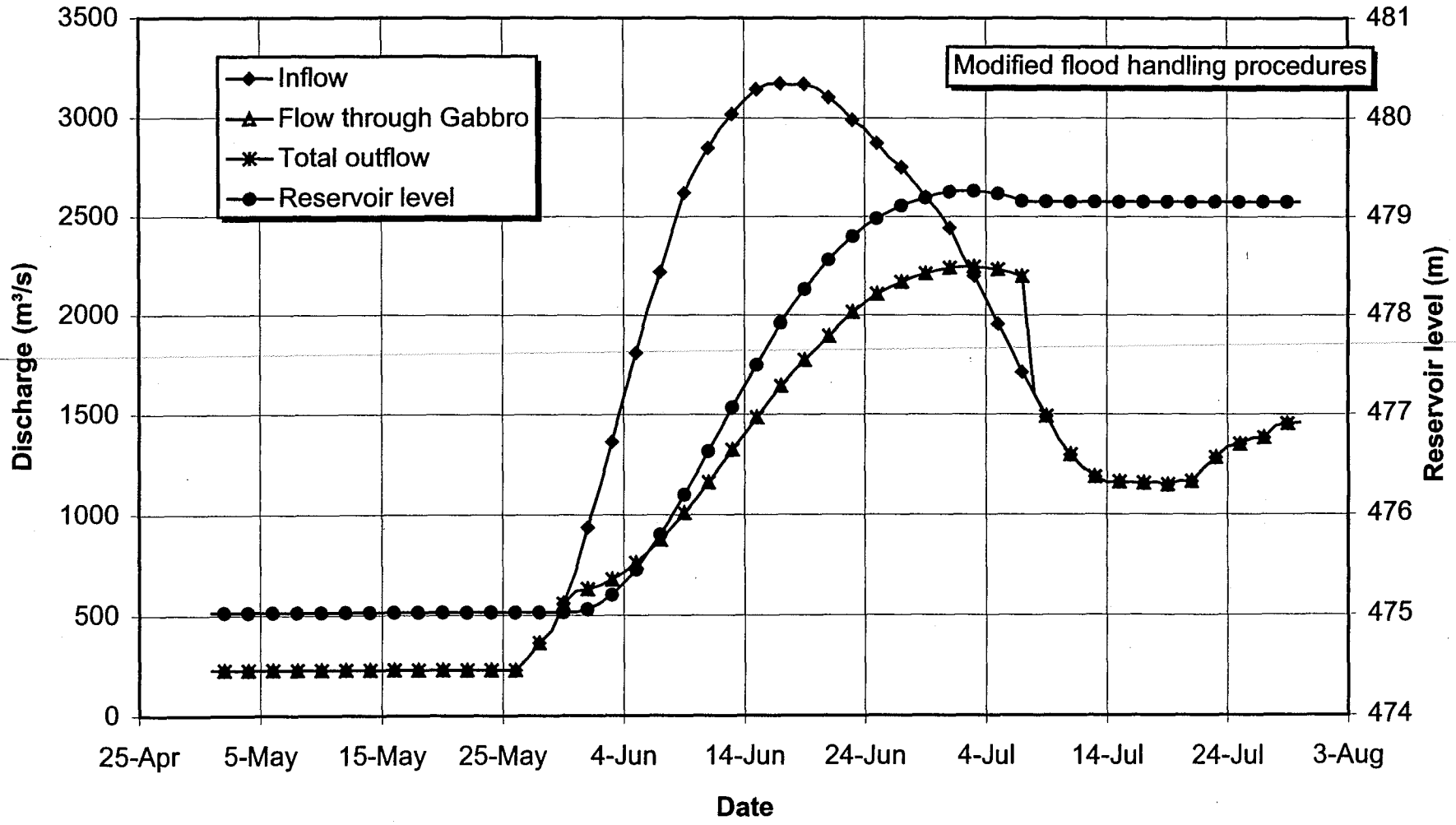


Figure 12

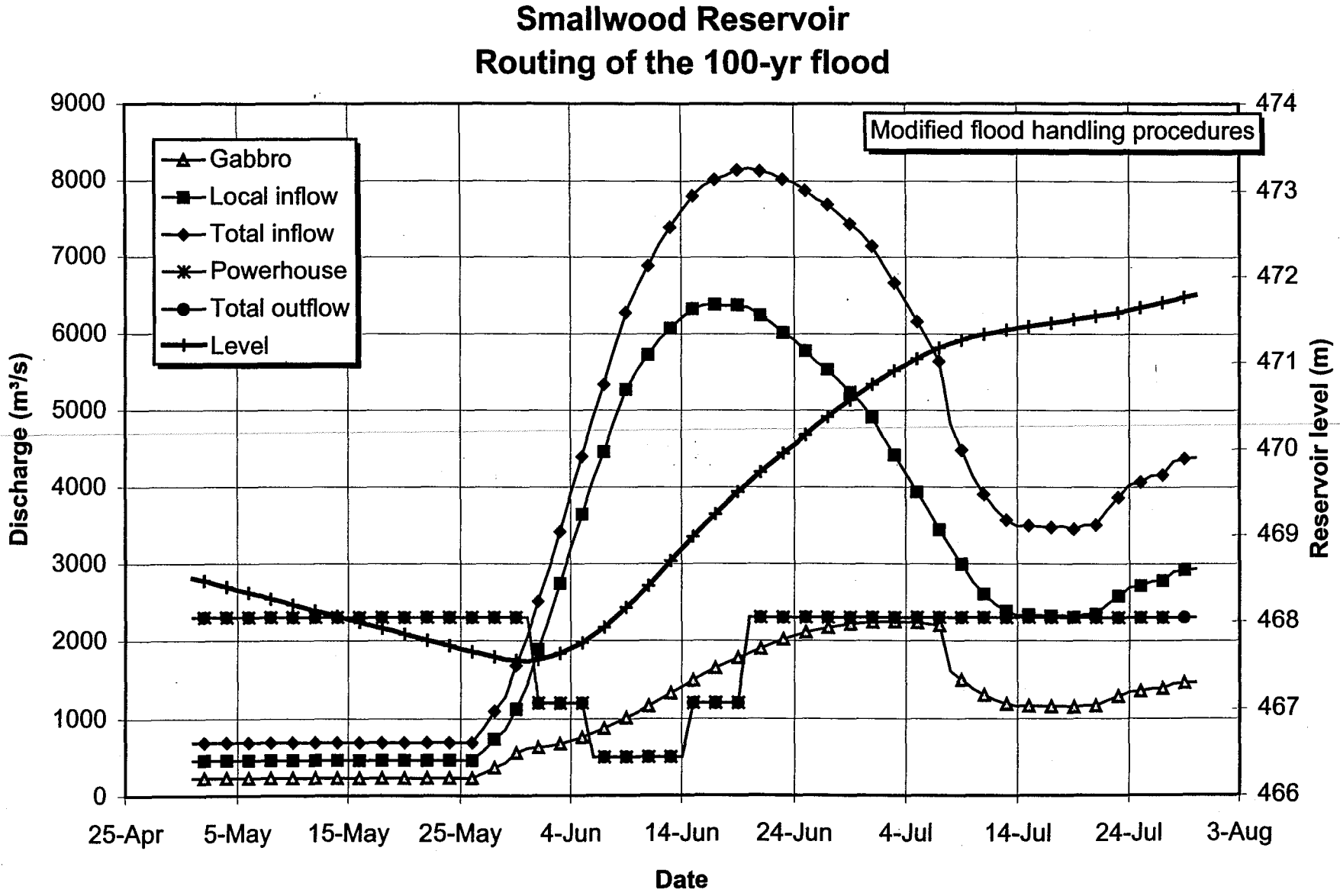

Smallwood Reservoir
Routing of the 100-yr flood
Modified flood handling procedures
Gabbro
Local inflow
Total inflow
Powerhouse
Total outflow
Level
Discharge (m³/s)
Reservoir level (m)
Date
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
474
473
472
471
470
469
468
467
466
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug

Figure 13

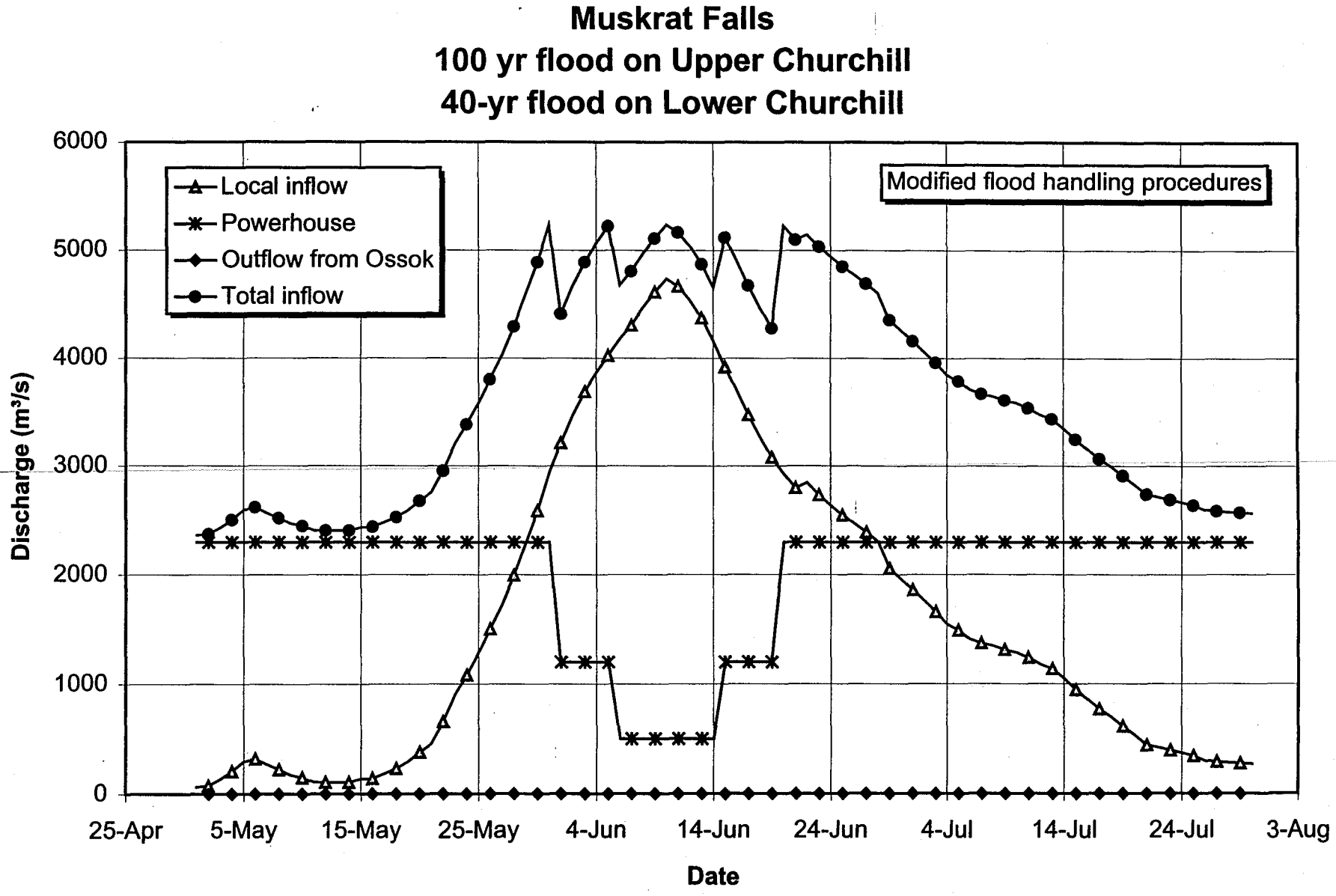

Muskrat Falls
100 yr flood on Upper Churchill
40-yr flood on Lower Churchill
Local inflow
Powerhouse
Outflow from Ossok
Total inflow
Modified flood handling procedures
Discharge (m³/s)
6000
5000
4000
3000
2000
1000
0
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug
Date

Figure 14

Ossokmanuan Reservoir

Routing of the 10 000-yr flood

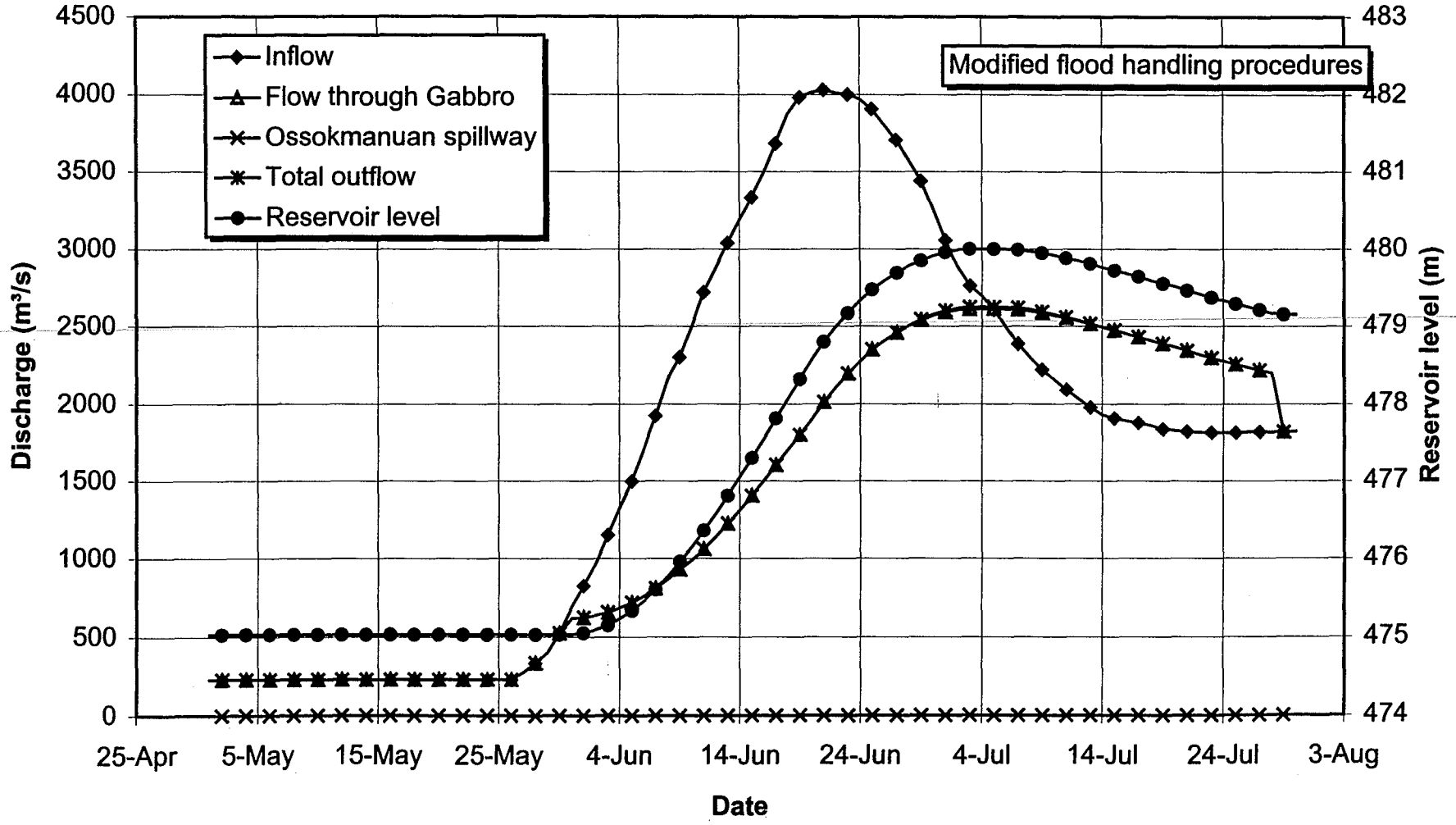


Figure 15

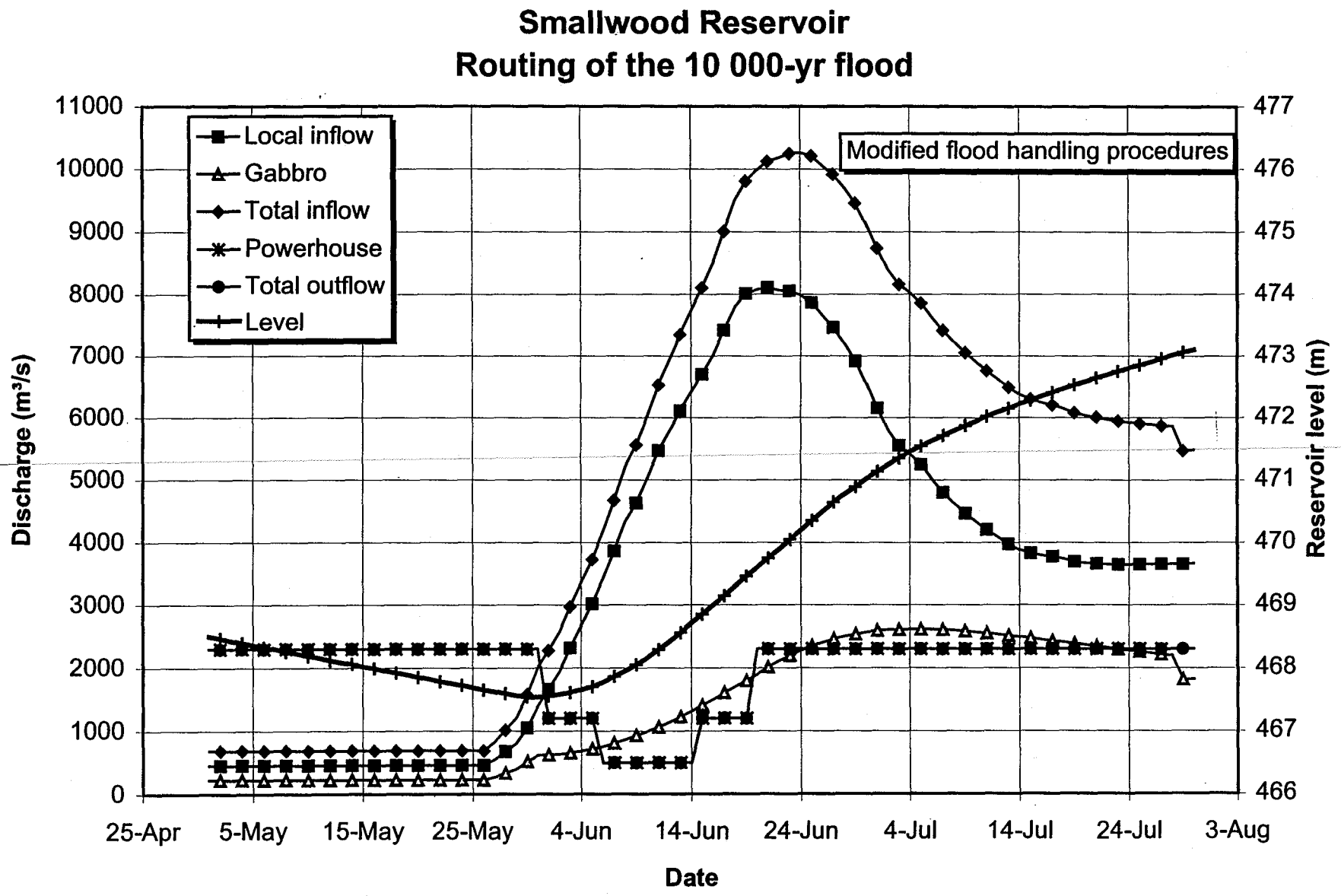

Smallwood Reservoir
Routing of the 10 000-yr flood
Local inflow
Gabbro
Total inflow
Powerhouse
Total outflow
Level
Modified flood handling procedures
Discharge (m³/s)
Reservoir level (m)
Date
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
477
476
475
474
473
472
471
470
469
468
467
466
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug

Figure 16

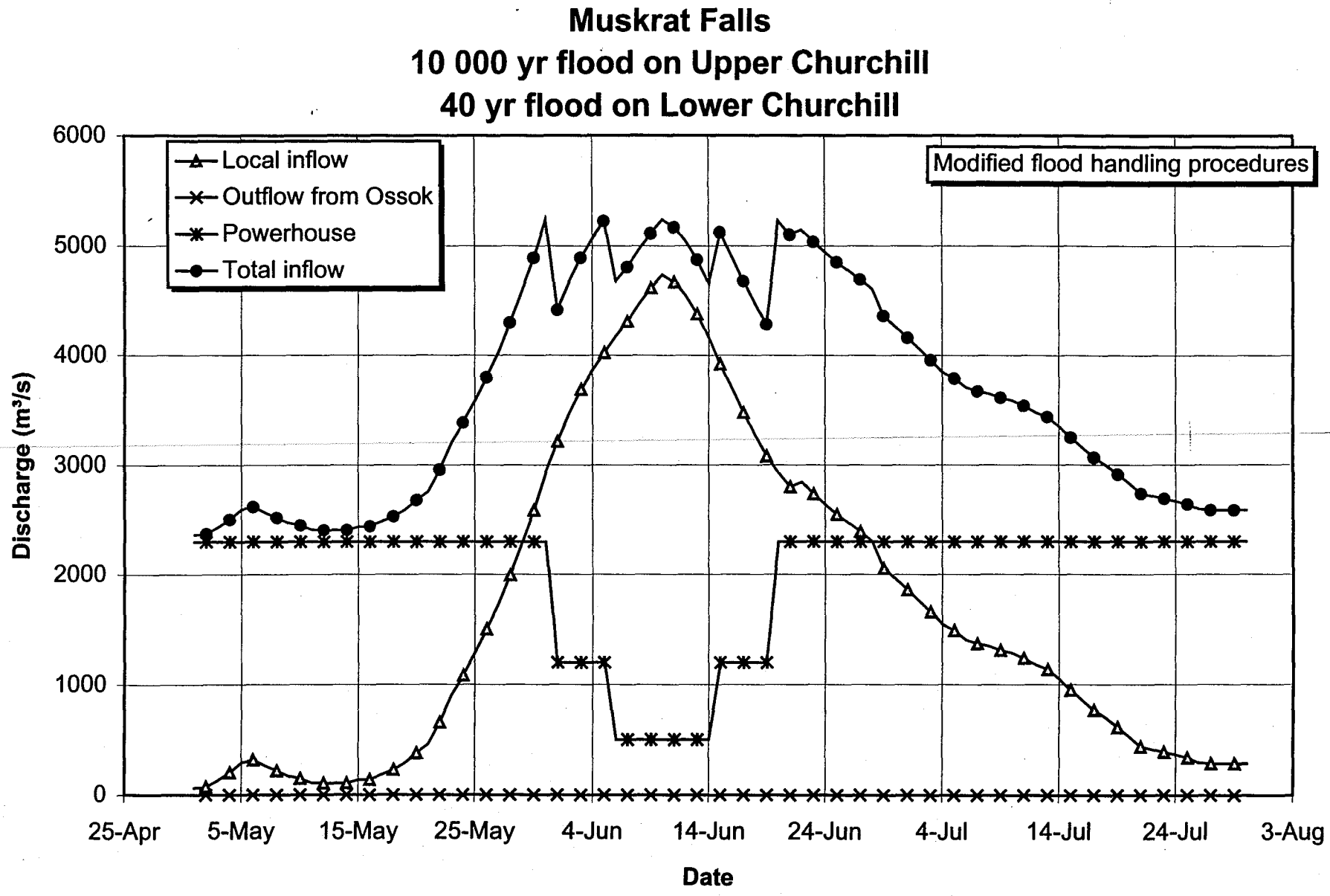

Muskrat Falls
10 000 yr flood on Upper Churchill
40 yr flood on Lower Churchill
Local inflow
Outflow from Ossok
Powerhouse
Total inflow
Modified flood handling procedures
Discharge (m³/s)
6000
5000
4000
3000
2000
1000
0
25-Apr
5-May
15-May
25-May
4-Jun
14-Jun
24-Jun
4-Jul
14-Jul
24-Jul
3-Aug
Date

Figure 17

APPENDIX E

OPTIMIZATION OF NUMBER OF GENERATING UNITS

APPENDIX E

OPTIMIZATION OF NUMBER OF GENERATING UNITS

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT
FINAL FEASIBILITY STUDY
PROJECT: 721397

INTERIM REPORT
ON
OPTIMIZATION OF NUMBER OF GENERATING UNITS

Submitted to:

Newfoundland and Labrador Hydro

SNC-AGRA JOINT VENTURE
OCTOBER 27, 1998

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APPENDICES

Appendix 1 - Memo regarding experience with concrete semi-spiral case at Gezouba.

Appendix 2 - Joint memo issued by Mr. Jacques Vezina of SNC-Lavalin and Mr. M.D. Roy of Hydro-Quebec.

1.0 INTRODUCTION

In the 1980 SNC-Lavalin Engineering Report[1], the scheme of development for the Muskrat Falls Hydroelectric Development, which was accepted by the Board of Consultants and the Client, LCDC, (Variant 5) comprised a power tunnel and surface powerhouse arrangement on the North bank of the Churchill River. This arrangement included a 618 MW capacity power plant with three propeller turbines, each with a steel spiral case. The 1989 SNC Engineering Report[2] reviewed four additional 'in-river' layouts for the Development, each comprising a close-coupled intake/powerhouse. These arrangements each included an 824 MW capacity power plant with four propeller turbines with concrete semi-spiral cases.

The Interim Report on Selection of Project Layout[3] reviewed all of the previous schemes of development and qualitatively and quantitatively compared these schemes with one additional scheme proposed by Hydro Quebec. This report recommended Variant 7, from the 1989 Report, with an in-river, close-coupled intake/powerhouse arrangement including four propeller turbines with concrete semi-spiral cases.

Acres International Ltd., carried out a parallel study to optimize the total plant capacity for Muskrat Falls. That study determined the optimum plant capacity for Muskrat to be 824 MW. Based on the total plant capacity of 824 MW, SNC-AGRA proceeded to optimize the number and size of units.

The following sections discuss the reasons for selection of the concrete semi-spiral case for the plant and the optimization process used for selection of the number and size of units.

[1] SNC-Lavalin Newfoundland Ltd., Muskrat Falls Power Development & 345 kV Transmission Intertie to Churchill Falls, Volume 1 – Engineering Report, March 1980.
[2] SNC, Muskrat Falls Power Development, Supplemental Development Layout Studies, April 1989.
[3] SNC-AGRA, Muskrat Falls Hydroelectric Development, Final Draft of Interim Report on Selection of Project Layout, September 10, 1998.

2.0 CONCRETE SEMI-SPIRAL CASE VERSUS STEEL SPIRAL CASE

With an in-river installation and a gross head of 37 metres, a close-coupled intake and powerhouse was found to be the most suitable layout for the power complex.

The advantages of a concrete semi-spiral case, over a steel spiral case, in a close-coupled intake/powerhouse arrangement for Muskrat Falls, are as follows:

- A narrower powerhouse (a shorter upstream/downstream dimension between the face of the intake and the centreline of the unit). A steel spiral case would require a longer intake water passage than a concrete semi-spiral, comprising:

  - a concrete transition section between the rectangular concrete gate opening and the steel penstock, and

  - a steel penstock between the transition and the steel spiral, including two vertical bends with an intermediate tangent section of penstock.

  With a concrete semi-spiral case, a shorter, sloped water passage with constantly decreasing rectangular cross-section, in the downstream direction, is constructed between the intake gates and the entrance to the semi-spiral.

- A smaller unit spacing for the same size of turbine. This results in an overall shorter powerhouse, mainly due to the frontal water feed into the unit.

- Smaller volumes of concrete and excavation and a correspondingly smaller superstructure.

- Generally, simpler construction procedure. Although the concrete semi-spiral requires the use of single-curved formwork, erected in place during the concreting process, it eliminates:

  – interruption of the concreting process for the introduction of additional trades, as required for erection of a steel spiral case,

  – the difficult and time consuming segment by segment erection required for a very large steel spiral, as would be required for Muskrat Falls,

  – the problems associated with welding of thick steel plates of such a large steel spiral, and

  – the subsequent stress relieving and pressure testing requirements, prior to concrete embedment.

- A lower overall cost of the erected turbines with the elimination of the steel spiral cases. This cost reduction is approximately equal to 15% of the turbine cost due to the savings on supply and erection of the steel spiral.

For a concrete semi-spiral at Muskrat Falls, with a turbine runner diameter of 9.18 m, it is recognized that a concrete semi-spiral case, would be very large. The main concern with such a large concrete semi-spiral case is the structural design required to withstand the bursting force exerted by the hydrostatic pressure of the water. The following Section 3.0 discusses the acceptability of a concrete semi-spiral for Muskrat Falls. Figures 3.1 and 3.2 illustrate the general arrangement of the powerhouse and concrete semi-spiral case.

3.0 OPTIMIZATION OF NUMBER OF GENERATING UNITS

With an optimum plant capacity of 824 MW determined in a previous study, the next step was to determine the optimum number of units to provide this capacity. To achieve this the methodology used must compare the total economic ranking of several alternatives, including capital and operating costs and energy production. In addition, it must consider the following power development features:

- type of power development, i.e. run of the river, large reservoir, etc.,
- type and characteristics of the turbine contemplated,
- flow duration curve applicable to the powerplant,
- upper and lower limit to the number of units to be considered, due to the site specific conditions and/or the unit constructibility.

The type of power development has a significant effect on the determination of the optimum number of units, as it qualifies the capacity of flow regulation available for the powerplant. With a powerplant equipped with a large reservoir, the energy production is much less sensitive to the "instantaneous" powerplant generating efficiency since the flow regulation capacity of the reservoir would allow the plant to operate the turbines at peak efficiency. This is not the case, however, with a "run of the river" development, without flow regulation, as this imposes the requirement for immediate use of the upstream incoming water flow. In this event, the "instantaneous" generating efficiency curve, as a function of powerplant flow, directly affects the energy production. This is the case for the Muskrat Falls Development.

The type and characteristics of the turbine being contemplated are necessary to compute the powerplant "instantaneous" efficiency as a function of flow. The plant efficiency curve, especially when considering the use of fixed blade propellers or high specific speed Francis machines, is greatly affected by the number of units, especially if the total number of units contemplated is small. Generally, the more units installed in a given powerplant, the better the total powerplant "instantaneous" efficiency curve.

The flow duration curve (Figure 3.3) provides the expected simulated long term incoming plant flow as a function of percentage of time. With a "run of the river" plant, this curve is used as a basis on which to determine the energy production for different alternatives.

Finally, in order to study only the number of alternatives that can be practically implemented at a specific site, it is important to fix, at an early stage, the maximum and minimum number of units to be considered. The minimum number of units is normally limited by physical sizing constraints, in manufacturing or transportation, or by the maximum unit capacity which can be dropped from the network at one time. The maximum number of units is sometimes limited by the total physical area available at a specific location.

In the case of the Muskrat Falls Development, with a "run of the river" powerplant and the use of fixed blade propeller turbines, as recommended in previous reports, the total energy production would be significantly affected by the number of units installed. In this situation, additional units could significantly reduce, if not cancel, the cost increase effect associated with

an increase in the number of units installed. To define the upper and lower number of units to consider, research was carried out to determine the largest axial unit ever fabricated, installed and in operation, first in our vincinity and then elsewhere in the world. In Canada, several large axial machines were "recently" installed, the most recent being La Grande 1, Laforge 2 and Brisay during the second phase of the James Bay Complex. These powerplants have the following features:

Plant	Unit Casing Type	Max. Gross Head (m)*	Runner Dia (m)
LG-1	Concrete semi-spiral	31.5	8.00
LA-2	Concrete semi-spiral	36.0	8.45
Brisay	Steel spiral case	53.3	8.60
Muskrat	Concrete semi-spiral	43.0	9.18 (4 units)

* At the distributor centerline.

From the above table, it can be seen that the use of a concrete semi-spiral case, in Canada, was limited to a powerplant with a 36 m gross head, at the distributor centerline, combined with a unit with an 8.45 m diameter runner.

Since the runner diameter of the 206 MW unit (turbine size = 212MW) already considered in the 1980 Study implies a runner diameter of approximately 9.2 m, the research was extended to include experience throughout the world and the major designs applicable to Muskrat Falls are tabulated as follows:

Plant	Unit Casting Type	Max. Gross Head (m)*	Runner Dia (m)
Gezouba 1	Concrete semi-spiral	29.4	11.3
Gezouba 2	Concrete semi-spiral	29.4	10.2
Iron Gate	Concrete semi-spiral	34.5	9.5
Muskrat	Concrete semi-spiral	43.0	9.18 (4 units)

* At the distributor centerline.

In order to evaluate the viability of design and construction of a concrete semi-spiral case associated with the Muskrat four-unit scheme, the above mentioned powerplants were listed in order of relative bursting force factor which is a function of the square of the runner diameter, times the gross head at the distributor centerline. The list is presented below:

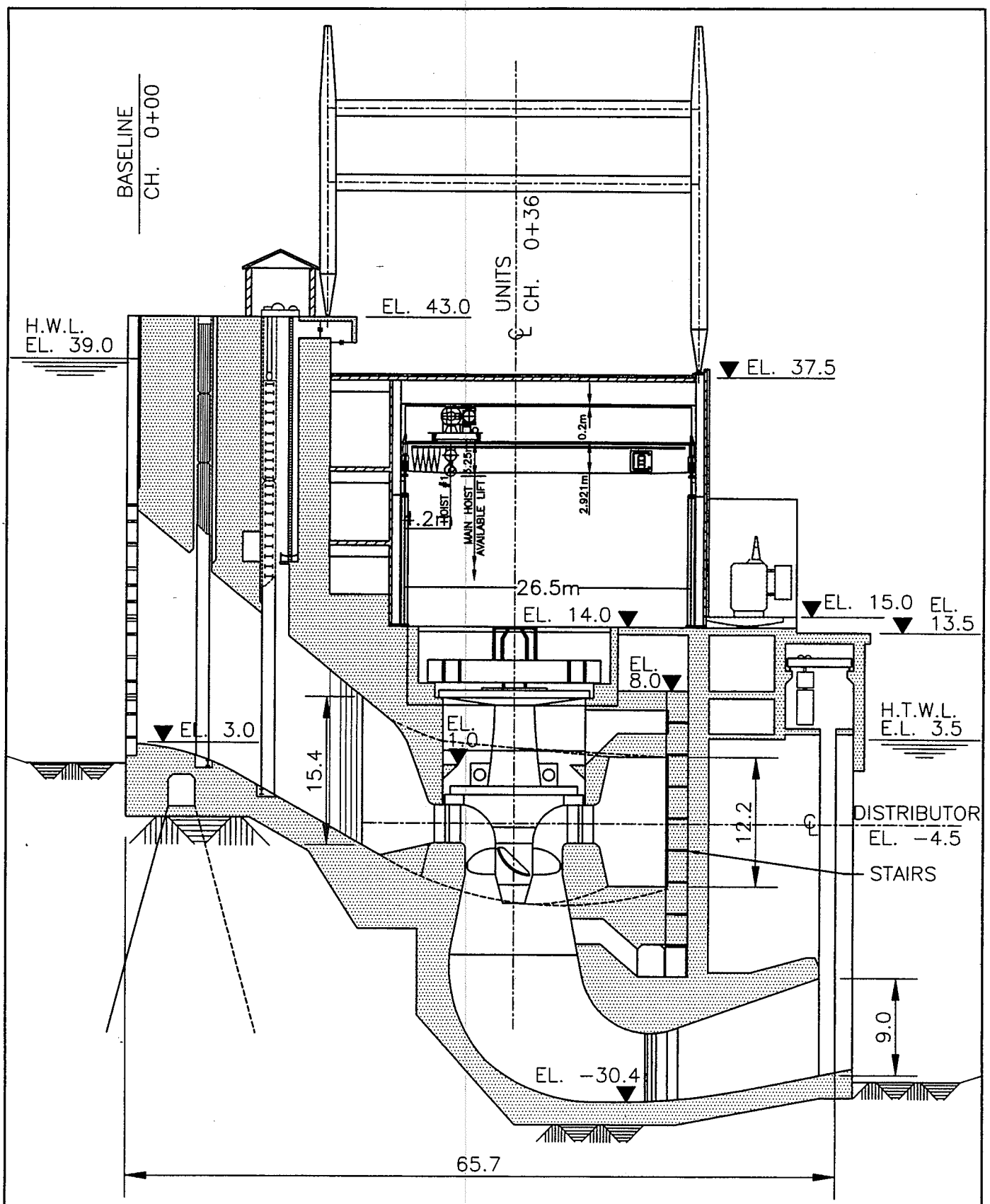
Powerplant	Bursting Force Factor
Gezouba 1	3754
Muskrat (4 units)	3624
Iron Gate	3114
Gezouba 2	3059
LA-2	2570
LG-1	2016

While the bursting force factor is on the high side when compared with the experience in Canada, plants elsewhere in the world present higher or similar characteristics. The Gezouba 1 powerplant in China, with a higher relative bursting force than Muskrat Falls, was commissionned in the early 1980's. The behaviour of the concrete semi-spiral case at Gezouba since construction was querried and Mr. Yang Guowei[1] advised that there have been no operating problems with the concrete semi-spiral to date. Specifically, the plant has not experienced any detrimental cracking or leaking through the walls, ceiling or floor of the concrete semi-spiral. Mr. Guowei also noted that another unit with similar gross head and runner diameter as Muskrat Falls was successfully model tested and is being constructed for the Gaobazhou project in China. A letter from Mr. Guowei is included in Appendix 1 of this report.

A preliminary finite element analysis was made of the concrete semi-spiral case for Muskrat and it was determined that a wall thickness of 4 m, with appropriate quantities of shear and flexural reinforcing, should be adequate. Based on this, it appears that the design and construction of the concrete semi-spiral case for the four-unit alternative at Muskrat Falls is within the present realm of technology.

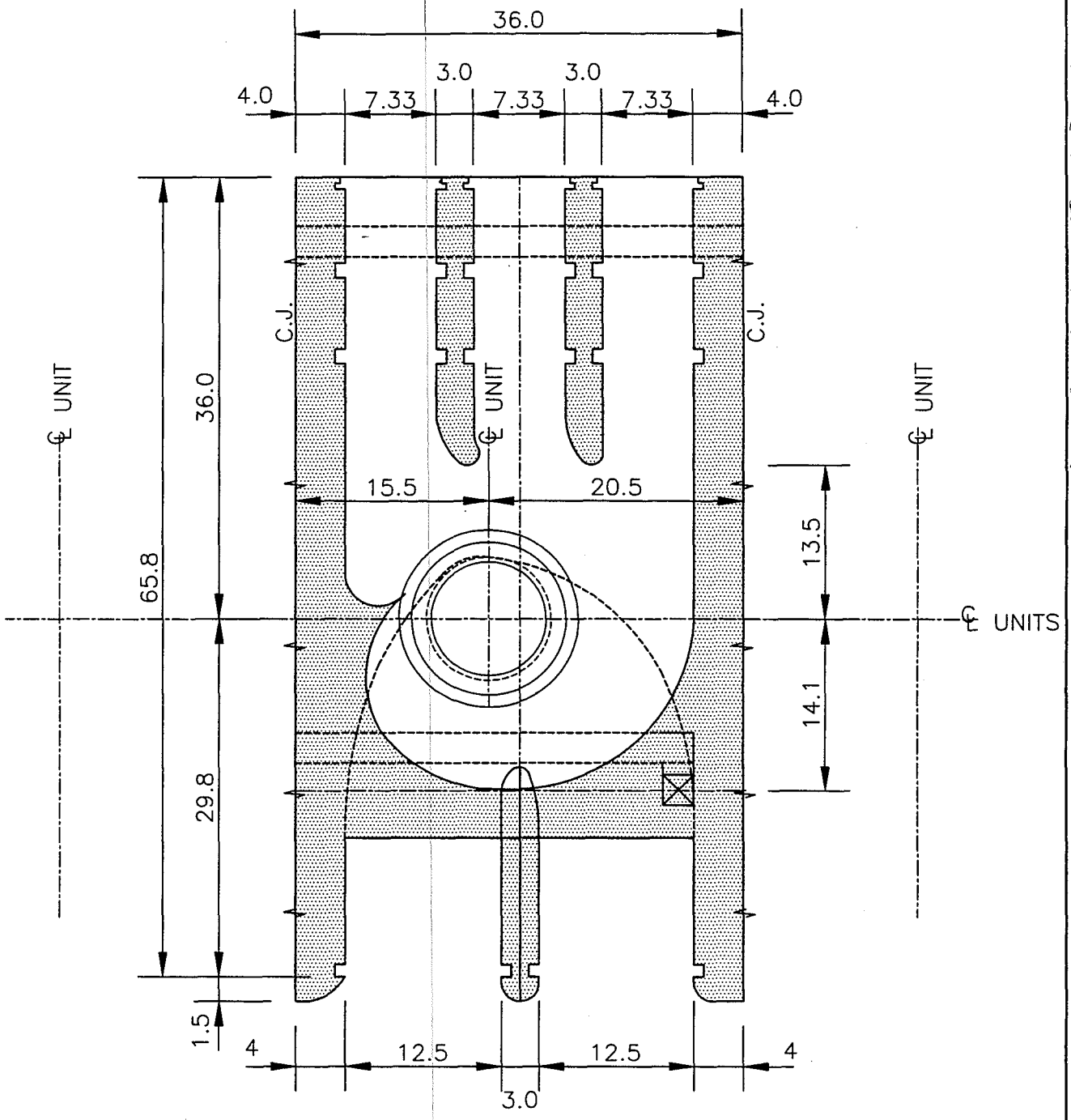
Based on the selection of a concrete semi-spiral case, it was then decided to study three alternatives to produce the installed capacity of 824 MW. The three alternatives included four, five or six turbines, recognizing that the machine for the six-unit alternative would require a runner diameter of 7.5 m, which is well within the diameter range of units already in operation.

[1] Mr. Yang Guowei, Director-General, Bureau of Science/Technology and International Affairs, Changjiang Water Resources Commission, Wuhan, P.R. China.



4 UNIT ALTERNATIVE
212 MW EACH
UNIT SPACING 36.0

FIGURE 3.1
POWERHOUSE - TYP. CROSS SECTION



4 UNIT ALTERNATIVE
212 MW EACH
PLAN OF WATER PASSAGE

FIGURE 3.2
POWERHOUSE - PLAN OF INTAKE & DRAFT TUBE

Flow Duration Curve - Muskrat Falls - Base Case

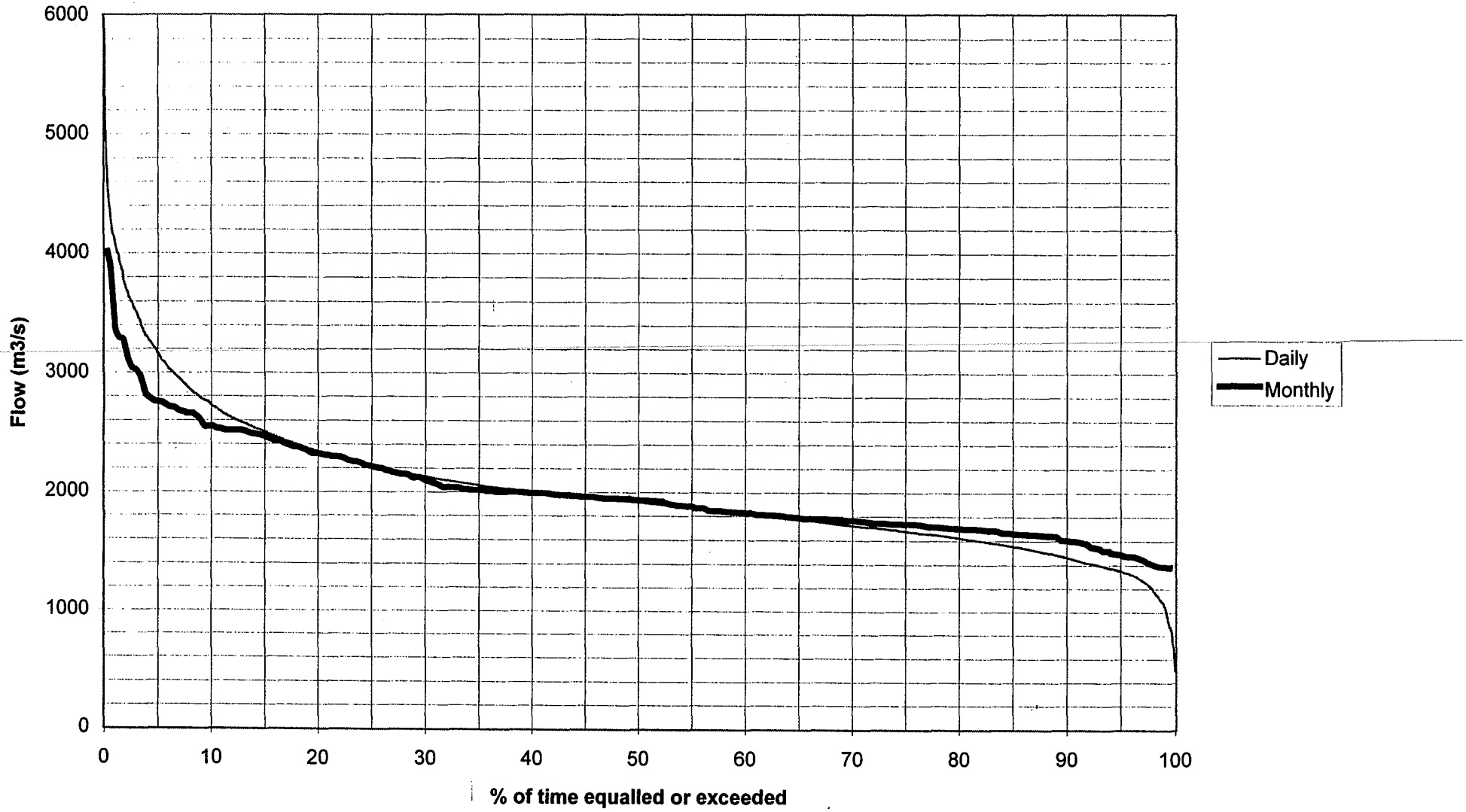


Figure 3.3

4.0 COST COMPARISON

The civil costs of each alternative were derived by manual calculations for excavation and the applicable formulae specified in a paper by Mr. J.L. Gordon[1] . The electro-mechanical costs of each alternative were computed using an in-house estimating reference data base. Tables 4.2 and 4.3 provide the comparative civil and electro-mechanical costs, respectively, for each alternative.

The energy production associated with each alternative, based on the use of propeller turbines, is as follows:

Alternative	Energy Production
4 units	5436 GWh/year
5 units	5500 GWh/year
6 units	5544 GWh/year

The energy production was calculated from the flow duration curve and the applicable plant efficiency curve associated with each alternative. Figure 3.3 provides the flow duration curve and Figures 4.1, 4.2 and 4.3 provide the efficiency curves for four, five and six units, respectively. The resulting comparative costs for each of the alternatives were computed using a present value of $225 million per TWh, as provided by Newfoundland and Labrador Hydro. Table 4.4 provides the comparative energy production, energy gain and present value of energy gain for the four, five and six unit alternatives.

4.1 ALL PROPELLER TURBINES

Table 4.1 provides the comparative total cost for the four, five and six unit alternatives, with all propeller turbines. On the basis of capital cost, the four-unit alternative is the lowest cost by almost $36 million. On the basis of capital and annual costs, the four-unit alternative is the lowest cost by over $59 million.

4.2 KAPLAN/PROPELLER MIX

In a previous analysis it was found that the use of one Kaplan unit in a four-unit alternative or a five-unit alternative would, for all practical purposes, produce the same energy as the six-unit propeller alternative. For simplicity of construction, the same synchronous speed was selected and the use of the same hydraulic passages were imposed on both types of turbine. With these constraints, the runner diameter of the propeller unit would be slightly less than the Kaplan unit but the civil works would be the same.

[1] J.L. Gordon, Powerhouse Concrete Quantitiy Estimates, Montreal Engineering Company Limited, Revised February 4, 1983.

The substitution of an optimized Kaplan turbine for an optimized propeller turbine would require a direct cost increase of about 25% of the cost of a propeller turbine, or about $4.5 million. Note 1 below Table 4.1 shows the cost difference with one Kaplan turbine in the four-unit and five-unit alternatives, and six propeller turbines in the six-unit alternative, would favour the four-unit alternative by over $73 million.

Furthermore, with an annual value of approximately $2,450,000, the additional annual energy produced with one Kaplan turbine would result in repayment of the additional cost of the Kaplan turbine in less than two years.

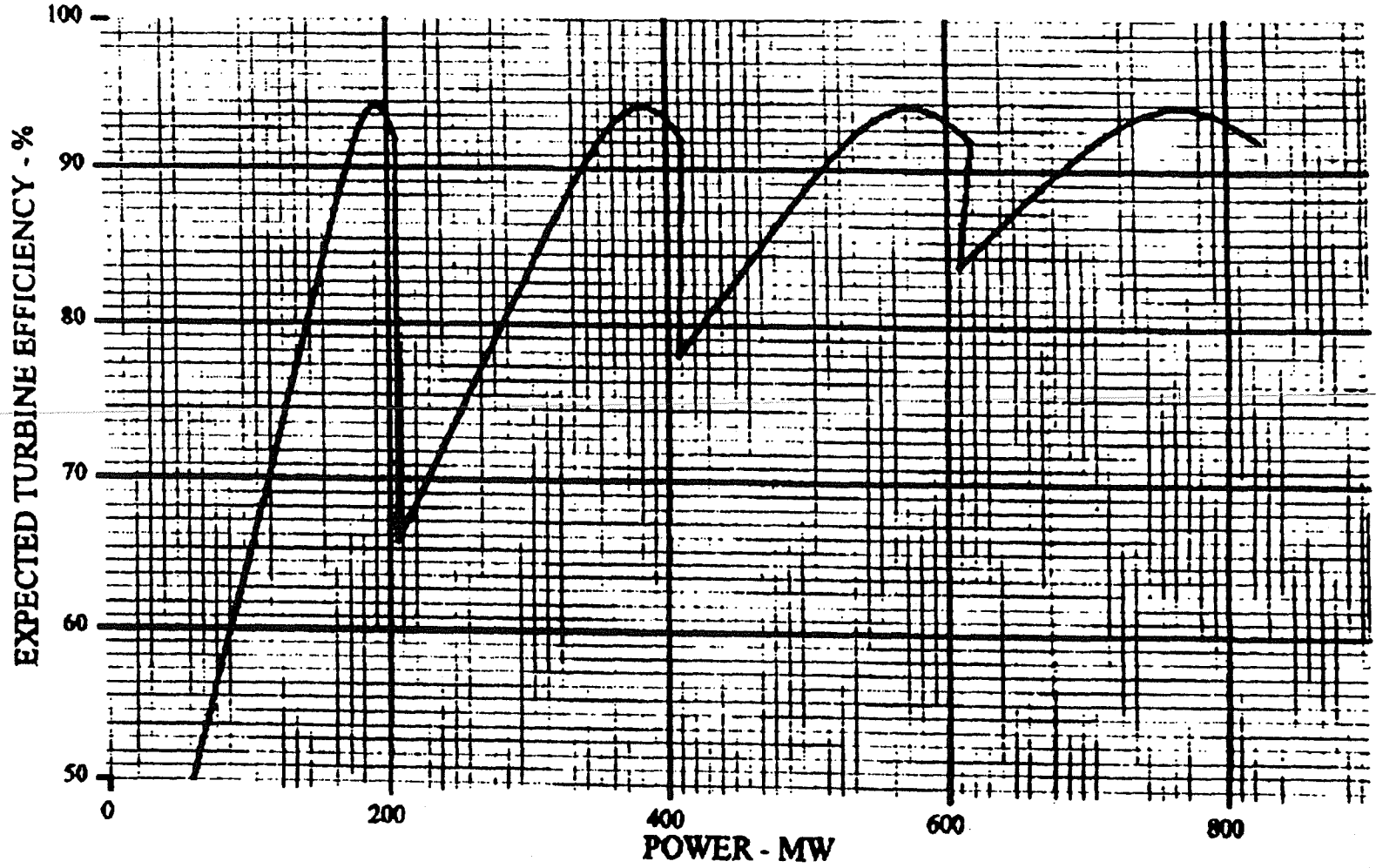
4.3 ALL KAPLAN TURBINES

With all Kaplan turbines in either the four-unit, five-unit or six-unit alternative, at an additional cost of about $4.5 million per unit, there would only be a small increase in total energy generation, as compared to the Kaplan/Propeller mixes in Section 4.2, above. Furthermore, each alternative would have the same energy generation. Note 2 below Table 4.1 shows the cost difference, with all Kaplan turbines in each alternative, would favour the four-unit alternative by obout $78 million.

MUSKRAT FALLS - 4 PROPELLER TURBINES

212 MW - 35.0 m net head - 81.8 rpm

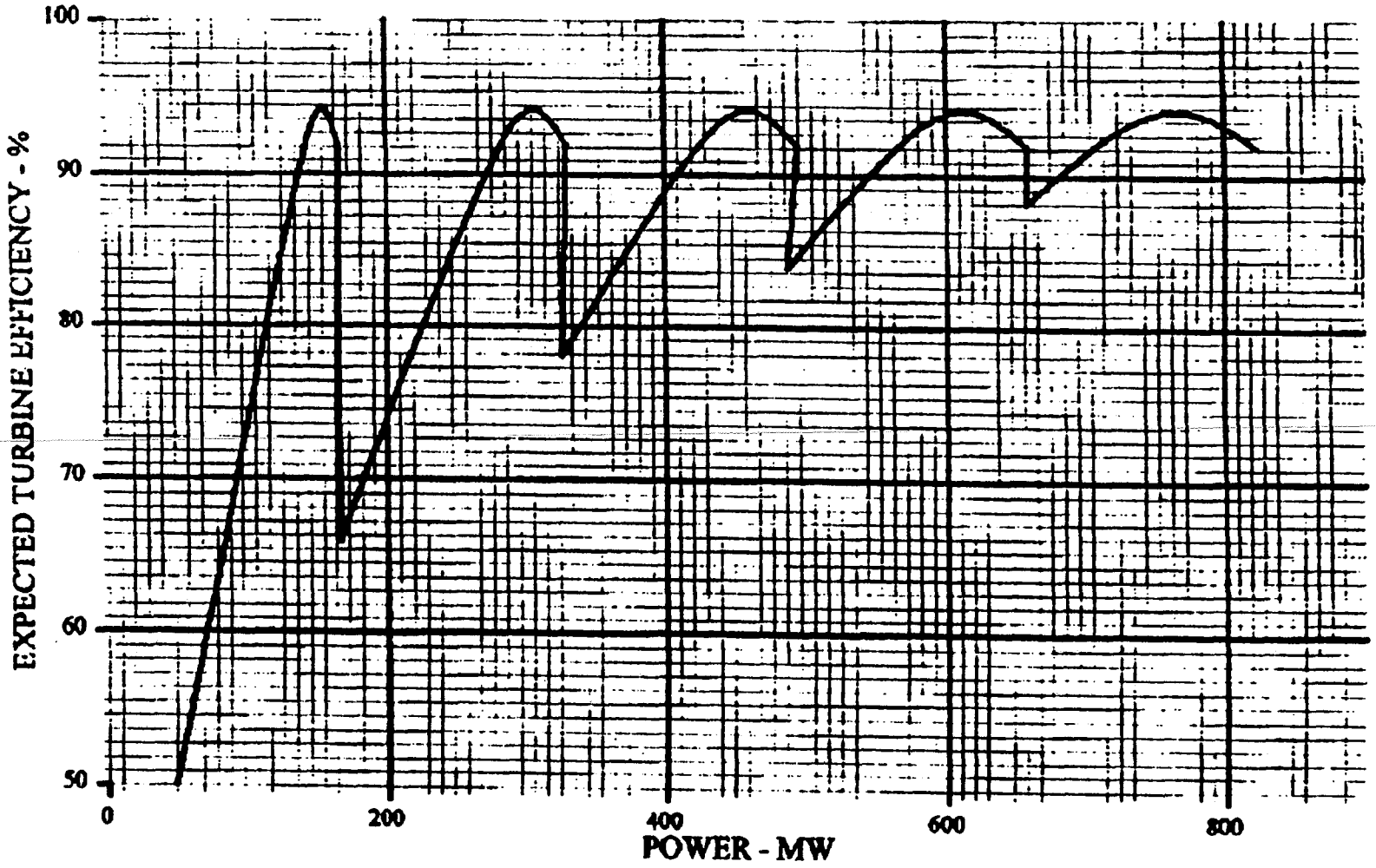
Figure 4.1



MUSKRAT FALLS - 5 PROPELLER TURBINES

169.6 MW - 35.0 m net head - 92.3 rpm

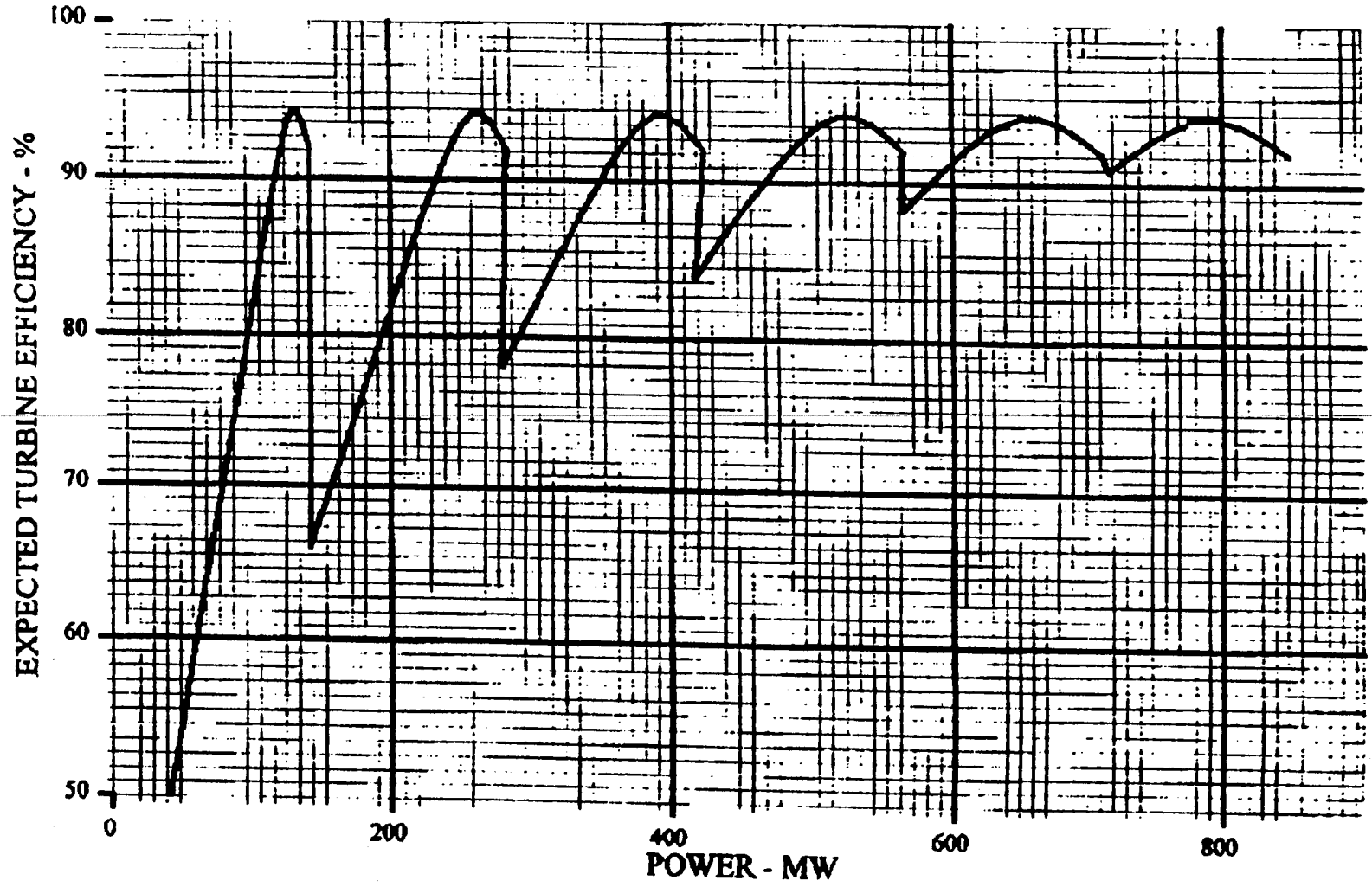
Figure 4.2



MUSKRAT FALLS - 6 PROPELLER TURBINES

141.3 MW - 35.0 m net head - 100 rpm

Figure 4.3



MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Unit Optimization - Size & Number

TABLE 4.1

COMPARATIVE TOTAL COSTS
(All Propeller Turbines)

	Description	**4 unit** 665 m^3/s 81.8 rpm 212 MW, ea.	**5 unit** 532 m^3/s 92.3 rpm 169.6 MW, ea.	**6 unit** 443 m^3/s 100 rpm 141.3 MW, ea.
1	Direct Costs:			
	- Civil costs (ref. page 2)	-	$4,500,000	$11,556,000
	- Electro mechanical costs (ref. page 3)	-	24,189,000	43,965,000
	Sub-Total difference - Directs	0	$28,689,000	$55,521,000
2	Indirect Costs:			
	- IDC @ 10%	0	7,172,000	13,880,000
	Sub-Total difference - Indirects	0	$7,172,000	$13,880,000
	Total $ difference - Capital Cost	**0**	**$35,861,000**	**$69,401,000**
3	Annual Financial Costs:			
	- Interest @ 10.00%	0	3,586,000	6,940,000
	- Sinking fund @ 0.026%	0	9,000	18,000
	- Insurance @ 0.10%	0	29,000	56,000
	- Interim replacement @ 0.20%	0	57,000	111,000
4	Annual O & M @ 0.30%	0	$108,000	$208,000
	Total $ difference - Annual Costs	0	3,789,000	7,333,000
5	**PV of Annual Costs** (i = 10%, n = 50 yrs)	**0**	**$37,567,000**	**$72,705,000**
	Total $ Difference in Costs	**0**	**$73,428,000**	**$142,106,000**
6	PV credit for Energy Gain (ref. page 4)	-	-$14,400,000	-$24,300,000
	Total $ Difference	**-**	**$59,028,000**	**$117,806,000**

Note 1: With the substitution of 1 Kaplan in the 4 unit and 5 unit installations, and the same energy credit for all three installations:			
Total $ Difference in Costs (compared to 4 propellers)	*$4,500,000*	*$77,928,000*	*$142,106,000*
PV credit for Energy Gain	*$0*	*$0*	*$0*
Total $ Difference	*$4,500,000*	*$77,928,000*	*$142,106,000*

Note 2: With all Kaplan turbines and the same energy credit for all three installations:			
Total $ Difference in Costs (compared to 4 propellers)	*$18,000,000*	*$95,928,000*	*$169,106,000*
PV credit for Energy Gain	*$0*	*$0*	*$0*
Total $ Difference	*$18,000,000*	*$95,928,000*	*$169,106,000*

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Unit Optimization - Size & Number

TABLE 4.2

COMPARATIVE CIVIL COSTS

Description	Unit	4 unit 665 m^3/s 81.8 rpm 212 MW, ea.	5 unit 532 m^3/s 92.3 rpm 169.6 MW, ea.	6 unit 443 m^3/s 100 rpm 141.3 MW, ea.
Parameters:				
N = number of units		4	5	6
Ne = equivalent number of units		4.45	5.47	6.48
R = repair bay length	m	29.70	27.30	25.70
S = unit spacing	m	32.97	28.90	26.87
d = turbine throat diameter	m	9.18	8.17	7.50
T = horizontal distance between u/s face of intake & vertical plane drawn at midway between the d/s face of the dt piers & the dt exit.	m	63.80	59.70	57.30
H = height of intake deck above foundation rock	m	46.00	45.50	45.00
h = gross head on the turbine	m	36.00	36.00	36.00
Concrete volume 1 unit (intake)	m^3	39,530	34,790	31,590
Concrete volume 1 unit (powerhouse)	m^3	26,590	20,100	16,370
Concrete volume 1 unit (total)	m^3	66,120	54,890	47,960
Concrete volume total	m^3	294,260	300,380	310,700
Cost @ $700/$m^3$		$205,982,000	$210,266,000	$217,490,000
$ difference in concrete			**$4,284,000**	**$11,508,000**
Powerhouse excavation	m^3	245,100	242,400	238,400
Powerhouse excavation difference	m^3	-	-2,700	-6,700
$ difference in excavation @ 30 $/$m^3$		-	**-$81,000**	**-$201,000**
Upstream approach channel excavation - dry	m^3	9,200	10,500	11,800
Tailrace channel excavation - dry	m^3	261,300	266,300	264,200
Tailrace channel excavation - wet	m^3	98,600	100,400	100,000
Channel excavation difference - dry	m^3	-	6,300	5,500
Channel excavation difference - wet	m^3	-	1,800	1,400
$ difference in excavation @ $30/$m^3$ dry & $60/$m^3$ wet		-	**$297,000**	**$249,000**
Total $ difference in civil		-	**$4,500,000**	**$11,556,000**

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Unit Optimization - Size & Number

TABLE 4.3

COMPARATIVE ELECTRO - MECHANICAL EQUIPMENT COSTS
(All Propeller Turbines)

	Description	4 unit 665 m^3/s 81.8 rpm 212 MW, ea.	5 unit 532 m^3/s 92.3 rpm 169.6 MW, ea.	6 unit 443 m^3/s 100 rpm 141.3 MW, ea.
1	Propeller turbines	$72,992,000	$85,526,000	$91,962,000
2	Alternators	62,557,000	71,639,000	79,658,000
3	Powerhouse crane	1,274,000	965,000	843,000
4	Transformers	11,196,000	11,844,000	12,400,000
5	Switchyard equipment	15,698,000	17,026,000	18,194,000
6	Aux. mechanical equipment	12,015,000	12,543,000	13,002,000
7	Aux. electrical equipment	11,077,000	11,077,000	11,076,000
8	Intake equipment	35,401,000	36,230,000	38,573,000
9	Draft tube equipment	4,549,000	4,098,000	5,016,000
	Total Costs	$226,759,000	$250,948,000	$270,724,000
	Total $ difference in E / M	-	**$24,189,000**	**$43,965,000**

MUSKRAT FALLS HYDROELECTRIC DEVELOPMENT

Unit Optimization - Size & Number

TABLE 4.4

COMPARATIVE ENERGY GAINS
(All Propeller Turbines)

	Description	**4 unit** 665 m^3/s 81.8 rpm 212 MW, ea.	**5 unit** 532 m^3/s 92.3 rpm 169.6 MW, ea.	**6 unit** 443 m^3/s 100 rpm 141.3 MW, ea.
	Average Annual Energy (GWh)	5,436	5,500	5,544
	Avg. Annual Energy gain (GWh)	-	64	108
	Present Value of Energy Gain (based on $225m/TWh/yr.)	-	$14,400,000	$24,300,000

5.0 CONCLUSIONS

- An integrated intake/powerhouse with a concrete semi-spiral case is the preferred arrangement for Muskrat Falls, for economy and constructability.
- A concrete semi-spiral case at Muskrat Falls, while large and with a high relative bursting force factor, is technically feasible and is not without presidence.
- An installation with four-units is the most economical, whether the installation comprises all propeller turbines, one Kaplan turbine in the four-unit and five-unit installations, or all Kaplan turbines. The summary relative costs are as follows:

Installation	4-Units	5-Units	6-Units
All propeller turbines	-	59,028,000	117,806,000
Å(relative to 4-units)	-	59,028,000	117,806,000
One Kaplan in 4-unit & 5-unit	4,500,000	77,928,000	142,106,000
Å(relative to 4 propeller units)	-	73,428,000	137,606,000
All Kaplan turbines	18,000,000	95,928,000	169,106,000
Å(relative to 4 propeller units)		77,928,000	151,106,000

- With the addition of one Kaplan turbine, at a cost of $4,500,000 and with an annual benefit of $2,450,000, the additional cost of the Kaplan turbine would be recovered in less than two years.

6.0 RECOMMENDATION

Based on the foregoing, the recommended installation for the Muskrat Falls Development is an integrated intake/powerhouse arrangement with a concrete semi-spiral case and with three (3) fixed blade propeller turbines and one (1) Kaplan turbine.

APPENDIX 1

Changjiang Water Resources Commission (CWRC)

1155, Liberation Ave. Tel: 86-27-82828607
Wuhan, 430010, P. R. China Fax: 86-27-82416125

Mr. David H. Brown

Lead Civil/Structural Engineer

SNC-AGRA Joint Venture

1133 Topsail Road, Mount Pearl,

NF A1N2W4

Fax: 709-368-5410

CC: Peter Mayers, M. Dessureault

October 22, 1998

Dear Mr. David H. Brown,

Your fax dated on Oct. 21, 1998 has been received.

With regard to the operating experience with the concrete semi-spiral case in the Gezouba Hydroelectric Plant, I would like to advise you that there is no any operating problems regarding the semi-spiral Gezouba Power Plant until now, that means Gezouba Power Plant has not experienced any cracking or leaking through the concrete walls, ceiling or floor of the semi-spiral. And the reinforced concrete semi-spiral case with the similar runner diameter and grass head to the proposed MF power plant has been recently used in the Gaobazhou project under construction in the Qingjiang River Hubei Province after the test of physical modeling.

I enjoyed my stay in Canada and am always happy to work with Canadian colleagues of AGRA Monenco and SNC-Lavalin.

With best regards.

Guowei, Yang Guowei Yang

Professor, General-Director

Bureau of Science/Technology & International Affairs

CWRC of MWR

APPENDIX 2

MUSKRAT FALLS DEVELOPMENT

RUNNER SELECTION

Joint Memo issued following a meeting held on October 7, 1998 with Mr Michel D. Roy, turbine expert from Hydro-Quebec.

1. Considering that the installation of four (4) axial turbine generating units has been selected as the optimal solution for the 35 meters net head, 824 MW capacity Muskrat Falls Powerplant, the proposed recommendation consisting of three (3) fixed blades Propeller turbines and one (1) Kaplan turbine represents a viable solution at this feasibility stage of the engineering studies.

2. During later design phases, a thorough analysis shall be undertaken to define the optimum combination and characteristics of the two turbine types including the case of four Kaplan type turbines. This analysis shall consider the possible "peak oriented" production scenario at the Churchill Falls powerplant with the additional two units actually foreseen.

APPENDIX F

LASALLE CONSULTING GROUP INC.
ICE STUDY

APPENDIX F

LASALLE CONSULTING GROUP INC.
ICE STUDY

REVISION

LaSalle Consulting Group Inc. issued the following report in October 1998. Revisions to the text were subsequently provided by LCG, following discussions with SNC-AGRA. These revisions are included on revised pages numbered 7, 8, and 9, dated November 1998.

LaSalle Consulting Group Inc.
9620, St.Patrick Street, LaSalle (Quebec) Canada H8R 1R8
Telephone : (514) 366-2970 / Fax : (514) 366-2971
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Study carried out for

SNC-AGRA

MUSKRAT FALLS

FINAL FEASIBILITY STUDY

ICE STUDIES

R.1364 **October 1998**

Prepared by : Nathalie Renaut

Nathalie Renaut, Eng.

Jean-Philippe Saucet, Eng.

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1. INTRODUCTION

A key problem involved in the development of Muskrat Falls is the formation of large ice jams in the river immediately downstream of the dam site. These jams are produced by the frazil ice generated upstream, in the fast flowing open water reaches. The jams raise the tailwater to reach levels that can be up to 17 m higher than under normal open water conditions.

During diversion, the downstream cofferdam has to be as low as possible in order for the construction costs to be minimized. This can be achieved by reducing the formation of ice and by stopping the frazil from being transported downstream. A numerical analysis of the ice conditions has established under what conditions this goal can be reached. The water level required to insure the closure of the ice cover upstream of Muskrat Falls has therefore been determined. A closure in early winter stops ice generation, and barely no ice transits through the diversion tunnels. The water level at the foot of Muskrat Falls which is to be considered in the design of the cofferdam is no longer affected by hanging dams.

The possibility that the thermal ice cover breaks in the spring time and accumulates against the closure cofferdam is of concern and has been considered in a separate chapter.

Since the diversion will take place after Gull Island becomes operational, all of these analyses but one were performed for the reach comprised between Gull Island and Muskrat Falls. This case, in which Gull Island is absent, involves an ice generating reach that is extended to Lake Winokapau and is the subject of a separate chapter.

2. WINTER DURATION AND FREEZING-INDEX

The relevant characteristics describing the winters used in the simulations have been established through a detailed analysis of the mean daily air temperatures at Goose-Bay for a period of 55 years (1942-1997). The winters are defined by the following parameters: the first and last days that air temperatures were below 0 °C, the duration and freezing-indices (number of freezing degree-days of these periods).

In the first phase of the analysis, the long term trend of the temperature variations were computed by averaging the daily temperatures over the 55 years of data. The averaged winter produced by these calculations begins October 31, when the temperature falls below freezing, and ends on April 22 as temperature rises above 0 °C. The total duration of this average winter is thus 174 days.

The first and last days of every winter have been established on the basis that the average of all the 55 first and last days should correspond to the first and last day of the averaged winter. In order to do so, two parameters have been adjusted. These are the number of consecutive days with negative temperatures that a winter must have at its beginning and at its end. By trial and error, we found that a sequence of 5 days is required before a winter can be considered to have started and that 8 days are needed for the winter to be over.

The resulting dates and freezing-indices are presented for the 55 winters in Table 1 and in Figure 1. The 1978-79 winter was discarded from the series because it exhibited non-typical length and freezing-index, probably resulting from a warm spell in September. For the remaining 54 winters, the mean freezing-index was found to be 1,920 °C-days and the standard-deviation 296 °C-days. For practical purposes, the 1963-64 winter may be regarded as a mean winter, while the 1971-72 and 57-58 winters represent extremely cold and warm winters, respectively.

The distribution of the observed freezing-indices closely match a normal distribution as shown in Figure 2, where the freezing-index of a specific winter has been plotted against the variable X according to the following equation:

$$erf(X) = 1 - i / (n+1)$$

where erf() is the error function and i the rank when sorting the n = 55 winters in decreasing order.

The trend line, established by a least-square fit is:

$$°C\text{-day} = 310.3\ X + 1920$$

and, for example, a winter having a return period of 40 years corresponds to 2,528 °C and is given by:

$$erf(X) = 1 - 1/40, \quad X = 1{,}96$$

The coldest winter observed, which was in 1971-72, had a slightly greater freezing-index of 2,551 °C-days and was used for simulation purposes when looking at extremely cold winters having return periods of 40 years or more. Two other scenarios of air temperature were also considered: 1968-69, a mild winter with its 1,414 °C-days, and 1963-64, a mean winter with 1,916 °C-days. The three scenarios are given in Figure 3.

3. FLOW SEQUENCES

3.1 Present Conditions

The ice regime prevailing for current conditions were studied using the two flow sequences given in Figure 4. One of them is for relatively low flows (1990-91) and the other one is representative of high flows (1981-82). These choices were made following a close examination of the flows at Muskrat given by the HYDAT database (Water Survey of Canada). During the 1990-91 winter, the discharge was fairly constant with a low value of 1,500 m^3/s and a peak of 2,000 m^3/s by the end of January and beginning of February. In 1981-82, the flow remained at constant value of 2,000 m^3/s during most of the winter.

3.2 Diversion Conditions

The flow sequences considered for diversion were based upon the results of simulations conducted on a monthly basis by Acres International Limited for 52 years (1943-1994). For practical purposes, the flows used in the simulations are those computed for Muskrat Falls when Churchill Falls I and II, at Gull Island and at Muskrat Falls are operational (CF Existing + 2*500MW+GI+MF+Fishery Rel. (Run 6)). In order to treat the case leading to the maximum ice generating surfaces and volumes, the year 1969, which had the greatest flows of all, was selected. The flow sequence, for which the daily flows have been linearly interpolated from the monthly discharges, is given in Figure 5.

Preliminary runs indicated that the intensity of the flows at the beginning of the winter, prior to freeze-up, had a major impact on the ice formation and progression processes, and consequently on the volume of ice arriving at Muskrat Falls. In order to consider these effects, the 1969 flow sequence was modified and two other flow sequences were established. These two scenarios, which are given in Figure 5, have fixed values at the beginning of the winter : sequence 2 has flows of 2,000 m^3/s until January 1 while in sequence 3 the flow is 2,500 m^3/s until January 10.

4. THE NUMERICAL MODEL

FRASIL is a freeze-up jam model developed by LaSalle Consulting Group. Appendix A presents this model in detail. Calculation of the hydraulic conditions (*backwater curves*) follows a one-dimensional procedure similar to the HEC-2 classical model formulation, with the difference that it explicitly takes into account the presence of ice : anchor ice, shore ice, the thermal cover and hanging dams. In another phase of the calculation process, the quantities and areas covered by each type of ice are calculated. A thermal balance is established as well as a longitudinal profile of the water temperatures. Open water zones are subsequently determined (a function of local velocity and rate of heat loss from water to air). Finally, the volume of frazil ice generated in these open areas is calculated. When encountering an ice cover closing the river from shore to shore, this drifting frazil either accumulates by juxtaposition or is carried under the cover, depending on the local hydrodynamic conditions. In the first case, there is a dynamic progression of the frontal edge upstream, whereas in the second case the frazil passing under the cover is deposited in zones where the water flow is slower, thus forming hanging dams according to a threshold velocity criterion for erosion or build-up [1].

The model usually operates on a daily time step and the data required every day are: discharge, upstream water level, air temperature and water temperature at the upstream edge.

---

(1) : The "mechanical stability" criteria do not apply to this study as it is considered, in accordance with the recommendations of the "Ad Hoc Committee on calculation criteria for the N.B.R. project (Hydro-Quebec)" that the thermal cover at the surface of a hanging dam provides ample mechanical stability. This does not hold during spring break-up and the accumulation of large ice floes formed during this period has to be analysed according to mechanical stability criteria.

5. BATHYMETRIC DATA

Simulations were carried out from downstream of Lake Winokapau to Muskrat Falls, but only the present conditions were studied for this whole reach. However, the conditions involving the operation of Churchill Falls II and Gull Island were studied for the reach comprised between Muskrat Falls and Gull Island since the ice which may be generated upstream from Gull Island would not flow past the Gull Island Station.

SNC-AGRA has performed HEC-2 backwater analysis for a 52.6 km reach described by 43 cross-sections surveyed in November 1975. The hydraulic conditions prevailing on November 3rd 1975, when the flow rate was 1,630 m^3/s, have been reproduced and the resulting data file, with the adjusted Manning and contraction coefficients, were establisehd by SNC-AGRA. Some further adjustments were required in order for FRASIL, the in-house model used to simulate the ice conditions, to produce the same results as HEC-2.

For the purpose of the ice study, the reach was prolonged by 4 km upstream in order to reach the Gull Island site. This was done by adding four estimated cross-sections and a cross-section located at the future dam site, obtained from the final report of the river closure model study (ref. 1).

The water elevation upstream of Muskrat Falls, which constitutes the downstream end of the reach, was determined from the rating curve provided to us by SNC-AGRA and given in Figure 6. This curve was established through a curve fit of field data. The elevations given on this curve are geodesic. There is a discrepancy of one meter between those elevations and the elevations of the surveyed cross-sections, and in order to reconcile the different data, it was decided to lower the cross-section elevations by one meter.

The upper reach running from the downstream end of Lake Winokapau down to Gull Island is 125 km long and defined by a set of estimated cross-sections that were established from the river width and water levels shown on the 1 :50,000 topographic maps. All elevations in this report are related to the geodesic datum.

6. STUDY OF THE CURRENT ICE CONDITIONS

LaSalle Consulting Group was provided with many reports of ice observation programs that were conducted over a period of years. A certain number of the observation programs, the most recent ones, were very detailed and accompanied by video tapes taken during helicopter flights. The surveys were made up to ten times throughout the winter, allowing a good appreciation of the evolution of the ice cover to be made. In addition to the detailed maps of the ice cover that were given in those reports, visual observations could be made using the video tapes. The two kinds of information complemented each other and have permitted us to obtain a satisfying degree of understanding of the ice formation processes involved from Cache River to Muskrat Falls.

The Churchill River was only studied downstream of Cache River because the information available indicated that the river is completely frozen from Churchill Falls up to this point, and therefore would not generate any frazil that could be transported downstream. Some reports have mentioned that for the winters of 1974-75, 1975-76 and 1976-77, when the flows were of 1,250 , 1,420 and 1,470 m^3/s, respectively, it was observed that an ice cover formed quickly from a point three miles downstream of Churchill Falls to Lake Winokapau. The video tapes show that the river was frozen upstream of Cache River, as no ice is seen drifting from upstream.

For the purpose of our study, we decided to explore two extreme conditions : the first condition gives the maximum generation of ice, while the second one produces the minimum. In the case where the maximum quantity of ice is to be generated, the hypothesis was made that the reach between Cache River and Gull Island is not covered with ice and thus generates large amounts of frazil, as appears to frequently be the case according to the ice observation programs. In the second scenario, we made the assumption that the river is frozen up to km 104, which is approximately 5 km upstream of Beaver Bk. The ice generating reach is therefore reduced by 40 km upstream of Gull Island.

The 1981-82 flows were maintained close to 2,000 m^3/s for the whole winter and are therefore considered high. In the first scenario, those flows were coupled with the 1971-72 temperatures. This winter, which has 2,551 °C-days, is the coldest in the past 45 years. The calculations performed under those conditions show that major hanging dams form in Sandy Island Lake and Gull Lake. In Gull Lake, the hanging dam has a maximum thickness of 30 m. Beside these accumulations, border ice is important but no thermal cover develops. Indeed, a channel remains ice free all winter long. Figure 7 shows the conditions on March 16 when the volume of ice in the reach is at its peak (270 hm^3 (1)). The downstream water elevation, upstream of Muskrat Falls, was varied from day to day according to the rating curve established at this location.

For the second scenario, we considered low flows combined with mild temperatures. The flow sequence for 1990-91 given in Figure 4 was used for these simulations. The average flow rate is approximately 1,500 m^3/s and the peak reaches 2,000 m^3/s on two occasions. The winter 1968-69 is considered to be mild since the total number of degree-days for the whole winter is only 1414, the lowest number of degree-days for the past 45 years. As in the precedent scenario, the ice is solely located in Sandy Island Lake and Gull Island Lake, but the volume of ice in the hanging dams is considerably reduced : 117 hm^3.

The computed volumes of ice evacuated at Muskrat Falls for those two scenarios are 418 hm^3 for high flows and cold temperatures and 236 hm^3 for low flows combined with mild temperatures. These volumes are definitely on the high side as they are computed based upon the skim-ice generated in the relatively slow flowing reach located downstream of Sandy Lake. The ice discharge is computed as the product of the skim-ice thickness and velocity mutiplied by the open water width, which is delimited by the border-ice. The width is generally overestimated since the border-ice progresses more slowly than in reality. This is on the safe side since the model's main purpose is to predict maximum ice generation and water levels.

---

(1) :1 $hm^3 = 1 \times 10^6 m^3$

The computations have been redone in order to obtain open channels having widths close to 400 m. The parameters involved in the progression of border-ice were adjusted accordingly, and the volumes of ice transiting through Muskrat Falls were reduced to 170 hm^3 in the first scenario and to 145 hm^3 in the second scenario.

A considerable quantity of the ice discharged downstream of Muskrat Falls accumulates in the bay in the immediate vicinity of the Falls, but since it is an accepted fact that the maximum slope under a hanging dam never exceeds 1/1000, or maybe 1,2/1000 (based for instance upon observations at LG2 prior to completion of the La Grande project, Québec), the frazil ice accumulation must therefore be 10 to 15 kilometers long in order to raise the water levels by 10 or 15 m or so. The hanging dam is thus not solely located in the hummocked area, but a significant volume of ice is stored further downstream, even if it is not revealed by a visual examination of the surface.

These results indicate that the range of ice volumes causing large water level increases downstream, at the foot of Muskrat Falls is of the order of 150 hm^3 depending on the winter flow and air temperature regime. However, we can maintain the downstream level at its low, open-water value if we reduce the volume of ice flowing through the diversion to 2 or 4 hm^3, which represents a few percents of the current value. A condition with, say, 50 hm^3 of ice going through the diversion would probably raise the downstream level and thus affect the cost of the required downstream cofferdam.

The original version of the model, which tends to overestimate the quantities of ice generated and therefore provides a safety factor, was used to perform the computations of the diversion conditions.

7. ICE CONDITIONS DURING THE MUSKRAT FALLS DIVERSION

7.1 The Need to Increase the Water Level at Muskrat Falls

The situation involving no water level control whatsoever at Muskrat Falls was analyzed first. The simulation was performed from just downstream of Gull Island to Muskrat Falls with the water level at the control section varied according to the flow and water level obtained from the rating curve (Figure 6).

The results were predictable : no ice cover formed and the volume of ice evacuated was 343 hm^3. This simulation clearly showed the need to raise the water level upstream of Muskrat Falls to a value high enough to allow an ice cover to form and to progress upstream.

7.2 Churchill Falls II and Gull Island in Operation

The diversion scheme retained at this point in time involves two tunnels that will regulate the water level in the reservoir. Our task was to determine at which elevation the reservoir should be set in order for a static ice cover to form and subsequently trap all of the incoming ice. We decided to simulate a certain number of cases involving different water elevations at Muskrat Falls as well as various air temperature and flow sequences. As it was assumed that during the diversion period, Churchill Falls II and Gull Island would be operating, simulations were started at Gull Island.

The simulations show that when the upstream water level at Muskrat Falls is sufficiently raised, a solid ice cover forms and extends from one shore to the other. This occurs at locations with low velocities : section 42 (km 2.5) and/or section 20 to 32 (km 14 to 17). Once a section is completely covered, it intercepts the incoming drifting ice, and the leading edge of the cover rapidly proceeds upstream. It then takes typically less than one

week to form a solid ice cover from the Falls up to km 20, while some fast flowing sections such as Flakes and Porcupine Rapids remain ice free for the remainder of the winter.

The ice generated in the open areas upstream of km 20, is easily accumulated and form hanging dams of moderate height (typically 5 m) in the region of km 10 to 20. No further ice is carried downstream regardless of the intensity of the winter.

The ten cases considered involve three water elevations at Muskrat Falls : +21, +22 and +23 m.

7.2.1 Level +21 m

The effects of air temperatures and flows have been studied for a water elevation of +21 m at Muskrat Falls through four combinations of the parameters.

Case 1 : - 1969 flow sequence with Q = 2,000 m^3/s in November and December, mean freezing index, 1,916 °C-days.

Results : - The ice cover forms by mid-November upstream of Muskrat Falls and rapidly progresses upstream.
- A total volume of **2.09 hm^3** flows through Muskrat Falls during the winter.

Case 2 : - 1969 flow sequence with Q = 2,000 m^3/s in November and December, extreme freezing index, 2,551 °C-days.

Results : - The ice cover forms at the beginning of November and a week is needed for it to progress up to km 20.
- A total volume of **2.34 hm^3** flows through Muskrat Falls during the winter.

Case 3 : - 1969 flow sequence (with flows of approximately 2,200 m^3/s in November and December), mean freezing index, 1,916 °C-days.

Results : - The ice cover forms by mid-December such that a considerable quantity of ice has already transited through Muskrat Falls.
- A total volume of **35 hm^3** flows through Muskrat Falls during the winter.

Case 4 : - 1969 flow sequence (with flows of approximately 2,200 m^3/s in November and December), extreme freezing index, 2,551 °C-days.

Results : - The ice cover forms by mid-December such that a considerable quantity of ice has already transited through Muskrat Falls.
- A total volume of **36 hm^3** flows through Muskrat Falls during the winter.

Case 5 : - 1969 flow sequence with flows of 2,500 m^3/s up to January 10, mean freezing index, 1,916 °C-days.

Results : - The ice cover forms by the beginning of January, such that a large amount of ice has flowed through Muskrat Falls by that date.
- A total volume of **88 hm^3** flows through Muskrat Falls during the winter.

7.2.2 Level +22 m

One situation has been studied for a water level of +22 m upstream from Muskrat Falls.

Case 6 : - 1969 flow sequence (with flows of approximately 2,200 m^3/s in November and December), extreme freezing index, 2,551 °C-days.

Results : - The ice cover forms by mid-November such that a small amount of ice has flowed through Muskrat Falls by that date.
- A total volume of **2.46 hm^3** flows through Muskrat Falls during the winter.

Case 7 : - 1969 flow sequence with flows of 2,500 m^3/s up to January 10, extreme freezing index, 2,551 °C-days.

Results : - The ice cover forms by mid-December
- A total volume of **33 hm^3** flows through Muskrat Falls during the winter.

7.2.3 Level +23 m

Case 8 : - 1969 flow sequence (with flows of approximately 2,200 m^3/s in November and December), mean freezing index, 1,916 °C-days.

Results : - The ice cover forms by mid-November
- A total volume of **1.17 hm^3** flows through Muskrat Falls during the winter.

Case 9 : - 1969 flow sequence (with flows of approximately 2,200 m^3/s in November and December), extreme freezing index, 2,551 °C-days.

Results : - The ice cover forms at the beginning of November
- A total volume of **2.31 hm^3** flows through Muskrat Falls during the winter.

Case 10 : - 1969 flow sequence with flows of 2,500 m^3/s up to January 10, extreme freezing index, 2,551 °C-days.

Results : - The ice cover forms at the beginning of November
- A total volume of **2.18 hm^3** flows through Muskrat Falls during the winter.

In Figure 8 are illustrated the ice conditions obtained on March 1 with case 9. Hanging dams form in regions of lower velocities and a thermal cover develops upstream from Muskrat Falls and extends up to km 20.

7.2.4 Conclusions

Table 2 describes the ten cases studied and gives the results obtained in terms of volume of ice evacuated during the winter and of the maximum volume of ice in the hanging dams.

The volume of ice arriving at the diversion site corresponds to the ice generated prior to the total closure of the upstream reach located between km 1 and 20. It therefore appears that the air temperatures and flows prevailing at the beginning of the winter have a major influence on the total volume of ice produced, while they have no effects on it once the ice cover is formed.

The results clearly show that a water level of +21 m is not appropriate when the flow exceeds 2,000 m^3/s in the beginning of winter and is therefore insufficient to retain all the flowing frazil. A water level of +22 m appears to be adequate, but a level of +23 m is required to insure the early formation of an ice cover in the event that the flow exceeds 2,220 m^3/s.

7.3 *The Effects of Air Temperatures on Ice Conditions in November and December*

In this section, ice generation prior to total closure of the upstream reach was considered. Ice is generated downstream of Gull Island [(1)], into the Porcupine and Flakes rapids and the reach leading to Muskrat Falls, which has not yet closed. The generating surface varies with the flow and the water level and air temperature are considered to be constant during a given week of November or December.

Figure 9, upstream level +20 m: With a flow of 2,500 m^3/s, the volume of ice generated each week increases as the air temperature drops and the heat-loss increases, even though the free surface due to border ice growth reduces. At -20 °C and below, the increased heat-loss is more than balanced by the surface reduction, and the ice discharge decreases even as it gets colder. Finally, at about -30 °C, the ice production drops suddenly once the border-ice extends from one shore to the other.

The ice generation is therefore maximum for a temperature of approximately -20 °C, whatever the flow, and the order of magnitude of this maximum is 15 to 25 hm^3 per week. An air temperature of about -20 °C for 2 or 3 weeks in a row in November or December, never dropping to -30 °C, would therefore produce enough ice to significantly raise the water levels downstream of the site. Such a sequence is not improbable and would raise the upstream level above +20 m.

The Figure 10 corresponds to an upstream level of + 21 m. Excessive generation is again observed when the air temperature remains close to -20 °C and the flow exceeds 2,000 m^3/s. At 2,000 m^3/s and less, the upstream reach closes as soon as the air temperature drops below freezing and the volumes of ice arriving at the diversion site are negligible. These results confirm the previous conclusion : a level of +21 m is required with the flow limited to 2,000 m^3/s for the first few weeks of the winter.

---

(1) : The water temperature at the Gull Island station is assumed to be 0°C, which is the worst case.

Flow between 2,000 and 2,375 m^3/s would require a level of +22 m (Figure 11), and flows between 2,375 m^3/s and 2,500 m^3/s would require a level of + 23 m (Figure 12) to attain total closure of the upstream reach as soon as the air temperatures are below zero.

7.4 Conclusions

An upstream water level raised to +23 m reduces the volume of ice flowing through the diversion works to a mere 1 % of its present value, and the water level downstream of Muskrat Falls remains at its low value all winter long [1], regardless of the flow (up to 2,500 m^3/s) and air temperatures throughout the winter.

This level is considered safe as it is established by computations based upon conservative criteria with respect to border ice formation and ice cover dynamics. However, considering the relative imprecision of the soundings and rating curve at the Falls, it would be advisable to raise this level by a safe margin. We therefore recommend that the upstream level should be raised to +24 m.

---

(1) : This level, to be considered for the design of the downstream cofferdam, should be evaluated by a backwater computation starting from Goose Bay at high tide, taking into account the presence of a solid ice cover along the reach, 1 m thick, and considering a flow of 2,500 m^3/s. This requires some sounding of the reach.

8. WITH CHURCHILL FALLS II BUT WITHOUT GULL ISLAND

Another scenario was investigated with Churchill Falls II and without Gull Island. For this case, the ice generation reach was extended 100 km to Lake Winokapau and the conditions used are identical to those of case no 9. The results of this simulation involving high flows (1969), cold air temperatures and a water level at Muskrat Falls of +23 m are very conclusive. The volume of ice evacuated at Muskrat Falls was found to be 2.48 hm^3, an increase of 0.17 hm^3 over case no 9 conducted with Gull Island upstream. We observed from the simulation that there is a significant quantity of ice that exits Sandy Island Lake which accumulates further downstream. It would therefore appear that, in the event that diversion takes place before Gull Island becomes operational, conditions will still be satisfactory because the ice cover upstream of Muskrat Falls will trap all of the incoming drifting ice.

9. STABILITY ANALYSIS

The stability of the ice covers that form under different conditions in the upstream reach has to be evaluated in terms of ice motion, breaking and jamming. The discharge variations that are likely to occur will raise or lower the water level in the upstream reach, and the ice cover will move accordingly.

Water level variations produce hinge cracks to form along the banks, which reduce adhesion at the cover and its capability to resist forces (shear stresses underneath the ice and its weight component along the hydraulic gradient). Theoretical analysis show that water level variations of 0.10, 0.22 and 0.31 m produce hinge cracking of ice covers of 0.10, 0.50 and 1 m thick. A flow variation of the order of 100 m^3/s would generally suffice to induce hinge cracking of the cover.

The driving forces are resisted by bank friction, and the final thickness of the accumulation of broken ice is governed by the classical bell-shaped curve [1]. In the remote hypothesis of a general failure and jamming of the cover, it can be easily proven using this curve that the cover would stabilize itself once the thickness of the jam has reached two meters : this corresponds to minor ice motion and jamming, as the initial thickness of the solid cover is 0.5 to 1 m (0.8 to 1.6 m when a void ratio of 0.4 is taken into account after initiation of the jamming process).

The results of this analysis are presented in Figure 13. Under a given water level, the risk of instability leading to shoving and thickening of the cover is larger when the velocity is high. We have therefore considered the maximum discharge under a given water elevation, with regard to the rating curve of the diversion tunnels (fully open

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(1) : Formation of Ice Covers and Jams in River, by E. Pariset and R. Hausser, Proceedings of the American Society of Civil Engineering, Journal of the Hydraulics Division, November 1966.

gates) [1]. The relative thickness t/H was computed assuming a thickness t = 2 m. The values for the width B, the Chezy C and the height H were established by a backwater computation and averaged throughout the downstream part of the reach, km 0.42 to 19.5. The stability index Y was then evaluated, using a conservative value of μ = 1,15. It is demonstrated in Figure 13 that the corresponding points X, Y are below the limiting curve, by a safe margin ; in other words, the cover, even under the remote eventuality of a general failure and collapse, would not thicken in excess of 2 meters, as the internal friction balances the driving forces before reaching this thickness.

Furthermore, when the X,Y point lies on the bell-shaped curve, the resisting forces are exactly equal to the driving forces. Inside the curve, the resisting forces are larger than the driving forces, and the safety factor is greater than 1. On Figure 13, curves corresponding to safety factors of 1.25, 1.50,... have been plotted. One can see that the safety factor increases when the characteristic points move from A to G, and the risk of severe jamming is reduced at large flow. This is the result of the rating curve of the tunnels, which provides significant raising of the water level and speed reduction into the upstream reach under large flow.

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(1) : In this section, it is considered that the diversion is made through 6 gates, 6 m wide (sill at 8.0 m). The maximum discharge corresponds to 6 gates fully open, with critical flow conditions at the intake. These levels have been provided by SNC-AGRA.

10. CONCLUSIONS

Numerical analyses of the ice conditions during diversion at Muskrat Falls were performed in this study and indicated that a water level of + 24 m is required upstream of Muskrat Falls to insure total closure of the ice cover in early winter, and thus stop the generation of ice. Under these circumstances, the tailwater level is not affected by any ice jams, and the downstream cofferdam can be designed at minimal costs. The risks of spring break-up have been considered in a stability analysis that showed that the ice cover formed under the recommended conditions can resist stresses induced by flows up to 5,300 m^3/s.

These conclusions were derived for conditions involving Gull Island, but a rough analysis conducted without Gull Island show that equivalent conditions will prevail.

TABLE 1 : Winter Duration and Freezing Index
GOOSE-BAY, 1942-1997

Year	First Day	Last Day	Freezing-Index
1942 -1943	11/7	5/3	2211
1943 -1944	11/1	5/4	1886.4
1944 -1945	10/26	5/1	1864.2
1945 -1946	10/22	4/21	2132.8
1946 -1947	11/1	5/14	1848.8
1947 -1948	11/5	4/25	2014.8
1948 -1949	11/12	3/28	1749
1949 -1950	10/31	4/17	2296.9
1950 -1951	10/21	3/28	1453.5
1951 -1952	10/23	4/25	1782
1952 -1953	10/18	3/23	1635.9
1953 -1954	10/29	4/28	2034.3
1954 -1955	10/24	4/16	1636.7
1955 -1956	11/12	5/13	1526
1956 -1957	11/9	5/3	2134.3
1957 -1958	10/22	4/26	1146.2
1958 -1959	10/17	4/18	2008.4
1959 -1960	11/6	4/24	1452.6
1960 -1961	11/5	3/29	1894.1
1961-1962	11/7	3/22	1534.9
1962 -1963	10/27	4/12	1916.8
1963 -1964	10/8	5/8	1916.1
1964 -1965	10/11	4/27	1885.1
1965 -1966	10/7	3/18	1451.1
1966 -1967	11/12	4/28	2081
1967 -1968	11/6	4/22	1714.5
1968 -1969	11/6	5/2	1414.4
1969 -1970	10/18	5/6	1634.1
1970 -1971	11/1	4/13	1829.1
1971-1972	11/1	5/13	2551.4
1972 -1973	10/23	4/22	2322.9
1973 -1974	11/5	5/7	2101.8
1974 -1975	10/15	5/7	2177.1
1975 -1976	11/2	4/27	2304.8
1976 -1977	10/22	5/3	1743.8

TABLE 1 (cont'd)

1977 -1978	11/19	5/6	1836.1
1978 -1979	10/16	1/22	1077.2
1979 -1980	10/27	3/19	1631.4
1980 -1981	11/16	4/18	1509.2
1981-1982	11/8	4/29	1945.6
1982 -1983	10/25	4/13	2215.8
1983 -1984	11/7	4/23	2220.2
1984 -1985	10/30	5/13	1974.9
1985 -1986	10/9	4/17	1988.1
1986 -1987	10/14	4/15	1963.8
1987 -1988	10/30	4/26	1887.9
1988 -1989	11/7	4/12	2145.2
1989 -1990	11/17	5/9	2214.5
1990 -1991	10/22	5/6	2199.6
1991 -1992	11/4	5/9	2296.2
1992 -1993	10/30	5/10	2389.5
1993 -1994	10/22	5/5	2291.8
1994 -1995	11/22	4/11	2079.1
1995 -1996	11/17	4/21	1658.2
1996 -1997	11/11	4/25	1934.5

Table 2: Simulations Results - Diversion at Muskrat Falls (with Gull Island)

Case	Downstream Water Elevation	Flow Sequence	Scenario of Temperature	Volume Evacuated (hm^3)	Max. Volume in Place (hm^3)
1	21	2	mean	2.09	163
2	21	2	extreme	2.34	166
3	21	1	mean	35	170
4	21	1	extreme	36	179
5	21	3	mean	88	171
6	22	1	extreme	2.46	171
7	22	3	extreme	32	198
8	23	1	mean	1.17	157
9	23	1	extreme	2.31	165
10	23	3	extreme	2.18	181

Flow Sequences:

1 : year 1969

2: year 1969, except that Q=2000 m^3/s up to January 1

3: year 1969, except that Q = 2500 m^3/s up to January 10

Scenarios of temperature:

extreme: year 1971 - 1972 (2551°C-days)

mean: year 1963-1964 (1916 °C-days)

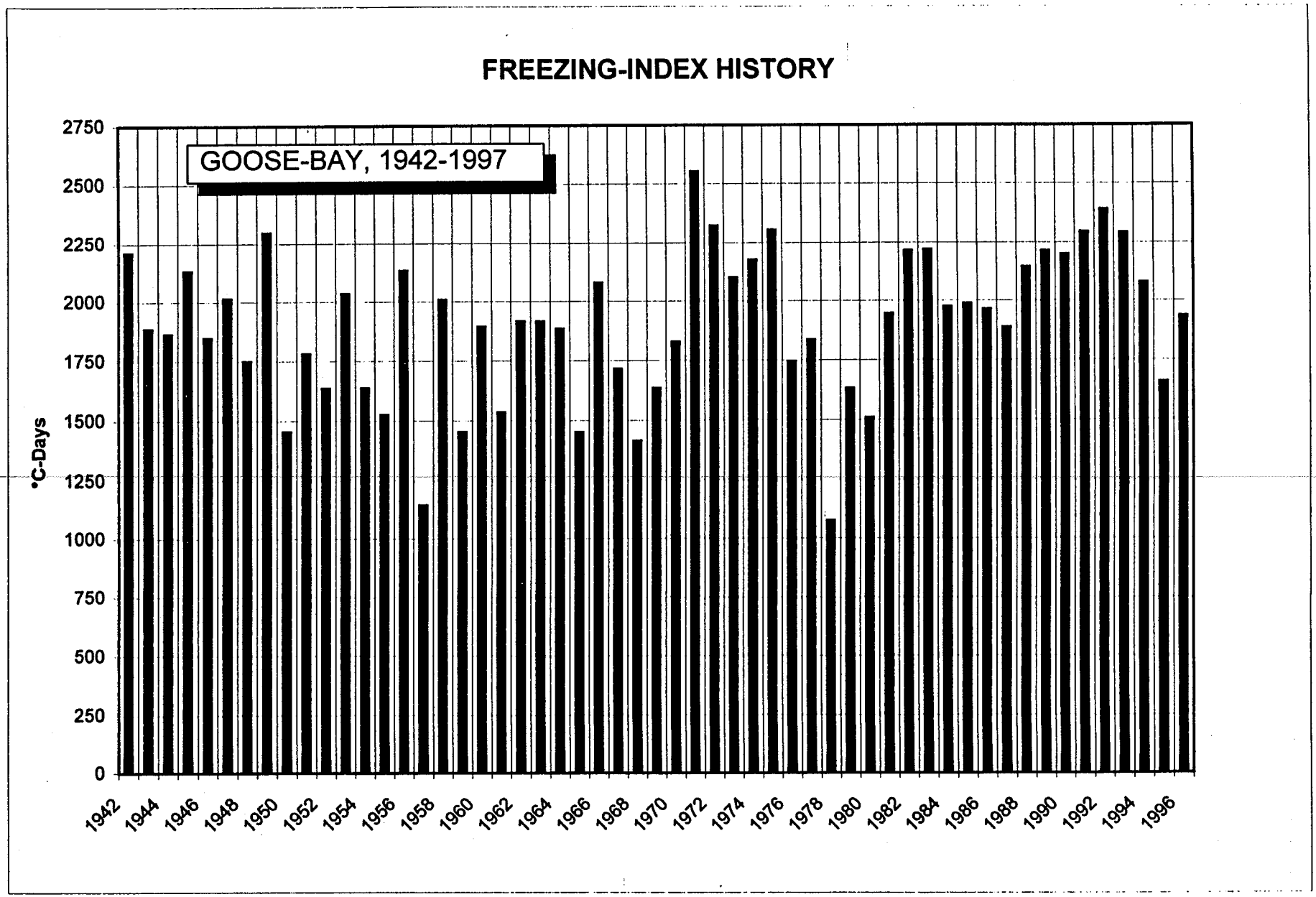

FREEZING-INDEX HISTORY
GOOSE-BAY, 1942-1997
°C-Days
2750
2500
2250
2000
1750
1500
1250
1000
750
500
250
0
1942
1944
1946
1948
1950
1952
1954
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
1996

FIGURE 1

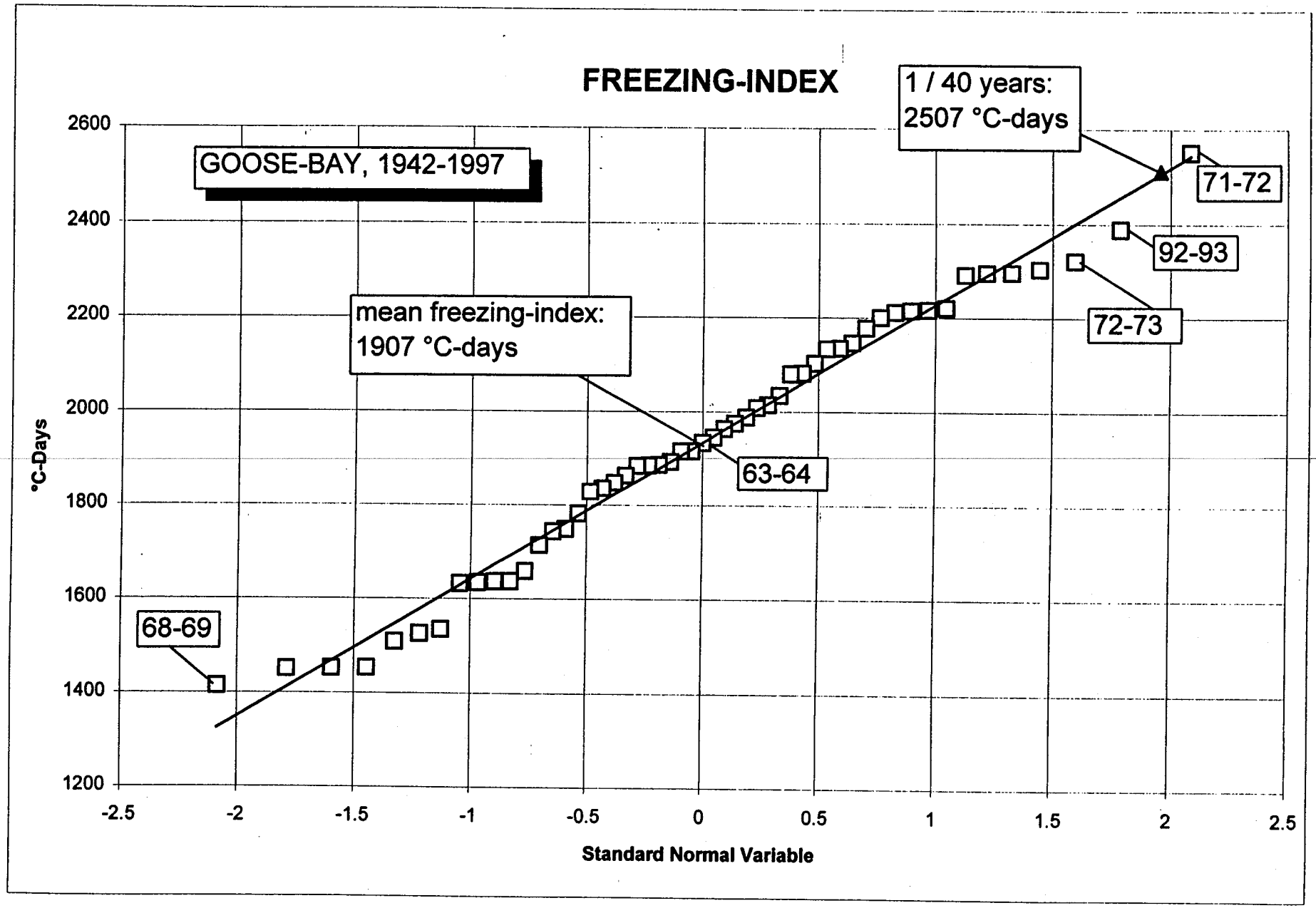

FREEZING-INDEX
GOOSE-BAY, 1942-1997
1 / 40 years:
2507 °C-days
mean freezing-index:
1907 °C-days
71-72
92-93
72-73
63-64
68-69
°C-Days
2600
2400
2200
2000
1800
1600
1400
1200
-2.5
-2
-1.5
-1
-0.5
0
0.5
1
1.5
2
2.5
Standard Normal Variable

FIGURE 2

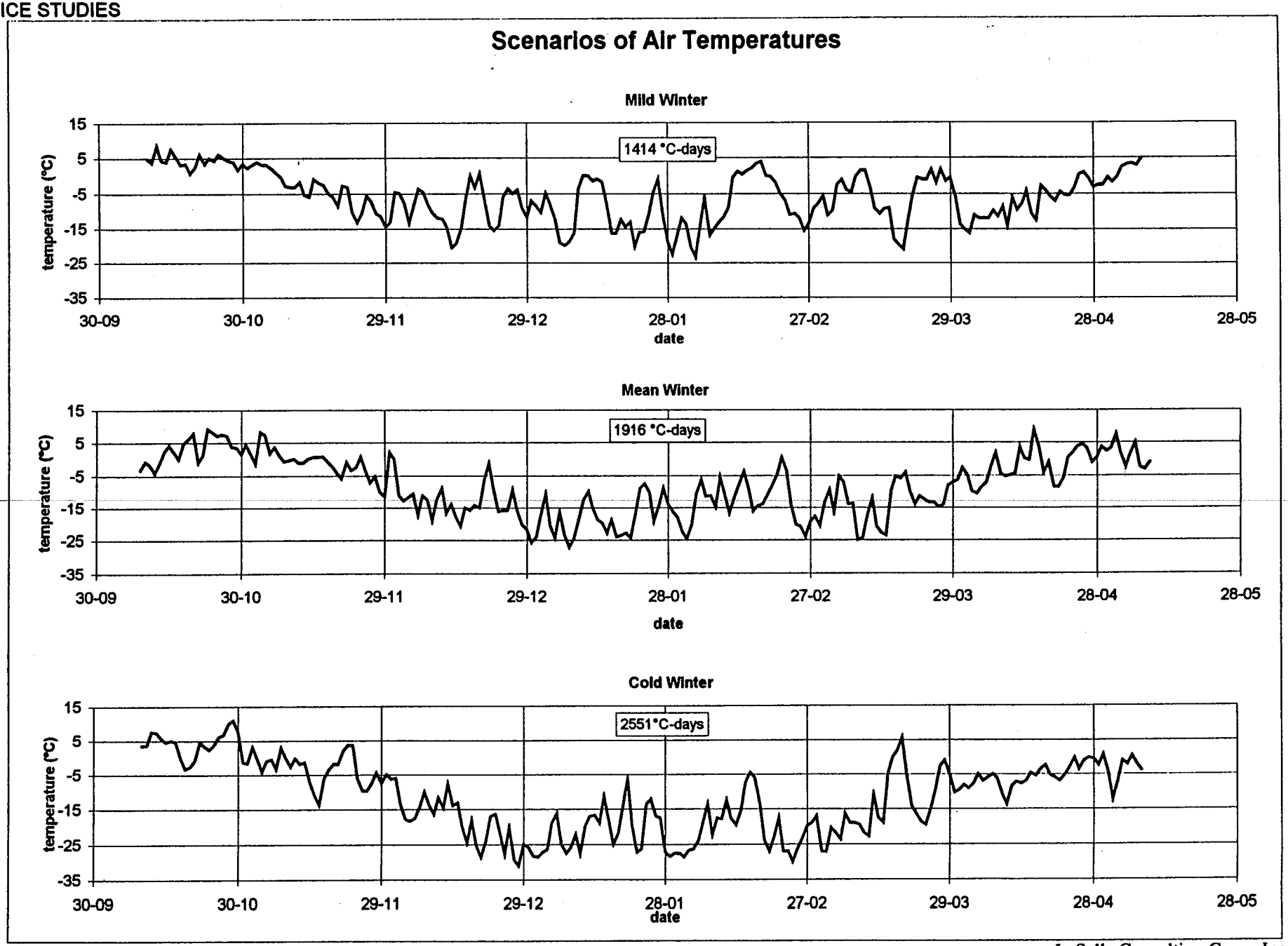


FIGURE 3

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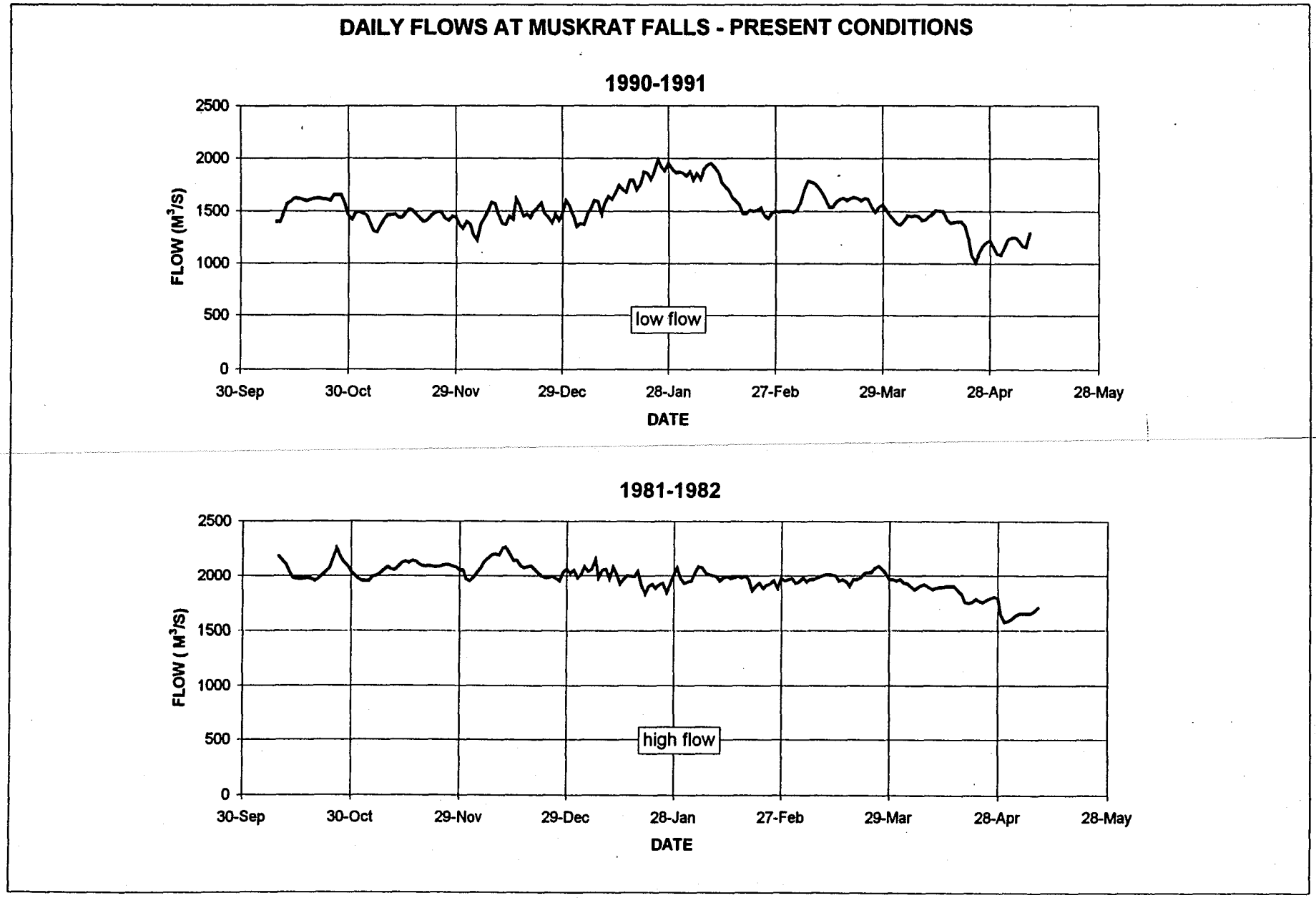
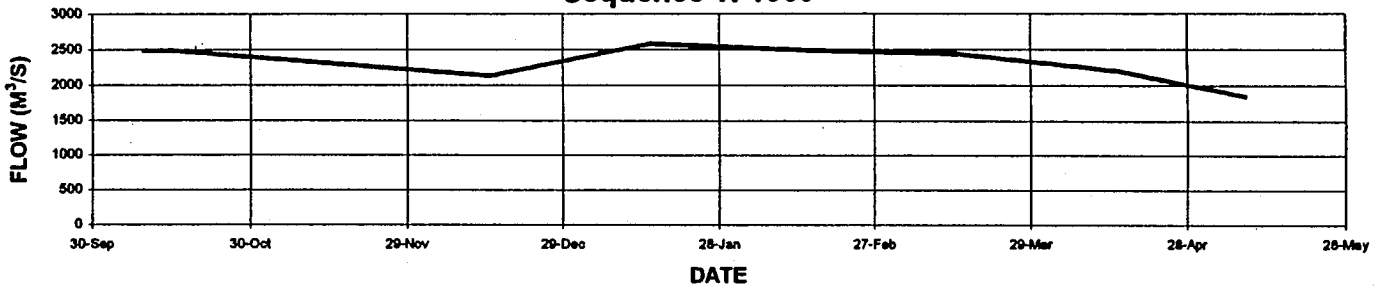

DAILY FLOWS AT MUSKRAT FALLS - PRESENT CONDITIONS
1990-1991
FLOW (M3/S)
2500
2000
1500
1000
500
0
low flow
30-Sep
30-Oct
29-Nov
29-Dec
28-Jan
27-Feb
29-Mar
28-Apr
28-May
DATE
1981-1982
FLOW (M3/S)
2500
2000
1500
1000
500
0
high flow
30-Sep
30-Oct
29-Nov
29-Dec
28-Jan
27-Feb
29-Mar
28-Apr
28-May
DATE

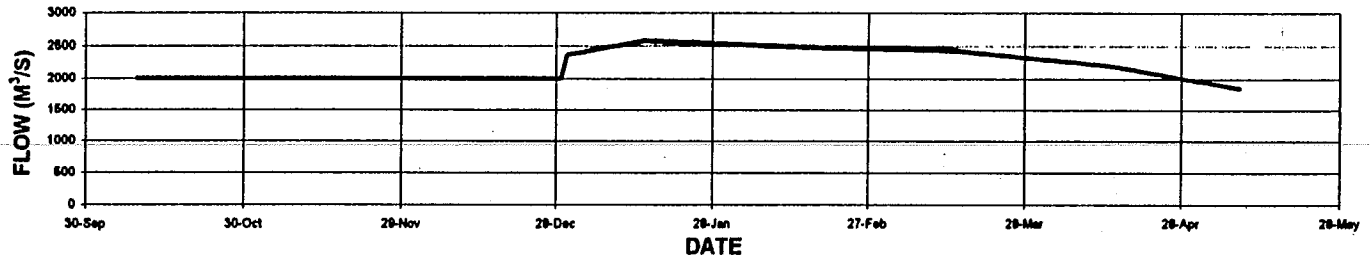
FIGURE 4

DAILY FLOWS AT MUSKRAT FALLS - FUTURE CONDITIONS

Sequence 1: 1969



Sequence 2



Sequence 3

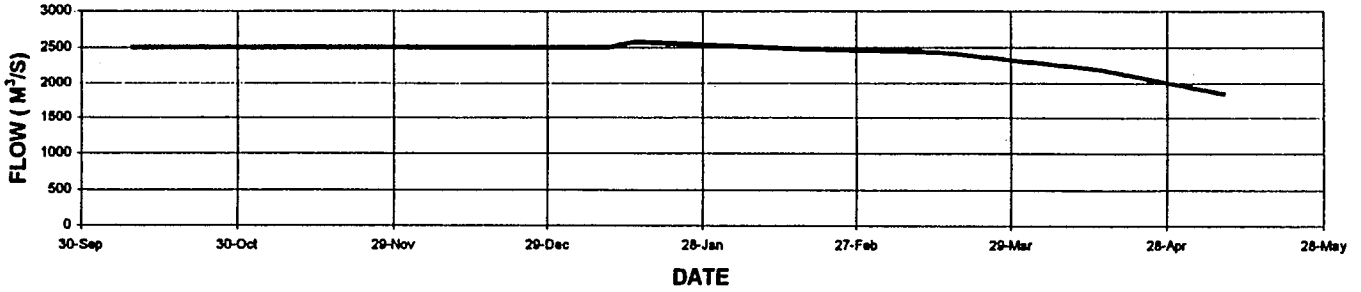


FIGURE 5

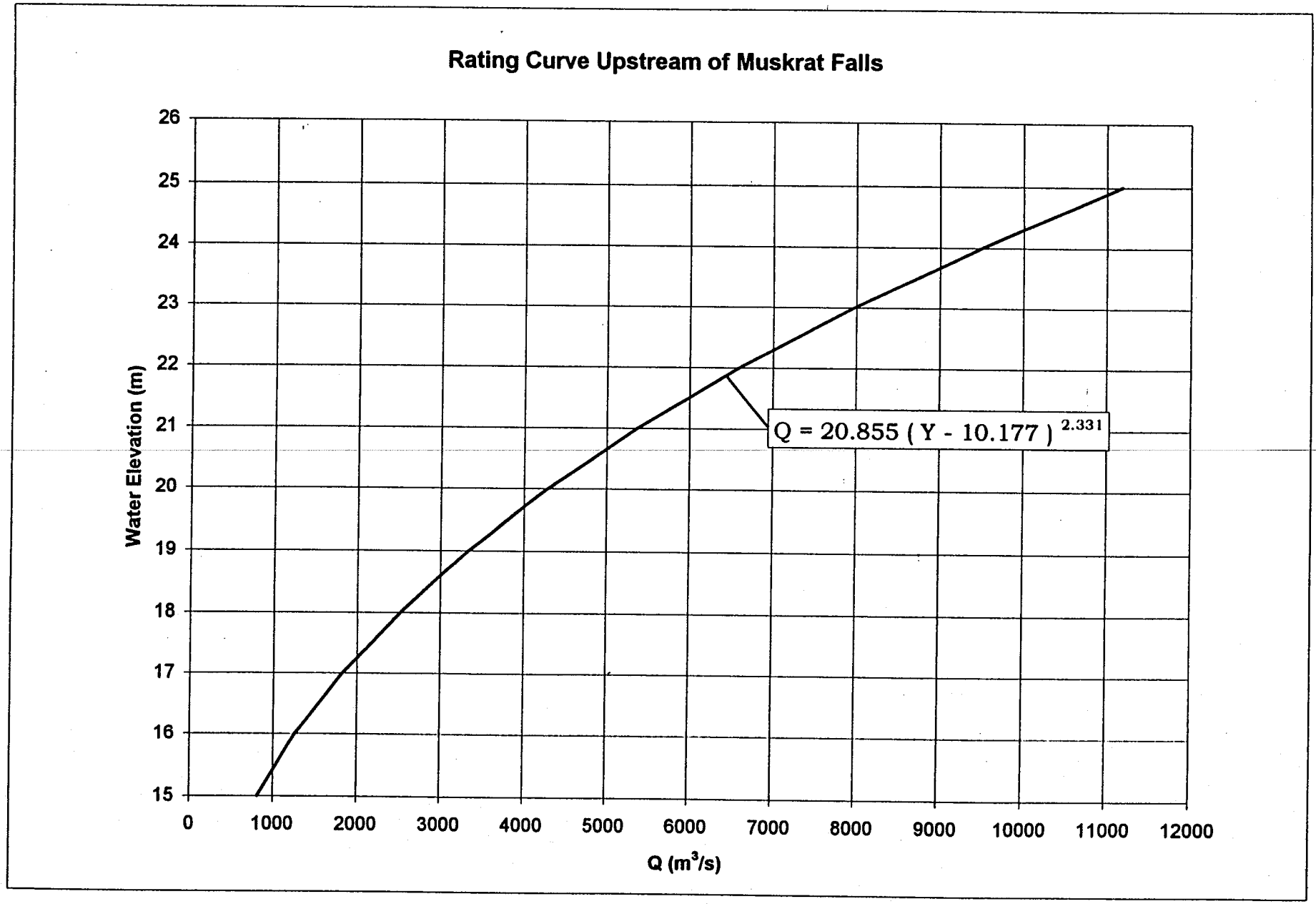


FIGURE 6

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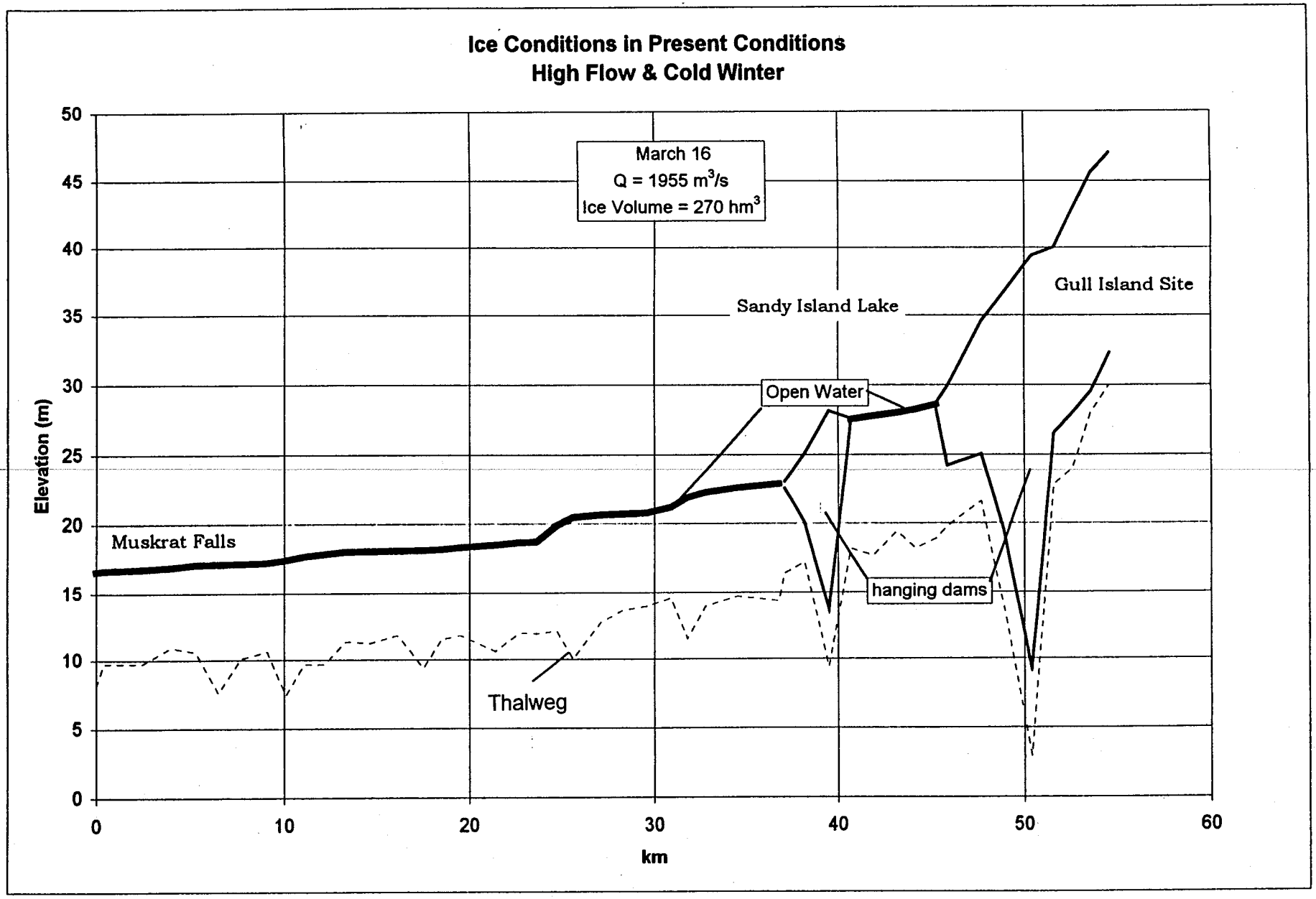

Ice Conditions in Present Conditions
High Flow & Cold Winter
March 16
Q = 1955 m3/s
Ice Volume = 270 hm3
Sandy Island Lake
Gull Island Site
Open Water
Muskrat Falls
hanging dams
Thalweg
Elevation (m)
km
0
5
10
15
20
25
30
35
40
45
50
0
10
20
30
40
50
60

FIGURE 7

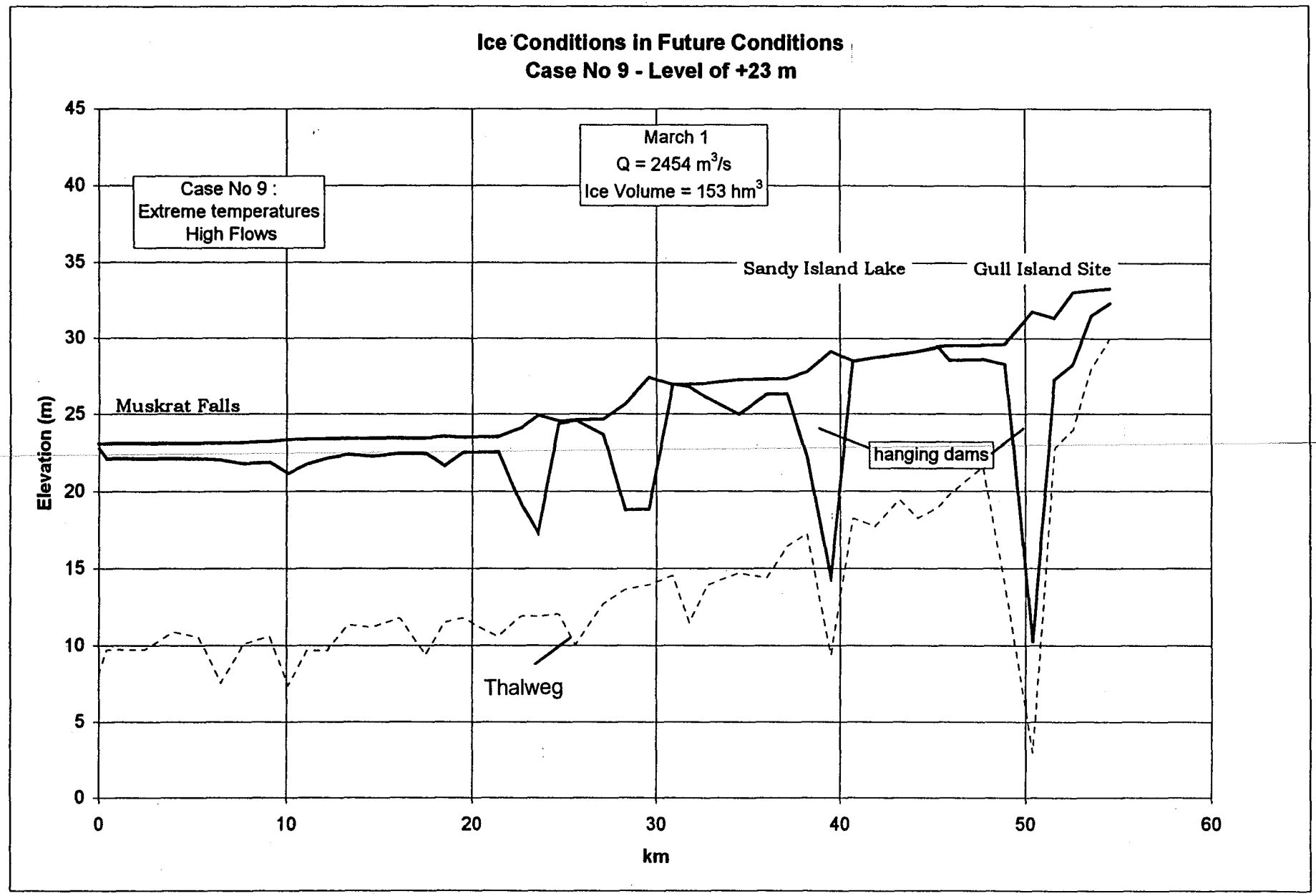


FIGURE 8

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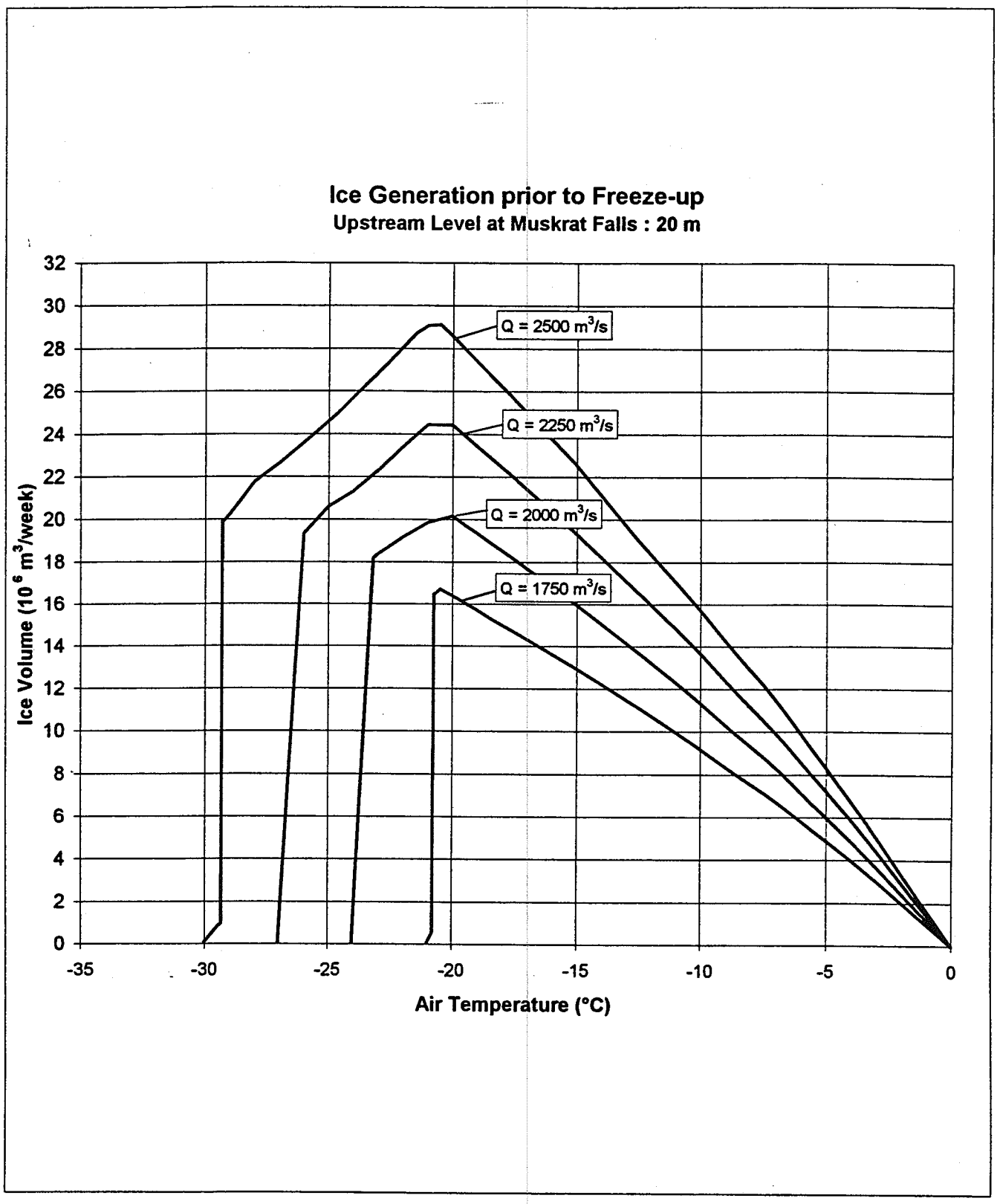

Ice Generation prior to Freeze-up
Upstream Level at Muskrat Falls : 20 m
Ice Volume (10^6 m^3/week)
Air Temperature (°C)
Q = 2500 m^3/s
Q = 2250 m^3/s
Q = 2000 m^3/s
Q = 1750 m^3/s
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
-35
-30
-25
-20
-15
-10
-5
0

FIGURE 9

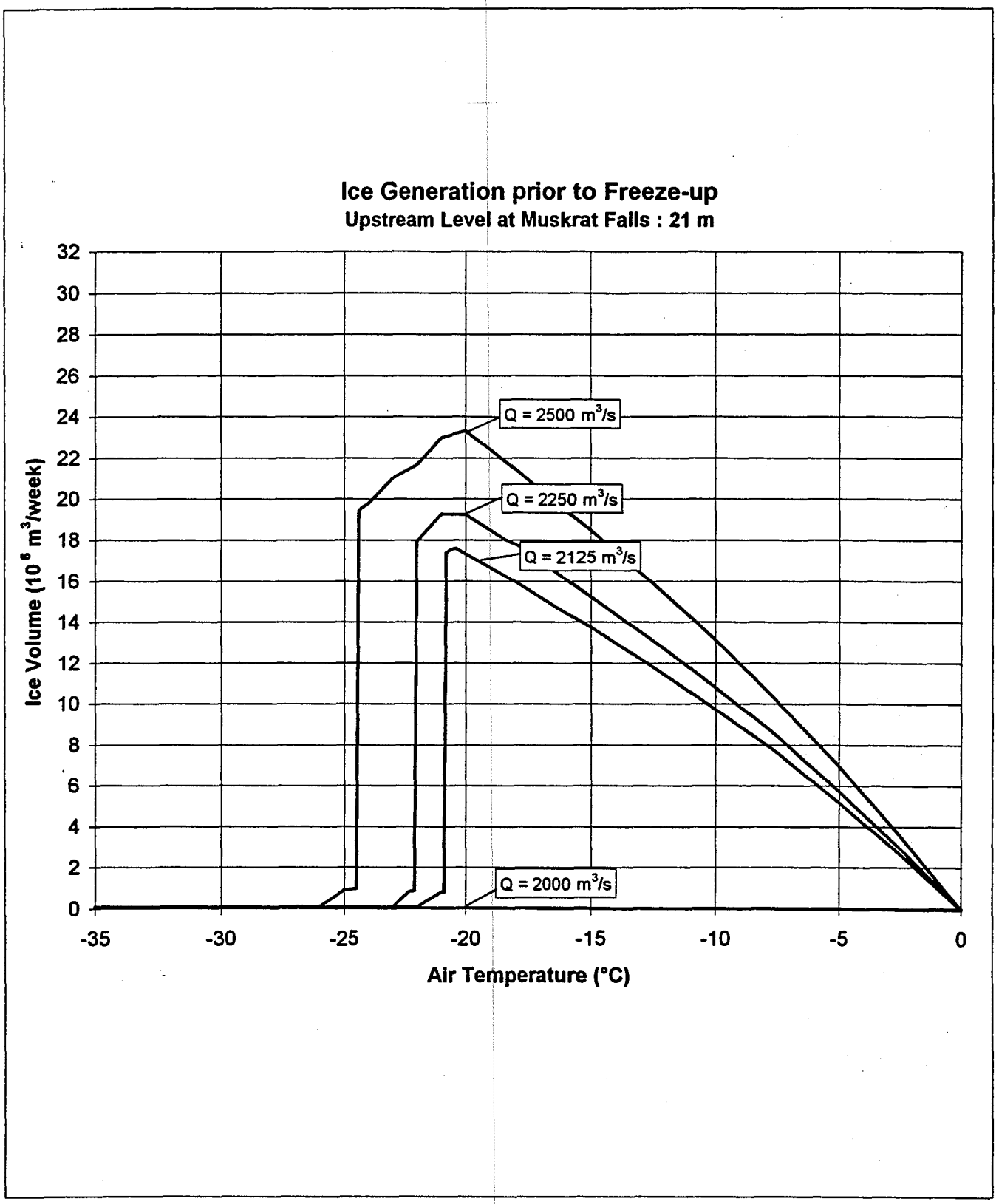

Ice Generation prior to Freeze-up
Upstream Level at Muskrat Falls : 21 m
Ice Volume (10^6 m^3/week)
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
Q = 2500 m^3/s
Q = 2250 m^3/s
Q = 2125 m^3/s
Q = 2000 m^3/s
-35
-30
-25
-20
-15
-10
-5
0
Air Temperature (°C)

FIGURE 10

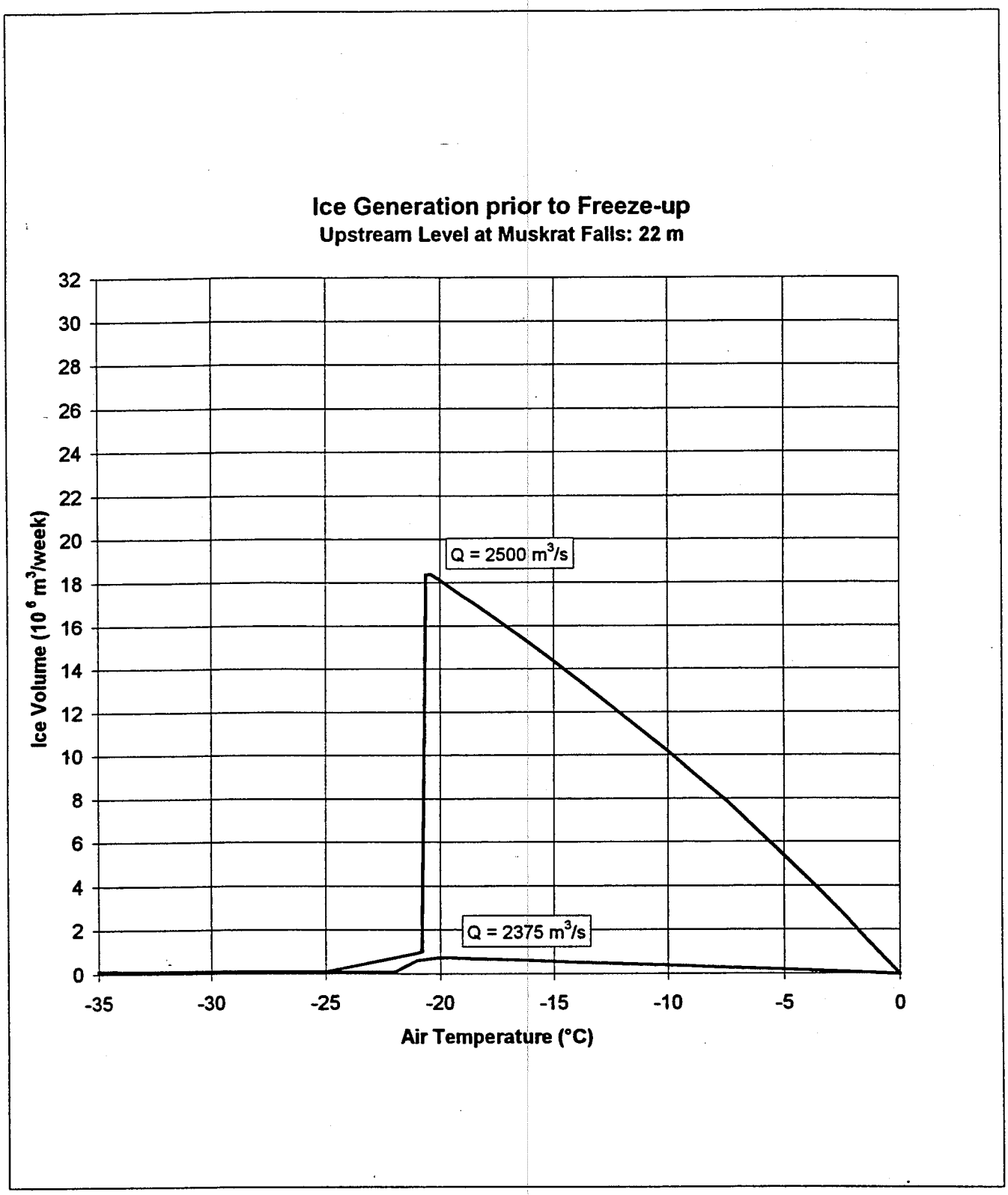

Ice Generation prior to Freeze-up
Upstream Level at Muskrat Falls: 22 m
Ice Volume (10^6 m^3/week)
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
-35
-30
-25
-20
-15
-10
-5
0
Air Temperature (°C)
Q = 2500 m^3/s
Q = 2375 m^3/s

FIGURE 11

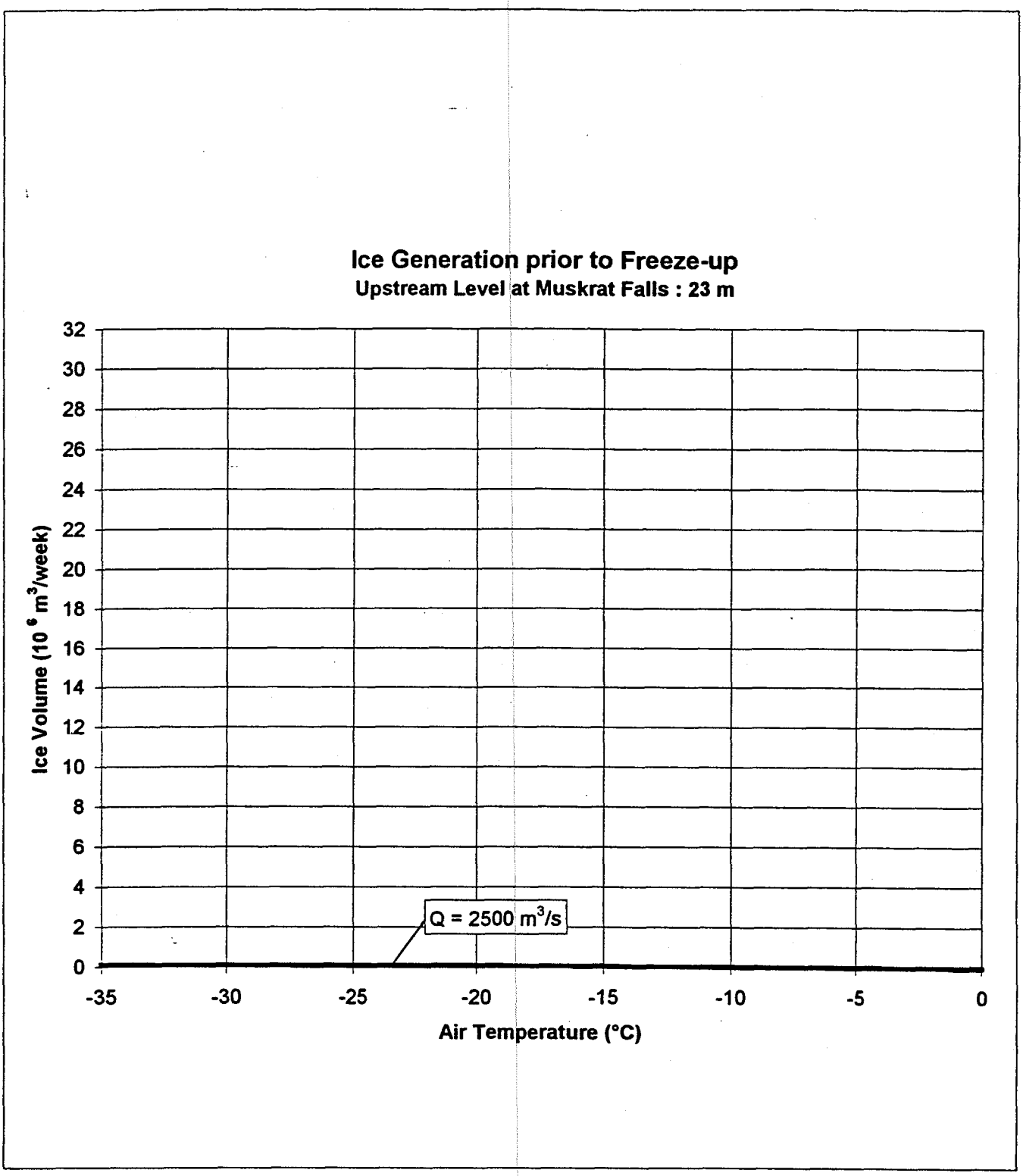
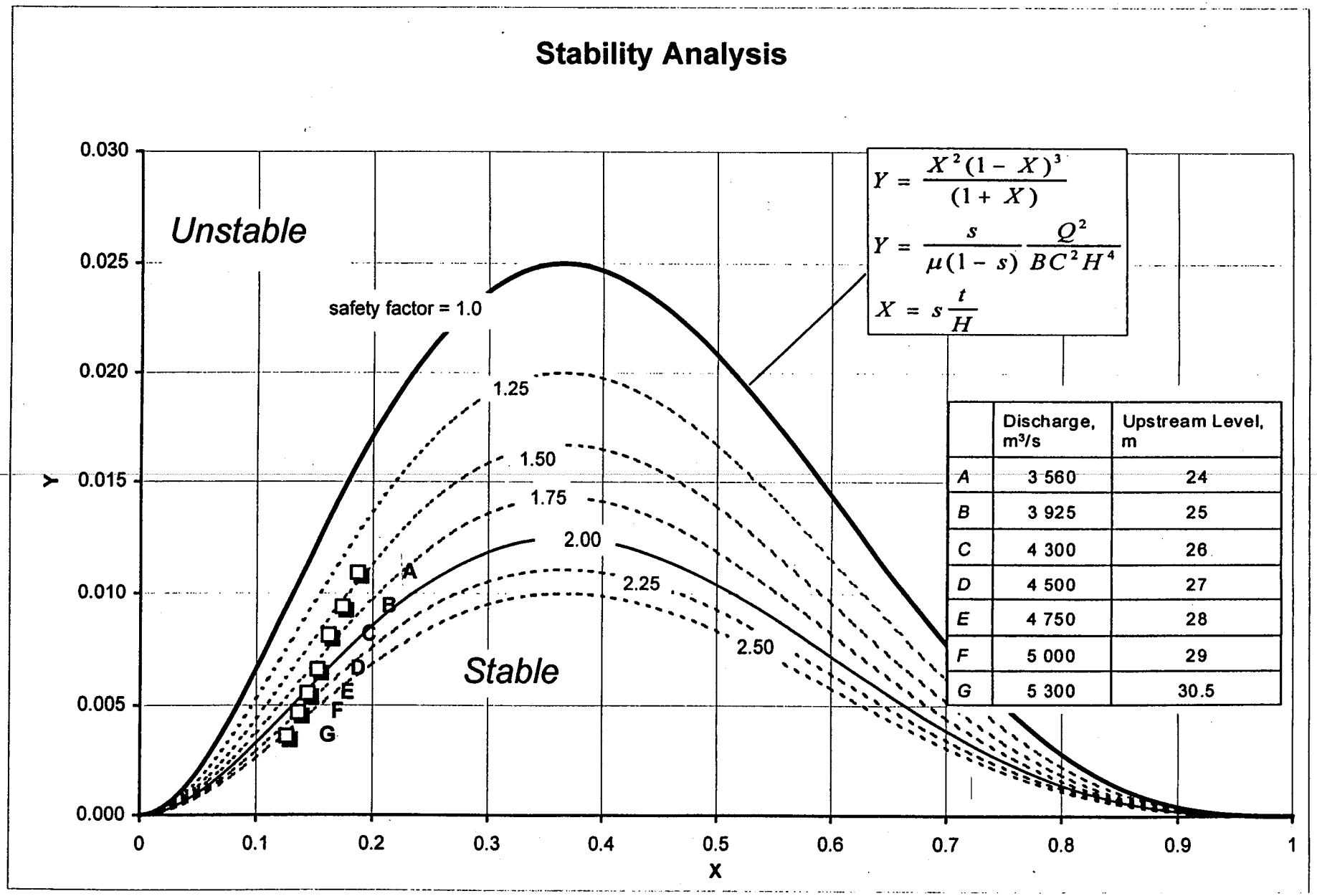
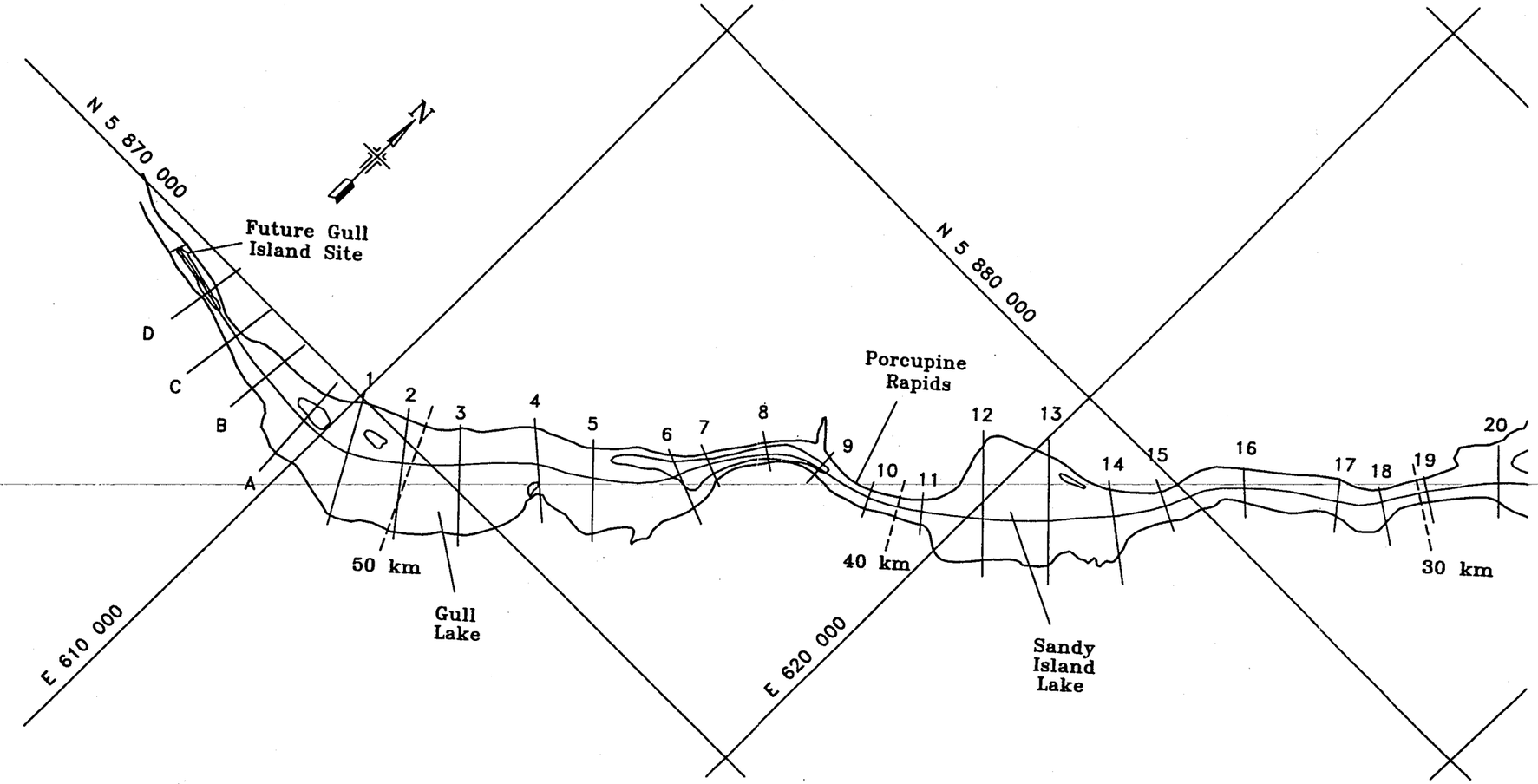

Ice Generation prior to Freeze-up
Upstream Level at Muskrat Falls : 23 m
Ice Volume (10^6 m^3/week)
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
Q = 2500 m^3/s
-35
-30
-25
-20
-15
-10
-5
0
Air Temperature (°C)

FIGURE 12



	Discharge, m³/s	Upstream Level, m
A	3 560	24
B	3 925	25
C	4 300	26
D	4 500	27
E	4 750	28
F	5 000	29
G	5 300	30.5

FIGURE 13



Legend

1 ——— surveyed cross-section

A ——— estimated cross-section

LaSalle Consulting Group Inc.

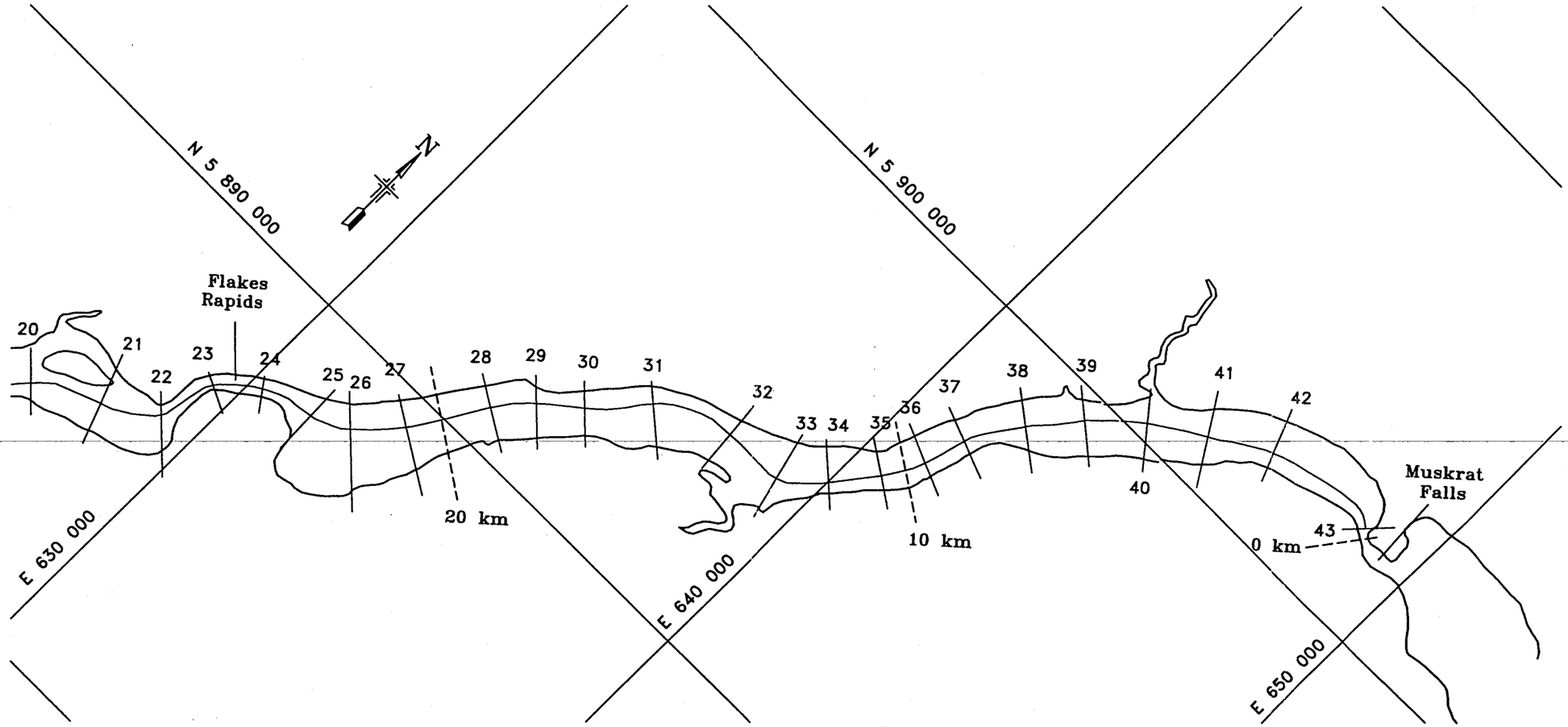
9820, Saint-Patrick
LaSalle, Quebec
Canada H8R 1R8

SNC-AGRA

PROJECT: Muskrat Falls
Final Feasibility Study
Ice Studies

TITLE: Churchill River Between
km 28 and Gull Island

DRAWN BY:	CLIENT REF.:	LASALLE REF.:
P.T.	N/A	028-107-01
SCALE	DATE:	PLATE:
1 : 100 000	OCTOBER 1998	01



Legend

1 ——— surveyed cross-section

A ——— estimated cross-section

LaSalle Consulting Group Inc.

9820, Saint-Patrick
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P.T.	N/A	028-107-02
SCALE: 1 : 100 000	DATE: OCTOBER 1998	PLATE: 02

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APPENDIX A

Presentation of the FRASIL model

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This document presents the FRASIL numerical model, a freeze-up jam model developed by LaSalle Consulting Group in 1997/1998 for the purpose of studying river hydraulics in nordic climates. The model calculates the formation of different types of ice during winter and their influence on water levels and flow velocity.

1. GENERAL PRESENTATION

The calculation of hydraulic conditions (backwater curves) follows a one-dimensional procedure similar to the HEC-2 classical model formulation, with the difference that it explicitly takes into account the presence of ice : anchor ice, shore ice, thermal cover and frazil dunes. In another calculation step, the quantities and area covered by each type of ice are calculated : a thermal balance is established as well as a longitudinal profile of water temperatures, then open water zones are determined (a function of local velocity and rate of heat loss between water and air), and lastly, the volume of frazil ice generated in these open areas is calculated.

When encountering an ice cover closing the river from shore to shore, this drifting frazil accumulates by juxtaposition with or without shoving or, on the contrary, is carried under the cover, depending on local hydrodynamic conditions. In the first case, there is dynamic upstream progression of the frontal edge, whereas in the second case, the frazil passing under the cover is deposited in zones where water flow is slower, thus forming hanging dams. These dunes can be subsequently eroded, by thermal or mechanical erosion.

The model usually operates on a daily time step, and data required every day are: discharge, downstream water level, air temperature and water temperature at the upstream boundary of the model. Each of these values can vary from one time step to another, but the calculation is carried out in a permanent regime within each time step.

2. DEFINITIONS

The expressions *thermal cover, smooth cover, thin cover* are used indifferently. In all cases they refer to the cover which forms locally in low velocity areas and cover thickness varies with the number of freezing degree-days, in accordance with a relation similar to Stefan's formula. *Shore ice* is a thermal cover which does not extend from one shore to another in a given cross-section, leaving a free flowing channel in the central part of the section, which is also deeper and faster. The same equations apply to thickening of shore ice as to thermal covers. A section is said to be completely *closed* when shore ice extends from shore to shore. An opening is formed by the open channel of one or more sections which are not completely closed. The leading edge of the cover is located at the downstream transition between an opening and a closed section.

Frazil is defined as any form of ice generated in the open water areas and drifting at the flow velocity. The model considers three types of drifting ice : skim ice, pancake ice and the frazil grains suspended in the body of the flow.

The frazil can be deposited under the thermal cover of one or more closed sections (i.e. without any open channel). It is then referred to as a *hanging dam*, a *suspended dam* or simply as an *accumulation*.

3. BACKWATER CURVES

There are four types of sections, as follows;

- open water (zero thermal cover thickness, open width equal to the total width),
- partially covered by shore ice (thermal cover thickness greater than zero, open width greater than zero),

- closed with thermal cover (zero open width),

- closed with thermal cover and accumulation (thermal cover thickness greater than zero, zero open width, accumulation thickness greater than zero).

No other case is considered ; in particular, there cannot be accumulation other than when the thermal cover is present from shore to shore.

The ice found at a cross-section, consisting of a combination of thermal cover and frazil accumulation, is always in hydrostatic equilibrium.

The bottom/ice composite Manning *n* is calculated by Sabaneev's formula. The ice friction, otherwise referred to as thermal cover or accumulation Manning *n*, such as the case may be, is defined by the user. The model does not calculate the Manning *n* of accumulations as a function of their thickness, as was proposed by different authors, and there is no simple way to vary the ice Manning *n* with time.

The backwater curve calculation is done with "fixed ice", which means the extent and thickness of the various types of ice have the values computed at the end of the preceding time step. The calculation proceeds downstream to upstream, the level of energy at the downstream section is known and the level at the current section is solved to verify the energy equation. The presence of ice is precisely accounted for while calculating the flow surface and the wetted perimeter. In particular, the wetted perimeter is calculated by adding the following components :

- bottom : assimilated to the total width of the ice

- ice : total width under the ice minus the open width

- shores : mean height multiplied by the number of shores (which may be greater than 2 where islands are found).

The backwater curve calculation does not allow for the separation of flows on each side of an island, but takes into account the presence of islands when evaluating the flow areas and the wetted perimeters.

The minor losses are calculated taking into account contraction and expansion coefficients specified by the user for each cross-section. The mean slope of the energy line between two sections is evaluated following the method used by HEC-2.

When the calculation does not converge towards a sub-critical solution, it is assumed that the flow is critical and the level is computed accordingly. There never is a true upstream-downstream backwater level calculation in a super-critical regime, and the calculated Froude numbers never exceed the value of 1.

4. THERMAL EXCHANGES FROM WATER TO THE ATMOSPHERE

In the course of the calculation, the rate of thermal exchange between the body of water and the atmosphere is evaluated following the equation :

$$Phi_eau = COEF1*(Tair-Teau) - COEF2 + Rayonnement_solaire_absorbe$$

The first terms COEF1*(Tair-Teau) - COEF2 represent in a linear fashion all exchange terms other than solar radiation (sensible heat, evaporation, infra-red radiation, precipitations) (Saucet, 1993).

Solar radiation absorbed by water is calculated as a function of latitude and date at the time step being considered. The equations which are used have been established by N. Marcotte and have, among others, been used in the RHIVER model (Marcotte, 1973, Saucet, 1985).

A simplified calculation mode can also be adopted, in which the solar radiation term does not appear explicitly but is globally taken into account in the COEF1 and COEF2 coefficients.

5. RATE OF THERMAL TRANSFER BETWEEN WATER AND ICE

When the water temperature of flow under the ice is not 0 °C, there is heat transfer from water to ice :

$$Phi_eau = h_{eau} * Teau , \qquad W/m^2$$

where h_{eau} is the rate of ice/water transfer, in W/m²/°C.

This heat flux is taken into account in the thermal balance of the body of water during the calculation of longitudinal profiles of water temperatures. The heat flux melts part of the hanging dams when it acts directly under a frazil accumulation. Finally, when this same flux acts directly under a thermal cover (that is, without accumulation), it contributes to the balance of thermal exchanges for that cover, thus making it thinner or reducing the speed of the thickening process.

Under sections which are completely closed, the rate of transfer h_{eau} is evaluated on the basis of a relation between Nusselt's number and Reynold's number:

$$Nu = 0{,}026 \ \mathbf{R}^{0,9}$$

6. STATIC COVERS AND SHORE ICE

As previously mentioned, the expressions thermic cover, static cover and smooth cover are used interchangeably. All cases refer to a solid ice cover (without inclusion of any voids) which is motionless, attached to the shores and whose thickness is controlled by thermal exchanges. Shore ice is but a particular type of static cover which does not extend from shore to shore but instead offers an ice-free channel in its midst. Inversely, FRASIL calculates the progression of shore ice, which forms a thermically closed section when it extends over the whole cross-section.

The thickness and extent of static covers are calculated by two distinct and independent mechanisms. In fact a thermal cover can completely disappear and the section reverts to an open-water status in two different ways. In the first case it melts completely whereby it continues to take up the whole width while its thickness becomes nil (this would be the case where warm water flows at high velocity under the cover). In the other case, its thickness remains constant but its attachment from one shore to another becomes nil (such would be the case, for example, if the cover was completely carried away because of stronger flow or higher water velocities). These two phenomena permanently act in combination and are correctly reproduced.

6.1 Shore Ice Thickness

Thickness variations are calculated by a generalisation of Stefan's law for cases where the cover is subjected to heat flux from warm water flowing below it and the surface subjected to exchanges with the atmosphere (see for example Lunardini, 1988).

When there is frazil accumulation under the thermal cover as well as warm water circulating underneath, the heat flux from water to ice melts the frazil but does not affect the thermal cover. This cover maintains its thickness as if in contact with 0 °C water as is effectively the case, since it is isolated from the warm water circulating in the frazil

mass. The presence of a snow cover on top of the ice acts as a thermal insulator and is taken into account by reducing the effective thermal conductibility of the ice. The model is adjusted in such a way that ice thicknesses are predicted by the classical Stefan law for large values of degree-days number and when flow under the cover is at 0°C.

6.2 Shore Ice Extent

Shore ice extent is adjusted at each time step and each cross-section as a function of flow velocity, rate of heat exchange with the atmosphere and water temperature.

Shore ice extent is first determined by calculating the maximum velocity that the cover can bear at the boundary between shore ice and the open water channel. This threshold velocity is calculated by four different equations. Progression of the cover towards the center is first distinguished from its retreat towards the shores and then mechanical aspects (the only ones that play a role when water is at 0 °C) are distinguished from thermal aspects when water is warm.

Therefore, when the temperature of a body of water is greater than 0 °C, shore ice may progress towards the middle of the river as long as it encounters water temperatures greater than 0 °C at the surface, and this fixes a threshold velocity. When the local velocity is too high, the rate of heat transfer from the body of water to the surface by turbulent diffusion becomes greater than the loss to the atmosphere such that the water temperature does not remain at 0 °C at the surface. This "thermal" limitation is accompanied by "mechanical" limitation which bars against progression of the ice when the flow is too high (Matousek, 1984).

When shore ice is retreating towards the shore, thermal conditions are different because water then flows under the ice and the rate of heat transfer from the body of water under the surface is different from that at an open surface. Furthermore, once ice is formed and thickened, it resists greater flow velocities than when it is progressing.

The distinction made between four types of laws in this process (thermal, mechanical, progression, regression) allows a satisfactory representation of freeze-up at the beginning of winter and its retreat at the end of winter.

Once the velocity at the boundary between the shore ice and the open channel is known, we calculate the width of the channel by a *quasi-2D* velocity field approximation in a given cross-section. The mean velocity is known from the calculation of the backwater curve and it is considered that the velocity at a given distance from the shore is proportional to the local depth raised to a certain power. The proportionality coefficient is determined in each section in such a way that the continuity equation is respected.

7. THERMAL EXCHANGES AND FORMATION OF DRIFTING ICE

When the water temperature is above 0 °C there is no drifting ice [(1)] and the temperature variation from one cross-section to the other is carried out according to a simple thermal balance of various heat fluxes.

When the water temperature is 0 °C, drifting ice can take two forms : at low velocities it is skim ice and at higher velocities, a mixture of pancake ice drifting at the surface and suspended frazil carried in the body of water.

7.1 Water Temperature

Three modes of thermal exchange account for water temperature variation in the current cross-section.

---

(1) : This a basic assumption of the model.

- Exchanges between water and existing ice, whether it be shore ice or solid ice cover or a hanging dam. The flux is the product of the exchange rate Phi_eau by the ice/water contact surface.

- Exchanges between the atmosphere, at a Phi_air rate applied to the open surface, that is the open-water surface of the channel (if it exists) minus the surface or pancake ice partially covering it.

- Head losses, in other words conversion of the total water energy into heat.

FRASIL does not consider heat received from underground water sources or by geothermal flux. These terms could be of some importance at the onset of winter, especially in small rivers.

7.2 Drifting Ice

When water temperature is nil, the heat exchange terms in sections with an open water channel are head losses and exchanges with the atmosphere. The analysis is not the same for drifting ice in the form of skim ice and in the form of pancake ice.

7.2.1 Skim Ice

When flow velocity is low, no pancake ice is formed but rather, the open water surface is covered by a thin plate of ice (skim ice) drift. The threshold velocity is a function of turbulence (a function of the Chézy *C* coefficient) and the heat loss rate with the atmosphere (Matousek, 1984). In such a case, drifting skim ice thickness varies with thermal conductibility, according to a formulation adapted to very small thicknesses.

In FRASIL, as indicated above, it is implicitly supposed that there is never any moving ice when water is at a temperature above 0 °C. Therefore, in this version of the model, one cannot find "skim ice on warm water" as one might find in nature.

7.2.2 Pancake Ice

When flow velocity is greater than the allowed threshold for skim ice formation, ice drifts in the form of pancake ice and suspended frazil. In this case we calculate, from one section to another, which proportion of the total volume of ice drifts while in suspension as well as the surface concentration and the thickness of pancake ice (Andres, 1995).

Two mechanisms bring about modifications to these values : thermal factors (thawing or freezing of new quantities of ice), but also basic continuity whereby concentrations would increase with reduced open width or lower flow velocity, since in the absence of any thermal exchange the total flow of ice must remain constant.

First we calculate the concentration variations stemming from continuity : the concentration of surface pancake ice can increase to the point where an ice bridge is formed. This phenomenon is represented very concisely in FRASIL : an ice bridge is formed when the concentration reaches the threshold value specified by the user. In this case the cross-section closes off and is then treated as all other cases where drifting frazil encounters a closed section (see further).

In most cases there is no ice bridge and the drifting ice volume varies with thermal exchanges. The open water surface in contact with the atmosphere is the total opening minus the surface occupied by pancake ice already formed. The mass of ice formed during the time step is calculated in proportion to this contact surface and the rate of water heat loss to the atmosphere. This mass of ice brings on an increase of the suspended frazil drifting into the body of water.

At the same time, this concentration is reduced by the fact that part of the suspended frazil emerges and stays at the surface of water. We assume that the rate of emergence is simply proportional to the concentration in suspension. The frazil emerging under pancake ice will increase pancake ice thickness whereas frazil emerging on open water will form new pancakes which then add to the surface already occupied by pancake ice.

An advantage to the calculation method adopted in FRASIL is that it follows more closely the physical mechanism of pancake ice. Upstream of a long reach, there is still very little suspended frazil and little pancake ice can be formed; therefore, heat loss will mostly increase the amount of suspended frazil. Not until having reached a certain distance downstream will the amount of suspended frazil have become important enough to create pancake ice in appreciable quantities. Further downstream, very little fresh suspended frazil is formed because the pancakes form a thermal screen between the atmosphere and the body of water. The pancakes which continue to emerge then reflect the suspended frazil which was formed further upstream. Therefore the total volume of drifting ice is divided in a constantly shifting ratio of suspended and pancake ice. This was not the case with previous models which assumed 84% of the volume was always in the form of pancake ice and 16% suspended (Hydro-Québec, 1989).

7.2.3 Anchor Ice

Formation and disappearance of anchor ice are not represented in FRASIL. However, the user can very succinctly take into account the effect of anchor ice on hydraulic conditions by imposing a heightening of the bottom of the river. This can be done on certain specific sections by modifying their reference level in the bathymetry file or, in a more global manner by inserting on the keyboard the desired heightening prior to reading the bathymetry file.

8. STABILITY OF UPSTREAM EDGE AND DYNAMIC UPSTREAM PROGRESSION

Dynamic progression of covers occurs when drifting ice is stopped by and accumulates at the upstream edge of a closed section, causing progression of the upstream edge in the upstream direction. This can happen in any of the following situations :

- The nearest downstream section is closed because shore ice has progressed from one shore to another (section closed by thermal cover), with or without accumulation of frazil under the cover.

- The nearest downstream section was closed by dynamic progression earlier in winter.

- The section is open but the frazil concentration is such that an ice bridge is formed.

- An ice boom was put in place in the section.

When one of these situations is encountered, we first check for stability of the upstream edge and if this condition is not met, the frazil continues downstream and deposits under the closed section. On the contrary, stability at the upstream edge is verified and the thickness at which the ice accumulates while progressing upstream is calculated. The volume required in order to progress to the nearest upstream section is then inferred and progression takes place if the available volume is sufficient.

The upstream edge stability condition is as follows;

$$\text{Froude number} < 0{,}06 \quad \textbf{or} \quad \text{Velocity} < 0{,}65 \text{ m}$$

The mean depth of the section being H and the Froude number F, the cover is said to progress upstream at a thickness t such as (Michel, 1971) :

$$F = \sqrt{2\,(1-e)\,(1-s)\,\frac{t}{H}\left(1-\frac{t}{H}\right)}$$

where s is the ice density, e the porosity of the progressing cover. We adopt e = 0,5, and the relative thickness t/H reaches a maximum of 1/3 when the Froude number reaches or is greater than 0,11, beyond which there cannot be any progression.

The thickness that is calculated by this relation is the thickness required for hydrodynamic purposes at the progressing upstream edge. In "narrow" rivers, this thickness is sufficient and the cover being formed does not undergo other transformations. In "wide" rivers, this thickness is not sufficient to ensure the "mechanical stability" of the whole, and the cover undergoes shoving which increases its thickness.

We know that most rivers are "wide", but refreezing of interstitial water at the surface results in a solid "crust" which offers enough stability. The user has the choice between two options :

- we can consider that refreezing happens quickly and therefore, without verifying with calculations, there is always mechanical stability. In this case, the thickness required in order for the cover to progress upstream is simply calculated according to the above equation;

- we can also consider that refreezing is not quick enough and we then calculate the thickness required to ensure mechanical stability (Pariset, Hausser, 1966). This thickening by shoving takes place only during upstream progression (mechanical stability is still assumed if the frazil dune forms under a thermal cover or if the frazil is deposited on a cover which has already progressed in the course of a preceding time step).

9. TRANSPORT AND BUILD-UP OF HANGING DAMS UNDERNEATH CLOSED SECTIONS

Frazil circulating under sections which are closed from shore to shore is liable to be deposited under the thermal cover or under the frazil dune that it encounters. Inversely, frazil deposited in a frazil dune is liable to be carried away downstream again if hydraulic conditions become more "erosive" than when it was first deposited. The frazil which is entrained from a hanging dam is then added to the global balance of frazil circulating underneath the cover. Finally, moving frazil can reemerge in a downstream opening and it then contributes to the concentrations of drifting ice in the same manner as newly formed frazil.

Generally, thresholds for build-up and for erosion are different because it is considered that frazil is deposited relatively easily but is more difficult to dislodge once deposited.

During the calculation of frazil dune thickness variation by build-up or erosion, the water level remains constant [(1)]. The hydraulic conditions under the dune are then a direct function of the total thickness of the dune. The thickness is adjusted so that a criterion (velocity, shearing stress underneath the dune or transport capacity) is respected or that the available volume is used up.

9.1 Ice Build-up and Erosion Criteria

The user has a choice between three types of criteria to control build-up and erosion of frazil. One criterion can be specified at build-up and another at erosion. It is up to the user to evaluate if the chosen combination is realistic. Traditionally, one specifies velocity criteria at build-up as well as at erosion.

---

(1) : The value of the level is the one obtained after calculating the backwater curve at the beginning of the time step.

9.1.1 Threshold Velocities

Frazil dunes are thickened or eroded until the flow velocity reaches the specified value (mean velocity is defined as the relation between total flow and flow area between bottom and the under side of the dune). The threshold velocity set at erosion is in general greater or equal to the velocity at build-up. Usual values are 0.5 to 1 m/s for build-up threshold, 1.5 to 2 m/s for threshold at erosion. Intuitively, lower thresholds at build-up reflect lower frazil grain sizes and shorter open water generating zones.

9.1.2 Limiting Shearing Stress

More than velocity, shearing is the variable that controls build-up or erosion and there can be different shearing values for the same flow velocity. Thus, the shearing threshold can be evaluated by the user with Shield's formula as a function of the size of drifting ice fragments (which is nevertheless not well known...). The effective shearing stress acting underneath the dune is evaluated by the model and compared to the specified limiting stress.

9.1.3 Transport Capacity

Finally, it can be considered that the frazil is deposited when the transport capacity is lower than the frazil discharge arriving from upstream. Pariset and Hausser (1961) had proposed using Meyer-Peter and Muller's formula for evaluation of this transport capacity, and Shen (1972) has shown through measurements taken on the Yellow River that transport of frazil particles (diameter of 2 to 3 cm) followed a relation very close to Meyer-Peter and Muller's original relation.

The relation established by Shen (1972) can be written as a relationship between velocity, frazil particle diameter and ice discharge, and the latter increases the effective velocity under which frazil starts to deposit.

The diameter is evaluated by empirical means. When frazil goes from an open section to a closed section, the thickness of frazil pancakes is known as well as the proportion of this frazil which is pancake ice or is suspended in the body of water. It is considered that the mean diameter of pancakes is equal to 2.5 times their thickness and that the diameter of hanging frazil is equal to 1 mm. The average of these two thickness values is made and weighted against relative quantities of each of the two types of frazil. This "virtual" diameter remains constant as long as frazil circulates underneath hanging dams.

Practically, the "transport capacity" calculation is done like a "threshold velocity" calculation, but FRASIL uses the threshold velocity calculated by Meyer-Peter & Muller's equation instead of the fixed value assigned by the user. The threshold velocity at build-up therefore varies at every section and every instant. It is usually low when the temperature is mild and the site is upstream of the reach under study (particle diameter being smaller) and high when the temperature is cold as well as downstream of the build-up site.

It is possible to use the same equation to calculate a velocity at erosion which varies with ice flow. This is the same as considering that the frazil dunes do not consolidate with time and that frazil is as mobile at erosion as at build-up.

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APPENDIX G

ROLLER COMPACTED CONCRETE (RCC) DAM BACKGROUND AND APPLICATION

RUBBER DAM EXPERIENCE LISTS AND TECHNICAL LITERATURE

APPENDIX G

ROLLER COMPACTED CONCRETE (RCC) DAM BACKGROUND AND APPLICATION

APPENDIX G

RUBBER DAM EXPERIENCE LISTS AND TECHNICAL LITERATURE

Roller Compacted Concrete (RCC) Dams - Background and Application

Roller-compacted concrete (RCC) dams are a significant breakthrough in the design and construction of dams. RCC was developed in the1970s, and its use for new dams and the rehabilitation of existing dams has made even greater progress during the 1980s and 1990's.

RCC dams have gained acceptance worldwide in a relatively short time due to their low cost, which is derived in part from their rapid method of construction.

RCC emerged as a respond to a declining interest in conventional concrete gravity dam construction. From 1950 to 1980, the number of concrete dams built had dropped significantly. The greatest decrease was occurring in wide-valley sites, where concrete gravity dams were being replaced by less costly earth and rock embankments. Their cost advantage over concrete dams was derived mainly from the greater efficiency of the equipment and methods used in construction. The increased popularity of embankment dams also coincided with the emergence of soil mechanics technology.

In the meantime, while embankment dams were reducing in cost relative to concrete dams, they were, and are, more prone to failure. By comparison, hundreds of earth embankments of all sizes have failed during the past 60 years The primary cause of embankment dam failures is overtopping and internal erosion of the fill material.

Taking note of the relative vulnerability of fill dams, the experts were searching for a new type of dam that would combine the safety advantages of concrete and the efficiencies of embankment dam construction. By a number of different routes, their searches in the early 1960s and 1970s led to the development of roller-compacted concrete dam building.

The RCC method evolved not only from the efforts of some influential concrete dam designers but also from the work of geotechnical engineers who have traditionally designed earth and rockfill embankments. Their combined efforts have produced a concrete dam built by methods usually associated with earth dam construction. The product is a less costly dam with the same inherent safety as a conventionally placed concrete dam.

RCC is more than a new construction material. It is a new construction method. RCC differs from soil-cement, which uses similar placing methods, primarily because RCC contains coarse aggregate greater than 19mm maximum size and because it develops material properties similar to those of conventionally placed concrete. Soil-cement generally uses pit-run sands as its aggregate and develops lower strengths than RCC .

There are also considerable differences between vertical, block-by-block construction of dams with conventional mass concrete and horizontal, lift-by-lift construction of dams with RCC. The construction plant for RCC is usually less costly and utilized at a higher capacity factor than that required for conventional concrete gravity dams. But the fast pace of placement required to achieve the economies possible with RCC imposes a greater discipline on the planning, scheduling, and coordination in the field between designer and contractor.

In the development of RCC technology, two philosophies, or approaches, have emerged with

respect to RCC mix design methods, They can be termed the soils, or *geotechnical,* philosophy, and the *concrete* philosophy.

There is no distinct line separating the two philosophies. Basically, RCC mixtures produced using concrete design methods have a more fluid consistency as measured by a Vebe or vibratory compaction (VC) test. These mixes may be described as being more workable than those developed using the soils approach, yet both philosophies will produce a concrete that is termed "zero slump".

RCC dam design was evolving in three different directions during the 1970s. In the United States, a lean-concrete alternative based on soils technology was being developed by the Army Corps of Engineers and other investigators. British engineers were focusing on the so-called high-paste alternative, a hybrid of conventional concrete mix design and earthfill dam construction methods. The Japanese research team set up to explore rationalized concrete dam construction methods was developing the third approach, what it called roller-compacted dam, concrete method, or RCD. Of the three, RCD is the most cautious departure from traditional concrete gravity dam design and construction practices.

The last two of these alternatives, the high-paste RCC dam and the Japanese RCD method, may be termed concrete approaches. Upper Stillwater Dam, Elk Creek Dam, and all Japanese RCD dams are examples of dams with RCC mixtures designed using a concrete philosophy. Dams such as Willow Creek, Copperfield, Middle Fork, and Monksville were built with lean RCC mixtures that fit into the soils approach.

The roller compaction of concrete for dams is now sufficiently developed to enable very large gravity dams to be designed. RCC can be designed to have sufficient dynamic tensile strength to withstand severe seismic loading and there are techniques available for the comprehensive analysis of the thermal stresses within the structure so that joint spacing and maximum placing temperatures can be defined with a reasonable degree of certainty.

With the particular form of RCC used in the majority of recent RCC dams, in which a high paste content is used and a high proportion of the cementitious content is pozzolan, it is necessary to allow sufficient time during the trial mix program in order to determine the long-term development of strength of the concrete and in order to be able to obtain all the data required as inputs to the seismic and thermal studies of the dam.

RCC dams have been found to be very economic compared to other forms of construction and with judicious selection of design criteria, careful selection of materials and optimization of the mixture proportions, there is no reason why RCC gravity dams of any reasonable height cannot now be designed. Nevertheless, the design of the dam must allow easy placement of the RCC so that the primary benefits - economy and speed of construction - can be realized.

The economic advantage of RCC, combined with the long-term safety record of concrete dams, has led to rapid acceptance of RCC dams throughout the world. RCC dams have been completed on all six continents.

The following table provides a list of RCC dams constructed world wide between 1980 and 1989.

Completed Roller-Compacted Concrete Dams

Dam/River	RCC Complete	State or Province	Owner	Ht. ft (m)	Length ft. (m)	RCC Volume yd3 (m3)	Total Volume yd3 (m3)
ASIA - Japan							
Shimajigawal Shirnaji	1980	Yamaguchi	Japan Ministry of Construction (JMC)	292 (89)	787 (240)	216,000 (165,000)	415,000 (317,000)
Tamagawa/Tarna	1986	Akita	JMC	328 (100)	1448 (441.5)	944,000 (757,000)	1,509,000 (1,154,000)
Mano/Mano	1987	Fukushirna	Fukushima Prefecture	226 (69)	784 (239)	135,000 (103,000)	286,000 (219,000)
Pirika/ Shiribeshitoshibetsu	1987	Hokkaido	JMC+Hokkaido Development Bureau	131 (40)	2986 (910)	212,000 (162,000)	471,000 (360,000)
Shiromizugawa/ Shiromizu	1988	Yamagata	Yamagata Prefecture	179 (54.5)	1204 (367)	186,000 (142,000)	412,000 (315,000)
Asahiogawa/O	1988	Tovama	Tovama Prefecture	276 (84)	853 (260)	217,000 (160,000)	472,000 (361,000)
Nunome/Nunome	1989	Nara	Water Resources Development Corp.	236 (72)	1056 (322)	144,000 (110,000)	492,000 (370,000)
ASIA - China							
Kengkou/Pingshan	1986	Fujian	Fujian Provincial Dept. of Water Conservacy & Hydropower	186 (57)	402 (123)	56,000 (43,000)	78,500 (60,000)
Tianshengqiao 2/ Nanpan	1988	Guizhou & Guangati	Guizhou & Guanani Province	193 (59)	1542 (470)	171,000 (131,000)	421,000 (322,000)
Mahui/ Jialing	1989	Sichuan	Sichuan Province	75 (23)	463 (141)	196,000 (150,000)	327,000 (250,000)
Longmentan/Luxi	1989	Fujiani	Dehua County, Fujian Prov.	189 (58)	515 (157)	95,000 (73,000)	132,000 (101,000)
Tongjiezi/Dadu	1989	Sichuan	Sichuan Province	285 (87)	3310 (1009)	589,000 (450,000)	3,544,000 (2,710,000)
Pajiakou/Luanhe	1989	Hebei	Bureau of Panjiakou Control Works	80 (24.5)	801 (244)	27,000 (21,000)	39,000 (30,000)
NORTH AMERICA - United States							
Willow Creek	1982	Oregon	U.S.Army Corps of Engineers	169 (52)	1780 (543)	433,000 (331,000)	433,000 (331.000)
Winchester/Upper Howards Creek	1984	Kentucky	Winchester Municipal Utilities	74 (23)	1192 (363)	32,000 (24.500)	35,000 (27,000)
Middle Fork/Middle Fork of Parachute Creek	1984	Colorado	Exxon Co. U.S.A.	124 (38)	410 (125)	55,000 (42,000)	60,000 (45.000)
Galesville/Cow Creek	1985	Oregon	Douglas County	162.5 (50)	950 (290)	210,000 (161,000)	223,000 (170,000)
Grindstone Canyon-Grindstone Creek	1986	New Mexico	Village of Ruidoso	139 (42)	1416 (432)	115,000 (88,000)	126,000 (96,000)
Munksville/ Wanaque	1986	New Jersey	N.Jersey Dist. Water Supply Comm. & Hackensack Water Co.	157 (45)	2200 (670)	287,000 (219,000)	304,000 (232,000)

Dam/River	RCC Complete	State or Province	Owner	Ht. ft (m)	Length ft. (m)	RCC Volume yd3 (m3)	Total Volume yd3 (m3)
Lower Chase Creek	1987	Arizona	Phelps DodgeMorenci Inc.	64 (20)	400 (122)	18,000 (13,800)	29,000 (22,200)
Upper Stilwater/ Rock Creek	1987	Utah	U.S. Bureau of Reclamation	294 (90)	2673 (815)	1,471,00 0 (1,125,000)	1,675,000 (1,281,000)
Elk Creek, Elk Creek	1988	Oregon	U.S. Army Corps of Engineers (as halted), Portland District (as planned)	83 (25) 249 (76)	1197 (1165) 2580 (786)	148,000 (266,000) 1,041,00 0 (796,000)	455,000)348,000) 1,100,100 (841,000)
Stagecoach/Yampa	1988	Colorado	Upper Yampa Water Conservancy District	150 (115)	1180 (115)	44,000 (14,000)	50,500 (118,500)
Stacy (now SW Freese) Spillway/Colorado	1989	Texas	Colorado River Municipal Water District	103 (31)	568 (173)	117,000 (89,000)	209,000 (160,000)
Marmot (replacement)/ Sandy	1989	Oregon	Portland General Electric	50 (15)	104 (59)	10,300 (8,000)	13,000 (10,000)
Mexico							
La Manzanilla /Iborilla	1987	Leon	SARH	118 (16)	492 (150)		65,400 (50,000)
AUSTRALIA							
Copperfield/ Copperfield	1984	Queensland	Kidston Gold Mine Ltd.	131 (40)	1115 (1140)	1,811,00 0 (140,000)	205,000 (151,000)
Craigbourne/Coal	1986	Tasmania	Tasmanian Rivers & Water Commission	82 (25)	810 (247)	29,000 (22,500)	29,003 (22,500)
Bucca Weir/Kolan	1987	Queensland	Queensland Water Resources Commission	39 (12)	420 (124)	31,400 (24,000)	39,300 (30,000)
Wright's Basin/ Point Hut Creek	1989	Australian Capital Territory	Australian Capital Teritory Government	59 (18)	282 (86)	11,900 (9,100)	11,900 (9,100)
EUROPE - Spain							
Castilblanco de los Arroyos/Rivera de Ca1a	1985	Seville	Junta de Andalucia	82 (25)	404 (123)	18,000 (14,030)	26,000 (20,0331
Erizana (dike)	1987	Galicia	Confederacion Hidraulica del Norte de Espana	50 (15)	361 (110)	15,700 (12,000)	85,000 (65,000)
Los Morales / Mora1es	1987	New Castile	Confederaci6n Hidro grafica del Tajo	92 (28)	656 (200)	29,000 (22,000)	
Santa Eugenia / Xal1as	1988	La Corufla	Soriedad Espanola de Carburos Matilicos S.A.	281 (85.5)	925 (285)	286,000 (219,000)	306,000 (234,000)

Dam/River	RCC Complete	State or Province	Owner	Ht. ft (m)	Length ft. (m)	RCC Volume yd3 (m3)	Total Volume yd3 (m3)
EUROPE - France							
I,es Olivettes / Tributary of Herault	1987	Bas Rhone	Conseil General de l'Herault	118 (36)	837 (255)	105,000 (80,000)	111,000 (85,000)
EUROPE - USSR							
Tashkumir/Narin	1988	Kirgizia	Gidroproject	246 (75)	1050 (120)	111,000 (85,000)	
AFRICA - South Africa							
De Mist Kraal Diversion/Little Fish	1986	Cape	Department of Water Affairs	98 (300)	984 (300)	46,000 (35,000)	85,000 (65,000)
Arabie	1986	Lebowa	Department of Cooperative Development	118 (36)	1493 (455)	132,000 (101,000)	185,000 (141,500)
Zaaihoek/Slang	1987	Natal	Department of Water Affairs	154 (47)	1729 (527)	127,000 (97,000)	175,000 (134,000)
Knellpoort (gravity arch(/Rietspruit	1988	South Eastern	Department of Water Affairs	164 (50)	656 (200)	59,000 (45,000)	77,000 (59,000)
Wolwedans (gravity arch)/Great Brak	1989	Cape	Department of Water Affairs	230 (70)	879 (268)	196,000 (150,000)	221,000 (169,000)
AFRICA - Morocco							
Ain Koreima/ Akruech	1988	Rabat	Direction des Amenagements Hydrauliques	85 (26)	407 (124)	34,700 (26,500)	39,000 (30,000)
Rwedat	1988	Rabat	Direction des Amenagements Hydrauliques	76 (23)	410 (125)	33,000 (25,000)	35,000 (27,000)
SOUTH AMERICA - Brazil							
Saco de Nova Olinda Gravata	1986	Paraiba	State of Paraiba(Icoplan	184 (56)	755 (230)	173,000 (132,000)	187,000 (143,000)
SOUTH AMERICA - Argentina							
Urugua/Urugua	1989	Misiones	Electricidad de Misiones Sociedad Anonima	249 (76)	2254 (687)	772,000 (590,000)	819,000 (626,000)

In addition, RCC has been used for the construction of several major, large volume, cofferdams and for the rehabilitation of existing dams. There are many applications of RCC in upgrading existing embankment dams as well as complete replacement of failed or distressed dams.

In Canada, there are only a few RCC dam applications. The most significant one is Lake Robertson RCC Dam completed in Quebec during the mid 1990's. Most recent is the overflow RCC dam construction in 1996 at Grand Falls, Newfoundland.

APPENDIX G

LISTS OF CANADIAN AND AMERICAN EXPERIENCE

PRODUITS HYDRO INNOVATION INC.
905, Michèle-Bohec, suite 220, November 1998
Blainville (Québec), Canada J7C 5J6
Tel.: 450-979-9999 • Fax.: 450-979-0777

BRIDGESTONE RUBBER DAM
LIST OF PROJECTS (CANADA only)

Project	Province	Year	Dimensions
Forrestville (RSP #1)	Québec	1998	Span 1 : H= 1,70m x L= 40,00m
Ste-Brigitte	Québec	1998	Span 1 : H= 1,10m x L= 32,00m Span 2 : H= 1,10m x L= 32,00m Span 3 : H= 1,10m x L= 45,00m
Chute Blanche	Québec	1998	Span 1 : H= 0.90m x L= 13,50m Span 2 : H= 0.90m x L= 45,00m
Sept-Chutes	Québec	1998	Span 1 : H= 0,85m x L= 6,10m
Chute Bell	Québec	1998	Span 1 : H= 1,40m x L= 60,00m
Rimouski	Québec	1997	Span 1 : H= 1,70m x L= 40,00m
Charlo River	New-Brunswick	1997	Span 1 : H= 0,82m x L= 60,00m
Lumsden	Nova-Scotia	1996	Span 1 : H= 0,97m x L= 45,80m
Mitis - 2	Québec	1996	Span 1 : H= 0,63m x L= 22,80m
Windsor	Québec	1995	Span 1 :H= 1,70m x L= 25,40m Span 2 :H= 1,70m x L= 39,68m Span 3 :H= 1,70m x L= 69,06m Span 4 :H= 1,70m x L= 34,28m
Nairn Falls	Ontario	1995	Span 1 :H= 0,60m x L= 39,00m
EUSTIS	Québec	1995	Span 1 :H= 1,15m x L= 20,75m
Forrestville (RSP #2)	Québec	1995	Span 1 :H= 1,70m x L= 28,43m
McDougall	Québec	1994	Span 1 :H= 1,90m x L= 25,50m
Glenford	Québec	1994	Span 1 :H= 1,70m x L= 75,00m Span 2 :H= 1,70m x L= 8,00m
Soo River	B-C	1993	Span 1 :H= 2,00m x L= 14,00m
Sissiboo Falls	Nova-Scotia	1992	7 Spans : H= 2,44m x L= 12,70m
Brazeau River	Alberta	1991	Span 1 :H= 3,35m x L= 82,00m

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BRIDGESTONE RUBBER DAM
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Forrestville (RSP #1)	Québec	1998	Span 1 : H= 1,70m x L= 40,00m
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Chute Blanche	Québec	1998	Span 1 : H= 0.90m x L= 13,50m Span 2 : H= 0.90m x L= 45,00m
Sept-Chutes	Québec	1998	Span 1 : H= 0,85m x L= 6,10m
Chute Bell	Québec	1998	Span 1 : H= 1,40m x L= 60,00m
Rimouski	Québec	1997	Span 1 : H= 1,70m x L= 40,00m
Charlo River	New-Brunswick	1997	Span 1 : H= 0,82m x L= 60,00m
Lumsden	Nova-Scotia	1996	Span 1 : H= 0,97m x L= 45,80m
Mitis - 2	Québec		

1996
Span 1 : H= 0,63m x L= 22,80m

Windsor
Québec
1995
Span 1 : H= 1,70m x L= 25,40m
Span 2 : H= 1,70m x L= 39,68m
Span 3 : H= 1,70m x L= 69,06m
Span 4 : H= 1,70m x L= 34,28m

Nairn Falls
Ontario
1995
Span 1 : H= 0,60m x L= 39,00m

EUSTIS
Québec
1995
Span 1 : H= 1,15m x L= 20,75m

Forrestville (RSP #2)
Québec
1995
Span 1 : H= 1,70m x L= 28,43m

McDougall
Québec
1994
Span 1 : H= 1,90m x L= 25,50m

Glenford
Québec
1994
Span 1 : H= 1,70m x L= 75,00m
Span 2 : H= 1,70m x L= 8,00m

Soo River
B-C
1993
Span 1 : H= 2,00m x L= 14,00m

Sissiboo Falls
Nova-Scotia
1992
7 Spans : H= 2,44m x L= 12,70m

Brazeau River
Alberta
1991
Span 1 : H= 3,35m x L= 82,00m

EXPERIENCE WITH BRIDGESTONE RUBBER DAMS

Project: Mamquam Hydro Generating Station
(near Squamish, B.C.)

Owner: Northern Utilities, Inc.

Description:

Mamquam forebay dam is equipped with a Bridgestone rubber dam, height = 3.4 m in two sections 20 m long. The dam is located on top of a low weir, so that during floods - bed load and debris can be flushed downstream over the deflated dam. Mamquam River is a steep mountain river that carries a very high bed load of cobbles and boulders (up to 0.9 m diameter). The dam must also cope with debris from logging operations and occasionally entire trees - some with girth of up to 3 ft. (0.9 m). Owner selected rubber dam after considering the following factors (among others):

- resistance to debris and bed load;
- "fail safe" behavior;
- ease of operation; and
- cost.

The Project has been in operation since 1996 and has already been tested by a couple of major floods.

Owner is very satisfied with performance to date.

For further information contact:

Stephen Collen
Northern Utilities Inc.
Suite 1640
1055 West Georgia Street
Vancouver, BC
V6E 3P3

Telephone: (604) 682-8700

Fax: (604) 682-8780

RUBBER DAM EXPERIENCE

Project: Rainbow Dam
Great Falls
Montana

Owner: Montana Power
Head Office
Butte
Montana
U.S.A.

Description:

This Project comprises a 11.0 ft (3.35 m) Bridgestone rubber dam in 2, 200 ft. (61.0 m) long sections. It is located in a region of the northern U.S.A. subject to severe winter conditions, low temperatures in the range -30°C to -40°C, not unlike Labrador. The dams were installed in 1991 and the owner is very satisfied with performance to date.

(Additional performance data has been promised from operating staff).

Contact: Larry Thompson
Montana Power
Butte
Montana

Telephone: (406) 497-3000

LIST OF PROJECTS AND REFERENCES

CARD N°1

Project	:	Chute Bell
Country	:	Canada
State or Province	:	Québec
City or region	:	Grenville
River	:	Rouge
Commissioning year	:	still under construction (oct.1998)
Number of Rubber Dam	:	1

Size : Section 1 : H= 1,4m x L= 60,00m

Description :

Spillway.
This is a Hydro-Québec site being rehabilitated by a consortium on the basis of a turn key project.

Contact :

Marc Richard
Soliger
Telephone : (418) 623-2306

CARD N°2

Project	:	Sept-Chutes
Country	:	Canada
State or Province	:	Québec
City or region	:	Québec city
River	:	Ste-Anne du Nord
Commissioning year	:	Still under construction (oct. 98)
Number of Rubber Dam	:	1

Size :

Section 1 : H= 0,85m x L= 6,10m

Description :

New construction, will serve as a sluice gate.
This is a Hydro-Québec site being rehabilitated by a consortium on the basis of a turn key project.

Contact :

André Guala
Tecsult
Telephone : (514) 287-8500 ext.8590

CARD N°3

Project	:	R.S.P. #1
Country	:	Canada
State or Province	:	Québec
City or region	:	Forestville
River	:	Sault-aux-Cochons
Commissioning year	:	1998
Number of Rubber Dam	:	1

Size :
Section 1 : H= 1,70m x L= 40,00m

Description :
Spillway

Contact :
M. Patrick Thibodeau
Boralex inc.
Telephone : (819) 363-5872

CARD N°4

Project	:	Ste-Brigitte
Country	:	Canada
State or Province	:	Québec
City or region	:	Ste-Brigitte
River	:	Nicolet
Commissioning year	:	1999
Number of Rubber Dam	:	3

Size :
Section 1 : H= 1,10m x L= 32,00m
Section 2 : H= 1,10m x L= 32,00m
Section 3 : H= 1,10m x L= 45,00m

Description :
Replacement of inflatable rubber dam home made from conveyor belts.

Contact :
Algonequin Power Systems inc.
Mark Unsworth, 905-273-8906
Graham Agnew, 905-273-8919

CARD N°5

Project	:	Mitis - 2
Country	:	Canada
State or Province	:	Québec
City or region	:	Mont-Joli
River	:	Mitis
Commissioning year	:	1996
Number of Rubber Dam	:	1

Size :

Section 1 : H= 0,63m x L= 22,80m

Description :

Spillway

Contact :

M. Denis Gauthier
Hydro-Québec, région Matapédia
Telephone : (418) 724-1176

CARD N°6

Project	:	La Pulpe
Country	:	Canada
State or Province	:	Québec
City or region	:	Rimouski
River	:	Rimouski
Commissioning year	:	1997
Number of Rubber Dam	:	1

Size :

Section 1 : H= 1,70m x L= 40,00m

Description :

Spillway

Contact :

M. Patrick Thibodeau
Boralex inc.
Telephone : (819) 363-5872

CARD N°7

Project	:	Charlo River
Country	:	CANADA
State or Province	:	New-Brunswick
City or region	:	Charlo
River	:	Charlo
Commissioning year	:	1997
Number of Rubber Dam	:	1

Size :

Section 1 : H= 0,82m x L= 60,00m

Description :

Replacement of flash boards

Contact :

M. Roger Pelletier
Avenor Maritime inc.
Telephone : (506) 684-9399

CARD N°8

Project	:	Windsor
Country	:	Canada
State or Province	:	Québec
City or region	:	Windsor
River	:	Saint-François
Commissioning year	:	1995
Number of Rubber Dam	:	4

Size :

Section 1 : H= 1,70m x L= 25,40m
Section 2 : H= 1,70m x L= 39,68m
Section 3 : H= 1,70m x L= 69,06m
Section 4 : H= 1,70m x L= 34,28m

Description :

Replacement of flashboards

Contact :

M. Pierre Legault
Hydro-Windsor inc.
Telephone : (450) 449-1046

CARD N°9

Project	:	Eustis
Country	:	Canada
State or Province	:	Québec
City or region	:	Sherbrooke
River	:	Coaticook
Commissioning year	:	1995
Number of Rubber Dam	:	1

Size :

Section 1 : H= 1,15m x L= 20,75m

Description :

Replacement of steel flashboards

Contact :

M. Raynald Lévesque
Hydro-Sherbrooke
Telephone : (819) 821-5748

CARD N°10

Project	:	R.S.P. #2
Country	:	Canada
State or Province	:	Québec
City or region	:	Forestville
River	:	Sault-aux-Cochons
Commissioning year	:	1995
Number of Rubber Dam	:	1

Size :

Section 1 : H= 1,70m x L= 28,43m

Description :

Spillway

Contact :

M. Patrick Thibodeau
Boralex inc.
Telephone : (819) 363-5872

CARD N°11

Project	:	McDougall
Country	:	Canada
State or Province	:	Québec
City or region	:	Pont-Rouge
River	:	Jacques-Cartier
Commissioning year	:	1994
Number of Rubber Dam	:	1

Size :

Section 1 : H= 1,90m x L= 25,50m

Description :

Spillway

Contact :

M. Patrick Thibodeau
Boralex inc.
Telephone : (819) 363-5872

CARD N°12

Project	:	Glenford
Country	:	Canada
State or Province	:	Québec
City or region	:	Ste-Christine
River	:	Sainte-Anne
Commissioning year	:	1994
Number of Rubber Dam	:	1

Size :

Section 1 : H= 1,70m x L= 75,00m
Section 2 : H= 1,70m x L= 8,00m

Description :

Spillway and sluice gate for debris

Contact :

Algonoquin Power Systems inc.
Mark Unsworth, 905-273-8906
Graham Agnew, 905-273-8919

Complementary information:

List of american sites in cold regions

Project	State or region
Altoona City	Pennsylvanie
Bolton Falls	Vermont
Broadwater Power	Montana
Centenial	Vermont
Dodge Falls	New Hampshire
Eagle Bend	Idaho
Essex-19	Vermont
Ford Motor	Minnesota
Forest Park	Pennsylvanie
Green Island	New York
Hazelton B	Idaho
Heuvelton	New York
Highgate Falls	Vermont
Hotwood	Pennsylvanie
Island Park	Idaho
Mill Run	Pennsylvanie
Mineral Ridge	Ohio
Palmer Falls	New York
Proctor	Vermont
Rainbow	Montana
Stony Brook	Connecticut
Susquehana River	Pennsylvanie
West Danville	Vermont
Winnooski	Vermont

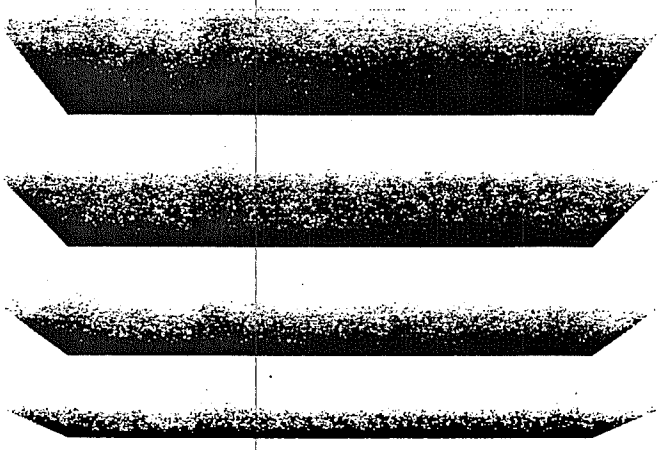
APPENDIX G

LIST OF WORLD EXPERIENCE

BRIDGESTONE

RD002-9706

SUPPLY RECORD OF BRIDGESTONE RUBBER DAM



International Supply Records of Bridgestone Rubber Dams

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1982	Chen Lili Xi	Taiwan	1.30× 8.00×1	Irrigation	SYS-1	
1982	Shashan	Taiwan	2.20× 25.00×1	Irrigation	SYS-1	
1982	Dongshi	Taiwan	1.00× 16.0 ×1	Irrigation	SYS-1	
1982	Indus River	Hong Kong	2.75× 30.00×1	Water Supply	SYS-1-1	(Fabridam Replacement)
1983	Vaca Dam	Philippines	2.00× 12.34×3	Irrigation	SYS-1	(Steel Gate Replacement)
1984	Susquehanna River #4	U.S.A.	2.44× 88.7 ×1	Recreation	SYS-1-1	(Fabridam Replacement)
1984	Peinan River	Taiwan	1.80× 10.00×1	Irrigation	SYS-1	
1985	Non Wai Dam	Thailand	0.60×125.00×1	Irrigation	SYS-1-1	(Crest-up)
1985	Felton Dam	U.S.A.	2.44× 22.40×1	Water Storage/Division	SYS-7	(Fabridam Replacement) V-Notch Control
1985	San Gabriel River #3	U.S.A.	2.38× 30.48×1	Groundwater Recharging	SYS-1-3	(Fabridam Replacement)
1985	Susquehanna River#5,#6	U.S.A.	2.44× 88.70×2	Recreation	SYS-1-1	(Fabridam Replacement)
1985	Songnae Chon	Korea	2.2 × 32 ×1	Irrigation	SYS-1	
1986	Weeks Falls	U.S.A.	2.50× 22.86×1	Hydropower	SYS-7(SCUL) +-50MM	5MW plant V-NOTCH CONTROL
1986	Mirani Weir	Australia	1.80×107.25×1	Irrigation	SYS-5(SCIP) 2.2M-M	(Crest-up) V-NOTCH CONTROL
1986	Susquehanna River#2,#3	U.S.A.	2.44× 88.70×2	Recreation	SYS-1-1	(Fabridam Replacement)
1986	Gon Lau River	Taiwan	1.50× 27.0 ×2	Irrigation	SYS-1	
1987	San Gabriel River #6	U.S.A.	2.13× 61.00×1	Groundwater Recharging	SYS-5	
1987	Palmer Falls	U.S.A.	1.83× 61.62×1 1.83× 45.48×1	Hydropower	SYS-5(SCIP)	(Flashboard Replacement)
1987	Los Angels River	U.S.A.	2.13× 39.62×1	Recharge/Irrigation	SYS-5	(Fabridam Replacement)
1987	San Gabriel River #4	U.S.A.	1.83× 61.00×1	Groundwater Recharging	SYS-5	
1987	Altoona City	U.S.A.	1.53× 35.70×1	Drinking Water	SYS-1	(Crest-up)
1987	Shing Chu River	Taiwan	1.60× 8.00×1	Flood Control	SYS-1	
1988	Detroit Sewage	U.S.A.	2.00× 4.42×2	Sewage Flow Regulator	SYS-1	Flow Regulator, Polluted Water
1988	Citrus Spreading Ground	U.S.A.	0.82× 9.15×1	Groundwater Recharging	SYS-5	
1988	Dae Am	South Korea	0.90× 31.80×1	Groundwater Recharging	SYS-1	
1988	San Gabriel River #8	U.S.A.	1.82× 73.15×1	Groundwater Recharging	SYS-1-3	

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1988	San Gabriel River #7	U.S.A.	1.82× 60.97×1	Groundwater Recharging	SYS-1-3	
1988	San Gabriel River #5	U.S.A.	1.82× 60.97×1	Groundwater Recharging	SYS-1-3	
1988	Broadwater Power	U.S.A.	3.35× 15.54×7	Hydropower	SYS-7(SCUL) +-50MM	(Crest-up) Winter temperature to-40C
1988	Boddington	Australia	0.90× 13.50×1	Mining Reservoir	SYS-3	(Crest-up)
1988	Pit River Dam #3	U.S.A.	1.83× 25.53×3	Hydropower	SYS-5(SCIP)	Large Arch Dam (Crest-up)
1988	Tin Shui Wai	Hong Kong	2.20× 52.50×1	Tide Barrier	SYS-3	CSCES
1988	Susquehanna River #1,#7	U.S.A.	2.44× 88.70×1 2.44× 50.60×1	Recreation	SYS-1-1	(Fabridam Replacement) World's Longest Completed
1988	Too Puu Kei	Taiwan	1.0 × 30.0 ×1	Irrigation	SYS-1	
1989	Rainbow	U.S.A.	3.65× 67.05×2	Hydropower	SYS-7(SCUL) +-50MM	Extreme Cold Weather Environment (Flashboard Upgrade)
1989	Fukubasen	Taiwan	2.20× 5.00×1	Irrigation	SYS-1	
1989	Tauranga	New Zealand	1.60× 16.00×1	Hydropower	SYS-1	
1989	Lunde	Norway	2.92× 64.40×1	Hydropower&Navigation	SYS-7(SCUL) +-50MM	(Needle gate Replacement)
1989	Alameda	U.S.A.	3.96× 88.8 ×1	Groundwater Recharging	SYS-5(SCIP)	Ceramic Cover
1989	Kjeldal	Norway	2.03× 84.08×1	Hydropower&Navigation	SYS-7(SCUL) +-50MM	(Needle gate Replacement)
1989	Sand Creek	U.S.A.	2.74× 24.38×1	Recreation	SYS-6 +-150MM	Ceramic Cover (Fabridam Replacement)
1989	Uirimji / Che-Chen	South Korea	2.50× 12.0 ×1	Irrigation(Creast-up)	SYS-1	
1989	Paek-Kok-Ji / Je-Cheon	South Korea	1.24× 25.0 ×1	Irrigation(Creast-up)	SYS-1	
1990	Hafslund.Vamma	Norway	1.70× 44.00×1	Hydropower	SYS-5(SCIP)+2.0 -0.3M	(Crest-up)
1990	Reimei Irrigation	Taiwan	1.40× 21.80×1	Irrigation	SYS-1	
1990	Brantas / Menturas	Indonesia	2.10× 7.20×2 2.10× 12.60×1 2.10× 16.20×1 2.10× 29.20×1 2.10× 63.60×1	Irrigation	SYS-2 (Auto Electrical Deflation)	OECF Loan
1990	Shin Chon #2	Korea	1.80× 25.00×1	Visual Recreation	SYS-1	
1990	Lam Pao	Thailand	2.00× 15.0 ×3	Irrigation	SYS-6 +-150MM	(Crest-up)
1990	Huey Sam Pad	Thailand	2.00× 17.80×3	Irrigation	SYS-7(SCUL) +-100MM	
1990	Yuen Long	Hong Kong	3.0 × 59.00×1	Sewage Control Tidal Barrier	SYS-5	
1990	Mill Run	U.S.A.	1.83× 12.00×2	Drinking Water	SYS-5(SCIP)	(Fabridam Replacement)

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1990	Gou Lau	Taiwan	1.00 ×27.0 ×1	Irrigation	SYS-1	
1990	Shin Chon #1	Korea	1.00 ×65.35 ×1	Visual Recreation	SYS-1	
1991	Shin Chon #4	Korea	1.50 ×50.65 ×1	Visual Recreation	SYS-1	
1991	Shin Chon #5	Korea	1.70 ×50.747×1	Visual Recreation	SYS-1	
1991	Sa Yeon	Korea	1.50 ×56.00 ×1	Irrigation(Crest-up)	SYS-1	
1991	Green Island	U.S.A.	0.61 × 7.32 ×1 0.61 ×54.29 ×1 0.61 ×55.82 ×1 0.61 ×57.34 ×1	Hydropower	SYS-5'(SCIP+ Oscillation Control)	(Flashboard Upgrade)
1991	Brazeau RIver	U.S.A.	3.35 ×82.00 ×1	Water Control Power canel Flood protection	SYS-1	Air Inflation & Water Deflation
1991	Bolton Falls	U.S.A.	1.525×59.475×1	Hydropower	SYS-6(SCIP)	(Flashboard Upgrade) V-NOTCH CONTROL
1991	Nairn Falls	Canada	0.61 ×39.65 ×1	Hydropower	SYS-6	(Flashboard Upgrade)
1991	Beas River BR8,9,10,11	Hong Kong	1.60 ×13.04 ×1 1.40 ×13.00 ×1 1.24 ×16.76 ×1 0.70 ×10.80 ×1	Irrigation	SYS-1	
1991	Shouka	Taiwan	2.70 × 9.00 ×1	Irrigation	SYS-1	
1991	Sylvan 2	U.S.A.	1.27 × 5.95 ×1	Hydropower	SYSTEM-1-1	(Flashboard Upgrade) Temp to -40°
1992	Beas River & Yuen Long BR16 & YLN65,117	Hong Kong	2.0 × 32 ×1 1.8 × 12.4 ×1 1.4 × 22.2 ×1	Irrigation	SYS-1	MARSHAL KARSON
1992	Kumhogang	Korea	1.5 × 30 ×1	Visual Recreation	SYS-1	
1992	Dairenpi	Taiwan	3.2 × 13 ×1	Irrigation	SYS-1	
1992	Shinchiku	Taiwan	1.75 ×24.247×1	Irrigation	SYS-1	
1992	Claude Wharton	Australia	1.65 ×79.1 ×2	Irrigation	SYS-5(SCIP) +1.9M & -1.95M Def and Manual inflation	(Crest-up)
1992	Dryden	U.S.A.	0.915×57.95 ×1	Fish Hatchery Diversion	SYS-1	Water Filled System
1992	Winooski	U.S.A.	2.59 ×6.1 ×1 2.95 ×40.1 ×1	Hydropower	SYS-5(SCIP)	
1992	Cleveland Sewer Gate	U.S.A.	2.9 ×2.9 ×1 3.05 ×3.05 ×1 1.42 ×2.44 ×1	Sewage	SYS-1	Expan Gate (Full Tunnel Closure)
1992	Sissiboo Falls	Canada	2.44 ×12.2 ×7	Hydropower	SYS-5(SCIP)	Ogee Crest(8-0')
1992	Banga South Cotabato (Epron Rubber)	Philippines	2.0 ×3.0 ×1	-	-	

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1992	Eina Dam	Norway	0.5 × 20 ×1	Hydropower	SYS-1	AX-FIXING
1992	Gubeng / Surabaya	Indonesia	2.85 × 12 ×2	City Flashing	SYS-3	OECF Loan Ceramic Caver Double Ancher
1992	Kam Tin & Yuen Long YLN162,189&191	Hong Kong	1.4 × 12.0 ×1 2.0 × 7.5 ×1 1.7 × 12.0 ×1	Irrigation	SYS-1	Luen Cheong Tai
1992	Hazelton B	U.S.A.	0.92 × 70.15 ×1	Hydropower	SYS-6	
1992	Highgate Falls	U.S.A.	4.57 × 66.45 ×1	Hydropower	SYS-7(SCUL)	Ogee Crest(15-0')Heavy ice
1992	Eagle Bend	U.S.A.	3.28 × 8.43 ×1	Water Storage	SYS-5	Air Inflation Water Deflation
1992	Shin Chon	Korea	1.5 × 50.65 ×1	Visual Recreation	SYS-1	(Replacement)
1993	Nan Dea Chon #2	Korea	2.0 × 50.0 ×1	Irrigation	SYS-1-1	
1993	Ohmurahaisui	Taiwan	2.6 × 8 ×1	Irrigation	SYSTEM-1-1	
1993	Heuvelton	U.S.A.	3.2 × 8.54 ×2	Hydropower	SYS-6	(Crest-up)
1993	Five Coves	U.S.A.	2.13 × 97.56 ×1	Groundwater Recharging	SYS-5	
1993	Yueng long YLN160	Hong Kong	1.5 × 11 ×1	Irrigation	SYS-1-1	
1993	Mitis-II	U.S.A.	0.6 × 22.86 ×1	Hydropower	SYS-5	
1993	Capay Diversion	U.S.A.	1.5 ×142.2 ×1	Irrigation	SYS-5	Ceramic Cover
1993	Dodge Falls	U.S.A.	0.61 ×22.86 ×1	Hydropower	SYS-5	(Flashboard Upgrade)
1993	Jati/Madium	Indonesia	3.0 ×21.2 ×2	Irrigation	SYS-2	OECF Loan
1993	Gombal/Ponorogo	Indonesia	2.4 ×23.22 ×1	Irrigation	SYS-1-2	OECF Loan
1993	Kori/Ponorogo	Indonesia	2.5 ×15.11 ×2	Irrigation	SYS-1-2	OECF Loan
1993	Kedung Celeng/Madium	Indonesia	2.0 ×17.76 ×1	Irrigation	SYS-1-2	OECF Loan
1993	Sungkur/Ponorogo	Indonesia	2.8 ×20.22 ×2 2.8 ×19.47 ×1	Irrigation	SYS-1-2	OECF Loan
1993	Tirtonadi/Solo	Indonesia	2.25 ×30.0 ×1	City Flashing	SYSTEM-1-2	Double Auchor
1993	Horseshoe Bend	U.S.A.	2.44 ×15.24 ×1	Hydropower	SYS-5 SYS-6	
1993	Cleveland	U.S.A.	RFo9'× ×1 RS6' 1 BF3'-10' 4'-3 1/2" 1 RF5'-1 3/8 1	Sewage	SYS-6	Flow Control
1993	Vanan	Sweden	1 ×27 ×1	Recreation	SYS-6	

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1993	Soo River	Canada	2.0 × 14 ×1	Hydropower Diversion	SYS-6	
1993	Tlaxcala	Mexico	1.5 × 13.8 ×1	Irrigation	SYS-1-1	
1994	Tegucigalpa	Honduras	3.5 × 63 ×1	Water Supply	SYS-6	Spillway control IBRD Loan
1994	Laba Lembangan	Malaysia	2.5 × 12 ×1	Irrigation	SYS-1	
1994	McDougall	Canada	1.9 × 25.5 ×1	Hydropower	SYS-6	
1994	Mount Piper Power Station Units 1&2	Australia	3.5 × 29.26 ×2	Hydropower	SYS-6'	
1994	Mineral Ridge	U.S.A.	0.45 × 78 ×1	Water Supply	SYS-6	(Flashboard Upgrade)
1994	Walnut Creek	U.S.A.	3 ×135 ×1	Groundwater Recharging	SYS-5	
1994	Proctor	U.S.A.	0.9 × 24.8 ×1	Hydropower	SYS-6	(Flashboard Upgrade)
1994	Glenford	Canada	1.7 × 74.7 ×1 1.7 × 8 ×1	Hydropower	SYS-6	
1994	Colusa	U.S.A.	1.5 × 16.5 ×1	Irrigation	SYS-5	
1994	Windsor	Canada	1.7 × 50 ×4	Hydropower	SYS-6	(Flashboard Upgrade)
1994	Stony Brook	U.S.A.	1.78 × 10.67 ×1	Drinking Water	SYS-1	Spillway control
1994	Forest Park	U.S.A.	1.35 × 19.5 ×1	Drinking Water	Owner supplied	Intake level control
1994	Sonoma County	U.S.A.	3.3 × 36.9 ×1	Groundwater Recharging	Manual	(Inbertson Water filled replace)
1994	Kumpul Kuista / Cirebon	Indonesia	3 × 34.5 ×1	Water Supply	SYS-1-2	
1994	Urepetiro	Mexico	1.5 × 20 ×1	Hydropower	SYS-1	Spillway upgrade
1994	Cisirih / Anyer	Indonesia	2.5 × 12.5 ×1	Water Supply	SYS-1-2	
1994	Santa Fe	U.S.A.	1.831×171.541×1	Water Supply	SYS-5	Longest single span in the world
1994	Forestville	Canada	1.7 ×28.43 ×1	Hydropower	SYS-6	
1994	Nusa Dua / Bali	Indonesia	1 ×9.25 ×2	Water Supply	SYS-1-2	Double Anchor
1994	Ohmurahaisui	Taiwan	2.25 ×8 ×1	Irrigation	N / A	
1995	Jejeruk / Magetan	Indonesia	2.3 ×28.65 ×1	Irrigation	SYS-1-2	OECF Loan
1995	Pulo / Ponorong	Indonesia	2.3 ×21.88 ×1	Irrigation	SYS-1-2	OECF Loan
1995	Beringin / Magetam	Indonesia	2.3 ×21.58 ×1	Irrigation	SYS-1-2	OECF Loan
1995	Sungai Tampines (HDB)	Singapore	2.2 ×24 ×1	Beatification	SYS-5+	

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1995	Kougo River	Taiwan	1.8 ×12 ×1	Irrigation	N / A	
1995	Anton River	Taiwan	1.7 ×10 ×1	Irrigation	N / A	
1995	Mamquan	U.S.A.	3.2 ×15.3 ×2	Hydropower	SYS-6	Diversion
1995	Holtswood	U.S.A.	3.3 ×11.3 ×1	Hydropower	SYS-7	Flushboard Emergency Spillway
1995	Essex -19	U.S.A.	1.5 ×21.6 ×1 1.5 ×49.5 ×1 1.5 ×52.5 ×1	Hydropower	SYS-6	(Flashboard Upgrade)
1995	Nairn Falls	Canada	0.6 ×39 ×1	Hydropower	SYS-6	(Flashboard Upgrade)
1995	Ban Nong Yang	Thailand	2.0 ×70 ×1	Irrigation	SYS-1-2	
1995	Centennial	U.S.A.	1.5 ×33 ×1 1.5 ×33 ×1	Hydropower	SYS-6	(Flashboard Upgrade)
1995	Ciberung / Ciregon	Indonesia	2 ×21.7 ×1	Water Supply	SYS-1-2	Double Auchor
1995	Ford Motor	U.S.A.	0.6 ×83.1 ×1 0.6 ×81.1 ×1	Hydropower	SYS-1-1	(Crest-up)
1995	Island Park	U.S.A.	0.3 ×30 ×2	Hydropower	SYS-5	(Crest-up)
1995	Kang Sanam Nang	Thailand	2.5 ×80 ×1	Irrigation	SYS-1-2	
1995	Wawan / Irianjaya	Indonesia	2 × 6 ×1	Irrigation	SYS-1-2	Double Auchor
1995	West Danville	U.S.A.	0.75× 7.96×1	Hydropower	SYS-1-1	(Flashboard Upgrade)
1995	Ranbatan / Indramaya	Indonesia	2 ×27.5 ×2 2 ×15 ×1	Water Supply	SYS-1-2	
1995	San Jacinto	Bolivia	1.5 ×10 ×4	Irrigation	SYS-5	(Crest-up)
1995	Sherbrook	Canada	1.15×21.6 ×1	Hydropower	SYS-6	(Flashboard Upgrade)
1995	Manantiales	Colombia	0.75× 9.9 ×1	Water Supply	SYS-5	
1995	Janeberang/Ujung Pandang	Indonesia	2 ×95 ×2 2 × 9.5 ×2	Irrigation	SYS-1-2	OECF Loan Side Spans:Ceramic Cover
1996	Hunts Point	U.S.A.	1.8 × 3 ×2	Sewage	By customer	Expansion Gate
1996	Bayong	Malaysia	2.0 ×30 ×1	Irrigation	SYS-6	
1996	Lumsden	Canada	0.97×45.8 ×1	Hydropower	SYS-1-1	
1996	Kamtin(DSD)	H.K.	2.4 ×28 ×1 2.4 ×21 ×1	Tidal Barrier	SYS-5	Double Anchor
1996	Lamchi Muang Ling	Thailand	4 ×73 ×1	Irrigation	SYS-1-1	
1996	Way Koko	Indonesia	2 ×20 ×1	Irrigation	SYS-1-1	OECF Loan

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1996	Madium	Indonesia	2.3 ×25 ×1	Irrigation	SYS-1-1	
1996	Centennial	U.S.A.	1.95×30.5 ×1	Hydropower	SYS-6	(Flashboard Upgrade)
1996	Los Esclavos	Guatemala	2.02×14.35×1	Irrigation	SYS-1-1	
1996	Castelnuovo Garfagnana	Italy	1.6 ×22 ×1	Hydropower	SYS-1	
1996	Anson	U.S.A.	4.08×12.19×1	Hydropower	SYS-6	
1996	Mapocho	Chile	1.5 × 14.4 ×1	Irrigation	SYS-6	
1996	Coors	U.S.A.	1.5 × 18 ×1	Hydropower	N / A	
1996	Mitis-2	Canada	0.62 × 23 ×1	Hydropower	SYS-6	
1996	Huntington Falls	U.S.A.	0.76 × 55 ×1	Hydropower	SYS-5	
1996	Centennial	U.S.A.	1.95 × 30.5 ×1	Hydropower	SYS-6	Flashboard Replacement
1996	Los Esclavos	Guatemala	2.02 × 14.35 ×1	Irrigation	SYS-1-1	
1996	Castel Nuvoe	Italy	1.6 × 22 ×1	Irrigation	SYS-6	
1996	Anson	U.S.A.	4.08 × 12.19 ×1	Hydropower	SYS-6	
1996	Mapocho	Chile	1.5 × 14.4 ×1	Irrigation	SYS-6	Extra Cover
1996	Coors	U.S.A.	1.5 × 18 ×1		N/A	
1996	Mitis-2	Canada	0.62 × 23 ×1	Hydropower	SYS-6	
1996	Huntington Falls	U.S.A.	0.76 × 55 ×1	Hydropower	SYS-6	
1996	Hunts Point	U.S.A.	1.8 × 3 ×2	Sewage	By customer	Expansion Gate
1996	Bayong	Malaysia	2.0 × 30 ×1	Irrigation	SYS-6	
1996	Lumsden	Canada	0.97 × 45.8 ×1	Hydropower	SYS-1-1	
1996	Kam Tin(DSD)	H.K.	2.4 × 28 ×1 2.4 × 21 ×1		SYS-5	
1996	Lam Chi Muang Ling	Thailand	4 × 73 ×1	Irrigation	SYS-1-1	
1996	Way Koko	Indonesia	2 × 20 ×1	Irrigation	SYS-1-1	
1996	Madium	Indonesia	2.3 × 25 ×1			
1996	Stone		0.838×127.99 ×1			
1996	Solo/Sedayu Lawas	Indonesia	3 × 25 ×4			

DELIVERY	PROJECT NAME	COUNTRY	SIZE(M) QTY	APPLICATION	OPERATION SYSTEM	NOTES
1996	Kounan	Taiwan	2 ×25 ×1			
1997	Tana Delta	Kenya	2.25×49.5 ×1			
1997	Angat	Philippines	2.5 ×76.9 ×3			
1997	Ban Tha Muang	Thailand	1.0 ×40.0 ×1			
1997	Klong Wang Sai	Thailand	1.5 ×56.0 ×1			
1997	Rimouski	Canada	1.7 ×40.0 ×1			
1997	Wilmington Water Capture	U.S.A.	1.6 ×37.64×1			
1997	Indianapolis	U.S.A.	3.0 ×2.1 ×1			
1997	Anson Dam Phase2	U.S.A.	1.68×60.91×1 1.68×55.89×1 1.68×43.86×1			
1997	Charlo River	Canada	0.82×60.9 ×1			
						As of June 1997

APPENDIX G

TECHNICAL LITERATURE

Public Power Profile

Vermont village tries out a new idea in hydro: rubber!

Imagine a dam that goes up and down automatically to keep the proper water level in the reservoir. It never has to be painted or rustproofed. It has a smooth top surface so that, in a storm or flood, trees and debris slide over it instead of getting caught in the gates. In cold weather, ice won't jam it up. It's environmentally friendly, too: it has an inconspicuous profile and can be made to fold down on itself now and then to let sediment pass downriver.

The municipal electric system in the village of Swanton, Vt., has installed one of these devices at its hydro plant in the town of Highgate. It's an inflatable dam, called a rubber dam by its manufacturer, a subsidiary of the Tokyo-based Bridgestone Corp. "Rubber Dam" is Bridgestone's name for it, but it's not really a dam, said George Lague, village manager in Swanton. It's a controllable crest gate, one made out of rubber instead of the traditional steel.

Bridgestone is the company that bought Firestone and the rubber dam looks as if it was made by a tire company. The smooth cylinder of black rubber lying on top of Swanton's concrete dam on the Missisquoi River looks like an oversized length of inner tube. It's 15 feet in diameter and 220 feet long, made of a nylon-rubber combination about one and a quarter inches thick. Air intake and exhaust tubes let air in and out. Other tubes monitor air pressure and drain moisture that builds up inside.

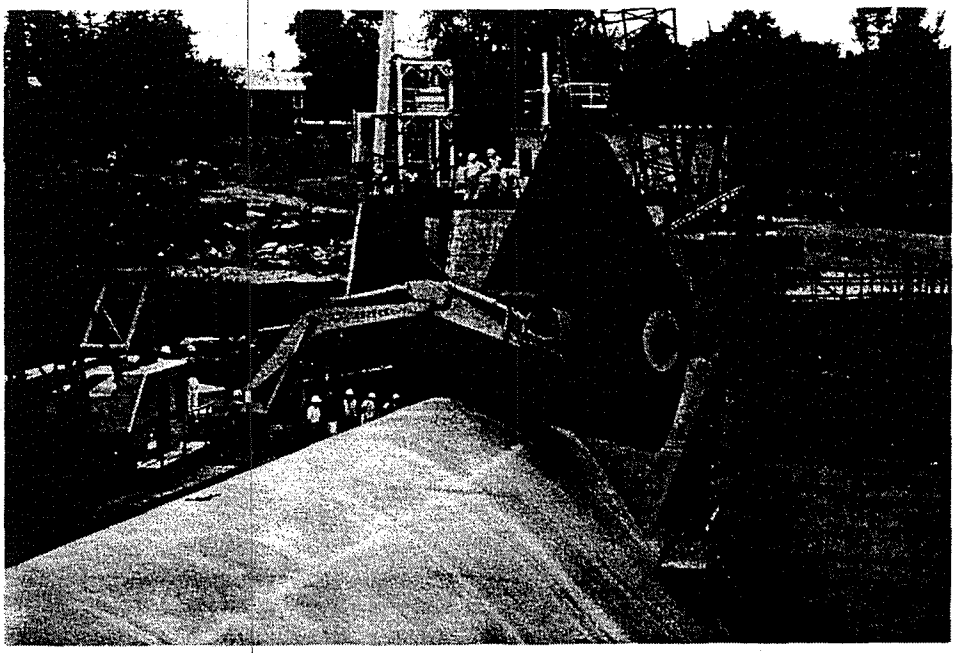
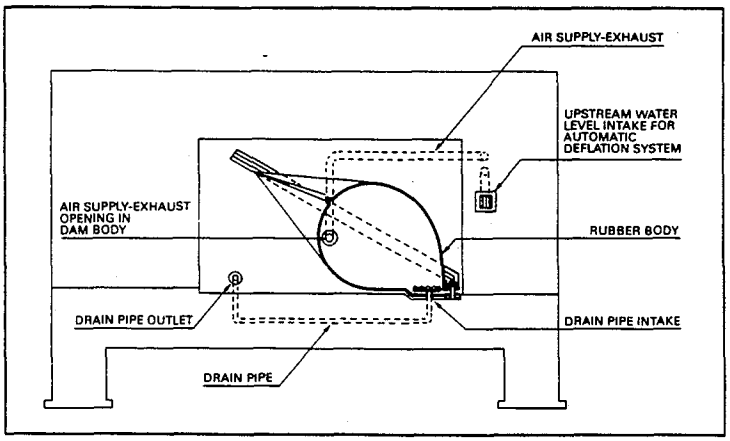


Photo courtesy of Bridgestone

Workers begin to unroll the huge piece of rubber onto the top of Swanton's dam on the Missisquoi.



Schematic drawing of the rubber dam.

Swanton's is the tallest rubber dam in North America and is among the first in the world to have a pressurized hatchway allowing dam operators to walk right inside. If a leak develops, it will be much easier to fix from the inside, Lague said.

There are a number of advantages to using the rubber dam, which has some give to it, rather than a steel crest gate, Lague said. In the harsh New England climate, steel requires de-icing in winter and causes expansion/contraction problems, he explained. The rubber dam, on the other hand, "has great, great, great flexibility," he said. It can be puffed up or flattened out simply by pumping air in or letting it out. There are no cumbersome steel gates to collect debris.

Last November, Swanton issued $8.75 million in bonds to finance the project. The dam is in place, but cannot be used yet because the municipal utility is in the process of acquiring some land around the existing impoundment at its Orman Croft Generating Station. After it secures all the land, it will cut trees and shrubs and raise the reservoir by 22 feet. This should happen by spring 1995, Lague said.

Swanton's peak load has been growing and is now about 10 MW, Lague said. "We're blessed for being on the Canadian border," he said. "There's a relatively steady migration of business from Canada into the United States to take advantage of U.S. markets and escape some Canadian taxes." Swanton also is a popular place to live because it is just a half-hour commute from Burlington, Vermont's largest city, he said.

The municipal utility has gradually been increasing the dam's capacity. In the early 1980s, utility officials planned to raise the dam by 32 feet, but the price turned out to be twice what they expected, so they tabled that idea. In 1989, the utility raised its capacity from three megawatts to four MW by adding a fourth generator at the hydro plant. Meanwhile, "we studied whether we could raise the dam a lesser amount and gain dollar savings," Lague said. The rubber dam option was suggested by the Turner

Group, a New Hampshire engineering firm. Raising the concrete portion of the dam slightly and adding the 15-foot rubber dam would create enough head to produce about 10 MW.

Lague was skeptical at first about the rubber dam. "I didn't know anything about rubber dams and didn't want to be a guinea pig," he told *Public Power Weekly*. But he was intrigued after visiting a Montana Power Co. hydro project, where one of Bridgestone's rubber gates was working well. The Montana Power official "was in love with them," Lague recalls. "He said they're very trouble-free, compared to the steel ones."

Montana Power has had two of the devices in place since 1989 at its Rainbow Dam on the Missouri River in Great Falls, Mont. Each is 12 feet high, three feet shorter than Swanton's, and 220 feet long, said Richard Halborson, operations supervisor for the utility's Region 2. They have performed very well, he told *Public Power Weekly*. The big rubber tubes control the water level in the reservoir quite well through a computer that instructs them to inflate or deflate. There have been some problems with a pressure transducer that keeps tabs on the reservoir's elevation, Halborson said. The transducer is in a steel pipe and mud gets in, preventing a correct read. If a utility needs to keep accurate river flows, the rubber dam will not take care of that, he noted; a separate monitoring device is required.

But the rubber device skims ice well and, since it is smooth on top with nothing to obstruct the flow of water, it lets trash pass over it. If a tree falls into the water and threatens to get hung up on the dam, the operator can simply deflate the dam and the tree will flow right over the top, whereas it probably would get stuck in traditional steel gates and flashboards. "In other places, we have to get chainsaws" to free the gates, Halborson said.

Rubber dams—also called inflatable weirs—have been used widely in Japan, where more than 1,000 are in operation. The technology is still quite new in the hydro power world, however. Most of the rubber dams in Japan are used for irrigation, tidal barriers and other forms of water control. Bridgestone has supplied about 600 of the rubber dams worldwide, said Jack Parent, a spokesman for the company. Of these, about 60 have been installed in the United States, he said. Sumitomo Electric of Japan also sells a rubber dam called Sumigate through the Rodney Hunt Co. in Orange, Mass., which has sold three of them over the last three years. All three were used for stormwater flow control, not hydro power. ■

—Jeannine Anderson

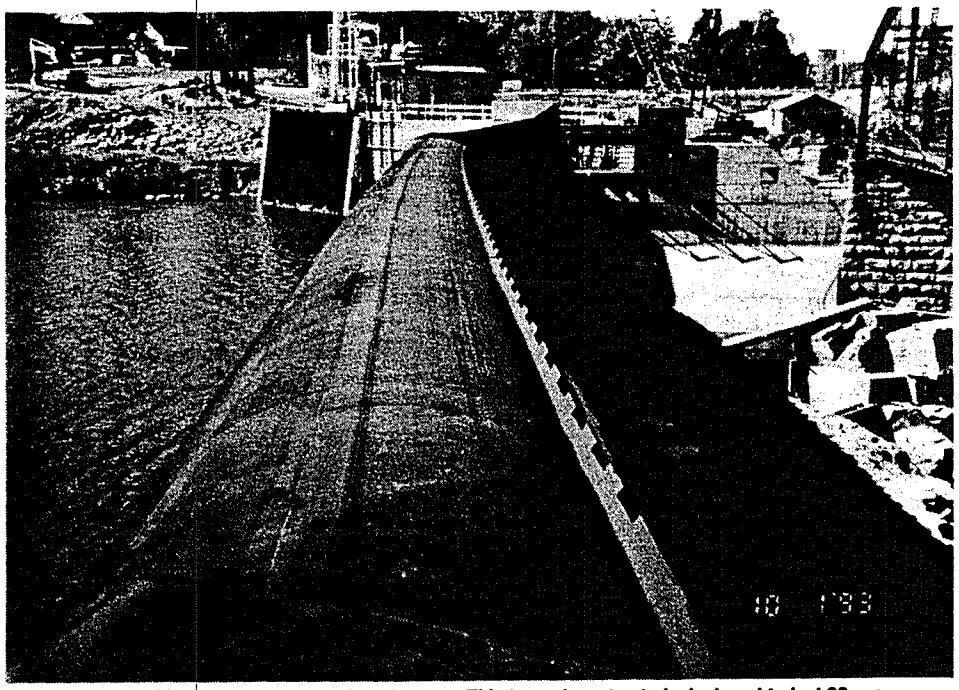


Photo courtesy of Bridgestone

The completed rubber dam, inflated and ready to go. This type of crest gate is designed to last 30 years.

At A Glance

Swanton is situated in the extreme northwestern corner of Vermont. It is six miles from Canada on Interstate 89, the highway that links the province of Quebec with Burlington and other New England cities. The Green Mountains are to the southeast; Lake Champlain and the Adirondack Mountains of New Yorkare to the southwest.

Swanton is located in Franklin County, known in Vermont for its dairy cows and maple sugar operations. The village is small, with a population of 2,360. Together, the village and town of Swanton are home to 5,636.

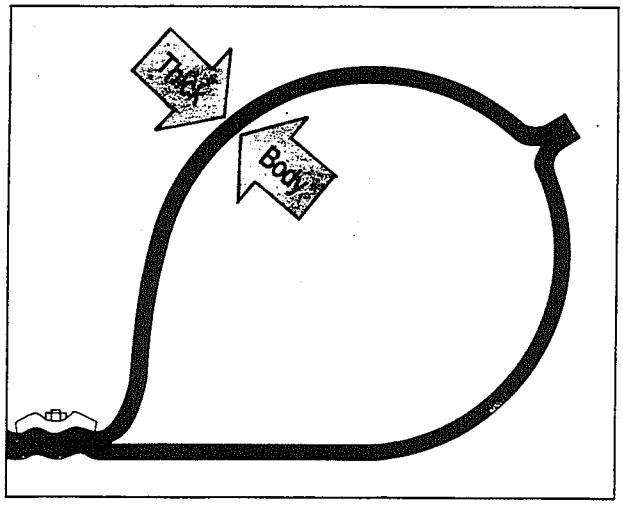
Winters are picturesque because of the snow but harsh, with the average temperature a chilly 17 F. In summer, the average is a moderate 72 F.

At one time Swanton had six railroad stations, with 25 trains a day passing through. Bog iron was mined, there was a woolen mill and a cotton mill. Later, several marble companies came to the area. During World War I, an ammunition plant employed 1,000 workers.

The community takes pride in its open green spaces and comfortable country atmosphere. "We like our community of agriculture, light manufacturing, tourists and recreation," Swanton proclaims in a leaflet distributed by the local Chamber of Commerce. "Progress isn't size, after all. On the other hand, we're not dragging our feet." Swanton has an industrial park and is interested in more industry—as long as that industry is clean and makes good use of the land, the chamber says. ■

Rubber Dam Features

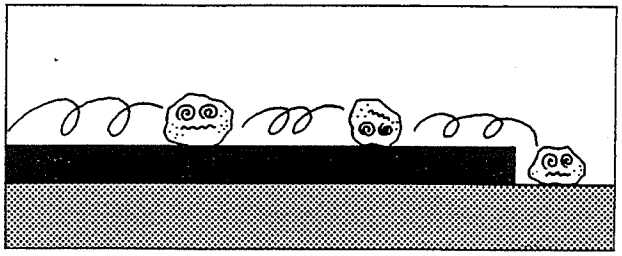
1 Thick Rubber Body



Superior Durability

Bridgestone rubber dams are manufactured with a high-pressure vulcanizing press, and feature a much thicker body than conventional fabric dams. In addition to outstanding abrasion resistance, the Bridgestone rubber dam features cut resistance, durability and easy installation.
Furthermore, the rubber body is made from EPDM (Ethylene Propylene Diene Monomer), which boasts superior resistance to weathering, ozone and heat. The adoption of this material has dramatically extended the life of the rubber dam, allowing it to be used in long-term applications. For heavy duty applications, a dam with a thicker rubber body is available. A ceramic armoured type can be supplied to protect against knife cuts.

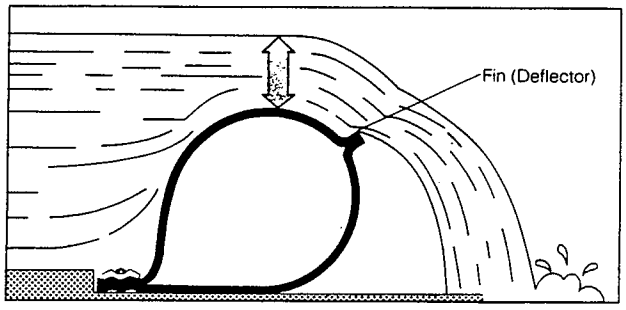
2 Resistance to Debris



Flat on Foundation

The fin structure provides lay-flat deflation which prevents damage from rolling stones or floating ice.
In addition, the lay flat structure prevents abrasion of foundation side bodey due to vibration caused by floating movement of bulged structure.

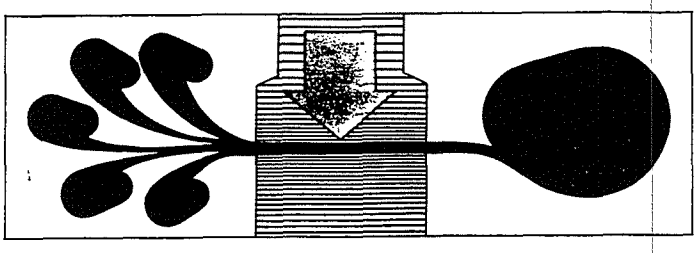
3 Oscillation Reduction



Fin (Deflector) Reduces Oscillation

The Bridgestone rubber dam features a special fin structure. The fin creates aeration below the fin when water flows over the fully inflated dam, virtually preventing any oscillation up to a 50% overflow.

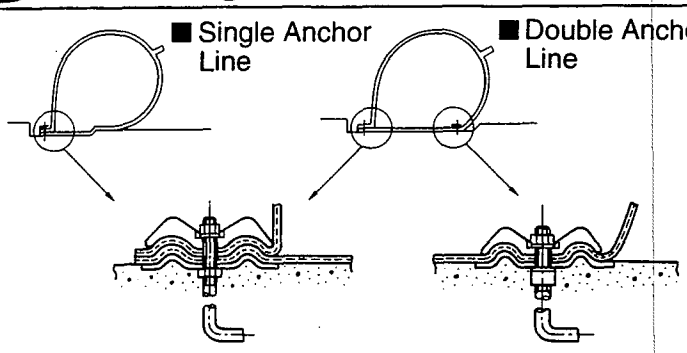
4 High-Pressure Vulcanizing



Long Term Quality

The rubber dam body is vulcanized by use of high temperature and uniform high pressure. This improves the adhesion between the rubber and the reinforcing fabric, providing more than adequate strength and consistently high quality for many years.

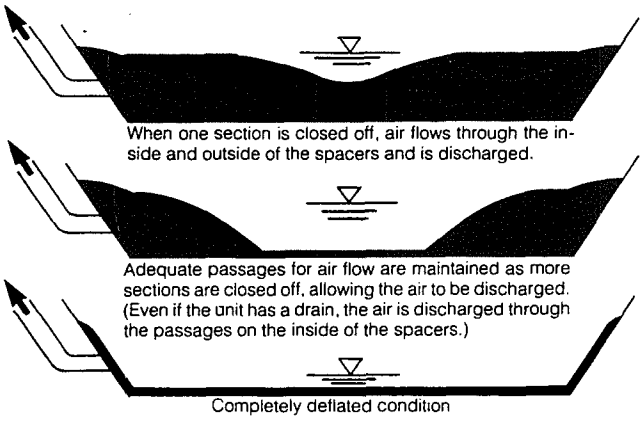
5 Fixing

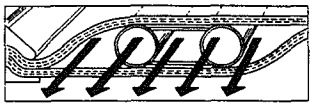


Easy Installation and Secure Anchoring

The installation procedure is easy — a uniform section of the rubber dam is secured by a clamping plate.
Normally, a single anchor line is applied, but a double anchor line can be provided when the dam is subject to a higher downstream water level.

6 Spacer





Quick Installation and Less Construction Work

Multiple hose spacers are installed inside the rubber dam body, allowing the dam to be completely inflated and deflated from one side, and eliminating the necessity of river bed piping. The elimination of the river bed piping makes it unnecessary to dig a deep pit for an additional operation room, representing substantial savings.
In addition, foundation work can be completed quickly and economically.

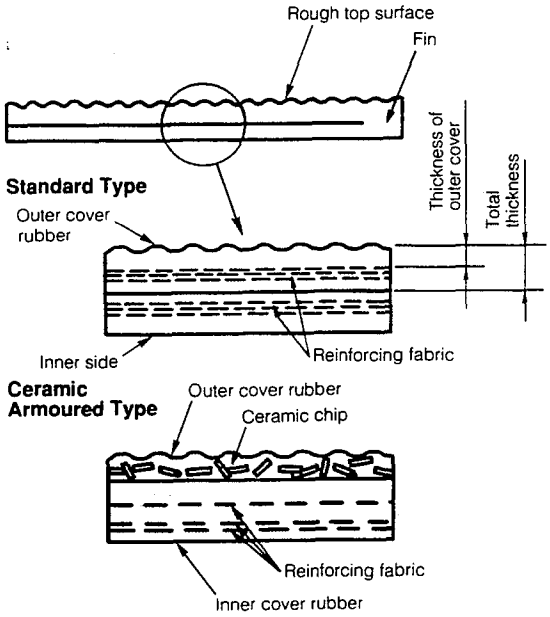
7 Operation System

Simple Operation

The rubber dam can be operated easily, safely and efficiently. Two types of mechanical auto deflation systems are available for different design conditions (refer to page 8 - 9).

Specifications of Rubber Dam Body and Fixing

● Construction of body



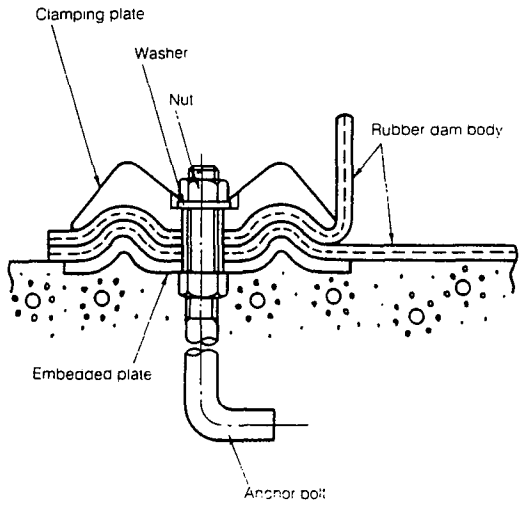
Extra cover is also available.

● Standard Thickness and Weight of Body

Height (m)	Thickness of Rubber		Weight (kg/m²)
	Total Thickness (mm)	Outer Cover (mm)	
Ho≦1.0	8.6	3.0	11
1.0<Ho≦2.2	10.6	5.0	13
2.2<Ho≦2.9	12.4	5.0	15
2.9<Ho≦3.5	13.8	5.0	17
3.5<Ho≦4.0	15.2	5.0	19
4.0<Ho≦6.0	22.5	5.0	28

Note: 1) Weight for one layer
2) Thicker rubber cover for heavy duty is also available.

● Construction of Fixing (Single Line)



● Fixing Materials

Parts	Standard	Option-A	Option-B	Option-C
Clamping Plate	FCD500 Galvanized	FCD500 Galvanized	FCD500 Galvanized	SCS13
Embedded Plate	SS400 or FCD500* Galvanized	SS400 or FCD500* Galvanized	SUS304	SUS304
Anchor Bolt	S45C Galvanized	SUS304	SUS304	SUS316
Nut	S45C Galvanized	SUS304	SUS304	SUS316
Washer	SUS304	SUS304	SUS304	SUS304

Note: Standard: All Galvanized Steel.
Option-A: Stainless Steel Bolt Set.
Option-B: Stainless Steel except Clamping Plate.
Option-C: All Stainless steel.
FCD500 in JIS is equivalent to ASTM A536 Gr.80
SCS13 in JIS is equivalent to ASTM A743
S45C in JIS is equivalent to ASTM A575 Gr.1045

*Embedded Plate for moro than 3.5mH will be SUS 304.