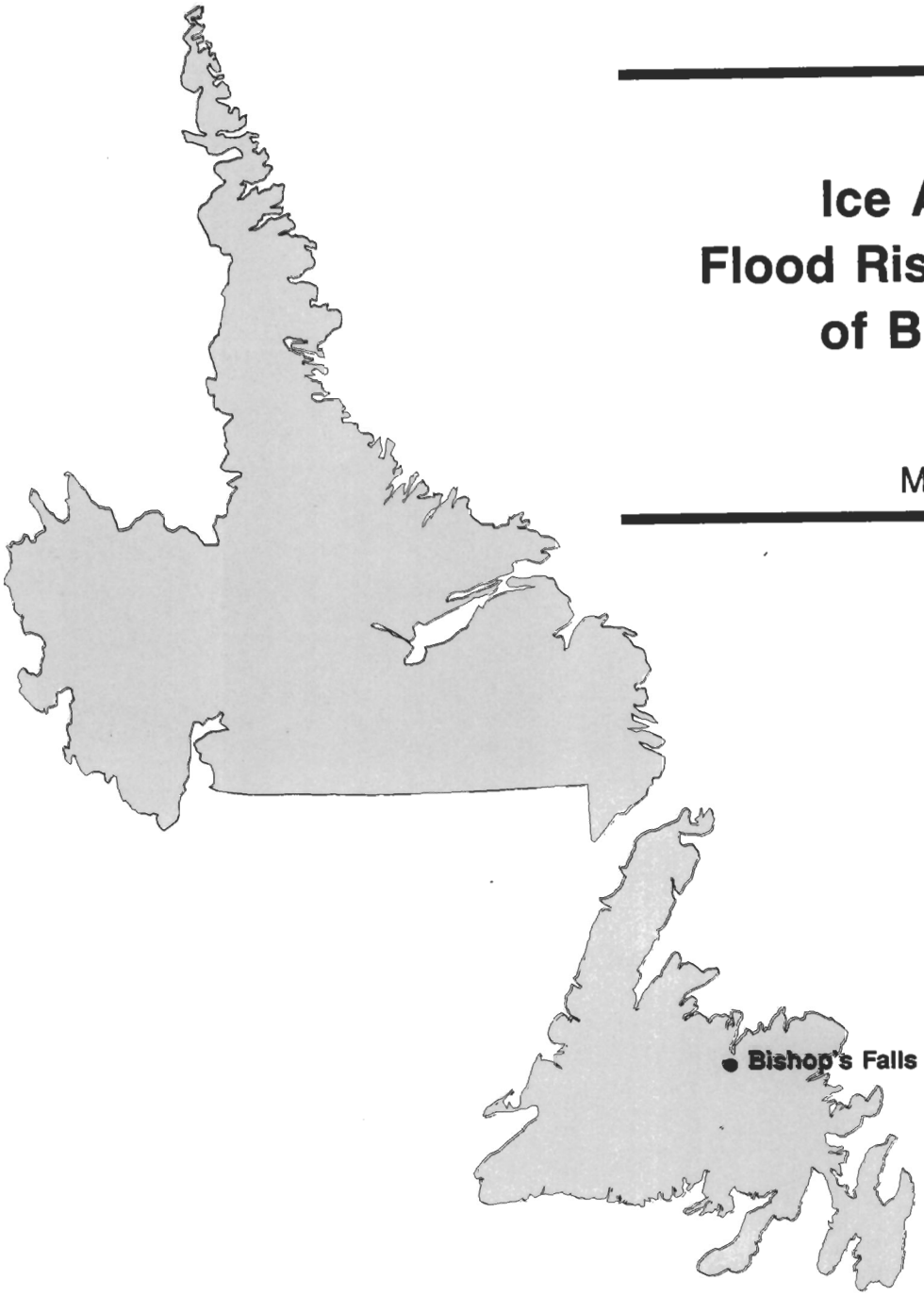
Canada – Newfoundland
Flood Damage Reduction
Program

Water Resources Division
Hydrological Modelling Section



Ice Analysis and Flood Risk Mapping Study of Bishop's Falls

MAIN REPORT

FENCO NEWFOUNDLAND LIMITED

Department of Environment and Lands

Environment Canada

Fenco Newfoundland

FENCO NEWFOUNDLAND LIMITED
189 WATER STREET, P.O. BOX 248
ST. JOHN'S, NEWFOUNDLAND, CANADA A1C 5J2 · TELEPHONE: (709) 754-1480
TELEX: 016-4936 · CABLE: FENCOENG · TELECOPIER: (709) 726-1145

52556-C1-380000

November 26, 1990

Canada-Newfoundland Flood Damage Reduction Program
Government of Newfoundland and Labrador
c/o Department of Environment and Lands
Confederation Complex
4th Floor West Block
P.O. Box 8700
St. John's, Newfoundland
AIB 4J6

Attention: Dr. W. Ullah, Ph.D., P.Eng.
Director, Water Resources Management

Re: Bishop's Falls - Ice Analysis and Flood Risk Mapping Study

Dear Dr. Ullah:

We take pleasure in submitting the enclosed Report and Technical Appendix on the above-titled study. The most up-to-date procedures were employed in determining the hydrologic and hydraulic regime of the Bishop's Falls area, and we anticipate that the enclosed will serve the Program well in the coming years.

The main report initially provides details about 1983 flood and the hydrologic modelling of the 20 and 100 year return period flood flows. These flows are then evaluated through hydraulic analysis to determine flood levels and the effect of ice accumulations. The last section of the report describes the flood-prone areas and flood damage reduction alternatives. A Technical Appendix (under separate cover) provides additional particulars about the study for use by your technical specialists.

It has been a distinct pleasure working on this project with members of the Flood Damage Reduction Program team. We particularly wish to thank you, Mr. Picco and Ms. Langley for the considerable assistance and constructive comments provided throughout the course of this study.

Yours very truly,

FENCO NEWFOUNDLAND LTD.

Clarence Hewitt, P.Eng.
Project Director

Douglas B. Hodgins, P.Eng.
Project Manager

Lavalin

Ice Analysis and Flood Risk Mapping Study of Bishop's Falls

MAIN REPORT

FENCO NEWFOUNDLAND LIMITED

BISHOP'S FALLS - FLOOD RISK MAPPING STUDY

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1.0 SUMMARY OF FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

Prior to the disastrous flood in January 1983, there was a period of about 80 - 90 years during which there were no reports of damaging floods at Bishop's Falls. Added to this are the recent 6 years since the flood when high flows have again been comparatively low. This length of dependable, non-problematic flows, punctuated by one major flood, points to the unpredictability of most flooding situations and the need for preparedness and vigilance along even the most "tame" river systems.

In view of these situations and of the potential for loss of life and damages resulting from floods and ice problems such as experienced at Bishop's Falls, the Province of Newfoundland and the Government of Canada have entered into an agreement to identify and delineate flood hazard areas. This study is part of that initiative and provides the necessary information for the identification of flood prone lands.

There is considerable background information on the 1983 flood, and this project began by collecting, reviewing and analyzing this information to obtain an understanding of this flood and the potential for future damaging floods. An hour-by-hour assessment of conditions in 1983 found, for example, that:

- ice was evident in the powerhouse forebay at midnight January 13, when it was heard to be hitting the stoplog gates

- considerable broken ice was observed, below the dam in the morning of January 14, but little was in evidence above the dam at 8:00 a.m. Ice appeared to have cleared the river below the dam by about 2:00 p.m.

- the flood flow peaked at about 11:00 a.m. January 14, about five hours before the earth dam failed and the erosion/loss of downstream buildings began in earnest

- the peak flow occurred at a time when there was discharge over the Ambersen Dam (spillway), through both sluice gates, over the powerhouse roof and over the un-eroded earth dam

The Terms of Reference for this ice analysis and flood risk mapping study note that one of the prime purposes of the work is to produce reliable estimates of the 1:20 year, 1:100 year and the historical flood profiles. Flood flow estimates to satisfy this objective were derived by various techniques including:

- single station frequency analysis of peak flow records from Bishop's Falls

- development and routing of dimensionless natural hydrographs through the river reservoir system and combining them with local inflows along the way to give an estimate of flood flow hydrographs which reflect the effect of reservoir operations

- regional frequency analysis of flow records within the general area of interest

The first analysis, a single station frequency analysis of over 50 years of flow data at Bishop's Falls, is considered most reliable for developing the 1:20 year and 1:100 year return period flood flows. The second, a hydrograph analysis, is somewhat theoretical but appropriate for confirming the reliability of the first method. The last approach, using regional flood flow regression analysis, is also a reliable method for (in this case) checking the results of the single station frequency analysis at Bishop's Falls.

The results of these three methods are listed below:

FLOOD FLOW ESTIMATES (M^3/S)

Return Period	**Frequency Analysis**	Hydrograph Approach	Regional Approach
1:20 year	2257	2171 - 2192	2186
1:100 year	2883	2980 - 3000	2919

The results of all methods give similar instantaneous flood flows. The hydrograph and Regional approaches give values which are slightly less than the more reliable results from the single station frequency analysis at the 1:20 year return period (2257 m^3/s). Alternatively, the 1:100 year frequency analysis result (2883 m^3/s) is slightly less than the estimates of the hydrograph and regional approaches. However, all values are within ±4% of the flood flow estimates derived from the single station frequency analysis at Bishop's Falls.

Given that the two approaches used to check the reliability of the single station frequency analysis give similar results to those from the frequency analysis, it is concluded that the frequency analysis results can be used with confidence to derive 1:20 year and 1:100 year flood flow profiles. The instantaneous flood flows for this analysis are:

1:20 year	2257 m^3/s
1:100 year	2883 m^3/s

Analysis of open water flood conditions observed at Bishop's Falls also determined the following flood flow:

1983 flood	3256 m^3/s

A number of field observations and hydraulic analyses of flood flows were assessed to determine the effect of ice conditions during the 1983 flood, and for other flood flows. These determined that:

- dynamic ice forces and action played a very small or negligible role in the failure of the earth dam during 1983

- the presence of ice contributed to slightly higher backwater levels upstream of the dam during 1983

- the effect of the ice concentration in 1983 was to displace a volume of water equivalent to approximately 256 m^3/s and, hence, super-elevate flood levels by about 0.2 m (or to the equivalent of a 3512 m_3/s flow rate)

The overall effect of a repeat of this ice condition with flood flows is summarized for several locations below:

Location	1:20 year Level (m)	1:100 year Level (m)	Level with 1983 Flood (m)
Ambersen Dam	14.61	15.10	15.53
Railroad Bridge	14.88	15.45	15.96
Six Km Upstream of the Dam	15.66	16.41	17.07

The above flood levels are those which would occur with the new dam and additional spillway gates which are now in place.

The greatest change with the new dam is the addition of the 10, 18 foot wide (5.5 m) spillway gates which supplement the overall spillage capacity over the Ambersen Dam. At the maximum high water level (14.5 m Geodetic - 136.5 ft Abitibi datum) each gate will pass about 280 m^3/s (10,000 cfs). All 10 gates will pass 2,800 m^3/s at this level.

Examination of the Operations and Maintenance Manual for the new facility determined that the operation of the gates has been protected by:

- heated gains in the gates to prevent ice build up on the downstream side

- a bubbler system on the upstream side to avoid adhesion of ice to the gates

- a back-up, standby generator, which is frequently tested, to operate the gates in the event of a power failure

- regular testing and maintenance of all gates

The objective of these protection measures and design features (which also include remote water level sensors and alarms) is to assure that the gate capacity will be available.

The flood levels given above were plotted on 1:2500 scale mapping which was prepared in October 1990 for this project. The new dam and spillway have the capacity to pass the 1983 flood and the flood line plots show that the flood risk area is very small at Bishop's Falls.

However, in view of the possibility of future development within the flood risk area, and in order to alleviate future damage problems:

1) It is recommended that the flood elevations advanced herein be adopted by the Municipality so that developable areas which are prone to flooding can be zoned in the near future for special flood risk restrictions and design consideration (e.g. no development, elevation flood proofing on fill, extended and reinforced foundation walls or piles).

2) It is recommended that current maintenance procedures be continued at the dam to ensure that ice protection measures and standby operations are in place to handle future flood flows.

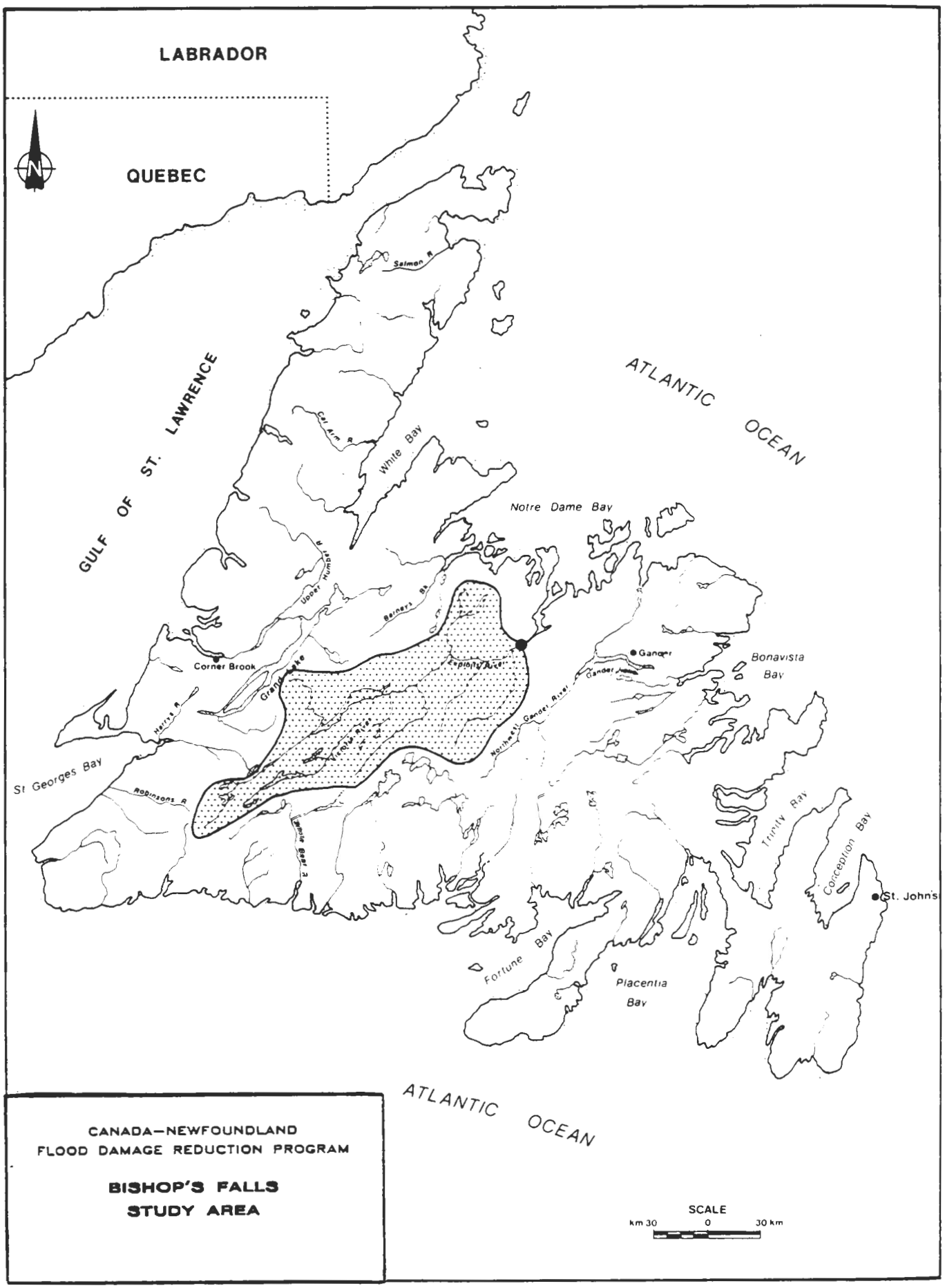
3) It is recommended that flood proofing by relocation or elevation on fill be considered for the garages, sheds, and uninhabited outbuildings in the flood fringe if it is desirable to provide physical flood protection for these existing buildings.

2.0 INTRODUCTION

Unprecedented runoff from storm precipitation and snowmelt in January 1983 resulted in Bishop's Falls being the centre of the worst flooding disaster ever experienced on the Island. Eight hectares (20 acres) of the Town and a number of buildings were swept away, the earth dam and mill buildings at the hydroelectric dam were destroyed, and overall direct damages in the area exceeded $33 million. The lesson learned from this disaster is that the threat of flooding is present in all riverside areas - even those, like Bishop's Falls, which do not have a history of flooding.

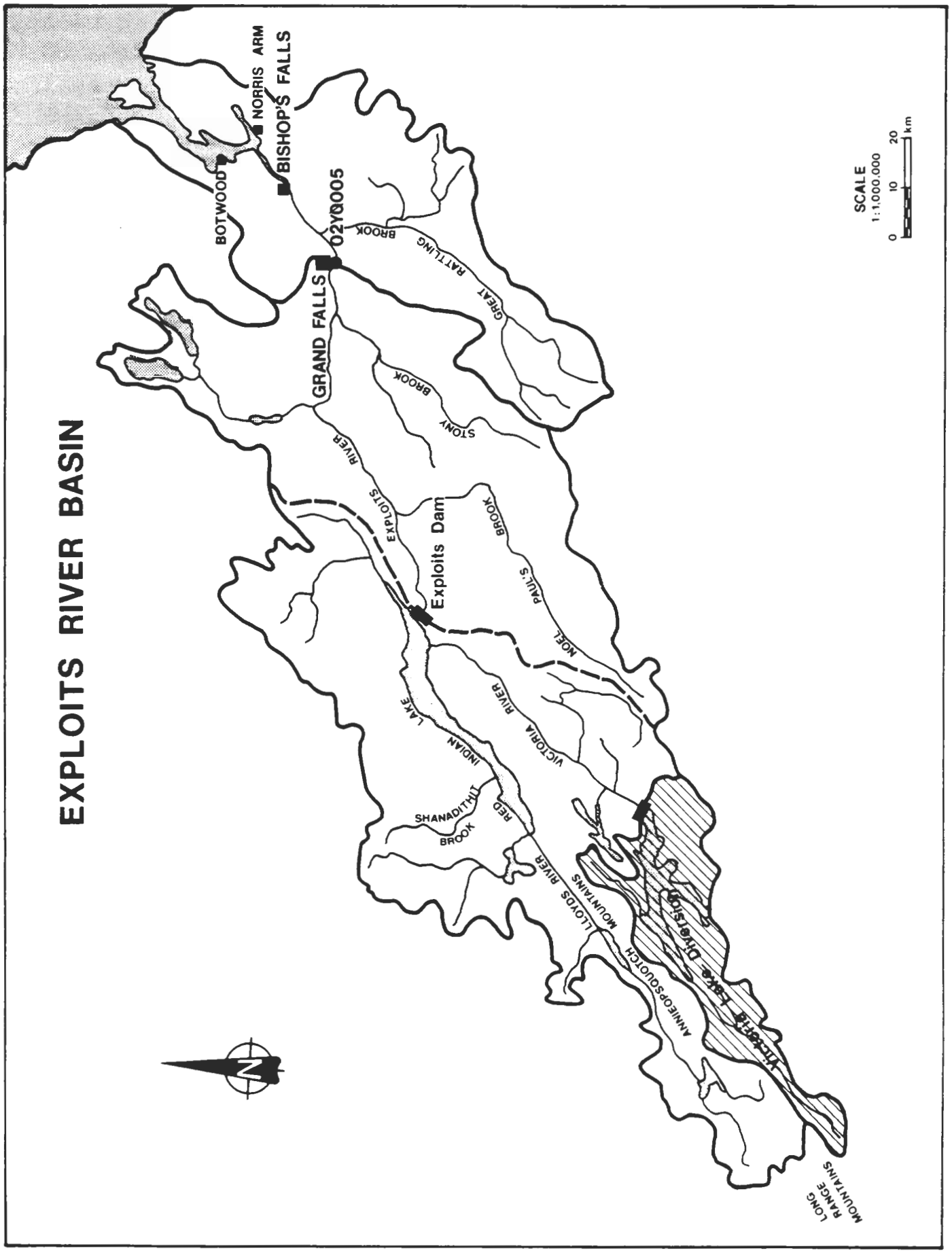
Figure 2-1 outlines the drainage area of the Exploits River and the location of its main tributaries upstream of Bishop's Falls. Figure 2-2 illustrates the basin and these tributaries in more detail, and shows the location of Exploits Dam on Red Indian Lake and a portion of the watershed which has been diverted for hydroelectric developments to the southeast. Most important to this study is Figure 2-3, which illustrates the extent of land area, buildings, earth dam and homes which were washed away or destroyed during the flooding disaster of January 1983.

In view of the potential for loss of life and damages resulting from floods, the Province of Newfoundland and the Government of Canada entered into a "General Agreement Respecting Flood Damage Reduction" on May 22, 1981. The objective of this Agreement is to reduce the potential flood damages on flood plains along the shores of lakes, rivers and the sea. This Agreement also recognizes that the potential for flood damages can be reduced by control of the uses made of flood hazard areas. This involves the identification and delineation of flood prone areas and ultimately the designation of these areas wherein only certain conforming developments may take place. As part of this initiative, a flood risk mapping program is being undertaken in Newfoundland. The mapping of a flood risk area consists of four main components: hydrology, hydraulics, topographic mapping and public information. The main purpose of this investigation is to provide the hydrologic and hydraulic components for the identification of flood prone land at Bishop's Falls.



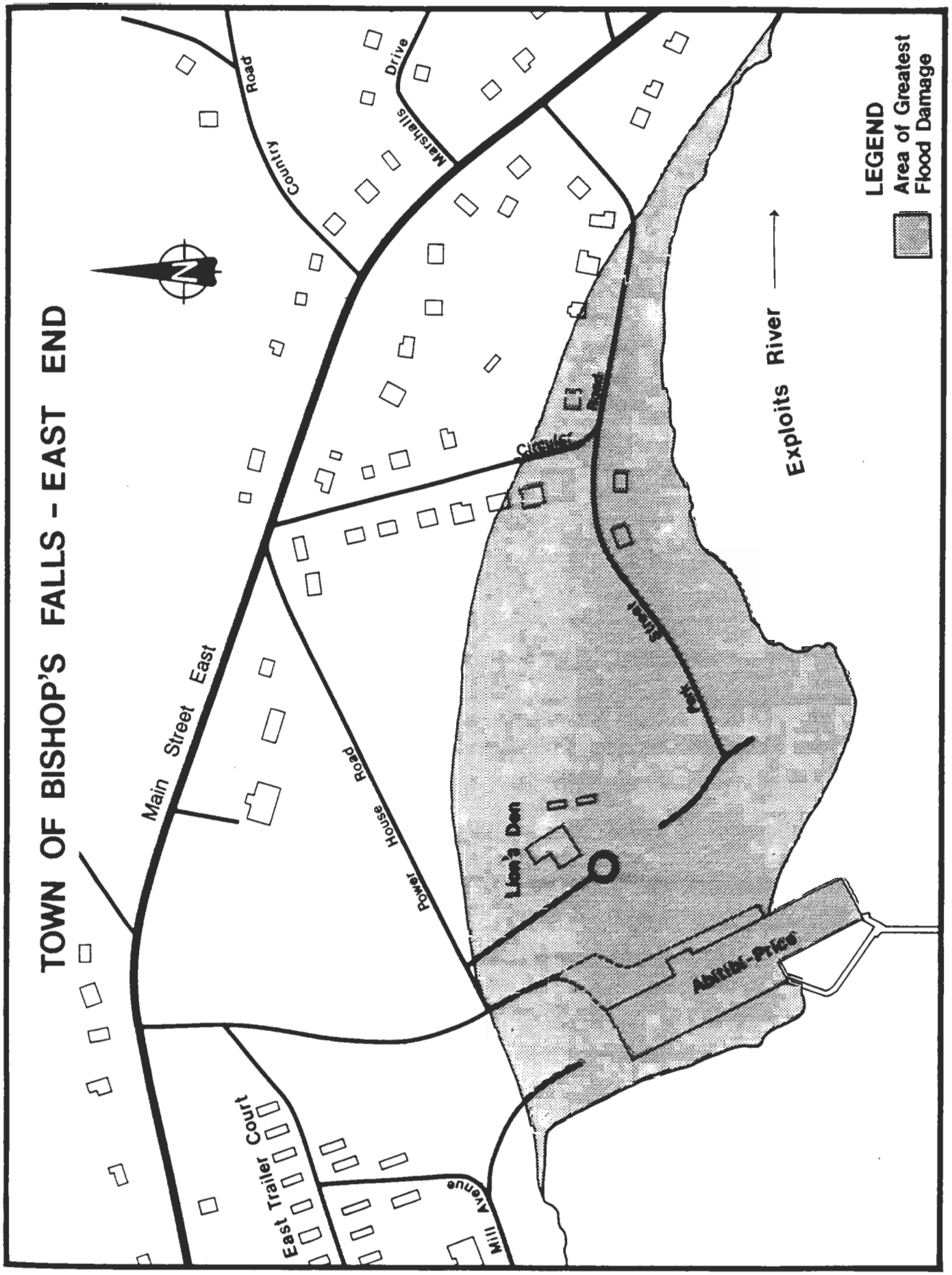
CANADA—NEWFOUNDLAND
FLOOD DAMAGE REDUCTION PROGRAM

BISHOP'S FALLS STUDY AREA



Exploits River Basin

FIGURE 2-2



Town of Bishop's Falls-East End

FIGURE 2-3

2.1 Study Objectives

The objectives of the study are succinctly summarized in the Terms of Reference prepared by the Canada-Newfoundland Flood Damage Reduction Program:

> "The basic purpose of this study is to assess the potential for ice related problems in the Bishop's Falls area, as shown in Figure 2-4, and to provide flood levels and to place these levels on available mapping."

This objective can be subdivided into the following tasks:

(1) To identify and assess the potential for ice-related problems at Bishop's Falls

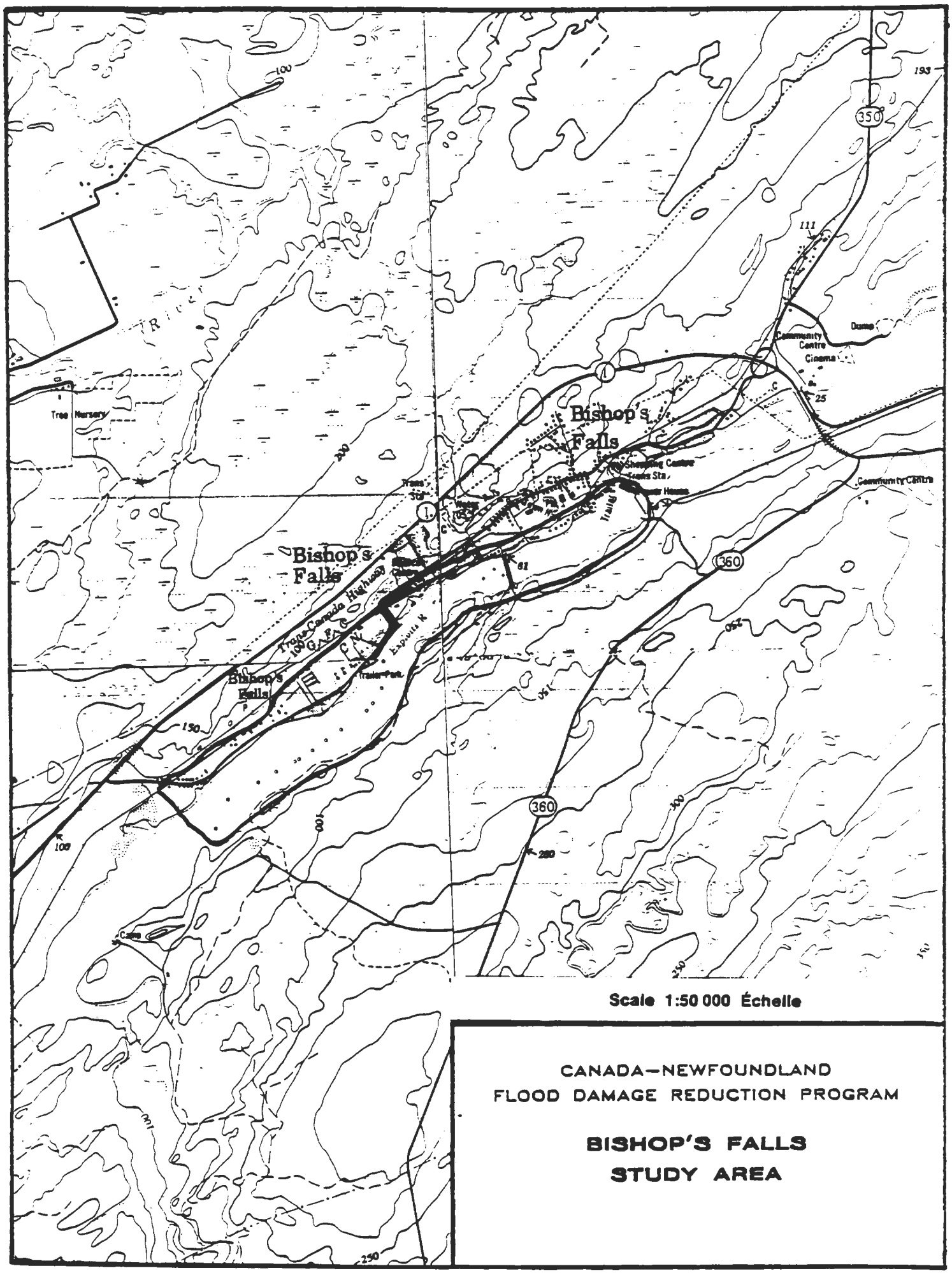
(2) To provide estimates of the 1:20 and 1:100 year recurrence interval flood levels, and the flood levels associated with the 1983 historical flood.

(3) To determine the extent of flooding associated with these ice-related or high flow conditions and place these levels on large scale topographic maps (1:2,500 scale with 1 m contours) provided by the Technical Committee of the Flood Damage Reduction Program.

(4) To suggest, for possible future studies and action, suitable remedial and preventive measure to alleviate potential flood damage problems in the future.

The study consisted of: a thorough review of existing information, the selection of appropriate mathematical models to simulate river flow and hydraulics, detailed evaluation of existing data, the identification of data voids, the collection and compilation of new data necessary for the study, the analysis and interpretation of the data, and derivation of the 1:20 and 1:100 year recurrence interval and historical flood profiles.

This volume (Volume 1) presents all the major finding of the study and examples of mapping



CANADA—NEWFOUNDLAND
FLOOD DAMAGE REDUCTION PROGRAM

BISHOP'S FALLS
STUDY AREA

FIGURE 2-4

of the flood prone areas in Bishop's Falls. Volume 2, which is a compendium of technical notes, drawings, field data and computer programs, is available to the interested reader from the Canada-Newfoundland Flood Damage Reduction Program office at Newfoundland Department of Environment and Lands, St. John's.

3.0 HISTORICAL OVERVIEW

3.1 General

The Terms of Reference prepared for this study note that:

> "The 1983 flooding in the Bishop's Falls was ironic since it resulted in the worst flood damages ever reported on the Island and occurred in an area which did not have a history of flooding."

Prior to the disastrous flood in January 1983, there was a period of about 80 - 90 years during which there were no reports of damaging floods. Added to this are the recent 6 years since the flood when high flows have again been comparatively low. This length of dependable, non-problematic flows, punctuated by one major flood, point to the unpredictability of most flooding situations and the need for preparedness for flood damage reduction along each and every river system.

The history of river development at Bishop's Falls appears to be unwritten and can only be traced through an incomplete series of drawings and sketches prepared for the A.E. Reed Company in the early 1900's. These drawings show the original Ambersen Hollow Dam, constructed in the period between 1905 - 1910, and the layout of the facility in 1922 (Figure 3-1).

Drawings in the archives of the current owners, Abitibi-Price Inc., also indicate that the Ambersen Dam was located along the same alignment as a set of earlier piers - presumably of a bridge or another mill dam which spanned the Exploits River before further developments by A.E. Reed Company. Unfortunately, no records have survived the passage of time to describe the form or nature of these earlier structures or observed streamflows before 1900.

It is known that flow records were kept at Bishop's Falls and Grand Falls after the development, and those from January 1913 to 1950 were supplied by Anglo-Newfoundland

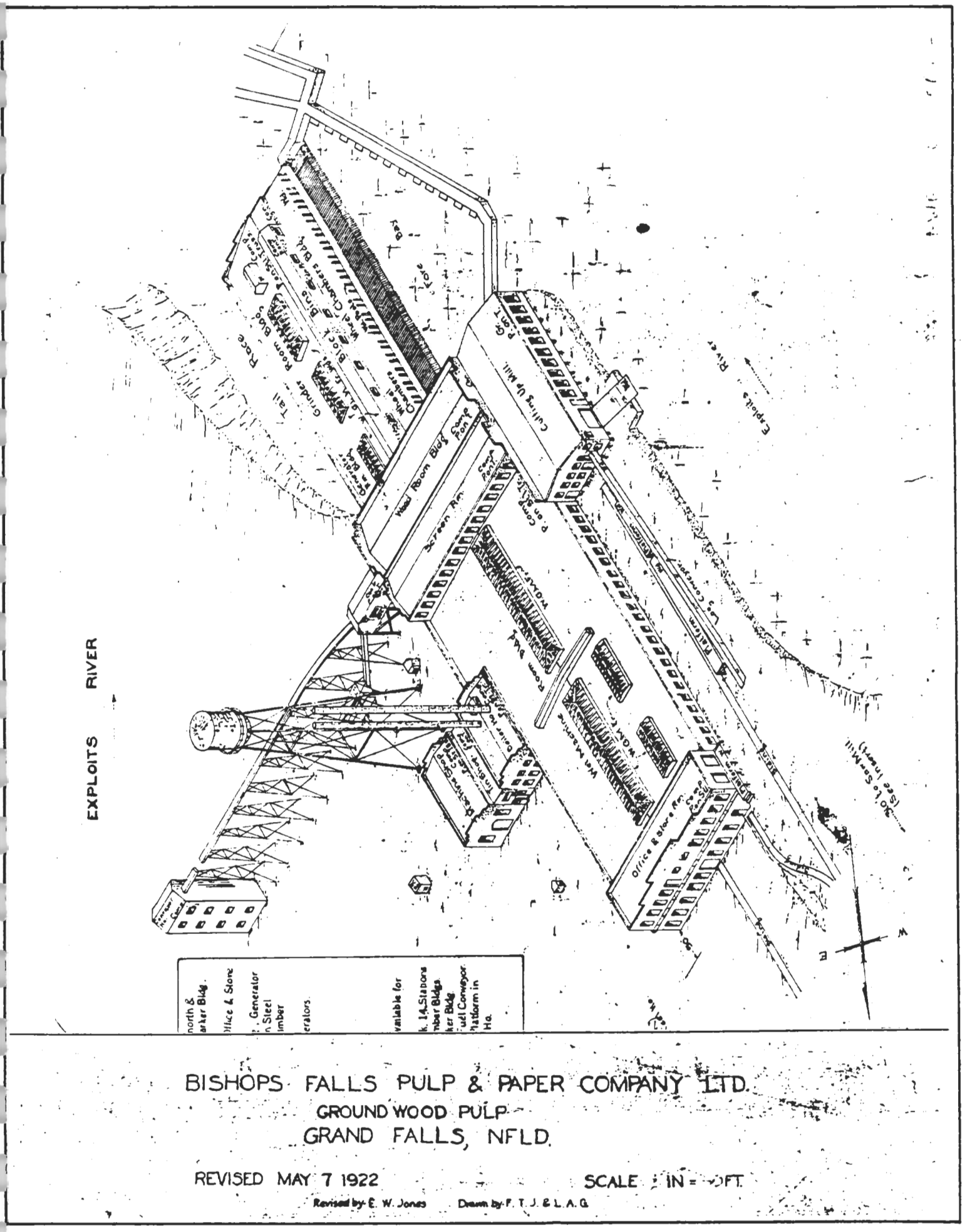

EXPLOITS RIVER
Head Room Bldg
Screen Rm.
Cutting Up Mill
Grinder Room Bldg
Tail Race
Wood Room Bldg
Office & Store Rm.
Wet Machine
Dry Machine
Boiler Room
Exploits River
BISHOPS FALLS PULP & PAPER COMPANY LTD.
GROUND WOOD PULP
GRAND FALLS, NFLD.
REVISED MAY 7 1922
SCALE
Revised by E. W. Jones
Drawn by F. T. J. & L. A. G.

FIGURE 3-1

Development Company for publication in 1952 (Canada Department of Resources and Development). Since 1932, flow records have been archived and those since 1944 are published by the Water Survey of Canada (Environment Canada, 1987).

Review of these records identifies the unprecedented nature of the January 1983 flood - a flood which took place when the ice cover on the Exploits River extended about 39 km upstream of Bishop's Falls (Fenco Newfoundland, 1985).

3.2 The 1983 Flood

There is considerable pertinent information already available in related studies which describe the events and aftermath of the 1983 flood. These were carefully reviewed to obtain an understanding of this flood and the contribution to problems relating to ice action. The principal data sources include:

1. "The Flood of January 1983 in Central Newfoundland" (Environment Canada - Newfoundland Department of Environment, 1985)

2. "A Note on the Flooding of Bishop's Falls" (Newfoundland Department of Environment, Water Resources Division, February 1983)

3. "Hydrotechnical Study of the Badger and Rushy Pond Areas" (Fenco Newfoundland for the Canada-Newfoundland Flood Damage Reduction Program, 1985)

4. Video tape from original 8 mm film taken by Mr. Snow of Bishop's Falls

5. CBC News footage taken at Bishop's Falls (plus other footage of the flood)

6. An Inland Waters Directorate video titled "A Friendly River?"

7. Photographs by local residents, and by commercial and newspaper photographers

TABLE 3-1

THE 1983 FLOOD SEQUENCE

BISHOP'S FALLS

DATE AND TIME (APPROXIMATE)	COMMENT ON FLOOD PROGRESSION, EXTENT AND DAMAGES
January 13 (Thurs)	
8:00 pm	- water levels rose about 0.6 m during the day[6] and flooding was reported on Riverside Avenue and west end Trailor Court[3]. Considerable ice flow observed in main channel
8:30 pm	- Abitibi-Price began to shut down power production at plant[6] in response to high water conditions. Pans of ice, 20-36 cm (8-14 in) thick, and some 3-3.7 m (10-12 ft) in diameter were observed hitting the forebay gates[6]. Most ice pieces were broken into smaller pieces prior to reaching the plant
	- evacuation of riverside residents was underway[7]
9:00 pm	- water level at west end Trailor Court was measured to be about 14 m and rising[5], and ice was moving in the Mill Pond[1] in the main stream of the river
	- TransCanada Highway closed because of flooding[3]
9:15 pm	- Generating Station taken off line because of backwater flooding in pits at the generating station[9]
11:00 pm	- water level was measured at about 15-15.5 m at west end Trailor Court - a rise of 1-1.5 m in two hours[5]
12:00 pm	- water levels reported to be "bank full"[1], and water and ice beginning to flow over the top of the forebay structure - but not over the earth dam[6] (about 3 m - 10 ft of water was flowing over the Ambersen, concrete dam and through the spillways). Flow still over Ambursen dam alone

DATE AND TIME (APPROXIMATE)	COMMENT ON FLOOD PROGRESSION, EXTENT AND DAMAGES
January 14 (Friday)	
1:30 am	- earth dam was still intact, with no spill or diversion around it[6]
3:00 am	- size and quantity of ice, and speed of river flow a growing concern[3]
2:00-6:00 am	- overflow or bypass of earth dam began[6]
4:00 am	- maximum instantaneous discharge recorded below Stony Brook (2400 m^3/s)[7]
8:00 am	- spill flow photographed at gate to powerhouse and over powerhouse roof[6]. Considerable debris and ice was reported to be pounding Sir Robert Bond Bridge[1], but little ice is shown in early photographs of spill flow[6,8]
	- considerable broken ice was observed below the dam (pieces about 0.6 m diameter at most[2]) in the backwater of the tail race (not in operation)
	- spill flow over roof of powerhouse was underway[2] and an estimated 3.7 m (12 ft) was passing over the dam[6]
~10:00 am	- water levels up to bottom chord of Sir Robert Bond Bridge[2,3] and bridge "pounded" by chunks of ice and pulpwood[3]
11:00 am	- water levels reach their peak at west end Trailor Park,[5] leaving 0.3 to 0.4 m(12-14 inch) thick pans behind. Peak level was measured at about 16 m, a rise of 0.5 to 1.0 m over preceding 12 hours
	- water was over Mill Road and scores of families were forced to evacuate[3]
by 2:00 pm	- new channel scour began as a result of spill, which was initially confined to a 40 m band between old groundwood building and transformer stations[2]
	- almost no ice observed in river at this time[4]

DATE AND TIME (APPROXIMATE)	COMMENT ON FLOOD PROGRESSION, EXTENT AND DAMAGES
~3:00 pm	- three transformer stations, poles and lines were lost to the flood[3], and erosion became rapidly destructive
after 3:00 pm	- earth dam and core were eroded from the downstream side and the facade of the office/store room of the old mill building collapsed[3]
	- appears that spill over powerhouse roof ceased[4]
	- washroom facility at the park was swept away[3] and erosion continued as the spill channel eroded a path to the northeast through the park and toward Circular Road
~3:30 pm	- two, 60 ton historical railway cars swept away
~3:40 pm	- Lions Club lost to the flood waters, followed by the Senior Citizens Club[3]
5:20 pm	- first dwelling was lost (Balsom's bungalow)[1]
5:30 pm	- Town Council declared Bishop's Falls a disaster area[1]
~6:00 pm	- home of Jack Butler was taken by the flood[3], although the flood flow was receding[1]
~8:00-9:00 pm	- home of Carl Budgell was lost[3]
	- in all, three houses lost on Circular Road[3] and two remain teetering on the eroded bank. Five others were left threatened by unstable banks (four on Circular Road and one on Main St.)

DATE AND TIME (APPROXIMATE)	COMMENT ON FLOOD PROGRESSION, EXTENT AND DAMAGES
January 15 (Sat)	
am	- Sir Robert Bond Bridge was re-opened as the water levels were much reduced[1]
	- 120-180 evacuated families began to return[1]
3:00 pm	- the emergency situation was lifted, but militia remained on hand until the following morning
Subsequent Period	- involved relocating families who lost their homes, moving several houses, providing new water and sewer systems, bank stabilization, and general clean up and reconstruction

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References

1 "A Friendly River?", video prepared by the Environment Canada, Inland Waters Directorate

2 8mm Super 8 movie of flooding by Bishop's Falls resident, Mr. Snow

3 reports of various newspapers, including the Grand Falls Advertiser, and St. John's Evening Telegram, and Daily News

4 CBC film footage of the flood event

5 observations of west end Trailor Court resident, Mr. Gill

6 observations of power plant operator, Mr. Goobie

7 Canada-Newfoundland report on "The Flood of January 1983 in Central Newfoundland"

8 photographic records of residents and Sweeney's Studio, Grand Falls

9 Abitibi-Price Inc. hydropower log

8. Water Survey of Canada records of streamflow

9. Abitibi-Price Inc. records of water levels and flows

10. Data on levels and the sequence of flood events obtained from site interviews and surveys

As background, perhaps the most succinct summary of conditions leading to the 1983 event are contained in information provided in the Terms of Reference for this study (1987). The following subsection is taken from the Terms of Reference.

3.2.1 Antecedent Conditions and the Storm of January 11 - 14, 1983

During the fall and early winter of 1982-83, total precipitation was about normal on the Island, with the snowfall portion of the total precipitation being quite variable. Although detailed information is lacking in Central Newfoundland, evidence suggests that at least 30 cm of snow has accumulated in the central and east-central areas prior to the onset of the storm on January 11.

Snowfall recorded from October 1982 to January 1983 at selected meteorological stations on the Island indicate that snowfall was mostly below normal, but a rectangular area through Central Newfoundland reported amounts some 10% to 30% above average.

The daily maximum and minimum temperatures at selected meteorological stations show that the highest temperatures for the month occurred between January 10 and 15, and were centered on the 12th and 13th. For the entire Island, the greatest daily maxima ranged between 5°C and 15°C during that period.

On January 11, there was a complex low pressure system over the Great Lakes with an occluding frontal system near Massachusetts. The frontal system moved to the vicinity of

Sable Island by the morning of the 12th. Rain associated with the front covered most of Newfoundland by 1200 GMT on the 12th. Another low south of Cape Cod on the morning of the 12th moved northeastward, and by Thursday morning, January 13, was located just east of Halifax. This new system had the effect of prolonging the precipitation which began on the 11th and 12th. By 1200 GMT on January 14, the system was located northeast of the Island and only eastern regions reported significant precipitation. Throughout the period of the storm, a mild moist airflow prevailed over the Island and temperatures remained well above freezing, night and day.

By mid morning on the 12th, 20 to 40 mm of rain had fallen on an area extending from southwestern Newfoundland to the Hermitage Peninsula. Rainfall increased on the 12th and 13th as the storm became more organized and moved across the Island.

On the 12th, rainfall totals for the day mounted to 116 mm at St. Albans, 113 mm at Bay D'Espoir, 125 mm at Pools Cove and 77 mm at Upper Salmon. On January 13, rain continued unabated and even heavier at some locations. Totals for the day at St. Albans amounted to 141 mm, at Bay D'Espoir 126 mm, at Pools Cove 73 mm and at Upper Salmon 142 mm. Lesser amounts were recorded elsewhere. By mid morning on January 14, except for eastern Newfoundland, precipitation had ended.

3.2.3 Sequence of Events

Site interviews and surveys conducted in July 1988, combined with detailed review of the data sources listed in Section 3.2 give almost an hour-by-hour description of the flood progression on January 13 and 14, 1983. Table 3-1 begins the sequence at 8:00 p.m. January 13 and concludes with the lifting of emergency conditions at 3:00 p.m. Saturday January 15, 1983.

Particularly noteworthy in Table 3-1 are the following points of hydrotechnical interest:

- water levels rose quickly with break-up ice involved in the initial flood flow on January 13

- ice was evident in the powerhouse forebay at midnight January 13, when it was heard to be hitting the stoplog gates

- considerable broken ice was observed, below the dam in the morning of January 14, but little was in evidence above the dam at 8:00 a.m. Ice appeared to have cleared the river below the dam by about 2:00 p.m.

- the flood flow peaked at about 11:00 a.m. January 14, about five hours before the earth dam failed and the erosion/loss of downstream buildings began in earnest

- the peak flow occurred at a time when there was discharge over the Ambersen Dam (spillway), though both sluice gates, over the powerhouse roof and over the un-eroded earth dam

Most important is the detailed summary provided in Table 3-1. It gives many valuable clues for evaluating ice and high flow conditions during the 1983 flood, and the potential for future ice and high flow problems - topics which are discussed in the following sections.

TABLE 4-1
STREAMFLOW DATA

LOCATION COMMENTS	DRAINAGE AREA (km^2)	TYPE OF DATA & RECORD		COLLECTING PERIOD	AGENCIES
Exploits River at Bishop's Falls*	10136	Water Level and Gate Settings (Regulated)	1934-date	Abitibi-Price Inc.	unpublished records of forebay records and gate settings
Exploits River at Exploits Dam*	4754	Water Level Gate Settings (Regulated)	1927-date	Abitibi	abstracted from records maintained by Abitibi and converted to flow
Exploits River at Grand Falls	8390	Water Level Gate Settings (Regulated)	1914-1933	Abitibi	approximate records
		Flow (Regulated)	1934-1961	Abitibi	accurate records
		Flow (Regulated)	1962-date	Abitibi	published by W.S.C.
Exploits River below Stony Brook*	8570	Flow (Regulated)	1969-date	Water Survey of Canada (W.S.C.)	
Sandy Brook at Powerhouse	508	Flow (Regulated)	1965-date	N.L.P.C.	published by W.S.C.
Rattling Brook at Powerhouse	378	Flow (Regulated)	1963-date	N.L.P.C.	published by W.S.C.
Hinds Brook near Grand Lake	529	Flow (Natural)	1957-1979	W.S.C.	
Lewaseechjeech Brook at Little Grand Lake	343	Flow (Natural)	1953-date	W.S.C.	

LOCATION COMMENTS	DRAINAGE AREA (km^2)	TYPE OF DATA & RECORD	COLLECTING PERIOD	AGENCIES
Sheffield River at Sheffield Lake	362	Flow (Natural)	1956-1966	W.S.C.
Sheffield River at Trans Canada Highway	391	Flow (Natural)	1973-date	W.S.C.
Lloyds River below King George Lake	469	Flow (Natural)	1979-date	W.S.C.
Peters River near Botwood	177	Flow (Natural)	1981-date	W.S.C.
Shoal Arm Brook near Badger Bay	64	Flow (Natural)	1982-date	W.S.C.

* This drainage area excludes the portion of the watershed (1056 km^2) upstream of the Victoria Lake Diversion. Areas revised through measurements in 1987-88 and confirmed by Water Survey of Canada, St. John's.

TABLE 4-2

EXPLOITS RIVER
OBSERVED AND INSTANTANEOUS FLOWS

	STONY BROOK MORNING FLOW	STONY BROOK MAX. INSTANTANEOUS FLOW	RATIO INST./MORNING
	(m^3/s)**	(m^3/s)	
1969	1420	1430	1.007
70	-	1010	-
71	1160	1180	1.017
72	659	787	1.194
73	785	793	1.010
74	619	625	1.010
75	802	818	1.020
76	961	963	1.002
1979	652	660	1.012
80	423	600	1.371
81	1250	1260	1.008
82	1100	1130	1.027
83	-	-	-
84	1080	1090	1.009
85	1560	1570	1.006
87 *	781	807	1.033
88	1560	1570	1.006
		Average ***	1.026

* Grand Falls data

** observed flow at 0300 hours at Stony Brook, taken to be equivalent in time to a 0700 hour reading at Bishop's Falls

*** average excludes 1980 anomolous ratio (very low flow peak)

_ missing or uncertain data

TABLE 4-3

BISHOP'S FALLS

REGIONAL FLOOD FLOW ESTIMATES

Northern Region

		log	20 year Coeff*	100 year Coeff*	20 year	100 year
Area	1566 km^2	3.1948	0.9458	0.9076	3.0216	2.8996
MAR**	750 mm	2.8751	1.5655	1.7432	4.5009	5.0118
Latitude	48.75°	1.6879	14.2157	14.6735	23.9957	24.7684
Total				31.5182	32.6798	
K Value*				-29.1469	-30.2744	
Difference (logarithm of flood flow)				2.3714	2.4054	
Flood Flow (Q)				235 m^3/s	254 m^3/s	

* parameters of regression equations

** MAR - mean annual runoff (mm)

Regional Regression Equation (Northern Region)

$$\log_{10}Q = \text{K Value} + \text{Coeff} (\log_{10} \text{Area}) + \text{Coeff} (\log_{10} \text{MAR}) + \text{Coeff} (\log_{10} \text{Latitude})$$

4.0 HYDROLOGY STUDIES

4.1 General

The Terms of Reference for this ice analysis and flood risk mapping study note that one of the prime purposes of the work is to produce reliable estimates of the 1:20 year, 1:100 year and the historical flood profiles. These flood profiles are generated by streamflow, and the following sections discuss the derivation of these flows which are later used to determine water level profiles (Section 7.0).

In summary, the objectives of the hydrologic analysis were as follows:

i) to determine 1:20 year and 1:100 year open water flood flows for the purpose of backwater modelling

ii) to provide flood flow estimates for use in evaluating the effect of ice on flood levels

iii) to determine historical flood flow data (January 1983) for use in assessing the historical flood profile

Flood flow estimates to satisfy the first objective can be derived by various techniques including:

- single station frequency analysis of peak flow records at the site of interest (or from a nearby site on the same stream by transposition)

- development and routing of dimensionless natural hydrographs through the river reservoir system, combining them with local inflows along the way to give a flood flow hydrograph at the site of interest

- regional frequency analysis of flow records within the general area of interest

- computer simulation using a mathematical model of the watershed and long-term weather records

The choice of method in a particular situation is governed by the availability and length of streamflow record at or near the point of interest, as well as by the "stationarity" of the watershed (e.g. the nature of changes in land use, reservoir development, diversions).

Given that there is a considerable length and number of flow monitoring sites within and surrounding the Exploits River System (Table 4-1), it was considered appropriate to focus on the first technique but employ all of the first three techniques for obtaining flood flow estimates. Although there have been changes in the stationarity of the watershed (brought about by flow diversion), it will be shown below that these changes can be assessed without requiring the additional development of a mathematical model of the watershed.

4.2 Streamflow Data Reduction

Of the data sources listed in the following Table, the most important records for estimating flood flows at Bishop's Falls are those at the dams at Bishop's Falls, Grand Falls and Millertown (Exploits Dam), and at the Water Survey Station at Stony Brook.

The hydrometric data from the dam sites were not in ready-to-use form for hydrologic analysis and data preparation consisted of:

a) abstraction of data from log books, hydropower log sheets, forebay operation sheets and gate setting records

b) conversion of levels and gate setting records into inflows and outflows from each dam

Additional information on this process at Exploits Dam and Grand Falls Dam is provided in the "Hydrotechnical Study of the Badger and Rushy Pond Areas" (Fenco Newfoundland Limited,

1985). At Bishop's Falls, discharge data was computed by local operators from November 1933 to 1951. Like Grand Falls, the calculation was based on a forebay reading, tabulated rating curves, and computed flow through the plant. This discharge calculation was discontinued in 1951, but continuous records of head water and tail water elevations and power production have been kept since then (with current records including a notation about gate openings at the new dam). In all, it was necessary to calculate discharge at this dam for the most recent 36 years to obtain information on inflows for the full period of record.

4.3 Record Analysis

As noted in Section 4.1, it was considered appropriate to conduct three analyses to develop and confirm flood flow estimates for Bishop's Falls. These are a single station frequency analysis, a hydrograph analysis, and a regional analysis - all of which are discussed below.

4.3.1 Single Station Frequency Analysis - Bishop's Falls

4.3.1.1 Data Development

The advantage of single station frequency analysis over other techniques is that the flow data for the analysis is specific for the monitoring point. If that point is at the study area (which is the case with monitored data at Bishop's Falls) the results of frequency analysis can be employed with considerable confidence because there is no uncertainty involved in transferring data from another site.

As noted above, Abitibi-Price Inc. has records of discharge at Bishop's Falls from 1933 to 1951 (19 years) and records of forebay water levels, power production and gate settings from 1952 to date (another 36 years). The latter were used to compute discharge for the years from 1952 to 1983, giving 51 years of peak flow records at Bishop's Falls. These data confirm earlier estimates (e.g. Newfoundland Department of Environment, 1983) that the peak flood flow in 1983 was 3256 m^3/s (115,000 cfs).

Prior to employing these data in a frequency analysis for determining the 1:20 year and 1:100 year flood flows, statistical tests were undertaken to determine the effect, if any, of the upstream diversion at Victoria Lake. This diversion was undertaken in the latter part of 1968 and directs flows from 1056 km^2 (about 9.4%) of the full drainage area above Bishop's Falls.

The Mann-Whitney Test was undertaken to assess the effect of this diversion on peak flows at Bishop's Falls. The test (which is appended) compared pre-diversion 1933-1968 flows with 1969-1983 post-diversion flows to determine if there is a difference in these records before and after the diversion.

The first test was performed to determine if there is a difference in the homogeneity of these two sequences of years (the test data is appended). The test statistic, Z, was found to be - 0.041, which determines that there is no statistical difference in the samples. In other words, the data available at this time indicates the Victoria Diversion does not effect flood flow **peaks** at Bishop's Falls.

Despite this result, a second test was undertaken to evaluate the effect of modifying the 1933-1968 record to reflect the reduced drainage area (i.e. 1933-1968 flood flows at 90.6% of observed). This gave a value of the test statistic, Z, of - 0.889, which is significant at the 15% level of significance. In other words, this adjustment does not improve the comparison between the two data sets, and indicates that such a modification would be incorrect.

Although it is certain that the Diversion has reduced the **volume** of flow at Bishop's Falls, these two Mann-Whitney Tests confirm that the Victoria Diversion has had no significant effect on flood **peaks** at Bishop's Falls. (This is also clearly illustrated later in the hydrograph analysis, which shows that it is the local inflows between Red Indian Lake and Grand Falls/Bishop's Falls which largely determine the flood peak.)

The peak inflow from Exploits Dam, which is affected by the Diversion, arrives downstream well after the peak at Bishop's Falls/Grand Falls - too late to have a significant effect on the flood peak.

4.3.1.2 Frequency Analysis

The flow data for 51 years of record at Bishop's Falls were analyzed to give 1:20 and 1:100 year flood flows by using the CFA-88 computer program (Pilon, et al., 1985), and a version of the FDRPFFA program (Condie, et al., 1977).

The CFA-88 program was selected because it provides an up-to-date model to allow the selection of a probability distribution which best fits the monitored data. The FDRPFFA program provides supplementary statistics to augment those in CFA-88, and the results of both analyses and the data are appended.

The Three Parameter Log Normal (3 PLN) distribution was found to be the most appropriate probability distribution of flood flows at Bishop's Falls. The resulting probability distribution of flood flows are shown in the following figure, and are given below:

RETURN PERIOD (YEARS)	MAXIMUM DAILY FLOOD FLOW (m^3/s)
1:20	2200
1:100	2810

For interest, a second run was undertaken to determine the sensitivity of these flood flow estimates to the above conclusion that the Victoria Diversion has had no effect on peak flows at Bishop's Falls. The second run decreased the 1938-1968 flood peaks in proportion to the diverted drainage area, and resulted in a 1:20 year peak of 2087 m^3/s, and a 1:100 year peak of 2767 m^3/s. These changes effect less than a 2% change in the 1:100 year flood flow estimate determined above, and about a 5% change in the 1:20 year flow estimate.

In all, the Mann-Whitney Tests and comparative frequency analyses lend confidence to a conclusion that the maximum daily return period flood flows at Bishop's Falls are 2810 m3/s (1:100 year) and 2200 m3/s (1:20 year).

4.3.1.3 Maximum Instantaneous Flows

A number of hourly water level observations are available for Bishop's Falls from November 1980 to the present. Unfortunately, the records: are intermittent; are all affected by hourly changes in power production (water usage); are only accurate to 0.1 foot (0.03 m); and, those since 1983 are considerably affected by spillway gate operations. In all, the gate and power operations cause water level fluctuations which mask the natural rise and fall of the flood water levels. What is not masked, however, is evidence that the flood peak at Bishop's Falls extends over many hours (like that at Stony Brook).

It is only possible from the available data at Bishop's Falls to assess the instantaneous flow regime for two floods - May 1981 and May 1982. Furthermore, it is only possible to do this in an approximate manner because the accuracy of the level reading (0.1 foot) translates into a discharge variation of ± 50 m^3/s.

The following table gives five lines of data for Bishop's Falls. The first is the maximum daily flow for the year and the second is the maximum instantaneous (peak hour) flow in that day. The third gives the ratio of these values and indicates that the instantaneous peak flow is about 8.6% higher than the peak daily.

The fourth line gives the observed flow (measured at 7:00 a.m.) and the last line gives its ratio to the instantaneous flow that day.

May 6, 1981 Bishop's Falls (flows in m^3/s)		May 2, 1982 Bishop's Falls (flows in m^3/s)	
Maximum Daily	1290	Maximum Daily	1099
Maximum Instantaneous	1402	Maximum Instantaneous	1194
Ratio Inst./Daily	1.087	Ratio Inst./Daily	1.086
Daily Observed	1367	Daily Observed	1194
Ratio Inst./Observed	1.026	Ratio Inst./Observed	1.00

This final ratio is the important one for this analysis because it is only the daily, 7:00 a.m. observation which has been archived in records kept since 1933. In 1981, it turned out that this observation took place slightly after the peak instantaneous flow and was 2.6% lower than the instantaneous value. In 1982, the observation was taken during the peak flow period (which extended over 10 hours) and was consequently the same as the peak flow. Hence, some of the observed flows used in the frequency analysis are likely to be the instantaneous peak flow, and some (perhaps most) will be lower.

The flow records at Stony Brook and Grand Falls were reviewed to determine how often an early morning flow observation would coincide with the peak flow, and to evaluate the ratio of morning flow to peak flow. Table 4-2 summarizes the results. In all, the table shows that a morning flow measurement will infrequently capture the peak flow, but that such measurements are only a few percentage points lower than the peak instantaneous flow (the reliable average ratio for peak flow events is 1.026).

Considering that this ratio is generally representative of high flow conditions at Bishop's Falls, and that the 1981 and 1982 ratios there are within this average, leads to a conclusion that the maximum instantaneous flows at Bishop's Falls average 2.6% higher than the maximum daily flows. The resulting instantaneous flood flow estimates for Bishop's Falls are as follows:

RETURN PERIOD (YEARS)	INSTANTANEOUS FLOOD FLOW (m^3/s)
1:20	2257
1:100	2883

4.3.2 Hydrograph Analysis - Bishop's Falls

As noted earlier, it is also possible to develop and route natural hydrographs through the Exploits River system to determine how they combine with tributary flows and add to give flood flows at Bishop's Falls. There are more uncertainties with this approach (than with the single

station frequency analysis completed above) because it requires some data which must be estimated, and it is somewhat more theoretical. Hence, this second approach is completed herein to provide a set of flood flow estimates to confirm the single station frequency analysis.

The analysis of flood events in this approach began at Exploits Dam and carried downstream to completion at Bishop's Falls Dam. The steps involved:

- assessing the effect of Victoria Diversion on the basin hydrology
- developing natural inflows to Exploits Dam
- routing of flood flows through the dam, considering realistic options for storing and spilling/releasing water from Red Indian Lake
- routing of outflows from Exploits Dam downstream to Grand Falls
- addition of local inflows at Grand Falls to provide a total inflow hydrograph
- routing of flood flows to Bishop's Falls
- addition of local inflows between Grand Falls and Bishop's Falls to give a total inflow hydrograph for a range of return period floods

The results of each step are summarized below. The details of each step in the analysis are provided in the Technical Appendices.

4.3.2.1 Exploits Dam / Millertown

The analysis of natural streamflow data was initiated at Red Indian Lake where water levels, gate settings and rating curves are available from previous studies and records of daily monitoring (Abitibi-Price, Grand Falls). The historical lake level data were initially screened to

identify periods of rapid infilling, and gate settings were reviewed for periods of significant outflow. The two were then combined to determine major inflow periods (in each of the spring, summer, fall and winter periods) for each year since 1948 (40 year record period).

The diversion of streamflow through the Victoria Diversion (since August 1968) was then factored into the analysis to give data to extend the historical inflow record at Exploits Dam. Details of the work are contained in the Technical Appendix and allow for completion of a 40 year period of peak flows into Red Indian Lake (1948-1987).

The majority of peak inflows occur in the spring and involve snowmelt with rainfall. Frequency analysis of these flows (using the three parameter lognormal distribution) gave the following estimates of the 1:20 year and 1:100 year return period inflows:

RED INDIAN LAKE AT EXPLOITS

FREQUENCY ANALYSIS OF SPRING INFLOWS

Return Period	Flood Flow	
(years)	(cfs)	(m^3/s)
1:20	81,577	2310
1:100	63,920	1810

Inflow Hydrographs, Starting Water Levels and Operations

A number of high flow, inflow hydrographs were then analysed to determine the typical way in which flows rise to their peaks and then recede. This made it possible to establish realistic hydrograph shapes and inflow volumes having peak flow values corresponding to these above.

Water level data in Red Indian Lake were also assessed to determine typical conditions before the initiation of spring inflows. This provided information on the quantity of inflowing water which could be stored before outflows would begin.

Last, the settings of the outflow gates were reviewed for a twenty year period to determine

gate operations before and during high flow periods. This analysis established the way that these gates would likely be operated in response to the 1:20 and 1:100 year inflows.

Reservoir Routing - Red Indian Lake

All of the above information - inflow hydrographs, lake levels, gate operations - were combined to determine how the 1:20 and 1:100 year flood flows would be modified (routed) in passage through Red Indian Lake. This analysis demonstrated that there would be a range of outflows associated with each return period flood.

- the 1:20 year outflow peak would range between 256 m^3/s and 548 m^3/s
- the 1:000 year outflow peak would range between 567 m^3/s and 897 m^3/s

4.3.2.2 Grand Falls Dam

The outflow flood flows from Red Indian Lake were routed down the river reach from Exploits Dam to Grand Falls Dam using river geometry data collected earlier (Fenco Newfoundland, 1985). This process also provided valuable information on the peak flows which come from the 3636 km^2 drainage area which lies between Exploits Dam and Grand Falls Dam.

This analysis determined that the local inflows from the 3636 km^2 area are as follows:

- 1:20 year local inflow, 1289 m^3/s
- 1:100 year local inflow, 1693 m^3/s

In all, the sum of the local inflows and those from Red Indian Lake combined to give the following range of inflows at Grand Falls:

- the 1:20 year total inflow peak would range between 1313 m^3/s and 1339 m^3/s
- the 1:100 year total inflow peak would range between 1797 m^3/s and 1812 m^3/s

4.3.2.3 Bishop's Falls

There are negligible reductions in peak flows in passage over Grand Falls Dam because there is little storage in the head pond. Hence, the 1:20 and 1:100 year outflows at Grand Falls are essentially the same as the inflows.

These outflows were transposed downstream to Bishop's Falls using river cross section and flow data collected in this study. This process also provided information on the peak flows which join the river from the 1746 km^2 area between Grand Falls and Bishop's Falls. (e.g. from Stony Brook and other tributaries).

It was determined from frequency analysis (Technical Appendix TA-2) that the local inflows are as follows:

- 1:20 year local inflow peak, 932 m^3/s
- 1:100 year local inflow peak, 1291 m^3/s

The sum of these local flows with the inflow from Grand Falls give the following range of return period inflows at Bishop's Falls:

- the 1:20 year total inflow peak would range between 2171 m^3/s and 2192 m^3/s
- the 1:100 year total inflow peak would range between 2980 m^3/s and 3000 m^3/s

As noted above, the "range" in the flood flow estimates is a result of a range of different ways in which the gates could be operated at Red Indian Lake. This range is relatively narrow and shows that spring flow peaks at Bishop's Falls are relatively insensitive to changes in typical operations at Exploits Dam.

The above results are all based on maximum daily flows which can be calculated with confidence from the daily records, and hourly routing of these flows. Hence, these hourly values are essentially equivalent to instantaneous peak flows. Of immediate interest is the

similarity between these values and those derived perviously from the single station frequency analysis (Section 4.3.1.2).

FREQUENCY ANALYSIS		HYDROGRAPH ANALYSIS	
1:20 year	2257 m^3/s	1:20 year	2171-2192 m^3/s
1:100 year	2883 m^3/s	1:100 year	2980-3000 m^3/s

The hydrograph approach, although somewhat more theoretical, gives much the same results as the single station frequency analysis, and confirms that the frequency analysis results are reasonable.

4.3.3 Regional Analysis - Bishop's Falls

A final check on the reliability of the flood flow estimates from the single station frequency analysis was made through use of the "Regional Flood Frequency Analysis for the Island of Newfoundland" (1984) and the "Users' Guide" for this analysis (1986).

These reports contain a description and background of the development of equations for estimating instantaneous streamflows (e.g. 1:100 year flows) on gauged and ungauged watersheds in the province. The applicability of the equations is widespread, except:

- where there is significant man-made regulation of the flow
- where certain watershed parameters lie outside the range for which the equations were developed (e.g. drainage area must be less than 4400 km^2).

The Exploits River at Bishop's Falls is affected by significant regulation at Red Indian Lake, and has a total drainage area of 10,136 km^2. Consequently, the equations for estimating flood flows are not directly applicable. However, they are applicable to the smaller, unregulated area between the Water Survey of Canada gauge at Stony Brook and Bishop's Falls (1566 km^2).

Hence, to develop flood flow estimates for checking the reliability of the single station frequency analysis at Bishop's Falls;

a) the regional flood frequency equations were employed to develop 1:20 and 1:100 year return period instantaneous flood flow estimates for the area between Stony Brook and Bishop's Falls.

b) a frequency analysis was undertaken on the observed instantaneous flows at Stony Brook to give 1:20 year and 1:100 year values.

c) the two were summed to give a total inflow to Bishop's Falls, assuming conservatively that the local drainage area contributed peak flows on the same day as those from Stony Brook (which appears reasonable from the appended hydrograph analysis of Bishop's Falls inflows).

The frequency analysis of Stony Brook flood flow data (Technical Appendix) gave the following return period flood flow estimates:

Return Period	1:20 years	1:100 years
Discharge	1951 m^3/s	2665 m^3/s

Flood flow estimates for the local area (1566 km^2) between Stony Brook and Bishop's Falls were developed by the regional equations (Table 4-3), which gave the following instantaneous flood flows:

Return Period	1:20 years	1:100 years
Discharge	235 m^3/s	254 m^3/s

Totalling the two gives the following flood flow estimates:

1:20 year	2186 m^3/s
1:100 year	2919 m^3/s

4.4 Hydrology Conclusion

Analyses in this chapter have described three approaches to determining flood flow estimates for Bishop's Falls. The first, a single station frequency analysis of over 50 years of flow data at Bishop's Falls, is considered most reliable for developing the 1:20 year and 1:100 year return period flood flows. The second, a hydrograph analysis, is somewhat theoretical but appropriate for confirming the reliability of the first method. The last approach, using regional flood flow regression analysis, is also a reliable method for (in this case) checking the results of the single station frequency analysis at Bishop's Falls. The results of these three methods are listed below:

FLOOD FLOW ESTIMATES (M^3/S)

Return Period	Frequency Analysis	Hydrograph Approach	Regional Approach
1:20 year	**2257**	2171 - 2192	2186
1:100 year	**2883**	2980 - 3000	2919

The results of all methods give similar instantaneous flood flows. The hydrograph and Regional approaches give values which are slightly less than the more reliable results from the single station frequency analysis at the 1:20 year return period (2257 m^3/s). Alternatively, the 1:100 year frequency analysis result (2883 m^3/s) is slightly less than the estimates of the hydrograph and regional approaches. However, all values are within $\pm 4\%$ of the flood flow estimates derived from the single station frequency analysis at Bishop's Falls.

Given that the two approaches used to check the reliability of the single station frequency analysis give similar results to those from the frequency analysis, it is concluded that the

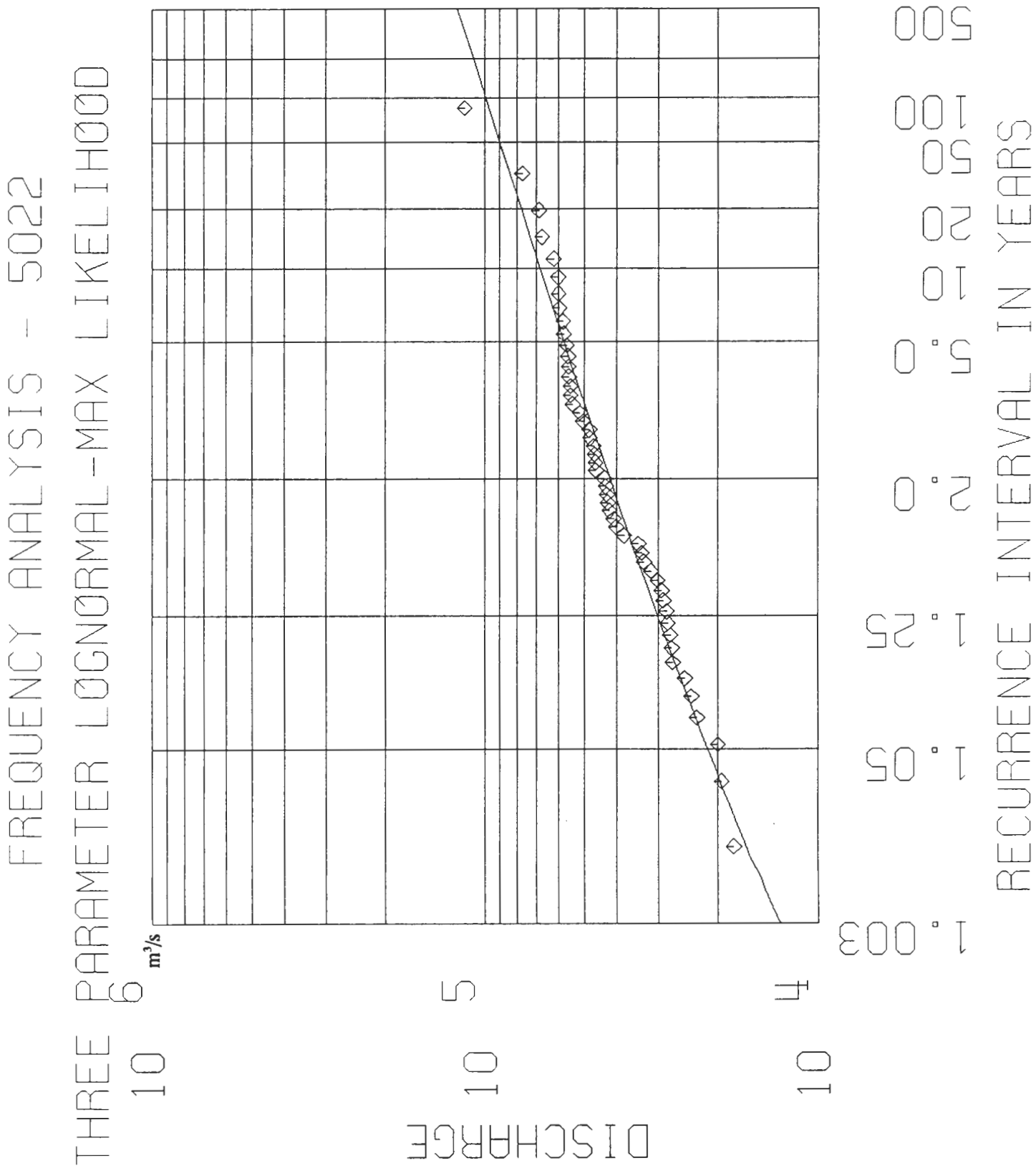


FIGURE 4-1 Frequency Analysis of Peak Daily Flow - Bishop's Falls

frequency analysis results can be used with confidence to derive 1:20 year and 1:100 year flood flow profiles. The instantaneous flood flows for this analysis are:

1:20 year	2257 m^3/s
1:100 year	2883 m^3/s

5.0 HYDRAULIC AND ICE ANALYSIS

5.1 General

It was noted earlier (in Section 3.0) that there was break-up ice in the powerhouse forebay in the night of January 13, 1983 and early the next morning. It is also known from historical records that there have been ice pieces lodged on the dam for short periods (e.g. April 1938), and that moving ice during spring break-up commonly destroys portions of the flashboards on the crest.

In order to gain an appreciation of ice and streamflow hydraulic conditions, a series of measurements and observations were undertaken in the spring and summer of 1988. These are described below and in the Technical Appendix and are followed by hydraulic modelling for ice and open water conditions.

5.2 Field Monitoring - Ice Conditions

The program of ice monitoring involved the following:

(a) field surveys of ice thickness and surface elevations in late March, 1988

(b) a daily photographic reconnaissance of break-up (March 28 - April 8, 1988)

(c) survey of river dynamics in mid-May 1988 and establishment of 1983 flood level locations

(d) survey of river/mill pond depth, cross-sections, and establishment of elevations of 1988 and 1983 ice and water levels

The first survey **(a)** in late-March involved measurements of ice thickness and description/photographs of ice conditions at six locations along the study reach (see Technical

Appendix). It was found that the near shore ice ranged in thickness from 13-73 cm (5 - 29 inches) - the maximum being at the western end of Station Road. The ice had begun to decay before the survey and approximately the top half of the ice cover was quite soft as a result of thermal influences.

The daily record (often twice daily) of break-up ice conditions **(b)**, covered the entire study area with photographs from five locations along the river. These and other photographs are provided in a separate Technical Appendix. The progression of break-up was also recorded in a series of notes, and the observer was in daily contact with our office. Regrettably for this data collection program, there was not a rapid increase in streamflow during break-up and the majority of the ice simply melted in place. As a result, no information could be obtained on the typical size, speed and trajectory of ice pieces as they move toward the dam during normal or premature break-up (e.g. 1983).

However, this program added considerably to our understanding of ice thickness because it chronicled the areas which opened first and last (thin and thick ice, respectively). Beginning at the dam, it appears that the ice cover gradually becomes thicker toward the CNR bridge. Ice on both sides of the bridge is relatively thin, but ice in the central area of the river upstream and downstream around the bridge is thicker. Further upstream at ARMCO Rd/Exploits Lane, the ice is also thick along the north bank, but relatively thin along the south bank. Upstream at the Pro Hardware/West End Trailor Court, it also appears that ice is thicker on the north bank and thinner on the south.

It is tentatively concluded from these observations of ice melt that there is a gradation in ice thickness in the mill pond with the south side being thinner than the north. Ice in the center of the channel at the CNR bridge also appears to be as thick as any north bank area, and might at times delay the downstream passage of ice.

Another interesting feature of the 1988 ice cover is that there were three to four shore parallel crack lines running along the bank over the length of the mill pond. These lines of cracks were crossed by another set of cracks which were spaced at about 100 m intervals and which

ran outward from the bank toward the opposite shore. It is expected that these cracks result from fluctuations in the water level in the pond (as a result of constantly changing inflows, releases and gradual deterioration of the flashboards for power), and that these cracks assist in the unobstructed clearing of ice from the mill pond.

The survey of river dynamics **(c)** and establishment of 1983 flood level locations was conducted over a three day period in mid-May. For the former, surface floats and coloured dye were used to give an indication of flow patterns approaching the dam. Again, however, because river flow was much less than typical flood flows, it was impossible to draw any substantial conclusions from this work.

The identification of 1983 flood levels was considerably more successful. Reviews of 1983 films and photographs prior to the survey, and discussions with residents in Bishop's Falls, enabled quite accurate estimates to be made of the maximum water level at 8 locations. These included the dam, Mill Road, Sydney Street and a progression of other locations up to the West End Trailor Court. At that location, one resident (Mr. Gill) had monitored the rate of water level rise in 1983 and the time and elevation of maximum levels.

The most important piece of information from these observations is that the maximum 1983 flood level occurred at the upstream edge of town about 3-4 hours after most of the ice had cleared the mill pond and was passing downstream and impacting Sir Robert Bond Bridge.

The final survey **(d)** was conducted to tie all of the preceding surveys into a common elevation datum (GSCD) and to take cross sections of the river and overbank areas. Elevations were taken in cross sections at the dam and at 10 other locations in the study area and upstream beyond Great Rattling Brook.

Overall, a considerable amount of valuable information was gathered during the data collection program. Flood levels for 1983 were well defined; ice thickness data was obtained and extended throughout the mill pond by daily observations; cross sections for ice/hydraulic models were taken at the desired locations; and, ice elevations were measured for use in

determining flood levels during typical ice conditions. Regrettably, the low flow conditions (which prevailed in the spring of 1988) and warm weather which combined to melt the ice in place were not conducive to monitoring typical break-up ice conditions.

5.3 Historic Break-up Ice Conditions

Until 1983, there were no reports of open-water flooding from the Exploit's River at Bishop's Falls and no ice-related flooding in the record period from about 1900. As a result, it appears from our data review and interviews that Abitibi-Price Inc. (and predecessor companies who originally constructed the dam) is the only group which has had a consistent, long-term program of assessing the effect of ice and water levels in the mill pond of Bishop's Falls. Their ice program was (and is) essentially one of monitoring conditions in the mill pond in order to maintain low flows and a stable ice cover through the winter period, so that ice does not interfere with flow at the dam. Their current (post 1983) program is much the same as it always has been, except now it is possible to regulate levels by gates at the new dam as well as to regulate inflow from upstream dams.

5.3.1 Typical Ice Conditions at Break-up

Local residents and Abitibi Price personnel described the following ice parameters as being typical of the average break-up:

- ice thickness about 0.6 to 1 m (2-3 feet) but only 0.3 to 0.4 m (12 to 16 inches) thick if break-up is early

- the maximum size of ice pieces is roughly 3 x 3 m (10 x 10 ft) with a few as large as 9 x 9 m (30 x 30 ft)

- ice at the dam is normally eroded/melted/not present at break-up in the area between the dam and about 100 m upstream

- ice pieces occasionally lodge on the crest of the dam but break in half and pass over the dam without significantly blocking the crest

- it typically takes about one day to clear all the ice from the mill pond and pass it downstream. It may take a week or more before all of the small pieces of broken ice from upstream are cleared from the river

5.3.2 1983 Ice Conditions at Break-up

It is interesting to compare and contrast the above conditions with those which are described for 1983:

- ice thickness was observed to be between about 0.3 to 0.5 m thick (12 to 18 inches), or about the same as the average for an early break-up

- the maximum size of ice pieces were about 3 to 3.7 m in diameter (10 x 10 ft to 12 x 12 ft) and most pieces were much smaller. Again, this piece size is about the same as that associated with a typical break-up. It is also noteworthy that large sheets in the range of 15 to 30 m in size (50 - 100 ft) were not reported (or observed) in the flow

- ice pieces were not observed to have lodged on the crest of the dam or to have blocked its discharge. Ice was heard hitting the outer forebay gates (around midnight January 13) but not the buildings or earth dam which collapsed about 16 hours later

- it again appears to have taken about one day to clear most of the ice from the mill pond. During this process, pieces were deposited on the banks and in yards along the way but only a small quantity of broken ice was present in the flow at the eroding earth dam.

In all, it appears that ice conditions in 1983 were similar to typical conditions for an early break-up. The only difference is that pieces of broken ice from upstream were swept clear of

the river system much faster than the norm as a result of high flows.

What is missing from the 1983 observations is a better understanding of ice conditions during the morning hours of January 14 - between the time that ice was hitting the forebay gates and the earth dam began to overtop. These are discussed in the following section.

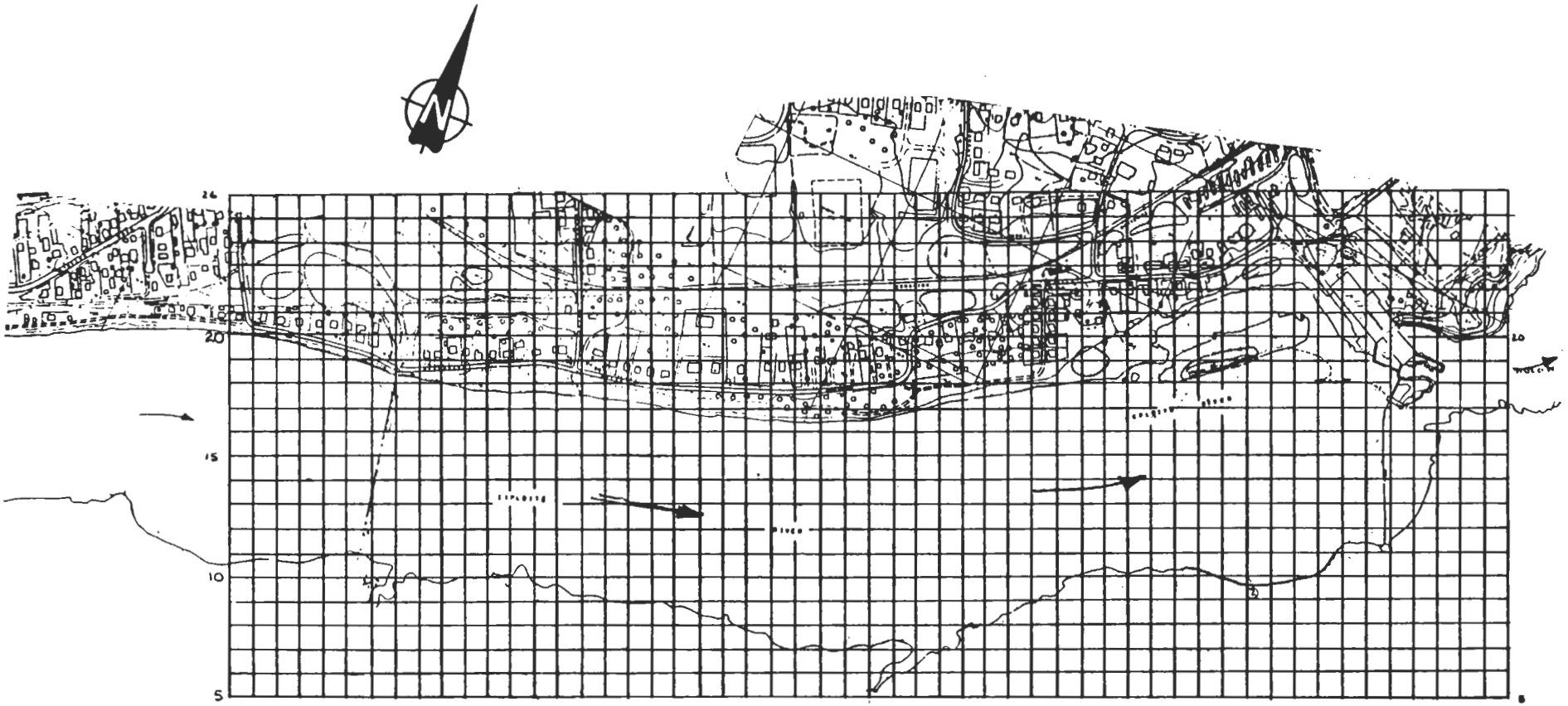
5.4 Hydraulic Modelling - Ice Dynamics

Two numerical models were used to evaluate water level and ice effects at Bishop's Falls. The first one is a sophisticated hydrodynamic model, which was employed to estimate the speed and direction of ice pieces in the headpond during the 1983 flood. The second is a backwater model, designed to give flood levels during open-water conditions.

5.4.1 Hydrodynamic Modelling - Old Dam

There are a wide number of models which can be used to simulate the hydrodynamics (and water quality) of rivers, lakes and coastal areas. One of the best of these for Bishop's Falls application is the vertically-averaged two-dimensional hydraulic model known as the RAND model. The model has a proven track record in diverse applications throughout the world, including Canadian rivers such as the St. Lawrence. It is a finite difference model which uses a square grid system to define the model geometry and it is well documented and supported by the RAND Corporation (Leendertse, 1970).

Figure 5-1 shows the two-dimensional square grid layout of the model at Bishop's Falls. The model simulates current speed and direction, and water level in each box of the square grid. The principal input to the model is streamflow (derived in Chapter 4), geometry of the dam and rating curves (provided in the Technical Appendix) and water depth from our field surveys (Appended). Depths are input into the model at every corner of each grid and the time-varying streamflow enters uniformly along the upstream boundary. The model was run for a 72-hour time span and generated current patterns every five seconds. This data was stored on computer tape so that the results could be plotted.



BISHOP'S FALLS HYDRODYNAMIC MODEL GRID

FIGURE 5-1

Two sets of simulations were conducted. The first evaluated current motion during the 1983 flood, using recorded water levels and the shape of the dam and spillway as it was at that time. The second set of simulations looked at present conditions with the new dam and gates.

Figure 5-2 shows condition at Bishop's Falls at about 4:00 a.m. in the early morning of January 14, 1983. At this time, broken ice was being carried with the flood flow, and a portion of the flow was spilling over the earth dam between the power house and the town.

The figure shows several aspects of interest. It shows the spill over the earth dam, overbank flooding along Riverside Drive, flooding of the island/bay just upstream of the dam, major flow over the Ambersen Dam and spill over the top of the powerhouse. Not shown are water levels, which were 3.7 m above the crest of the Ambersen Dam (crest elevation 11.753 m Geodetic and flood level 15.453 m).

Of principal importance is that the submerged island is shown to have deflected the high velocity currents and ice away from the earth dam and in a course which directed the flow over the Ambersen Dam. Ice and flow velocities over the Ambersen Dam were about 2.2 m/s (7.2 feet/second), which explains why local observers could hear the ice hitting the forebay gates.

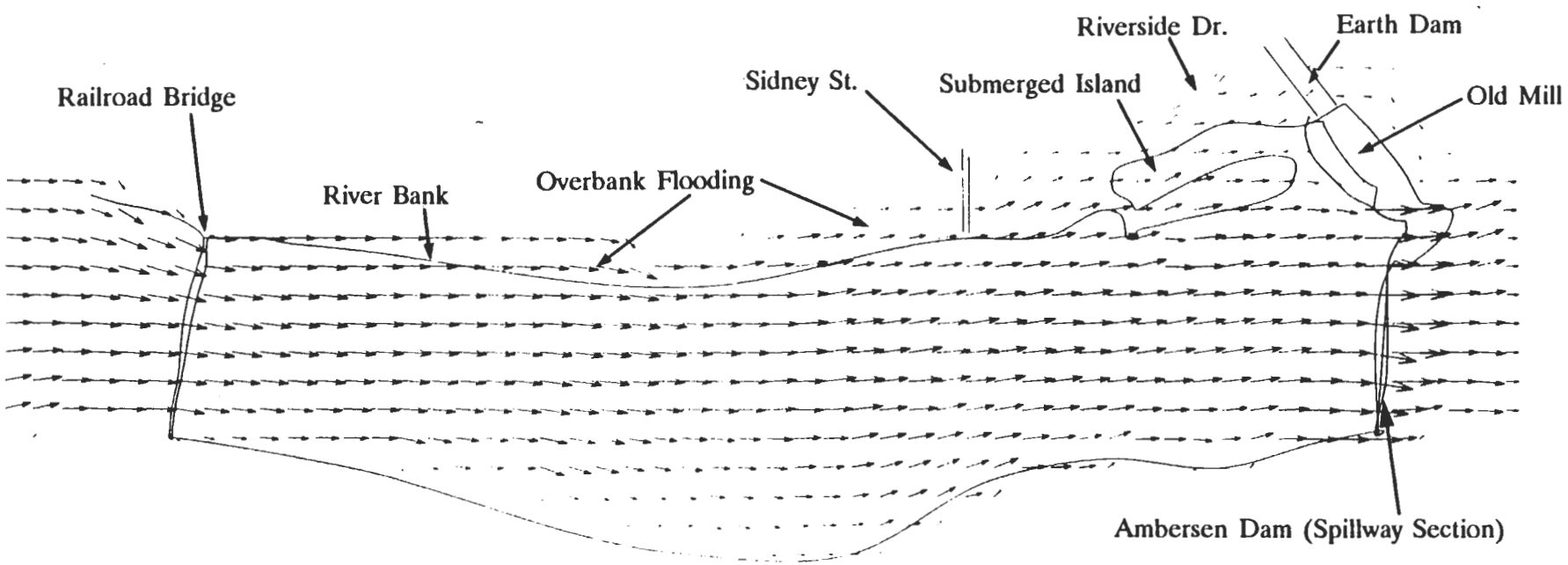
Ice and flow velocities approaching the earth dam (which was ultimately eroded) were very small in comparison to those at the main dam. The protection provided by the submerged island reduced ice and water speeds to about 30-35 cm/s (about 1.0 foot/second). This speed would have been sufficient to carry ice into the yards along riverside drive but was not sufficient to have significantly affected the integrity of the concrete cap on the earth dam - particularly because the largest pieces from the headpond were carried over the dam on the previous day (Jan. 13).

It is quite likely that small ice pieces from the upper river did dislocate some of the soil on the upstream face of the earth dam and did dislodge some granular material in passing over the dam. Overall, however, it was the erosive force of the overflowing water which caused the

BISHOP'S FALLS

January 14 1983 - 4:00 a.m.

BISHOP'S FALLS 2D-HYDRODYNAMIC MODEL FOR ICE 34.6M.GRID



BISHOP'S FALLS HYDRODYNAMIC MODEL

FIGURE 5-2

undermining and ultimate failure of the earth dam and adjacent buildings.

5.4.2 Hydrodynamic Modelling - New Dam

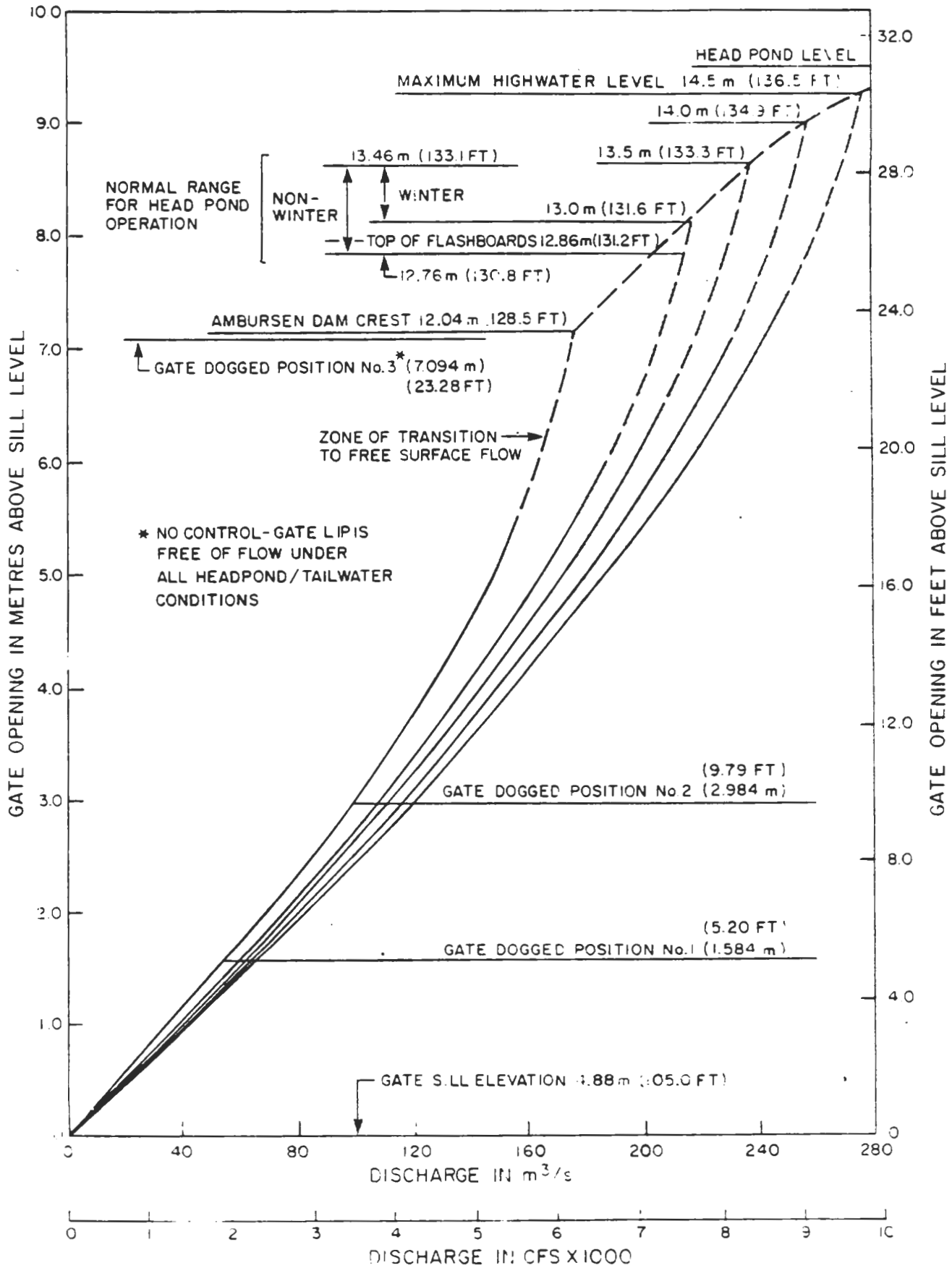
Rehabilitation of the dam at Bishop's Falls was quickly initiated after the 1983 flood and the new specifications are appended. The main elements of the work consisted of construction of ten new spillway gates, located along a length of the old Ambersen Dam, and construction of a replacement embankment in the area between the powerhouse and the north bank.

The greatest change is the addition of the 10, 18 foot wide (5.5 m) spillway gates which supplement the overall spillage capacity over the Ambersen Dam. Figure 5-3 shows the discharge capability for each gate as it increases with gate opening. The figure shows that at the maximum design high water level (14.5 m - 136.5 ft Abitibi datum) each gate will pass about 280 m^3/s (10,000 cfs). All 10 gates will pass 2,800 m^3/s at this level.

Figure 5-3 was taken from the Operations and Maintenance Manual (Acres, 1984) for the new facility. This manual notes that the operation of the gates has been protected by:

- heated gains in the gates to prevent ice build up on the downstream side
- a bubbler system on the upstream side to avoid adhesion of ice to the gates
- a back-up, standby generator, which is frequently tested, to operate the gates in the event of a power failure
- regular testing and maintenance of all gates

The object of these protection procedures and design features is to assure that the gate capacity will be available during flood conditions. Such flood situations can be detected by water level sensors at Bishop's Falls and Grand Falls, which also monitors downstream levels by remote sensors.



NOTE

ELEVATIONS IN METRES - CANADIAN GEODETIC DATUM
ELEVATIONS IN FEET - ABITIBI - PRICE DATUM
CONVERSION: - EL m = (EL. FT ÷ 3.281) - 27.12

FIGURE 5-3

ABITIBI PRICE INC
BISHOP'S FALLS GENERATING STATION

ACRES

PARTIAL GATE DISCHARGE CAPABILITY FOR ONE SPILLWAY BAY

As well as these gates, there is also the Ambersen overflow dam, some 177 m long, which has its crest at 11.75 m (Figure 5-3). This portion of the dam also spills flood flows. Its capacity is also substantial and approaches that of all 10 gates at high flows.

The overall effect of spillway flow and gate openings are shown in Figure 5-4, the spillway rating curves for Bishop's Falls. The curves show the total discharge obtained by flow through the gates and over the Ambersen dam spillway, and the individual contribution made by the spillway and the gates alone. The axis on the right side of the figure (in feet) corresponds to the elevations (in feet) given on the curves shown earlier in Figure 5-3.

The curve is read by determining the discharge along the bottom of the figure, moving vertically to the "total discharge" curves and then moving laterally to the axis at the side giving the reservoir elevation which would be reached to pass this discharge. The example sketched on the figure shows that a flood flow of 3500 m^3/s (more than the 1983 flood) would pass downstream at a flood level less than 135 feet (Abitibi Datum) - or with just about six feet (2 m) of flow on the dam. This level is about nine feet (2.7 m) below the top of the replacement earth dam and about six feet (2 m) below the 1983 flood level.

The hydrodynamic effects of the new dam and spillway gates are shown in Figure 5-5 and Figure 5-6, which show the 1983 and 1:100 year flow cases, respectively. Both show some spill over the Ambersen Dam and a considerable volume passing through the new spillway gates. The figures show the new geometry of the "island" and bay (dashed line) which were re-established after 1983.

Noteworthy in these figures are the high velocities (6-7 m/s) of flow through the gates and the unobstructed passage of flow over the Ambersen spillway crest. The open crest (177 m wide) allows even the largest ice pieces to pass unobstructed over the crest, and the high velocities will cause crushing and breaking of the ice without blockage.

It is also appropriate to note that stability analyses by Geocon (1983) and Shawinigan Lavalin

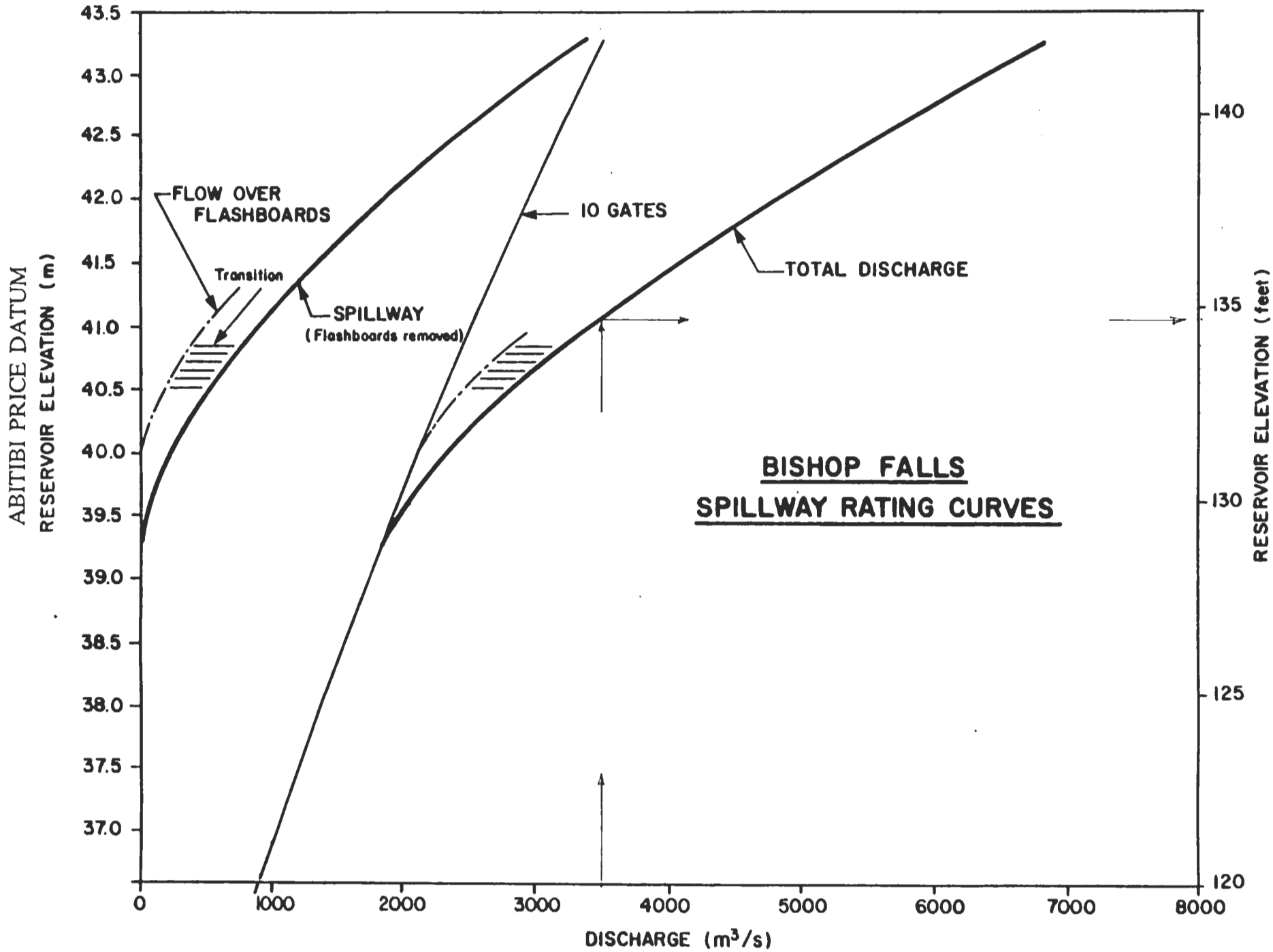
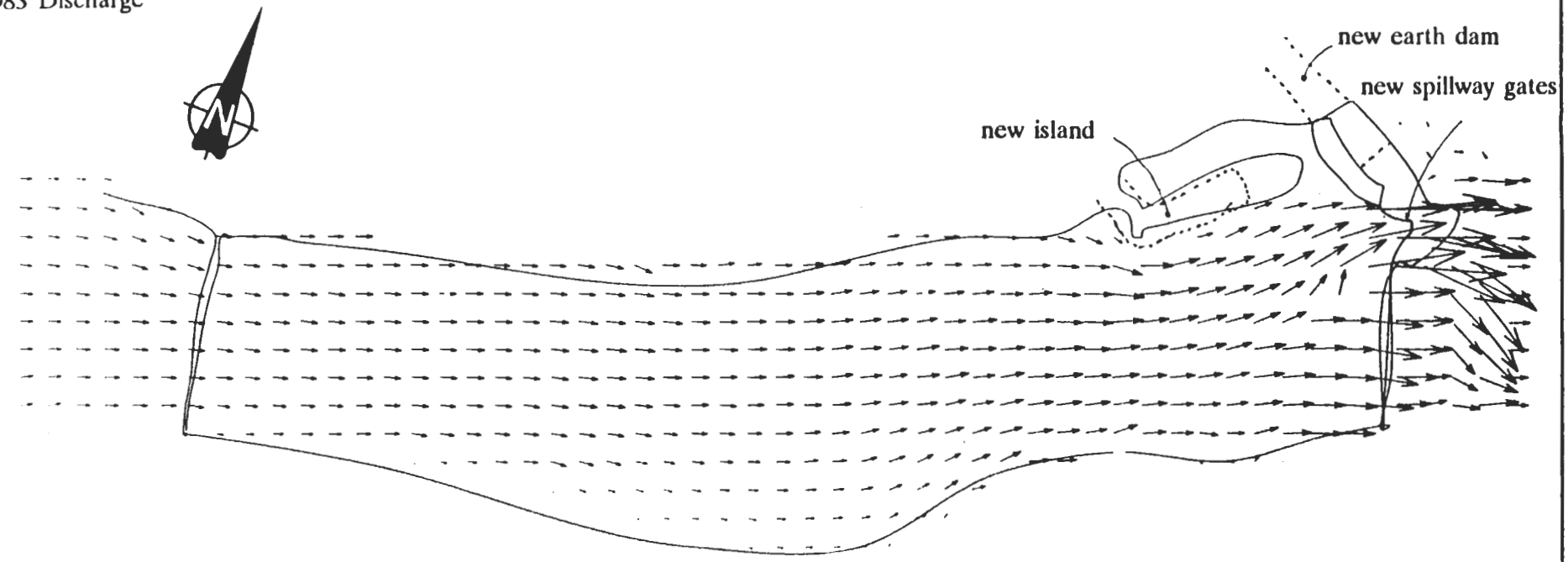

BISHOP FALLS
SPILLWAY RATING CURVES
FLOW OVER FLASHBOARDS
Transition
SPILLWAY
(Flashboards removed)
10 GATES
TOTAL DISCHARGE
ABITIBI PRICE DATUM
RESERVOIR ELEVATION (m)
RESERVOIR ELEVATION (feet)
DISCHARGE (m³/s)
43.5
43.0
42.5
42.0
41.5
41.0
40.5
40.0
39.5
39.0
38.5
38.0
37.5
37.0
140
135
130
125
120
0
1000
2000
3000
4000
5000
6000
7000
8000

FIGURE 5-4

BISHOP'S FALLS 2D-HYDRODYNAMIC MODEL FOR ICE 34.6M.GRID

1983 Discharge

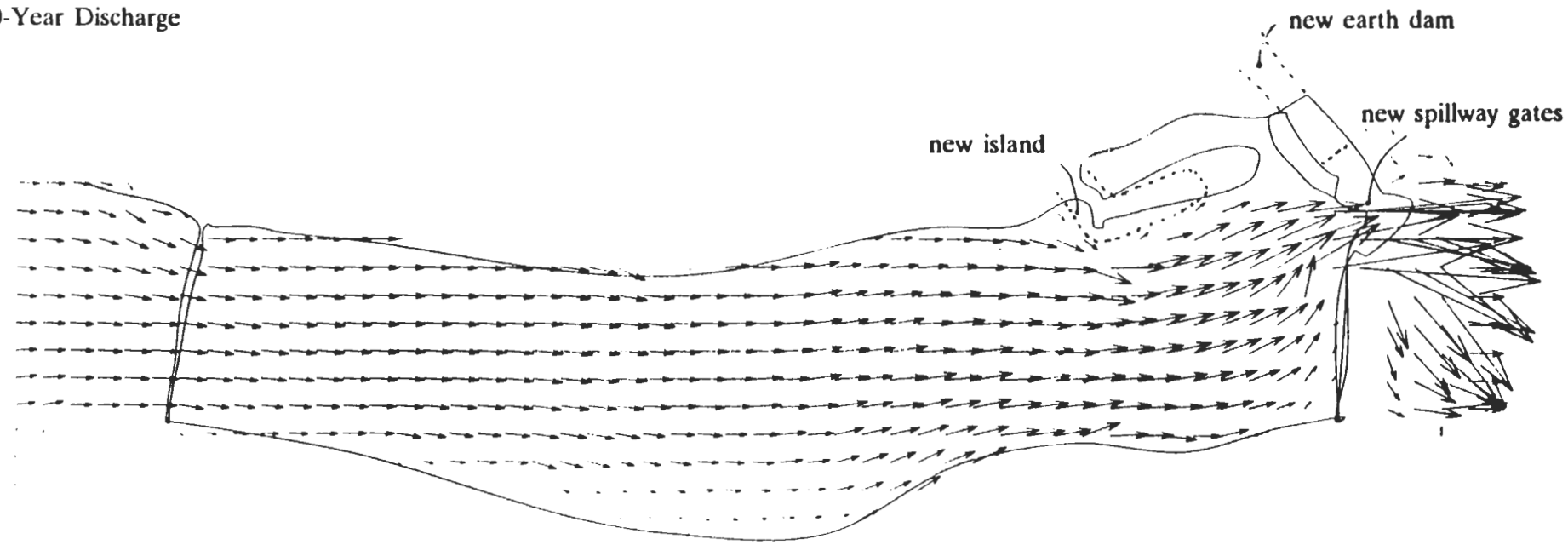


BISHOP'S FALLS HYDRODYNAMIC MODEL

FIGURE 5-5

BISHOP'S FALLS NEW DAM 2D-HYDRODYNAMIC MODEL FOR ICE 34.6M

100-Year Discharge



BISHOP'S FALLS HYDRODYNAMIC MODEL

FIGURE 5-6

(1988) confirmed that the dam is stable for ice loads. In accordance with Canadian practice, this involved ice loads taken to be 10,000 lb/ft (15,000 kg/m) for concrete structures and 5000 lb/ft (7460 kg/m) for stoplogs and gates.

In summary, the hydrodynamic analysis indicates that dynamic ice action and forces played a very small/negligible role in the failure of the earth dam during the 1983 flood. It may have contributed to some soil dislocation, but the dam and its concrete and steel sheet pile interior failed because of erosion caused by overflowing water. Hydrodynamic ice analysis for the new dam show that much of the river ice will pass freely over the Ambersen dam. Other ice will break in contact with the new "island", or in its rapid passage through the new spillway gates. Dynamic forces are too great to cause ice blockage at the gates and the dam is stable under static ice forces.

The very presence of ice, however, may have contributed to increasing backwater levels upstream of the dam during the 1983 flood - the topic of the next section.

6.0 HYDRAULIC BACKWATER ANALYSIS

6.1 General

The purpose of the hydraulic investigation is to derive the 1:20 and 1:100 year open water surface profiles along the study reaches using the results of the hydrologic information provided in Chapter 4.0

To carry out the investigations, the HEC-2 (HEC, 1982) computer model was used. This model was selected for this study because it represents the state-of-the-art for the computation of open water surface profiles for steady state conditions which are reasonable to assume in this study. It has been successfully used in similar applications in the U.S. and Canada, is well-documented, is parameter efficient for calibration, and is flexible in use.

The model was developed by the U.S. Corps of Engineers, Hydrologic Engineering Center to compute water surface profiles for natural or man-made channels, assuming that such flow is steady and gradually varied. The model estimates the change in water surface elevation between given river cross-sections with special computations accounting for bridge structures and other flow obstructions in the flood plain. The basic computational procedure used in the model is the solution of the one-dimensional energy equation with energy loss due to friction evaluated with Manning's equation.

Full details of the HEC-2 model and its underlying theory are given in the user's manual (HEC, 1982). The release used for this study was issued November 1976 updated May 1984.

6.2 Set-Up of the HEC-2 Model

The HEC-2 model for the Exploits River was set-up using river cross-sectional data obtained from field surveys (undertaken during the spring and summer of 1988) and topographic data from Abitibi-Price soundings. The field surveys included measurements of representative channel cross sections and structures across the watercourse, and photographs at each

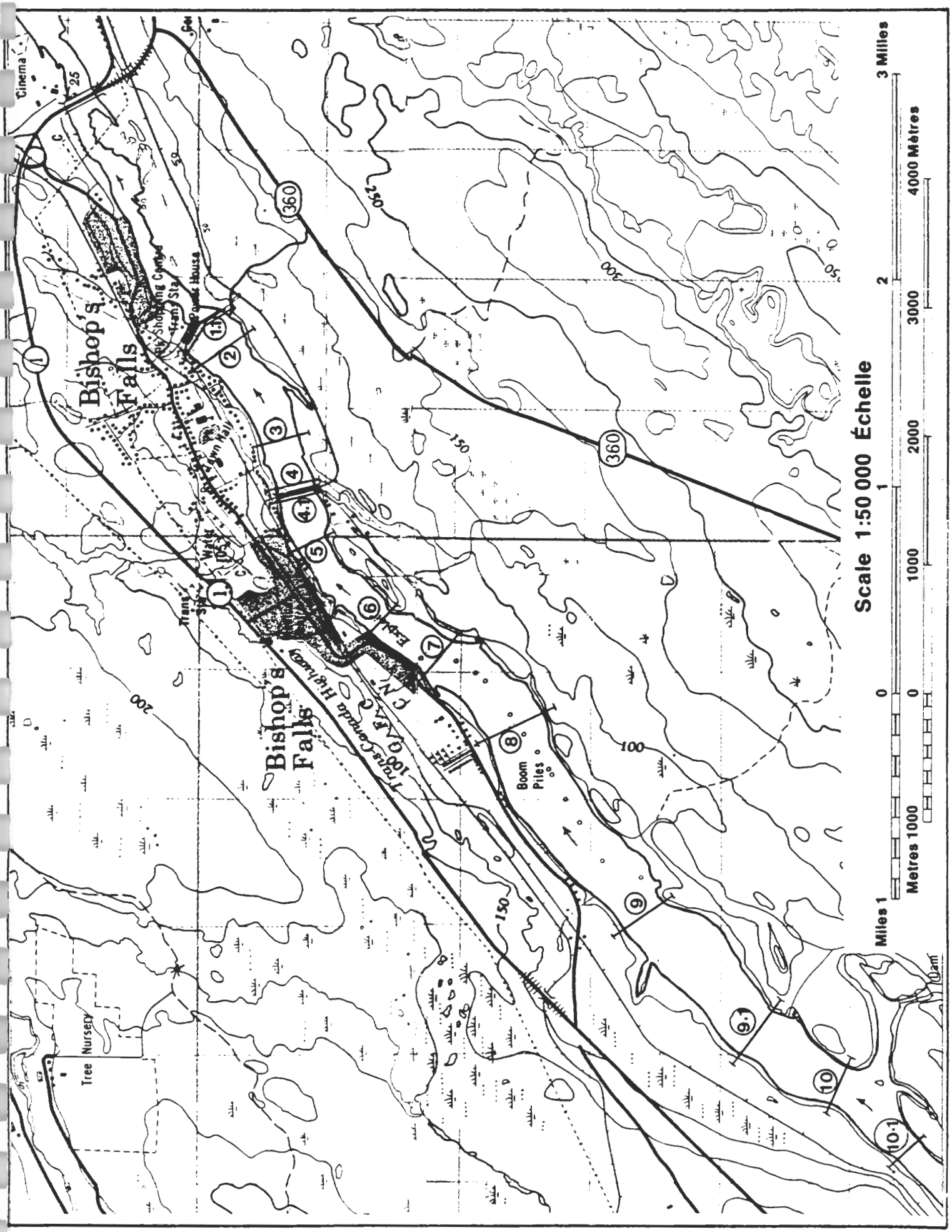
section.

A total of 10 representative river valley and structure cross-sections were surveyed along the river from the dam to a point about 200 metres upstream of the confluence of Great Rattling Brook. The total distance surveyed along the river was about 8.6 km.

The cross-sections were surveyed above and below the waterline and in most cases beyond the top of bank. Each cross-section was referenced to a datum at the Post Office. This local datum was subsequently referenced to geodetic datum following surveys in 1989 and mapping in 1990.

The structures across the watercourse were coded using the guidelines outlined in the HEC-2 Users Manual. For example, the skewness of the railway bridge structure was taken into account in the coding because the structure is not perpendicular to the flow. In addition, four cross sections were added to assist in coding of the dam and the bridge to give a total of 14 cross sections to model the river. Two are bridge-related sections and 12 are river sections between the dam and the upstream study limit. The locations of these sections are shown on the following figures (Figure 6-1). The overbank portion of these surveyed sections were compared with the elevations presented on the mapping to ensure that geodetic datums were correct and that both sets of topographic data were compatible. Our surveyed elevations and the topographic mapping were found to be a good fit.

Plots of all cross sections and bridge/culvert structures are presented in the Technical Appendix and all the cross-sections were coded adopting the convention of looking downstream from left to right bank. The Manning's roughness coefficients for the left and right overbanks and channel are shown in the cross section and are listed for each section in the Technical Appendix. Initially the roughness coefficients were estimated from photographs and soundings taken of each section during the field work, air photos taken after the mill pond was drained (1983), and from available mapping. Subsequently, some of the values were adjusted during the calibration of the model.



Cross Section Locations 1988 Survey

FIGURE 6-1

6.3 Calibration of the HEC-2 Model

6.3.1 General

In order to obtain confidence in the computed water surface profiles, the HEC-2 model was calibrated and validated with observed water levels and flow data. For this study, the information was obtained from:

(i) water level monitoring conducted during initial field investigations between 17-19 May 1988.

(ii) water level monitoring during the cross section surveys, undertaken between 11-14 July 1988.

(iii) evaluation of photographic records, television tapes, film by a local resident and interviews with streamside residents to obtain estimates of 1983 high water levels.

(iv) old and recent drawings giving the configuration of the dam and the shoreline.

6.3.2 Model Calibration - Open Water Conditions

The HEC-2 backwater model was calibrated to observed water levels measured during the cross-section survey of 14 July 1988. During this period, the flow rate as reported at the Bishop's Falls power dam was 253 m^3/s. A total of seven measured water level observations located along the reach were employed to calibrate the model. Estimates of the channel and overbank roughness coefficient (Manning's n) as determined during the surveys were used during the initial stages of the model calibration.

Several computer simulations were made to calibrate the HEC-2 model. After each run adjustments were made to the Manning's roughness values until good agreement was obtained between the simulated and observed water levels. The results presented in Table 6-1 show

Table 6-1

Calibration of Exploits River
HEC-2 Model - 14 July 1988

Cross-Section Number	Distance from Dam (m)	Peak Flow (m^3/s)	*Water Surface Elevation (m) Observed	Simulated
1.1	10	253.0	13.36	13.32
2	265	253.0	-	13.32
3	645	253.0	-	13.32
3.1	1075	253.0	-	13.32
4	1475	253.0	-	13.33
4.1	1481	253.0	-	13.33
5	1836	253.0	13.33	13.33
6	2731	253.0	13.31	13.33
7	3211	253.0	13.31	13.34
8	3931	253.0	13.36	13.34
9	5841	253.0	13.36	13.35
9.1	7250	253.0	-	13.36
10	8000	253.0	-	13.38
10.1	8650	253.0	13.46	13.42

* Note: All elevations are Geodetic

excellent agreement between observed and simulated results.

The Manning's roughness values used in the calibration are presented in the cross-section plots in the Technical Appendix. The values are given for the left and right overbanks and channel, and are reasonably close to anticipated theoretical values (Chow, 1959) and those found in other practical applications (U.S. Geological Survey, 1976).

6.3.3 Validation of the HEC-2 Model

After the HEC-2 model was calibrated, it was validated to ensure it was predicting reasonably accurate results. The observed water levels from the July 11 and July 12, 1988 surveys were used to validate the backwater model. Both validation events provide fairly good agreement between observed and computed water surface elevations as shown in Table 6-2 and Table 6-3. The 11 July 1988 water level observations were collected over a span of 8 hours and as shown in the dam records the actual flow rate and water levels were increasing throughout that day. Consequently the simulated water levels in the upstream sections of the reach are slightly lower compared to the observed. However, conditions remained relatively more uniform during the July 12, 1988 event and the agreement between observed and simulated water levels is much closer.

1983 Flood

A final verification employed the results of observed open water levels (from photographs, film, interviews) following the 1983 flood. These "observations" of open water levels covered a span of many hours during that flood (14 January) and are subject to some inaccuracy as a result of this inexact timing and changes in land-use along the river bank.

Despite the challenges involved in obtaining consistently good estimates of 1983 water levels, the verified model provided good results in this simulation. A total of six reliable open water level observations for the 1983 flood event along with the results of the simulated levels are presented in Table 6-4 and Figure 6-2.

Table 6-2

Verification of Exploits River
HEC-2 Model - 11 July 1988

Cross-Section Number	Peak Flow (m^3/s)	*Water Surface Elevation (m) Observed	Simulated
1.1	266.0	13.39	13.39
2	266.0	-	13.39
3	266.0	-	13.39
3.1	266.0	-	13.39
4	266.0	-	13.39
4.1	266.0	-	13.39
5	266.0	-	13.40
6	266.0	13.38	13.40
7	266.0	13.41	13.40
8	266.0	13.44	13.41
9	266.0	13.49	13.42
9.1	266.0	-	13.43
10	266.0	-	13.45
10.1	266.0	-	13.49

* Note: All elevations are Geodetic

- Water level and flow rate was not uniform throughout the period of observed water level elevations

Table 6-3

Verification of Exploits River
HEC-2 Model - 12 July 1988

Cross-Section Number	Peak Flow (m^3/s)	*Water Surface Elevation (m) Observed	Simulated
1.1	269.0	13.38	13.38
2	269.0	-	13.38
3	269.0	13.37	13.38
3.1	269.0	-	13.38
4	269.0	13.38	13.39
4.1	269.0	13.38	13.39
5	269.0	13.39	13.39
6	269.0	-	13.39
7	269.0	-	13.40
8	269.0	-	13.40
9	269.0	-	13.41
9.1	269.0	-	13.43
10	269.0	-	13.44
10.1	269.0	-	13.49

* Note: All elevations are Geodetic

Table 6-4

Verification of Exploits River
HEC-2 Model - 1983 Open Water Flood Event

Cross-Section Number	Peak Flow (m^3/s)	1983 Flood Observation Site	1983 Flood *Water Surface Elevation (m) Observed	Simulated
1.1	3256		-	15.81
2	3256	H	15.88	15.89
		E	15.85	15.93**
3	3256		-	15.96
3.1	3256			16.04
		F	16.04	16.05**
		C	15.98	16.05
4	3256		-	16.11
4.1	3256		-	16.11
5	3256	X	16.21	16.18
6	3256		-	16.43
7	3256		-	16.59
8	3256			16.69
		G	16.77	16.79**
9	3256		-	16.98
9.1	3256		-	17.33
10	3256		-	17.58
10.1	3256		-	18.09

* Note: All elevations are Geodetic

** interpolated from simulations

EXPLOITS RIVER – BISHOP'S FALLS

1983 FLOOD EVENT–OPEN WATER CONDITION

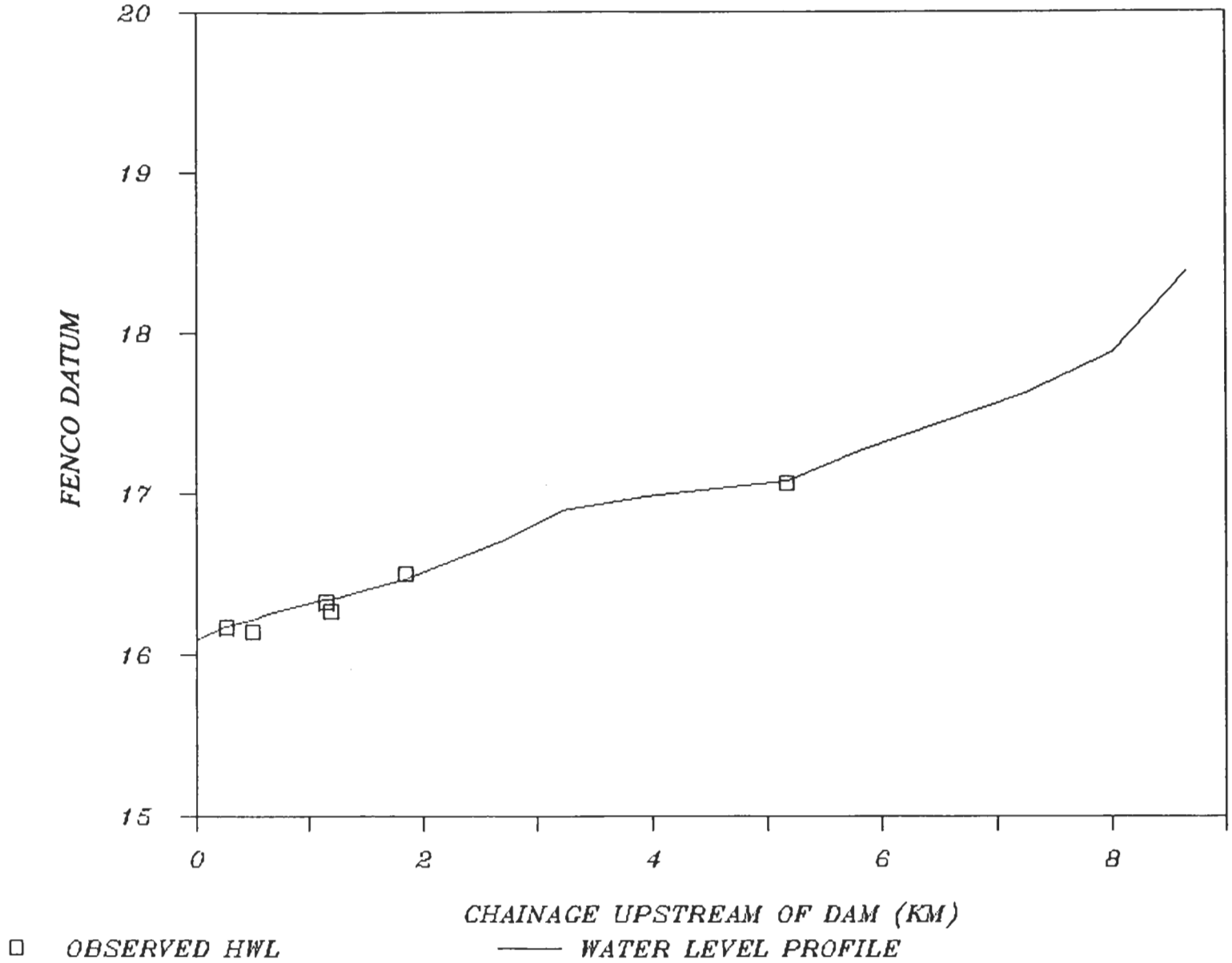


FIGURE 6-2

6.3.4 Calibration/Verification Conclusions

From the results presented in the previous tables, it can be seen that the observed and computed water levels are in reasonably close agreement. Consequently, it is concluded that the model is capable of accurately predicting open water surface profiles for flood flow events.

6.4 Sensitivity Testing for the HEC-2 Model

6.4.1 General

In order to test the sensitivity of the computed water levels to variations in parameters within the model, a number of computer simulations were made by varying one parameter while holding the others at their previously calibrated values. The parameters changed in the modelling were: channel invert by ± 0.15 m; discharge by ± 15%; starting water level by ± 0.15 m; Manning's "n" values by ± 10%; and expansion and contraction coefficients by ±10%.

The 1:100 year flood condition (2883 m^3/s) was used to conduct the sensitivity analysis and the results are presented in Table 6-5.

6.4.2 Sensitivity to Changes in Channel Invert

Lowering or increasing of channel inverts by 0.15 m has a minimal effect on the computed water surface elevations particularly in the downstream portion of the study reach. However, the effect becomes slightly more pronounced upstream of the railway bridge crossing where water level changes ranged as much as ± 0.10 metres at the upstream limit.

6.4.3 Sensitivity to Peak Discharge

Variations of the peak discharge rate of ± 15% altered the water levels within the reach by as much as ± 0.47 metres at the upstream study limit. Yet, within the Town of Bishop's Falls

Table 6-5

HEC-2 Sensitivity Analysis for Exploits River
Change in Water Surface Elevation (metres) at Cross-Section

Parameter	Parameter Variation	Cross-Section No. 1.1	2	3	3.1	4	4.1	5	6	7	8	9	9.1	10	10.1
Channel Bottom	+ 0.15 m	0.00	0.00	+.01	+.01	+.03	+.03	+.03	+.05	+.05	+.06	+.07	+.09	+.10	+.10
	- 0.15 m	0.00	-.01	-.01	-.01	-.02	-.02	-.03	-.04	-.05	-.06	-.07	-.07	-.09	-.10
Discharge	+ 15%	0.00	+.02	+.05	+.08	+.10	+.11	+.12	+.19	+.23	+.25	+.31	+.37	+.39	+.45
	- 15%	0.00	-.02	-.05	+.08	-.09	-.09	-.11	-.17	-.21	-.23	-.30	-.36	-.40	-.47
Starting Water Level	+ 0.15 m	+.15	+.14	+.14	+.13	+.13	+.13	+.12	+.11	+.11	+.10	+.09	+.08	+.06	+.05
	- 0.15 m	-.15	-.15	-.14	-.14	-.13	-.13	-.13	-.11	-.10	-.10	-.09	-.07	-.07	-.05
Manning's "n"	+ 10%	0.00	+.02	+.06	+.09	+.13	+.13	+.24	+.25	+.28	+.30	+.37	+.41	+.44	+.47
	- 10%	0.00	-.02	-.05	-.09	-.11	-.11	-.15	-.23	-.27	-.27	-.35	-.40	-.44	-.48
Expansion and Contraction Coefficients	+ 10%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	+.01	+.01	+.01	+.01	+.01	+.01
	- 10%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-.01	-.01

Note: Base Condition is 1:100 year flood event (i.e.) 2883 m^3/s, and calibrated Parameters

(cross section # 1.1 - 9.0) the deviation was only ± 0.31 metres. It was noted during the calibration of the model that the simulated water levels for lower discharge rates (i.e. 253 m^3/s, 14 July 1988) were significantly less sensitive to changes in the discharge (even at variations of ± 25%) as compared to the 1:100 year flood flow (2883 m^3/s).

6.4.4 Sensitivity to Starting Water Level

By varying the downstream starting water level elevation ± 0.15 metres at cross-section No. 1.1 of the backwater model the impact was shown to become less significant at cross sections further upstream (i.e. an increase in starting water level of 0.15 metres resulted in an increase of only 0.05 metres at the upstream limit).

6.4.5 Sensitivity to Roughness Coefficient

Variation of the roughness coefficient Manning "n" proved to significantly effect simulated water levels. An increase or decrease in the calibrated roughness coefficients by 10% resulted in changes in water level elevations by as much as ± 0.48 metres at the upstream study limit. Fortunately, the model was calibrated for high flow conditions so this sensitivity is not a concern.

6.4.6 Sensitivity to Expansion and Contraction Coefficients

The effect of varying the Expansion and Contraction Coefficients by ± 10% resulted in a minimal change of ± 0.01 metres to the upstream sections of the study reach.

7.0 FLOOD LEVEL PROFILES

As indicated previously, the purpose of this study is to derive water surface profiles for the 1:20 and 1:100 year return period flood flows and for the historical flood (1983).

In Chapter 4, it was determined from frequency analyses and 1983 observation that the return period flood flows of interest are:

1:20 year	2257 m^3/s
1:100 year	2883 m^3/s
1983 Flood	3256 m^3/s

7.1 Starting Water Level

The spillway rating curve (Figure 5-4) is shown on the next page (Figure 7-1) with additional notes describing further information about operations at Bishop's Falls.

This figure includes a curve called, "Flow Over Flashboards" and lines titled, "Transition". These lines/curves describe the increase in flow rates over the top of flashboards with increases in reservoir elevation, and show the "transition" or rapid increase in flow which takes place when these flashboards are removed or swept downstream.

The "flashboards" are a short, fragile wooden wall which is placed along the crest of the Ambersen Dam to increase the head and power production during normal flow conditions. Before 1983 and the construction of the new gates at the dam, the flashboards were frequently swept downstream during high flows in the summer, fall, and every year at break-up. When they failed, there was a "transition" or rapid increase in outflow from the dam.

Replacing the flashboards was an expensive and time consuming process and thus, the new gates and their operation is partly geared toward saving them from repeated failure. This is achieved through the new operation guidelines for the dam (Acres, 1984) which are structured

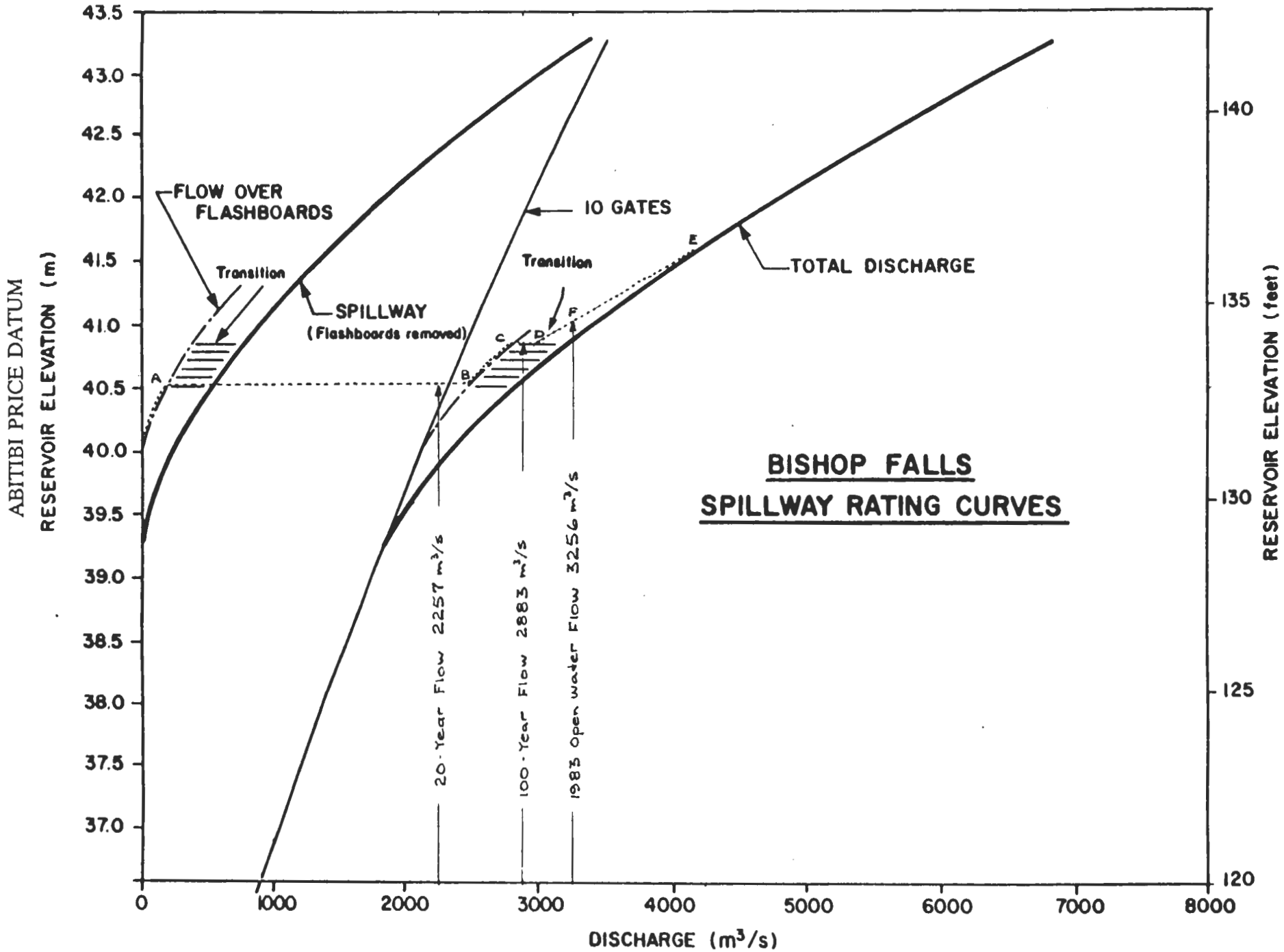

BISHOP FALLS
SPILLWAY RATING CURVES
FLOW OVER FLASHBOARDS
Transition
SPILLWAY
(Flashboards removed)
10 GATES
Transition
TOTAL DISCHARGE
A
B
C
D
E
F
20-Year Flow 2257 m³/s
100-Year Flow 2883 m³/s
1983 Open Water Flow 3256 m³/s
ABITIBI PRICE DATUM
RESERVOIR ELEVATION (m)
RESERVOIR ELEVATION (feet)
DISCHARGE (m³/s)
43.5
43.0
42.5
42.0
41.5
41.0
40.5
40.0
39.5
39.0
38.5
38.0
37.5
37.0
140
135
130
125
120
0
1000
2000
3000
4000
5000
6000
7000
8000

FIGURE 7-1

to keep water levels no higher than 0.3 m (about 1 foot) above the top of the flashboards (i.e. water levels below the maximum normal operating level of 133 ft - Abitibi datum). To do so, the new gates are opened in sequential manner until 9 gates are fully open. If levels and flows continue to rise the 10th gate is opened and the flashboards are allowed to fail.

The process is shown in Figure 7-1. Water levels are allowed to rise to point A (elevation 40.54 m - 133 ft Abitibi datum) before any gates are opened. The gates are then opened progressively to maintain this low level in the mill pond until all 10 gates and the flashboards are flowing full (point B). At this point, water levels begin to rise and exert increasing force on the flashboards. At about point C (elevation 40.84 m - 134 ft Abitibi datum) there is a transition to a point where about 50% of the flashboards are lost (point D). More and more of the flashboards fail as the water level rises and historical data indicates that they are all removed/lost by point E (a level of 41.3 m - 135.5 ft Abitibi datum). This elevation on the "Total Discharge" curve provides a discharge of 4248 m^3/s (150,000 cfs) - much more than the 1983 flood flow yet at a water level which is much lower than the 1983 level.

Overall, this operation procedure establishes that:

- the 1 in 20 year flood flow will be passed at an elevation of 40.54 m/133 ft (Abitibi datum) or less at the dam - near point B

- the 1 in 100 year flood flow will be passed through the gates and over the spillway at the dam at an elevation which will not exceed 40.84 m (134 ft Abitibi datum) - near point D

- a repeat of the historic 1983 open-water flood would be passed at the dam at an elevation which will not exceed 41.00 m (134.5 ft Abitibi datum) - near point F on the spillway rating curve

7.2 Flood Levels

7.2.1 Open Water Condition

The calibrated/verified HEC-2 model was run with the above-noted starting water levels and corresponding peak flows. Table 7-1 summarizes the flood levels obtained upstream of the dam for the 20 and 100 year flows and for the historical 1983 flood. Noteworthy, is that a repeat of the 1983 flood flow (without ice) would result in lower levels than before.

7.2.2 Backwater Conditions With Break-up Ice

Measurements of the surface level/elevation of water flowing in a channel or over a dam are obtained by Abitibi-Price Inc. and the Water Survey of Canada in their process of estimating streamflow. These measurements of level are compared to calculations (or other measurements) which translate the water level reading into a streamflow value. This approach is reasonably accurate when there is no ice within the flow, but can inaccurately measure streamflow if the volume of ice is large enough to block or displace a large volume of water.

Typically, the volume of ice which passes through Bishop's Falls is relatively small in comparison to the volume of streamflow, and the rate of break-up is relatively long (i.e. 7-10 days). Given this time span and the volume of ice which is typically present above Bishop's Falls (about 35 x 10^6 m^3, Fenco Newfoundland 1985), indicates that only 1% to 3% of the "streamflow" during typical break-up composed of ice. All of the above hydrologic and hydraulic analyses have accounted for this percentage through the use of water level observations as the basis for each computation.

The effect of dynamic ice action during the break-up flood of January 1983 was shown in Section 5.4 to have likely had little effect on flood damage at the powerhouse or old earth dam. However, this is not the whole story, because the break-up ice did raise water levels.

Available records of water levels and streamflow show that there was a rapid rise in water

Table 7-1

Bishop's Falls Open Water Flood Levels
(New Dam Structure)

Cross-Section Number	*Water Surface Elevations (m) 1:20 Yr Flood	1:100 Yr Flood	1983 Historical Flood
1.1	14.32	14.81	15.08
2	14.37	14.88	15.16
3	14.45	14.98	15.28
3.1	14.53	15.08	15.39
4	14.58	15.15	15.47
4.1	14.59	15.16	15.47
5	14.64	15.23	15.56
6	14.87	15.51	15.87
7	15.00	15.69	16.07
8	15.08	15.79	16.18
9	15.37	16.12	16.53
9.1	15.71	16.51	16.95
10	15.96	16.79	17.24
10.1	16.48	17.35	17.82

* Note: - All elevations Geodetic

levels in the early morning of Wednesday, 12 January. This rise was sufficiently above the freeze-up level to initiate break-up and downstream movement of ice on the Exploits River and tributaries (Beltaos, 1983). Photographs and film of the river above the dam show that there was little ice within the mill pond at about 8 a.m. on Friday, 14 January, 1983. Thus, practically all of the river ice in the Exploits Basin below Exploits Dam was carried over the spillway at Bishop's Falls Dam in about the 50 hours before the final erosion failure of the earth embankment.

The best estimate of the peak streamflow at Bishop's Falls is obtained from water level observations on the early morning of January 14, 1983. At that time, there was little ice within the flow, which was estimated to be 3256 m^3/s (115,000 cfs) from the open water calibration curves for the dam. Photographs along the banks at Bishop's Falls indicate, however, the presence of stranded ice pieces above the observed level of flow that morning. This suggests that either the flow rate was higher during the night of January 13, <u>or</u> that the concentration of ice within the flow raised water levels that night near the falls.

However, the water level observer at the West End Trailor Court (Mr. Gill) determined that flows were not higher that night than the following day, and hence, that it was <u>indeed</u> the <u>break-up ice</u> which <u>caused the highest levels near Bishop's Falls dam</u>.

<u>Effect of Ice Concentration</u>

Figure 7-2 presents the open water, surface level profile for 1983 conditions for a discharge of 3256 m^3/s. The square symbols on the profile are the estimated maximum water level determined from field surveys. These points, which were estimated from photographic evidence of the location of stranded ice pieces, indicate that the volume of break-up ice within the flow was likely responsible for raising water levels in the lower portion of the study reach by about 0.10 m to 0.20 m.

The volume of ice on the river in 1983 can only be estimated for such a large river. A recent study by Fenco Newfoundland (1985) determined that the ice volume upstream of Goodyear's

EXPLOITS RIVER – BISHOP'S FALLS

1983 FLOOD EVENT–OPEN WATER CONDITION

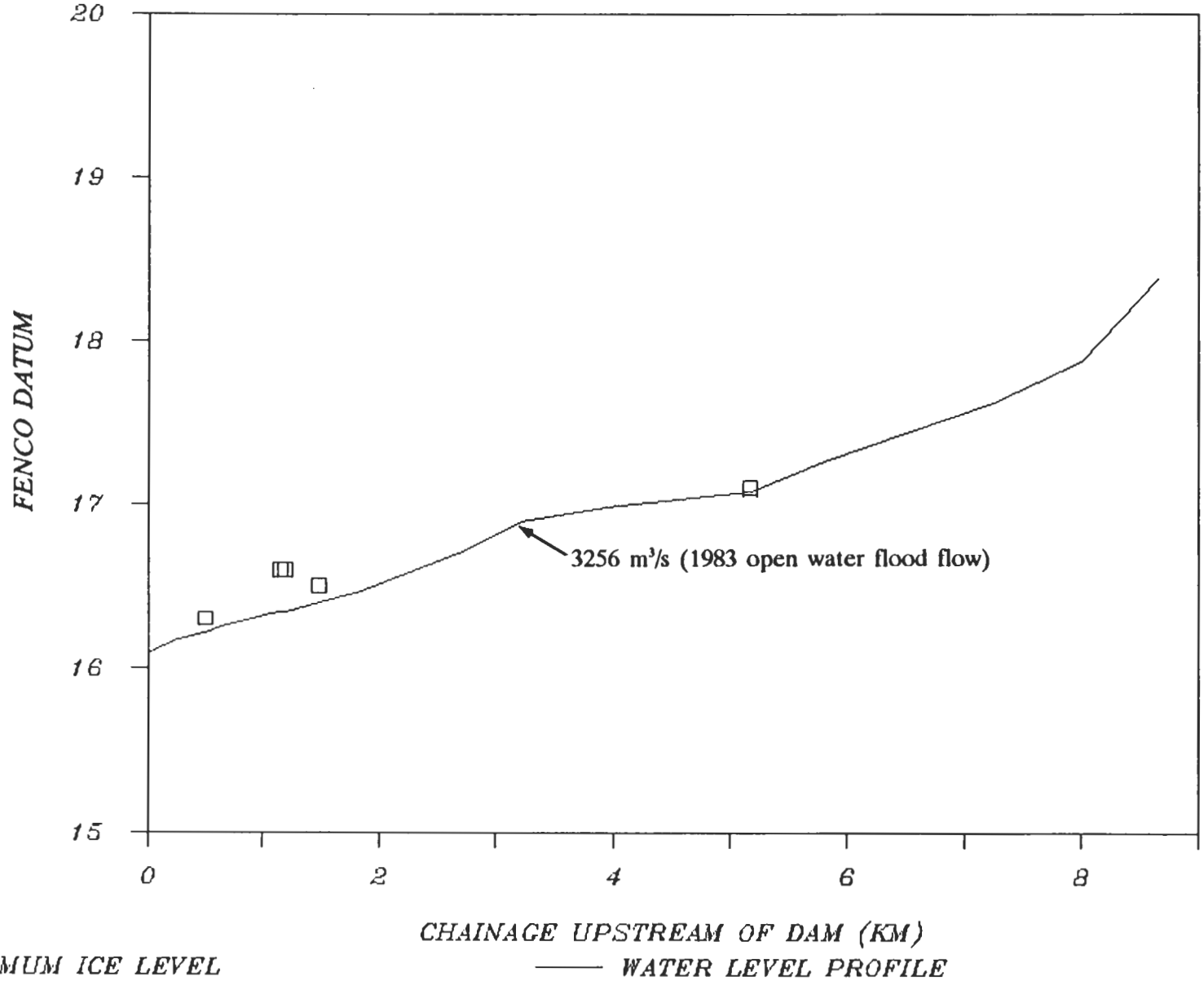


FIGURE 7-2

Dam (to Badger) was about 17 x 10^6 m^3, which is slightly more than the average (13.5 x 10^6 m^3) determined in the same study. The volume of ice between Goodyear's and Bishop's Falls, and on the major tributaries (Stony Brook and Great Rattling Brook) is estimated to be about 6 x 10^6 m^3 - based on surface area and 1983 ice thickness observations given in Section 5.3.2. Hence, a total of about 23 x 10^6 m^3 of ice was carried over Bishop's Falls Dam in approximately 50 hours. The average rate of ice discharge in this period was 128 m^3/s, but the peak rate of ice discharge may well have been more. For example, if most of the ice cleared in 24 hours (half the time), the ice discharge rate would be about double - or 256 m^3/s.

The effect of displacement of water by this ice was simulated by increasing the 1983 flood flow estimate (3256 m^3/s) by 128 m^3/s - the average effect of ice displacement (i.e. to 3384 m^3/s) over a 50 hour period. However, given that it appears to have taken nearly a day to clear the ice from the Mill Pond area (2 x 10^6 m^3 ice), it is quite possible that the average ice flow during the discharge period was up to twice the above rate or 256 m^3/s. This effect was also simulated, and the results of both ice conditions are shown in Figure 7-3.

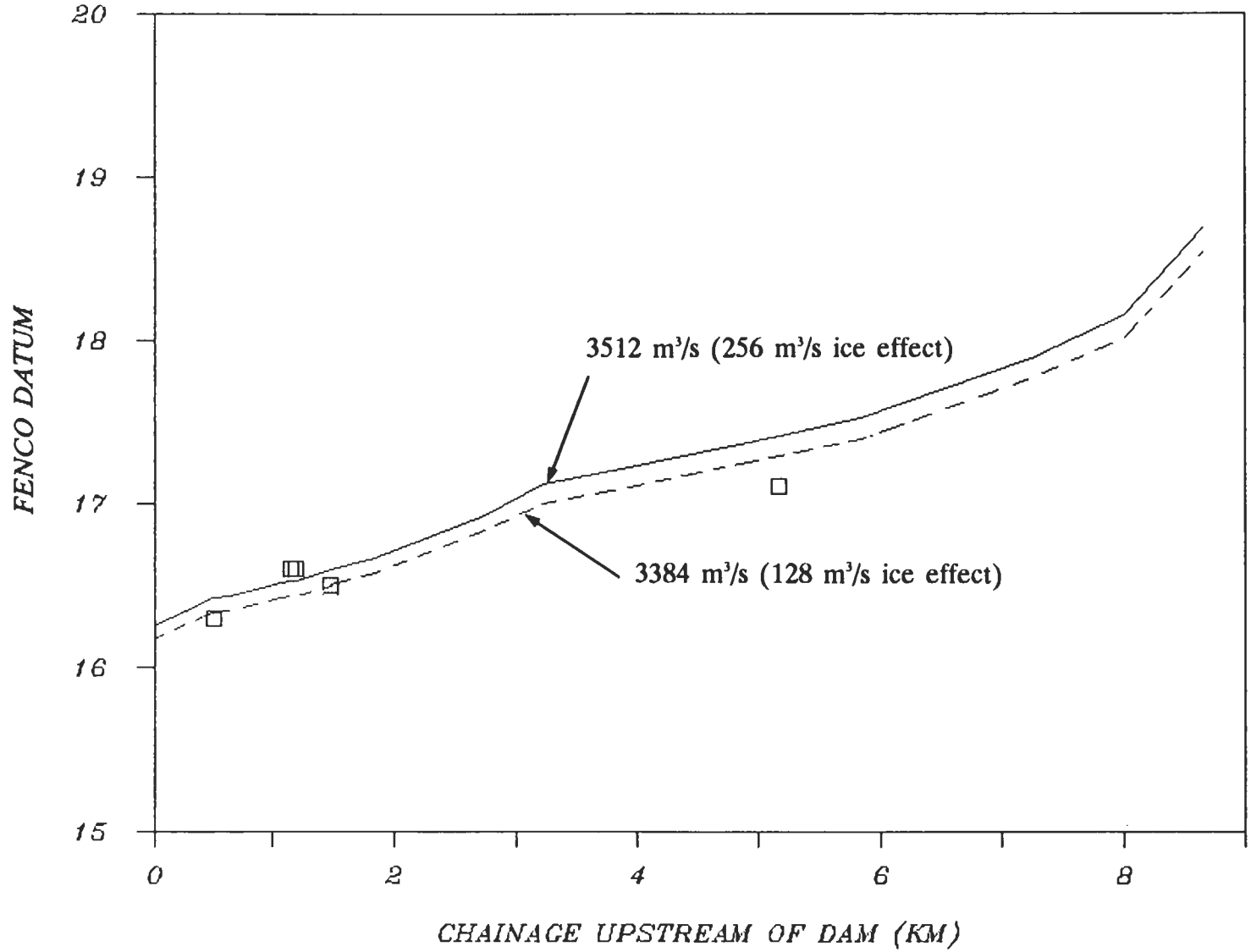
The upper curve in Figure 7-3 corresponds to ice displacement of 256 m^3/s (i.e. total effective flood flow of 3512 m^3/s). The lower curve is the effect of 128 m^3/s ice displacement. The curves show reasonably good agreement in the lower 2 km reach of Bishop's Falls, where the field estimates of ice elevation are slightly above or below the lines which are estimated from modelling. The upstream level (at 5 km upstream of the dam) does not appear to fit the curves. However, the observer at this location (Mr. Gill) noted that ice levels in the middle of channel were actually higher than at the banks at this location, and that this simulation is quite reasonable.

The overall effect of the ice concentration/displacement and (super elevation) is summarized at each section in Table 7-2. It is noted in this table that there are uncertain or no observations of ice-effect levels upstream of Section 9. However, it is reasonable to assume that water level rises of 0.12 and 0.24 m occurred from Sections 9 to 10.1.

There is no reason to believe that this additive effect of ice is changed by the replacement

EXPLOITS RIVER – BISHOP'S FALLS

1983 FLOOD EVENT– ICE DISPLACEMENT



□ MAXIMUM ICE LEVEL

---- ICE EFFECT=128 CMS

—— ICE EFFECT=256 CMS

FIGURE 7-3

Table 7-2

Expoits River - Effect of Ice Displacement

Section	Effect of Ice Concentration (Water Level rise in metres)	
	Ice Effect = 128 m^3/s	Ice Effect = 256 m^3/s
1	0.08	0.16
2	0.08	0.16
3	0.09	0.18
3.1	0.09	0.18
4	0.09	0.19
4.1	0.09	0.19
5	0.10	0.19
6	0.11	0.21
7	0.11	0.23
8	0.12	0.24
9	± 0.12*	± 0.24*
9.1	± 0.12*	± 0.24*
10	± 0.12**	± 0.24**
10.1	± 0.12**	± 0.24**

* unknown with certainty (poor data)

** unknown (no historical observations)

dam. Thus, to account for this effect, the level rises shown above for an ice effect of 256 m^3/s were added to the open water levels generated for 1983 (Section 7.2.1) to project the "historical" flood level for current conditions at Bishop's Falls.

A summary of the final 20-year, 100-year and historical flood levels at each cross section is given in Table 7-3.

7.3 Flood Level Delineation

The water level data from the design flood flow Table (Table 7-3) were transferred to a set of topographic maps, prepared in 1990.

A typical example of one such map is given in Figure 7-4, which is a photocopy-reduction of the area near the dam. Each surveyed cross section is marked as a line on the map, the circle connecting the line gives the 1:20 and 1:100 year flood water level. The solid and dashed lines connecting the cross sections delineate the 1:20 year and 1:100 year flood hazard areas, respectively. There are many locations along the river where the 1:100 year flood levels are only slightly higher than the 1:20 year levels. In these locations, only the 1:20 year flood line has been plotted. The historical flood level is delineated by a line of short dashes and dots. Complete versions of the flood risk maps may be obtained from the Department of Environment and Lands, St. John's.

The mapping shows that there are now no homes within the floodplain of the 1:20 or 1:100 year return period flood risk area, but that there are two buildings on the 100-year fringe. There are also several storage sheds/garages/out buildings which are within the 1:20 or 1:100 year flood risk areas. These are described below.

Table 7-3

Bishop's Falls Design Flood Levels

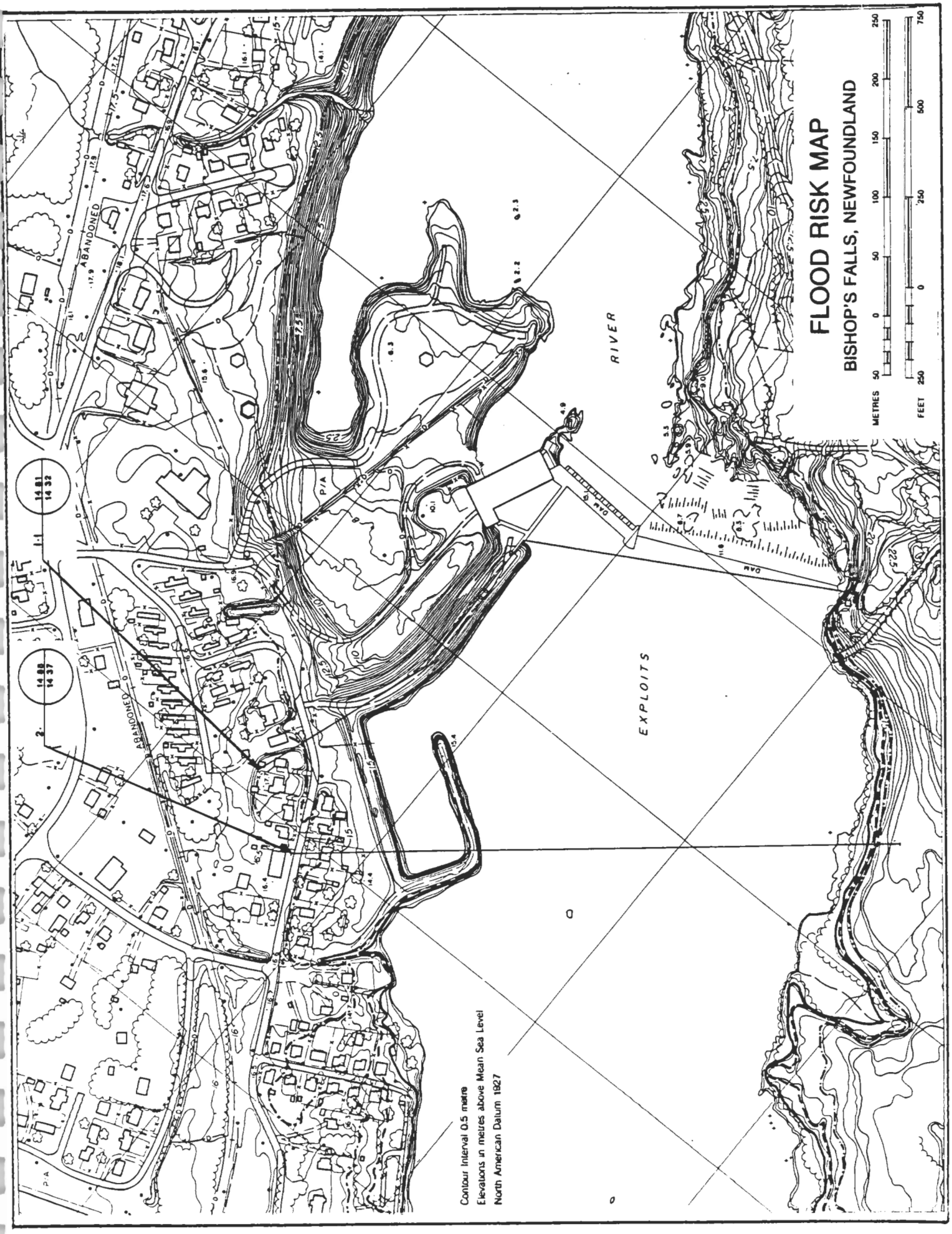
(New Dam Structure)

Cross-Section Number	*Water Surface Elevations (m) 1:20 Yr Flood	1:100 Yr Flood	**1983 Historical Flood
1.1	14.32	14.81	15.24
2	14.37	14.88	15.33
3	14.45	14.98	15.46
3.1	14.53	15.08	15.58
4	14.58	15.15	15.66
4.1	14.59	15.16	15.66
5	14.64	15.23	15.76
6	14.87	15.51	16.09
7	15.00	15.69	16.31
8	15.08	15.79	16.43
9	15.37	16.12	***16.78
9.1	15.71	16.51	***17.20
10	15.96	16.79	***17.49
10.1	16.48	17.35	***18.07

* Note: - All elevations are Geodetic

** Assuming Ice Effect due to additional 256 m^3/sec (see Table 7-2)

*** Assumed maximum rise due to ice = 0.24 metres (Table 7-2)



PORTION OF FLOOD RISK MAPPING SHEET

FIGURE 7-4

East side of Sidney Street	• building on fringe of 1:100 year floodplain • shed in 1:20 year floodplain
South side of river east of Railway Bridge	• building on fringe of 1:100 year • garage in 1:20 year
East of Ball Field Greenridge Road	• shed on fringe of 1:100 year
West Trailor Court along river bank	• several storage sheds in fringe of 1:20 or 1:100 year floodplain
Just west of ruins at west edge of town	• large storage barn in 1:20 year floodplain

Outside of this limited number, there are no other buildings in the floodplain at Bishop's Falls. The building near the railway bridge and that at the end of Sidney Street are unlikely to be damaged by the 1:100 year flood and hence, it is only the storage sheds/barns/garages which are of current concern.

8.0 FLOOD DAMAGE REDUCTION OPTIONS

The final step in this study calls for preliminary identification of flood damage reduction alternatives which could be employed along the river at Bishop's Falls. Noteworthy in considering these alternatives is that the floodplain mapping identifies a "floodway" and a "flood fringe".

The floodway is that part of the flood risk area, including the area normally occupied by the river, in which most of the flood waters are conveyed. This is an area where current speeds and flood depths are typically high and damages are often large. The floodway is defined as that area flooded on an average of one in 20 years.

The floodway fringe is that portion of the flood risk area lying between the floodway and the outer limit of the area which is flooded on an average of once in 100 years or which would be flooded by a repeat of the historical flood. This zone generally receives less damage from flooding than the floodway.

Proven ways to reduce flood damages can be broadly categorized into two groups. The first contains alternatives which accept that high water levels will occur from time to time but mitigate damages from these levels by a preventive approach which emphasizes long range planning for flood damage reduction. This preventive approach includes:

- floodplain regulations
- acquisition
- flood proofing

The second group of alternatives contains approaches which attempt to modify or reduce damages by methods designed to reduce the flood level (or modify the river hydraulics). Included here are structurally oriented work such as:

- flood control dams

- channelization or dyking
- bridge opening expansions

An important factor which enters into the decision making process at this stage is that it is always desireable to pursue alternatives which are economically justifiable. A benefit-cost analysis, (which is beyond the scope of this project), is generally employed to give guidance in weighing all the possible alternatives. However, from experience on similar projects and that the damage-prone structures are distributed along the full length of the river, it appears that the second group of alternatives would present highly unattractive benefit-cost ratios. Hence, the focus of the following is on the first group of alternatives.

8.1 Flood Damage Prevention

Although there are currently only a few out-buildings now within the flood-prone area of the Exploits River at Bishop's Falls, there are many flood-prone areas which might otherwise be considered as desirable river-side locations.

8.1.1 Floodplain Regulations

The primary alternative to reduce the potential for flood-related damages over the long term is to adopt a preventive approach which emphasizes long range planning in the flood-prone area. Measures such as zoning by-laws, building codes and subdivision regulations can be used to control and direct land use within the flood hazard areas. For example, no new buildings should be erected in the floodway where damage potential is high - although it is often desireable and acceptable to use this area for recreational purposes.

Within the floodway fringe, the objective of reducing future damages can be achieved if effective flood-proofing measures are incorporated in the design of new structures and subsequently carried out. This also applies to existing buildings in the floodway fringe where flood-proofing measures can substantially reduce the amount of future damage during a flood situation.

Overall, this option of damage prevention is recommended for immediate consideration. Regulations to control the design and type of structure located in the flood hazard area can ensure no adverse effects to new structures or to upstream or downstream residents, and can benefit existing buildings in the floodway fringe. The cost of introducing regulations is low and the flood damage reduction benefits for future development is high.

8.1.2 Flood Proofing

Flood proofing includes a wide variety of adjustments, additions, and alterations to structures (or their immediate environment) which attempt to reduce or eliminate potential flood damages. These measures include:

- installing permanent or temporary closures at low level openings in structures;
- raising structures on fill, columns or piers;
- constructing floodwalls or low berms around structures;
- relocation outside of the flood risk area.

Permanent closure, as its name implies, involves permanently closing and sealing all possible openings in a structure through which flood waters could enter. Flood proofing by permanent closure is usually limited to large industrial/commercial structures, or buildings on the outer fringe of flood prone areas where flood depths are less than about 0.3 m.

The elevation of buildings above flood levels is used for structures when permanent closure is difficult or impossible. As with permanent closure, no human intervention or flood warning is required to make the flood proofing effective.

Construction of floodwalls or berms around flood prone structures is most effective for structures with sufficient space surrounding them to put these works in place, and where soil permeability is low or flood peaks rise and fall quickly. This form of damage protection also reduces the potential for debris/ice damage to the building itself. Its principal limitation is that

special consideration must be given to building access if the berm of floodwall is higher than about 1 m.

8.2 Recommended Options

The following options to alleviate future damage problems are recommended for consideration:

1) It is recommended that the flood elevations advanced herein be adopted by the Municipality so that developable areas which are prone to flooding can be zoned in the near future for special flood risk restrictions and design consideration (e.g. elevation flood proofing on fill, extended and reinforced foundation walls or piles).

2) It is recommended that current maintenance procedures be continued at the dam to ensure that ice protection measures and standby opertions are in place to handle future flood flows.

3) It is recommended that flood proofing by relocation or elevation on fill be considered for the garages, sheds and out-buildings in the flood fringe, if it is desirable to provide physical flood protection for these existing buildings.

In conclusion, there are damage reduction options which may be applied to reduce future and existing problems in the Bishop's Falls area. Most may be carried out by the individual owners, but regulations to minimize future developing problems and to facilitate correction of existing ones rests in the hands of local government.

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