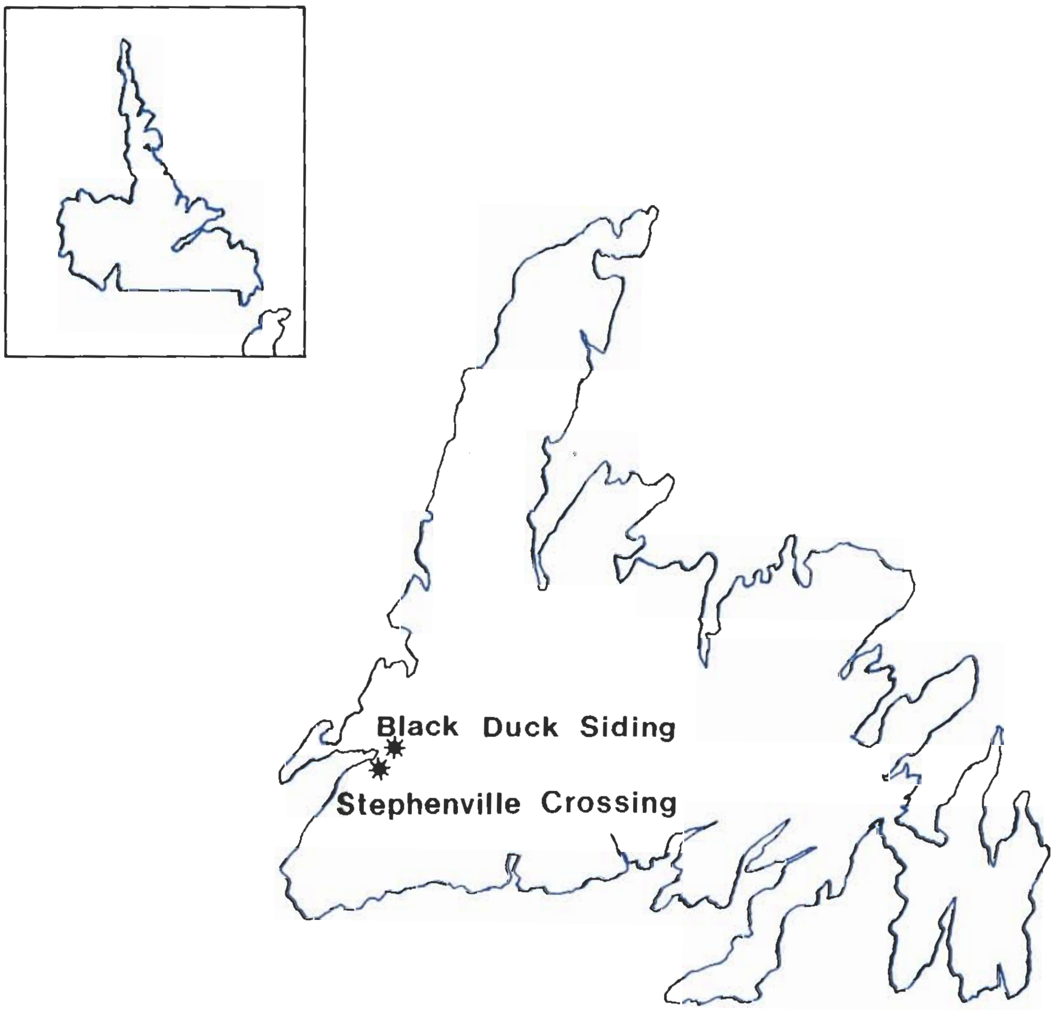
Canada – Newfoundland
Flood Damage Reduction
Program

Hydrotechnical Study of the Stephenville Crossing and Black Duck Siding Areas

Main Report



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Department of Environment

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HYDROTECHNICAL STUDY
STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

MAIN REPORT
VOLUME I

FOR

CANADA NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

MAY 1988

ACRES

May 19, 1988
P7907.00

Canada-Newfoundland Flood Damage
Reduction Program
c/o Department of Environment and Lands
P.O. Box 4750
St. John's, Newfoundland
A1C 5T7

Attention: Mr. R. Picco, P. Eng.
Project Engineer

Dear Sir: **Hydrotechnical Study - Stephenville Crossing/ Black Duck Siding Areas**

We are pleased to submit 50 copies of our final report on the above study. Ten copies of the technical appendices are also submitted.

The report details all work carried out during the course of the study, the methods of analysis, results and recommendations for continued monitoring and investigations. Floodlines for events having return periods of 1 in 20 and 1 in 100 years have been plotted and have been previously submitted. An executive summary accompanying the main report highlights the essential findings of the study.

The study constitutes a major piece of work in flood analysis and was particularly interesting because of the innovative approaches required for the frequency analysis both at Stephenville Crossing and Harry's River. We trust that the methodologies applied in the work will be of benefit to future analyses of both river and coastal flooding problems.

We would like to thank the members of the Technical Committee for their cooperation throughout the study. We would also like to acknowledge the assistance of the residents of Black Duck Siding in ice monitoring and in providing historical information.

We trust that you will find the report to be satisfactory and the findings of importance to the goals of the Flood Damage Reduction Program.

Yours very truly,

Maurice S. Mills
Regional Manager

RJG:jap

encl.

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TABLE OF CONTENTS

VOLUME 1 - MAIN REPORT

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

LIST OF TABLES

LIST OF TABLES (continued)

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

SUMMARY

The purpose of the Hydrotechnical Study of the Stephenville Crossing and Black Duck areas was to identify the flood prone lands for events having recurrence intervals of 1:20 and 1:100 years. The specific objectives were

- to identify the mechanics, physical processes and factors (physical, hydrometeorologic, oceanographic and hydraulic) responsible for flooding in the two areas, and

- to provide reliable estimates of the 1:20 and 1:100 year recurrence interval flood levels and the extent of flooding associated with each.

The study involved both technical analyses and field work to provide data and support for the analyses.

Different processes are responsible for the flooding in Black Duck Siding and in Stephenville Crossing, and consequently separate analyses were carried out for the two areas.

Conclusions

The principal conclusions of the study are as follows.

1. Black Duck: In both the 1:20 and 1:100 year events, the areas known as Tanglewood Ranch, Hickey's Farm, Hobbs' Farm and Dhoon Lodge will be flooded due to high water levels. The predicted levels presented in this report assume that the channel improvements made in November 1986 are maintained. These improvements result in lower water predicted levels at some locations, but flooding is still predicted to be widespread.

2. **Stephenville Crossing:** Stephenville Crossing is vulnerable to flooding from both west and east. To the west, high tides and waves in Rothesay Bay will lead to overtopping of the highway embankment in the low area near the Main Gut with maximum runup to elevations of 3.0 m and 3.2 m in 1:20 and 1:100 year events respectively. To the east, high levels in St. George's River result from high tide levels in Rothesay Bay combined with high fresh water inflows from Harry's River, Southwest Brook and Bottom Brook. Estimated 1:20 and 1:100 year flood levels in St. George's River are about 1.1 m and 1.5 m respectively.

Summary of Technical Analyses

Part A: Black Duck

Black Duck Siding is a small community located on a low lying area adjacent to Harry's River, near Stephenville Crossing on the west coast of the Island of Newfoundland. Historically, the worst flooding has resulted from ice-related events, but high open water flows can also cause some flooding. In November of 1986, the channel was improved by excavation in order to reduce future flooding.

For this study, computer models of Harry's River were set up to simulate conditions in both open water and ice-related events. The HEC-2 backwater model was used for the open water simulations, and Acres in-house model ICESIM was used for freeze-up and break-up ice events. River cross-sections and a water level profile were obtained in a field program. Winter field observations and ice surveys provided additional data.

Because flood levels can arise from three separate types of events, open water, freeze-up and break-up, the frequency analysis required a joint probability approach. Historic flood levels where available were used for calibration and validation of the computer models, but could not be used directly for frequency analyses because of the recently changed channel configuration. Rather, a calculated series of flood levels for each flood type for each year was generated, based on the new channel geometry.

An annual series of flood levels resulting from each type of event at each cross-section was prepared by simulating the historic temperature and flow conditions. The three series of maximum levels were then analyzed to estimate a frequency curve for each type of event. A combined frequency curve was prepared from the results of the joint probability analysis, and used to estimate the 1:20 and 1:100 year recurrence interval flood levels.

Part B: Stephenville Crossing

Stephenville Crossing is located on a barachois at the mouth of a shallow tidal basin known as St. George's River. A narrow opening, known as Main Gut, provides the outlet to Rothesay Bay in the Gulf of St. Lawrence. Flooding can occur from either the St. George's River side due to high tides and high fresh water inflows, or from the Rothesay Bay side as a result of high tides and high onshore winds.

Water level data were available from the Marine Environmental Data Service (MEDS) for some coastal locations, but not for St. George's River. As part of the field program for this study, a water pressure gauge was deployed for a period of about three weeks to obtain water level data in St. George's River.

Rothesay Bay: Flooding from the Rothesay Bay side results from the combination of atmospheric and astronomic forcing. Atmospheric forcing due to high winds causes superelevation of the water level in the form of surge, set-up, and runup. Astronomic forcing results in high tide levels. The analysis of flooding from the Rothesay Bay side required the development of a frequency curve for the atmospheric components, which could then be convolved with the probability density function (pdf) of tides.

Developing the frequency curve of the atmospheric components required an innovative approach. Hindcasting the offshore waves was done using standard techniques; however, the configuration of St. George's Bay and Rothesay Bay prevents penetration of long-period waves (greater than about 10 sec) to the coastline. An energy spectral approach was thus adopted to predict the proportion of energy in the offshore wave spectrum that could be transmitted inshore. The inshore wave heights were then estimated from this transmitted energy. Runup and setup were estimated from the wave heights, and a frequency curve was prepared for runup/setup components.

A frequency curve for surge was developed concurrently using the MEDS record for Lark Harbour, a station located on the west coast of Newfoundland about 100 km north of Stephenville Crossing. The two frequency curves, both representing atmospheric forcing components, were added. They were then combined mathematically with the tidal probability density function to produce the required estimates of 1:20 and 1:100 year flood levels, which are 3.0 m and 3.2 m respectively.

It should be noted that the resulting floodlines for the Rothesay Bay side do not apply if the highway embankment fails.

St. George's River: Flood level estimates on the St. George's River side were made considering high levels in Rothesay Bay due to high tides and surge/set-up, and high fresh water inflows into St. George's River itself. A routing model was developed to assess the interchange of water through the Main Gut. It was assumed that high tides, being a non-random process, could coincide with either high surge/set-up levels or high inflows.

Various combinations of surge/set-up and fresh water inflows were modelled. From the results of the modelling, frequency curves were prepared for each of the two types of events, which were dominated by either surge/set-up or fresh water inflows. The two curves were added to produce an overall frequency curve. This combined curve indicated that at the return periods of interest, the fresh water flow effect dominates. The resulting 1:20 and 1:100 year recurrence interval flood levels in Stephenville Crossing are 1.1 m and 1.5 m respectively.

1 - INTRODUCTION

1.1 - General

In 1981, the Government of Newfoundland and Labrador and the Government of Canada entered into a "General Agreement Respecting Flood Damage Reduction". The objective of this agreement is the reduction of flood damages on floodplains along the shores of lakes, rivers and the sea. The agreement recognizes that the potential for flood damage can be reduced by control of the areas prone to flooding. This is accomplished by the identification and delineation of flood prone areas and ultimately the designation of these areas by flood risk mapping wherein only certain conforming development should take place.

The flood risk mapping program consists of four major components: hydrology, hydraulics, topographic mapping and public information. Generally the hydrologic component involves determination of the response, in terms of flood flow, of a watershed to major climatic events such as rainstorms and/or rapid snowmelt. The flood flows for specific probabilities are then used in the hydraulic analysis to define the response of the river reaches under consideration to these flows. The output from the hydraulic studies, in the form of water surface profiles for the 1:20 and 1:100 year recurrence interval floods, is applied to detailed topographic mapping to delineate the areal extent of flood water levels on the floodplain. The final component involves the development of maps, brochures and other interpretative information for the purposes of informing the public, government agencies, private companies, and others of the flood hazard.

The main purpose of this study is to provide the hydrologic and hydraulic components for the identification of flood-prone lands for the communities of Stephenville Crossing and Black Duck

Siding on the west coast of Newfoundland. The study area is shown on Figure 1.1.

1.2 - Scope of Study

Historically, the most severe flooding in the Black Duck Siding area has resulted from ice jams while in the Stephenville Crossing area flooding has been caused predominately by high tides and wave attack.

The Terms of Reference for the study identified two overall objectives.

1. Identify the mechanics, physical processes and factors (physical, hydrometeorologic, oceanographic, hydraulic) responsible for flooding in the Stephenville Crossing and Black Duck Siding areas.

2. Provide reliable estimates of the 1:20 and 1:100 year recurrence interval flood levels and the extent of flooding associated with each.

In carrying out the work, desk top and computer analyses have been combined with field surveys to supplement existing hydrologic, meteorologic, ice and sea state data for the study areas. The study determined the flood levels at Black Duck Siding due to both open water flows and ice restricted flows on Harry's River, and flood levels due to ocean flooding and river flooding in the Stephenville Crossing area.

In this report, the two areas are treated separately. Part A (Sections 2 to 9) describes the flooding in the Black Duck Siding area, and Part B (Sections 10 to 12) describes the flooding at Stephenville Crossing.

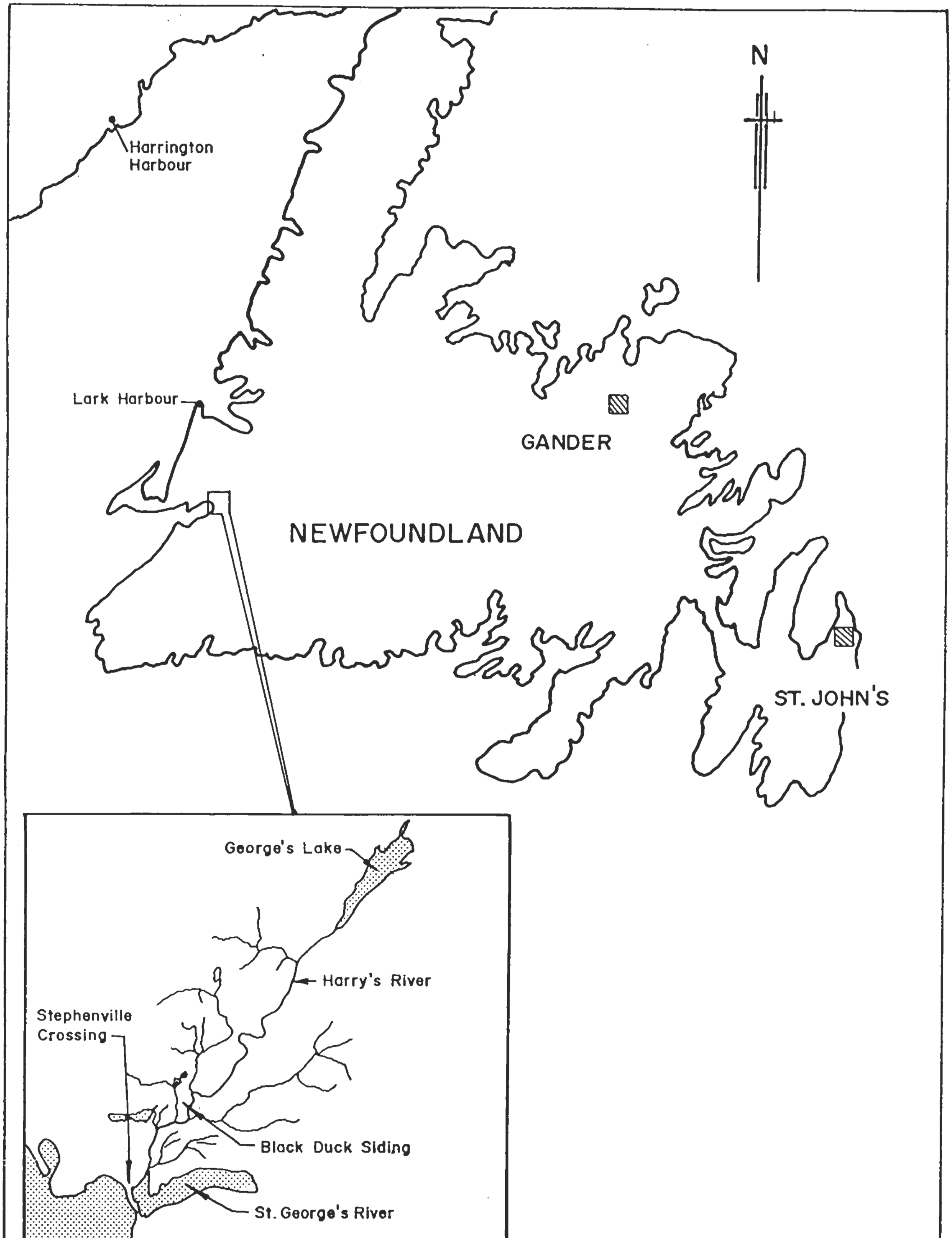

FIG. 1.1

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

STUDY AREA LOCATION

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PART A: BLACK DUCK SIDING, HARRY'S RIVER

2 - STUDY APPROACH

Historically, flooding on Harry's River has occurred in three different modes:

1. Open water flow, usually during the summer or fall.

2. Ice jams during ice formation in early winter.

3. Ice jams during the break up of the ice cover in mid-winter or spring.

Since flooding is caused by ice jams in some years, the maximum annual flood level is not necessarily associated with the maximum flow.

The approach adopted in this study was to carry out frequency analyses for the different types of floods and combine these to obtain the return period/water level relationship. The maximum water level for any particular year may result from one of the principal processes (open water, freeze-up, or break-up) or from a combination of two or three processes, each causing the maximum level at a different location of the study reach.

The guiding methodology adopted in this study is described as follows:

1. For each year

- determine the maximum open water flood levels using a standard backwater model (HEC-2)

- determine whether a freeze-up event is likely to occur in each year. If so, determine the associated water levels using Acres River Ice Simulation Model (ICESIM).

- determine whether a break-up event is likely to occur in each year. If so, determine the water level using Acres River Ice Simulation Model (ICESIM).

2. Perform a frequency analysis at various locations along the reach for each type of event. Obtain the 1:20 and 1:100 year water levels by combining the probabilities to determine the probability of exceedence/elevation curve.

The water levels obtained by mathematical modeling take into account changes made to the river in late 1986 as a result of excavation and berm construction by the Department of Environment, and therefore do not duplicate historical flood levels.

The following sections describe the background information available (Section 3), the field program (Section 4), the channel modifications (Section 5), and the hydrologic analysis undertaken (Section 6). The subsequent two sections describe the methods and the results from the open water (Section 7) and ice related (Section 8) flood level determination. Section 9 summarizes the hydrotechnical analysis for the Black Duck Siding area including the combination of the various events to determine the 1:20 and 1:100 year flood lines.

3 - BACKGROUND INFORMATION

3.1 - General

Black Duck Siding is a small community located approximately 15 km east of the town of Stephenville on the west coast of Newfoundland. There are approximately 20 houses in the community, with farming the main occupation of most residents. Two dairy farms are in operation in the area, as well as a motel which is open during the summer tourist season.

The community has been built along the banks of Harry's River, which flows from George's Lake to St. George's River in Stephenville Crossing. The study area is shown in Figure 3.1. This area is very flat with few hills, and many areas on the north bank have been cleared for farming. Harry's River itself has an average width of approximately 60 m and an average flow of 27 m^3/s. There are numerous shoals and, during low flow conditions in the summer, it is possible to walk across the river in many locations.

Residents report that substantial flooding has occurred four or five times since 1936, although some earlier floods may have been the result of log jams. The river has not been used for log drives for many years.

The most severe recent flood event was an ice-jam related flood which occurred during February 1984. Damage caused by this flood was extensive; estimated costs were reported to be of the order of $100,000. An ice jam related flood also occurred during the spring of 1979, although no estimate of damages was reported. The Department of Transportation has also documented an ice related event in February 1971. Ice jammed at the piers of the Route 460 bridge causing scour at the footing and a subsequent collapse of the bridge. Since the 1984 flood, there have been

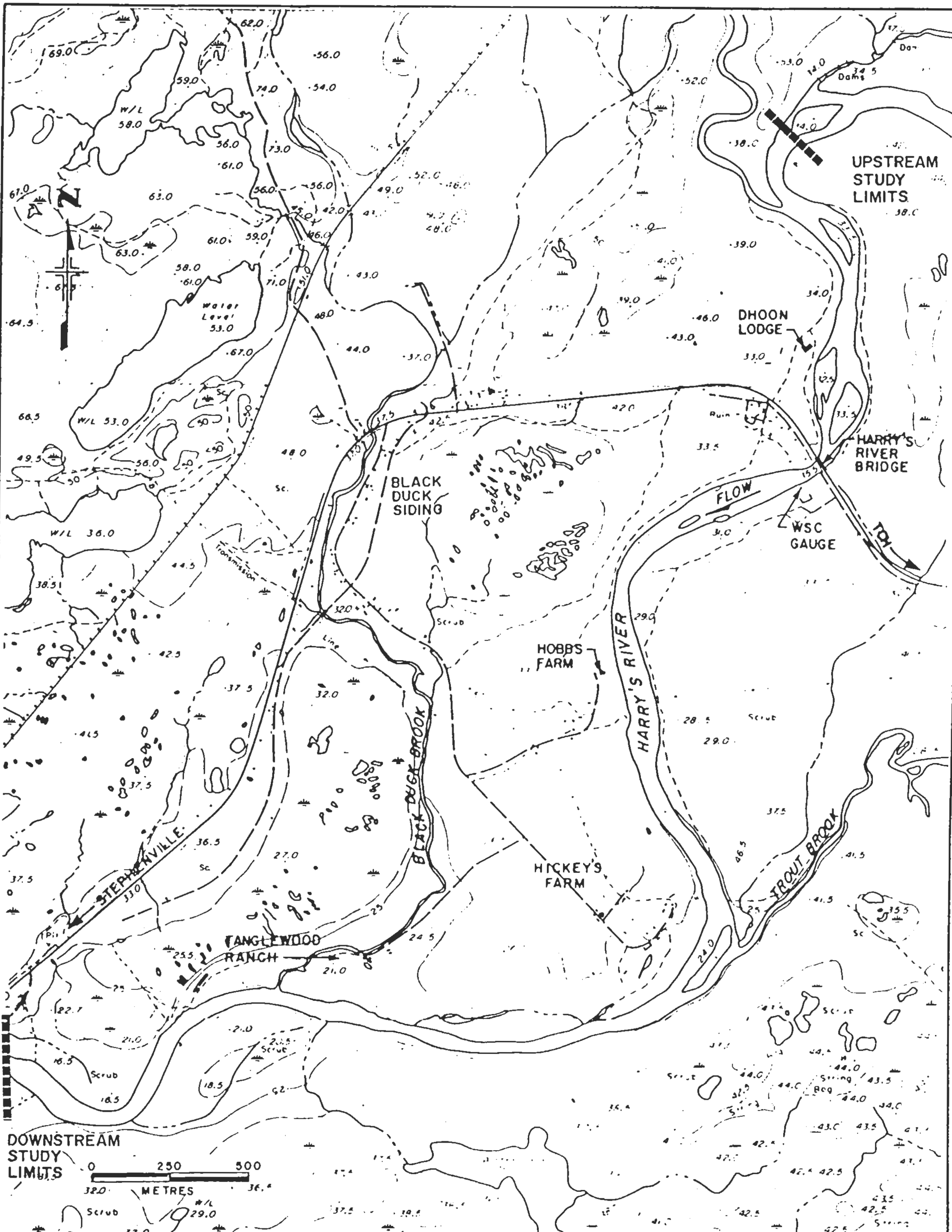

FIG. 3.1

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

BLACK DUCK SIDING STUDY PLAN

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two more ice-related floods, in January 1985 and January 1986. These floods were the result of the ice formation process rather than a break-up jam, and flooding, although significant, was not as widespread as the 1984 event and no damages were reported. Detailed descriptions of these floods are presented in Section 3.2.

The maximum instantaneous open water flow recorded for Harry's River is 688 m^3/s, which occurred on August 3, 1973. Discussions with residents of the area indicate that during this event flooding occurred at Dhoon Lodge upstream of the bridge, and at Hickey's Farm in the lower Black Duck area.

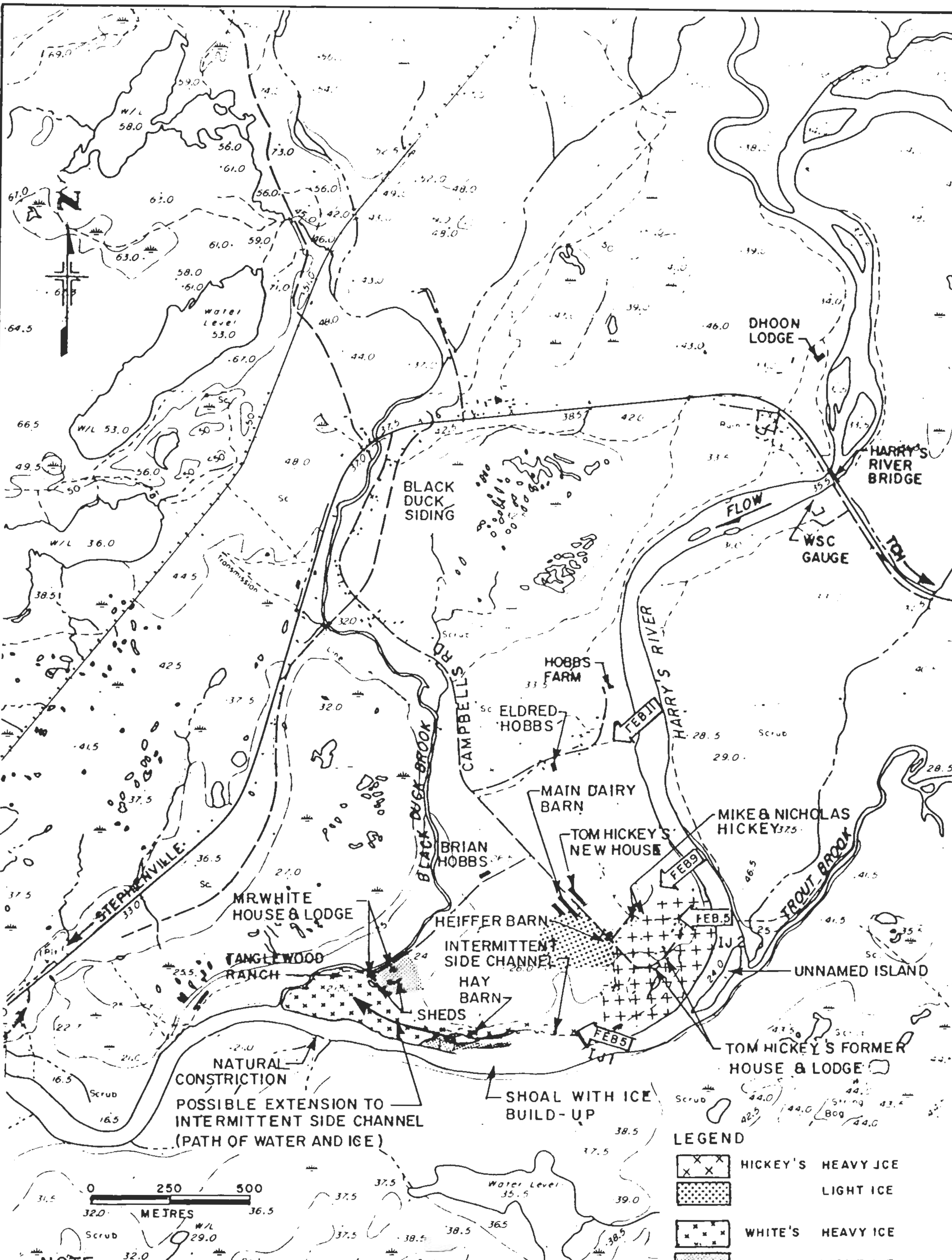
3.2 - Description of Historical Flooding

The following descriptions of the historical ice-related flood events are taken from documentation provided by the Department of Environment (13,14). Reference is made to actual water levels, either recorded or observed, as well as to ice thicknesses and the location of the ice front, where possible.

February, 1984

January, 1984, was colder than average in the Black Duck Siding area, allowing for the formation of a substantial ice cover on Harry's River and its tributaries. Early February brought a mild spell with temperatures reaching a high of 7 C on February 5. On February 4 and 5, a total of 58.3 mm of rain fell, and this was enough to cause ice in the river to break up and move downstream, causing ice jams to form at two locations in the area of Black Duck Siding, as shown in Figure 3.2, as well as at Harry's River Bridge.

The first of two jams which formed at the highway bridge started sometime during the night of February 4. Records from the



NOTE:
INFORMATION FROM NEWFOUNDLAND
DEPARTMENT OF ENVIRONMENT REPORT (1984).

FIG. 3.2

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

AGRES

DOCUMENTED FLOODING FEBRUARY 1984 BREAK-UP EVENT

streamflow gauge show that the water level rose approximately 0.48 m in 10 hours. This ice broke through at 0600 hours and moved downstream, forming a jam at about 1000 hours at the lower part of Hickey's farm, near the inlet of the intermittent side channel. As a result of this jam, water was diverted across Tanglewood Ranch* where it flooded a machinery shed, a lodge and the basement of Mr. White's residence. Residents of the area estimated that the ice depth at the jam was about 2.5 m.

After the first jam was free at the bridge, a second more severe jam formed there, causing the water level to rise 1.38 m in about 45 minutes. This jam caused flooding at Dhoon Lodge, upstream of the bridge. The lodge building and several motel units were damaged by flood waters and moving ice.

At about 1245 hours, the ice broke through and moved downstream, jamming in front of the unnamed island at Hickey's farm. This caused water and large sheets of ice to be diverted onto Hickey's farm, destroying a log cabin in its path. Sheets of ice 25 cm thick and 2 m square were found 300 m from the river channel. The depth of this ice jam was estimated to be approximately 3.5 m. Over the following days, the jam continued to consolidate and, by February 11, water was flowing around the jam over Hobbs' property. This water eventually crossed Campbell's Road and entered Black Duck Brook.

Blasting was initiated in an attempt to help clear a path for the water and ice. By February 16, the blasting operations, combined with higher temperatures, had created a channel through the jam.

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* Tanglewood Ranch is also known as White's property. In this report it is referred to as Tanglewood Ranch, to avoid confusion with White's farm, several kilometres downstream of the study reach.

Although no official figure for damage is available, estimated damages were in the order of $100,000.

January, 1985

Prior to January 4, 1985, there was a cold period of approximately two weeks during which an ice cover had formed from the river's mouth to the lower Black Duck area. On January 4, water was flowing over the fields at Tanglewood Ranch. Since the river channel was blocked with ice, the water had no flow path to the river and hence water levels started to rise as more water flowed over the land. By January 7, the ice front had passed the island in front of Hickey's farm and water was flowing onto this land as well.

Figure 3.3 indicates the formation of the ice cover and the extent of flooding for the January, 1985 event. There is limited information available on actual water levels and ice thicknesses during this period of flooding. The water levels indicated on the map are approximate and were obtained through review of information and photographs provided by the Department of Environment, and through consultation with local residents.

Blasting was initiated during the 1985 event in the hope that, by opening a channel to allow free flow of water, a flood similar to the 1984 event could be avoided. The blasting operation itself concentrated on opening a channel around the island at Hickey's farm. After 5 days of blasting, water levels were reduced by 0.46 m in the flooded areas.

Data obtained during the ice monitoring of Harry's River by the Department of Environment indicates that by January 18, the ice cover had advanced upstream past Harry's River Bridge. Ice thicknesses on January 15 were observed to be approximately 0.3 m downstream of the bridge and 0.8 m at Hickey's farm and Tangle-

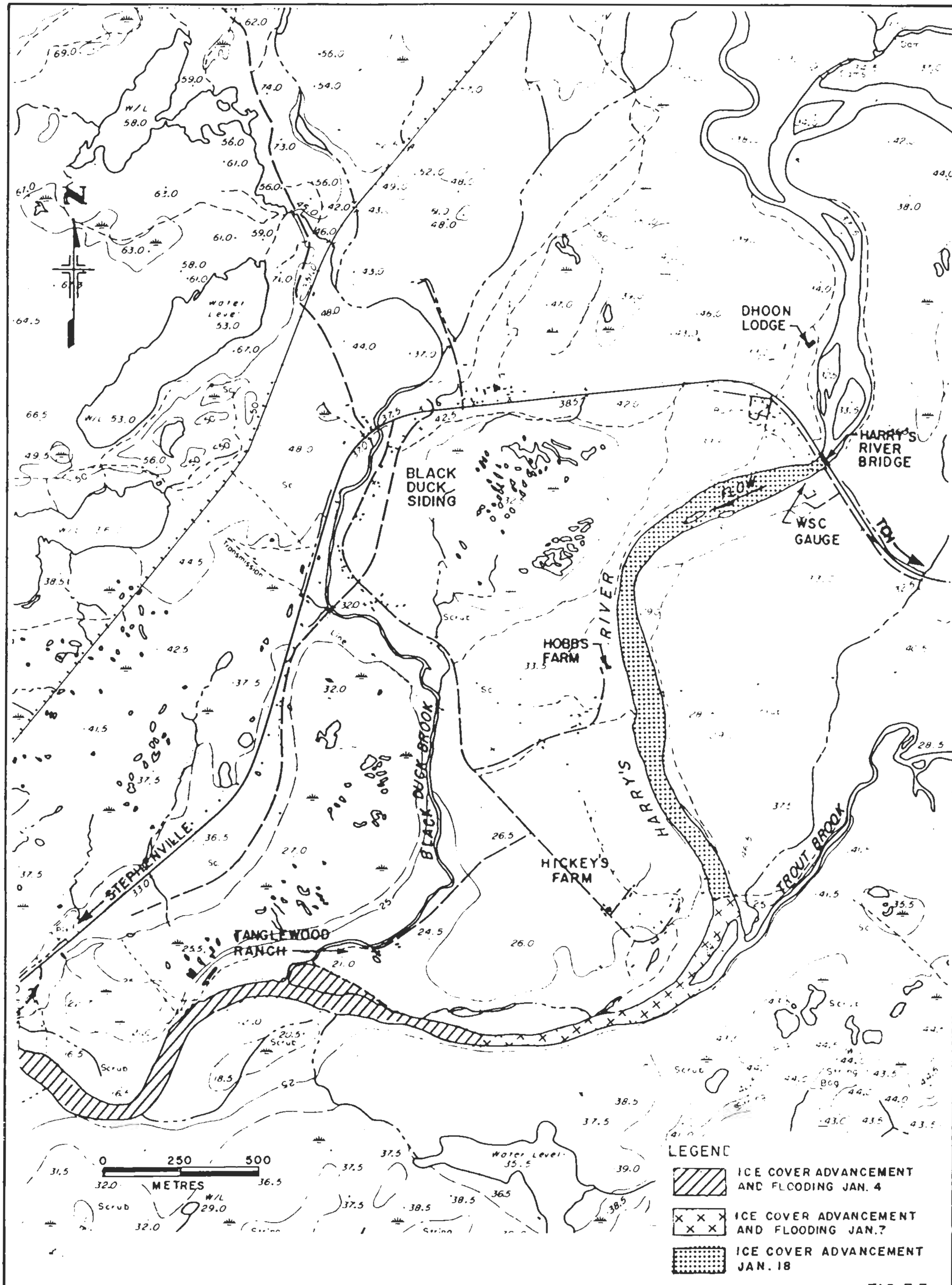

FIG. 3.3

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

DOCUMENTED FLOODING JANUARY 1985 FREEZE-UP EVENT

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wood Ranch. These measurements were taken at selected locations and may be considered representative of typical ice thicknesses in the river, although local accumulations due to shoving and jamming are likely.

January, 1986

Flooding in January, 1986, was similar to that experienced in January, 1985, and was the result of ice accumulation and buildup during formation. However, in 1986, the ice was thicker, resulting in more extensive flooding than in the previous year.

There is limited documentation available for the river ice conditions prior to this event except for information provided by the Department of Environment's Ice Monitoring Program. These observations show that up to December 23, 1985, the river was essentially free of ice except for some build up of frazil and slush ice. By January 6, there was sufficient build up of ice in the river to cause water to be diverted onto Tanglewood Ranch and Hickey's property, as well as onto Hobbs' farm at the transmission line. By January 20, a stationary ice cover had formed up to the bridge and extensive flooding was being experienced downstream. Observations indicate that the ice was about a metre thick in the river.

Blasting was again initiated in an attempt to clear a channel for the ice and water to flow. This time the blasting operation was successful in clearing the entire channel, reducing water levels in the area by 1.37 m. However, by February 11, the channel was again filled up with ice and was completely jammed. Water was being diverted over the floodplain at Hobbs', Hickey's and Tanglewood Ranch with the water level at its highest since the February, 1984 flood. Figure 3.4 shows the extent of flooding and approximate water levels on February 11. The river remained jammed for the remainder of the winter, but this time an emer-

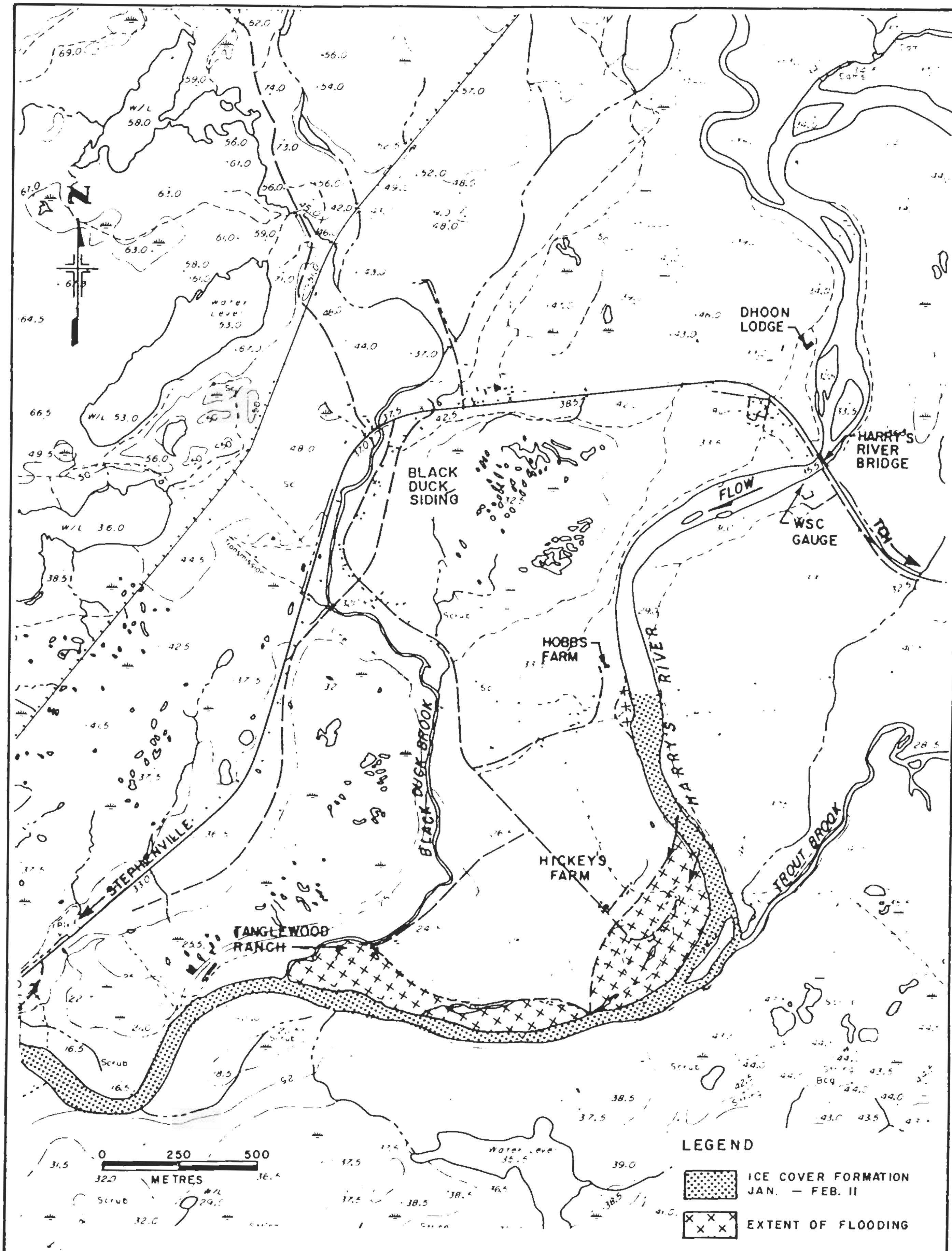

FIG. 3.4

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

DOCUMENTED FLOODING JAN.-FEB. 1986 FREEZE-UP EVENT

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gency was not declared. The water levels gradually dropped as the channel opened up.

3.3 - Existing Data

The following hydrological and meterological data were used in this study:

- Flow data from the Water Survey of Canada (WSC) gauge 02YJ001 on Harry's River at the highway bridge. The processed record includes daily values as well as summaries of maximum instantaneous flows for each year. Processed data was available from March 1968 to December 1985, and unprocessed from 1986.

- Atmospheric Environment Services (AES) temperature data for Stephenville Airport (Station 8403800). This includes hourly temperatures as well as freezing and thawing degree days and was available for February 1942 to March 1986.

- AES precipitation data for Stephenville Airport (Station 8403800). This includes total daily rainfall, snowfall, and total precipitation as well as snow cover on the ground for the period January 1951 to April 1986.

Also, the following reports and drawings documenting historical flooding and recent changes to the river were used:

- Newfoundland Department of Environment documentation of the breakup flood in 1984 entitled "Report on the Flooding in Black Duck: Feb. 5 - Feb. 12, 1984" (14).

- Newfoundland Department of Environment report entitled "Case Study of Ice Related Flooding of Harry's River at Black Duck Siding Newfoundland" (13).

- Newfoundland Department of Environment documentation of the monitoring of ice conditions in Harry's River in the Black Duck Siding area for the winters of 1985 and 1986. This consists of approximately weekly reports of the changes in ice conditions and information on ice thickness and type of cover when available.

- Proposed and "as built" drawings and sections of the channel modifications prepared by Nolan, Davis and Associates.

4 - FIELD PROGRAM

4.1 - Fall Field Survey

A field survey of Harry's River was carried out during the period of October 8 to October 25, 1986. The purpose of the survey was to obtain river cross-sections and water surface profiles to be used in defining the channel geometry and dimensions in the HEC-2 and ICESIM models. A total of 52 cross sections were surveyed along the study reach, over a distance of 5.8 km as indicated in Figure 4.1.

Cross sections were surveyed at locations where the river widened or narrowed, at islands, at slope changes and at locations where tributaries entered the river. Plots of the measured cross-sections are given in Appendix A.

During the field survey, photographs were taken of each cross-section to assist in the selection of appropriate roughness coefficients (Manning's "n") for backwater modeling purposes. A careful record was also kept of the nature of the cross-sections such as the overbank conditions, vegetation and tree cover, and locations of past erosion.

A description of the general floodplain and channel characteristics is also given in Appendix A, along with photographic references.

4.2 - Winter Ice Surveys

During the winter of 1986-1987, a number of field observations of Harry's River were carried out. The purpose of these winter observations was to obtain a better understanding of the

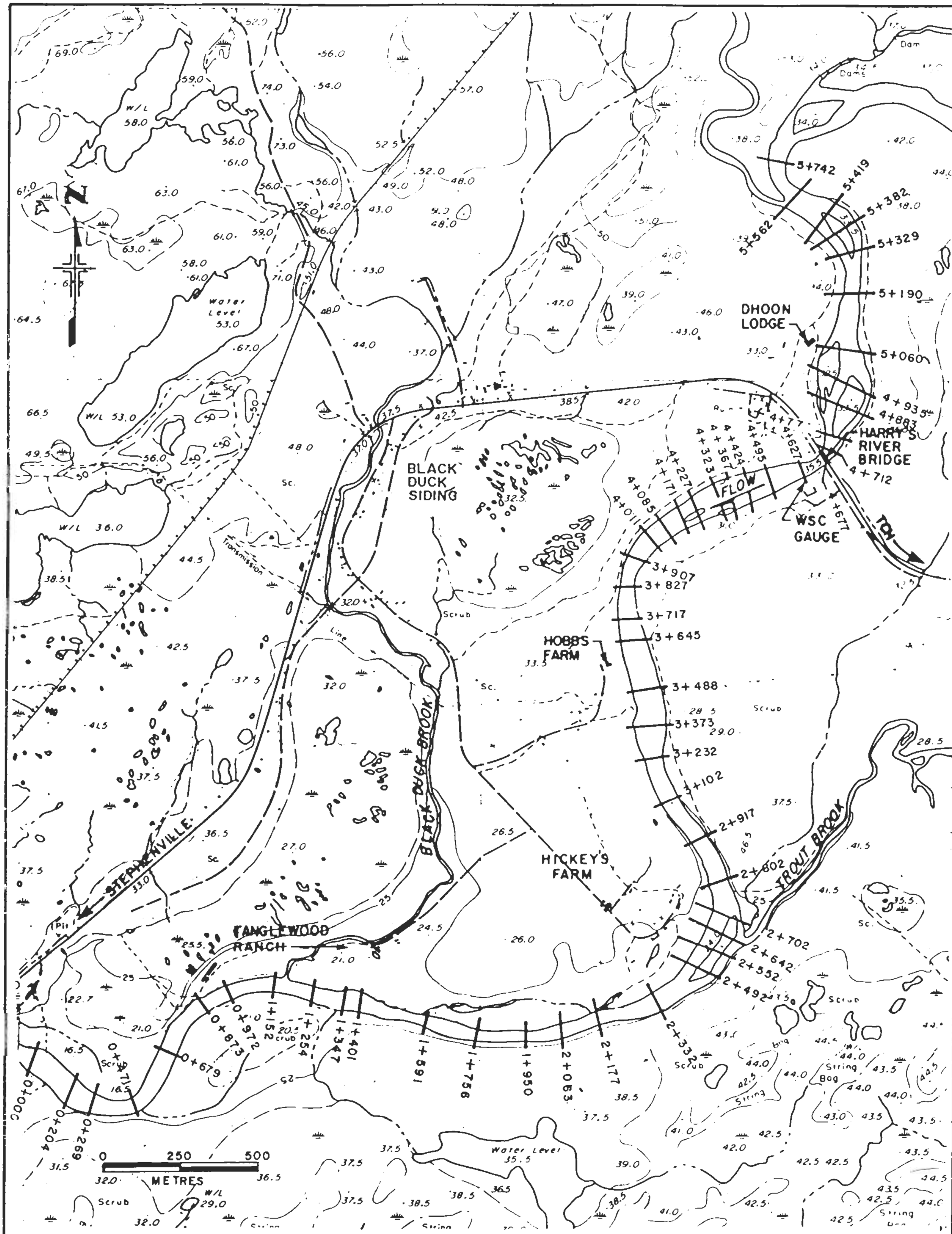

FIG. 4.1

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

SURVEYED CROSS SECTION LOCATIONS - HARRY'S RIVER

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mechanics and processes involved in the ice events. The winter field program consisted of

1. a walkover reconnaissance during freeze-up in December. Pictures were taken and observations were made of approximate ice thicknesses and formation characteristics.

2. a field survey in mid-January. During the survey, a helicopter fly-over was also made from the mouth of the river to the headwaters to determine the extent of ice formation. Observations were documented in the form of photographs and a videotape.

3. a mid-winter survey completed in late February. During this survey, measurements were made of ice thicknesses, snow cover and water elevations at selected locations along the reach. Photographs were also taken of each measured section.

During the course of the winter, continuous contact was maintained with residents of the area to obtain up-to-date information on the ice conditions.

Appendix B contains detailed descriptions of the observations and results of the winter field surveys along with photographic references.

4.2.1 - December 1986 Freeze-Up Reconnaissance

During the period of December 23 to December 31, a walkover survey of Harry's River was completed. The purpose of this survey was to

- observe the mechanisms and characteristics of ice formation
- identify possible control sections within the river reach

- identify locations for measurement of high water levels should a flood occur.

Observations were made where possible along the river reach from Roberts Brook upstream of Dhoon Lodge to Black Duck Brook in the lower study reach. Observations are detailed in Appendix B.

4.2.2 - January 1987 Field Reconnaissance

The January, 1987 field reconnaissance included a helicopter fly-over of the river, followed by walkovers of particular areas of interest. The fly-over was documented in a video recording, as well as in photographs. Appendix B contains a schematic of the river indicating the video footage at different locations as well as a sketch showing general limits of the cover.

A major jam was observed near White's farm, about 1.5 km downstream of the study reach. Considerable shoving appeared to have occurred in this jam. The cover had not progressed very far. Numerous open leads were observed upstream of the jam. Between this area and the bridge, the river was approximately 50 percent covered primarily with border ice with some ice apparently grounded in shallower sections. Upstream of the bridge the cover varied from nearly 100% cover to less than 10% near George's Lake.

4.2.3 - February 1987 Mid-Winter Field Survey

During the period of February 24-25, a field survey was completed to determine the average ice cover thickness and water levels along the river reach. Ice and snow thickness measurements were made at five cross sections. Since no ice events occurred in 1987, this information was not used directly for calibration. The ice thicknesses were typical of those expected during pre-freeze up conditions, i.e. with no thickening due to shoving or deposition. Profiles of the ice and snow thicknesses in these areas are included in Appendix B.

5 - CHANNEL MODIFICATIONS

5.1 - Description

In December 1986, the Department of Environment carried out channel modifications to Harry's River. The purpose of these channel modifications was to improve sections which were believed to cause ice jams and thus reduce the ice related flooding.

The changes to the river consisted of excavating shallow sections of the reach and using the fill material to create berms along the banks of the river.

The main features of the modifications were

1. Removal of the un-named island near Hickey's Farm.

2. Excavation of the shallow section of the river near Tanglewood Ranch.

3. Construction of a berm at Tanglewood Ranch.

4. Construction of a berm at Hickey's farm, upstream of the excavated island.

5. Filling in of a gully on Hickey's farm that previously formed part of a temporary channel during high flows.

The locations of these modifications can be seen in Figure 5.1.

5.2 - Impact of Modifications on Analysis

All events used to calibrate the HEC-2 and ICESIM models occurred prior to these channel improvements and therefore the old

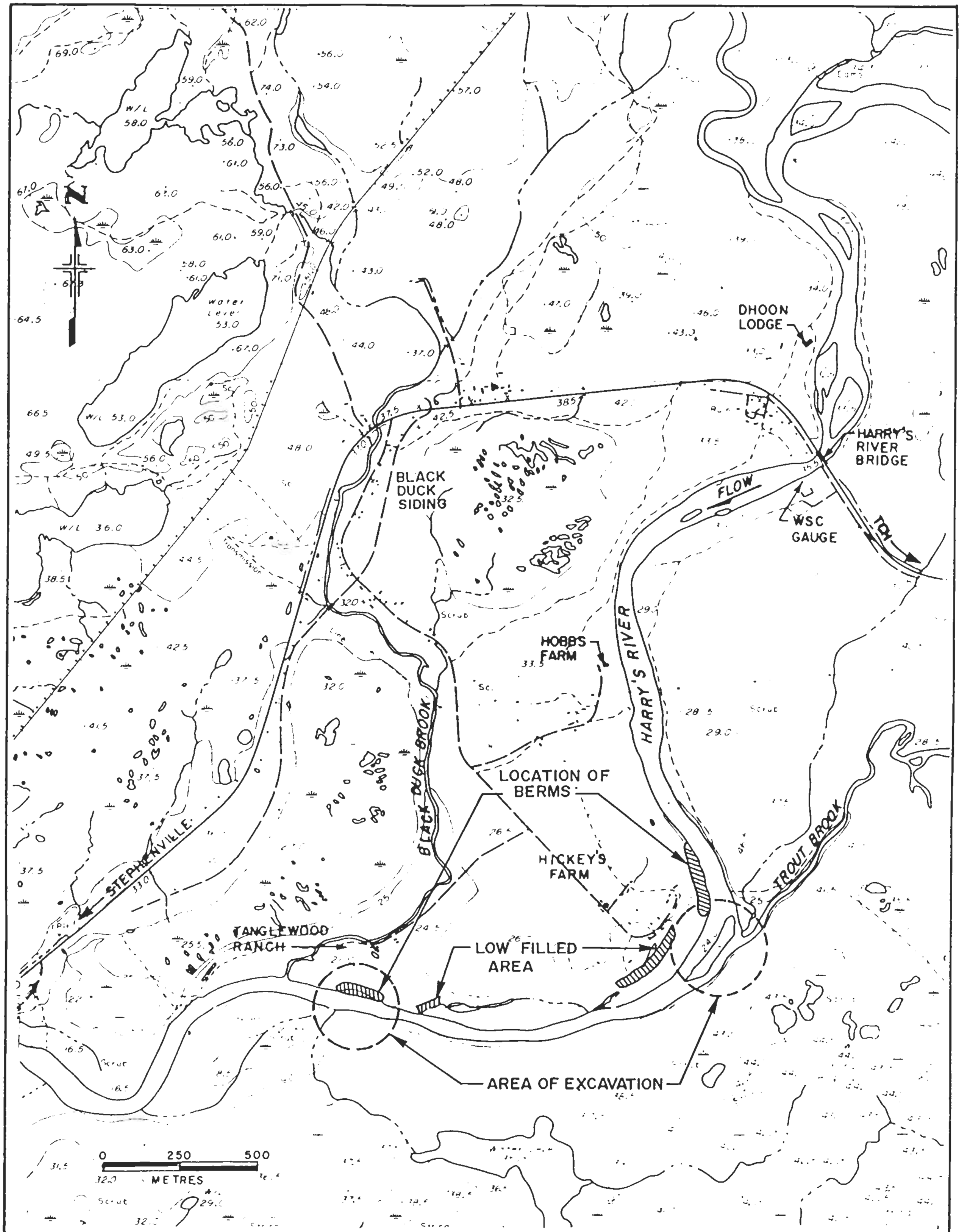

FIG.5.1

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

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NOVEMBER 1986 CHANNEL MODIFICATIONS

sections, as measured in the field survey of October 1986, were used for calibration purposes.

The new channel modifications were then incorporated into the sections in order to derive a revised annual series of high water levels.

The new sections were prepared as follows.

1. Changes due to the excavations were derived from as-built profiles by Nolan, Davis and Associates. The old sections were altered and one section was added to fully define the excavations.

2. Changes due to the berms were obtained from proposed or "as-built" drawings and photographs of the completed work by Nolan Davis. A number of sections had to be altered to fully describe the berms.

In both the hydraulic and ice mechanics analysis, the effect of these channel modifications on the flood levels was evaluated. Sections 7.5 and 8.5 describe the results of this work.

6 - HYDROLOGIC ANALYSIS

6.1 - Methodology

The general approach to this study was to determine high water levels due to 3 types of events, open water, freeze-up and break-up. The 1:20 and 1:100 year recurrence interval flood lines were then determined by combining the results of the three frequency analyses at various locations along the study reach. This section and the following one (Section 7) describe the hydrologic and hydraulic analyses required for the frequency analysis of open water levels.

The analysis proceeded as follows.

1. The open water and ice seasons were delineated based on the streamflow record from the WSC gauge on Harry's River.

2. The inflows from the main tributaries, Trout Brook and Black Duck Brook, were estimated for each of the flows in the annual maximum open water series.

3. Stage discharge curves at various section locations along the river were prepared, taking into account the effect of the inflows from the tributaries. The annual series of open water flood levels at each section was then obtained from the stage discharge curves.

Steps 1 and 2 above are described in the following sections. Step 3 is described in Section 7.

6.2 - Delineation of Ice and Open Water Seasons

The ice and open water seasons were delineated by examining the WSC streamflow gauge records. A "B" in the record indicates that ice is affecting the water levels at the gauge; this was taken to mean that ice is present in the river. Once the open water season was identified for each year, the annual maximum mean daily and instantaneous flows were extracted from the data.

Table 6.1 shows the dates of the first and last recordings of backwater effects due to ice, as indicated in the WSC flow records. The results indicate that the ice season for Harry's River is generally from December to April, and the open water season from May to November, with variations from year to year.

The streamflow data were then analyzed for each year to select the maximum daily flows for the open water season. Table 6.2 presents these results.

Since the guidelines for floodplain mapping (12) recommend that the maximum instantaneous flows be used for this work, a factor of 1.7 was applied to the maximum daily flows to obtain the corresponding maximum instantaneous flow for those years where the maximum instantaneous flows occurred during the ice season or was not measured. This factor was derived by averaging the ratios of the recorded instantaneous flows and the corresponding mean daily flows when records of both were available.

6.3 - Estimation of Open Water Flood Flows

Flow records were available for Harry's River from the WSC gauge (02YJ001), located just downstream of Harry's River Bridge. The annual series of maximum flows could thus be taken directly from this record, with a suitable adjustment for the contribution of the tributary flows from Black Duck Brook and Trout Brook.

TABLE 6.1

FIRST AND LAST RECORDINGS OF ICE - HARRY'S RIVER

Year	Freezing °C-D	Date of First Ice	Date of Last Ice	Comments
68-69	82.9	Jan 05	Feb 11	
69-70	128.4	Jan 19	Apr 07	Open water flows Feb 05 - Feb 13
70-71	60.7	Dec 16	Mar 30	Jan 07 start of continuous ice recordings. Open water flows March 16 - 21
71-72	120.3	Dec 27	Apr 25	
72-73	96.8	Dec 13	Apr 03	
73-74	64.0	Dec 29	Apr 14	
74-75	44.6	Dec 23	Apr 07	
75-76	160.4	Jan 01	Mar 30	
76-77	23.9	Dec 03	Mar 31	Jan 06 start of continuous ice recordings.
77-78	21.3	Dec 06	Apr 20	Jan 08 start of continuous ice recordings.
78-79	88.1	Dec 09	Mar 07	Open water flows Jan 04-06,Jan 08-1 Jan 28-Feb 07.
79-80	55.7	Dec 15	Apr 11	Jan 04 start of continuous ice recordings.
80-81	28.5	Dec 13	Feb 23	
81-82	26.1	Dec 27	Apr 24	
82-83	51.0	Dec 11	Mar 12	Open water flows Dec 18-20,Dec 28 - Jan 03,Jan 11-19, Jan 24-27,Feb 6-7
83-84	82.7	Dec 24	Mar 21	Open water flows Jan 06 -12, Feb 06 -11, Feb 14-Mar 06
84-85	48.8	Dec 21	Apr 15	

TABLE 6.2

MAXIMUM FLOWS FOR OPEN WATER SEASON - HARRY'S RIVER

Year	Date of Max Open Water Flow	Max Mean Daily Flow (m^3/s)	Max Instantaneous Daily Flow (m^3/s)	Ratio of (2)to(1)++
1969	May 21	333	436	1.3
1970	May 27	184	281	1.5
1971	Apr 16	121	183+	-
1972	May 16	228	*	-
1973	Aug 03	260	688	2.6
1974	Nov 01	121	326	2.7
1975	Dec 23	261	430	1.6
1976	Nov 01	166	379	2.3
1977	Nov 14	152	**	-
1978	Jun 09	186	340	1.8
1979	Nov 11	156	244	1.6
1980	Nov 06	219	442	2.0
1981	May 03	131	187	1.4
1982	Apr 30	267	281	1.0
1983	Jun 02	116	**	-
1984	May 15	137	208	1.5
1985	Jun 07	279	402	1.4
			AVERAGE	1.7

* Maximum instantaneous flow not recorded for this year.
** Maximum instantaneous flow occurs in ice season.
\+ Maximum instantaneous flow occurs on October 11.
++ Ratio of max. instantaneous to max. daily mean flow if both occur on same day.

NOTES: 1. Maximum mean daily flows obtained from corrected WSC gauge records.
2. Maximum instantaneous flows determined from WSC publication of streamflow summaries.

Direct proration by drainage areas is acceptable for low and average flows but may not apply to high flows.

The approach taken in this study to adjust for the additional drainage areas was as follows.

1. Estimate the 1:20 and 1:100 year flood flows on Harry's River at 3 locations: the gauge site, the confluence with Trout Brook, and the confluence with Black Duck Brook using the regression equations from the Regional Flood Frequency Analysis of Newfoundland (RFFA) (6).

2. Calculate the ratio of these flows in order to obtain a suitable factor to apply to the annual series of maximum flows at the Harry's River gauge. (An equivalent procedure is to calculate the exponent to be applied to the ratio of drainage areas.)

3. Apply the appropriate ratio to each of the annual maximum open water flows to obtain the flows to be used in the backwater modelling.

This approach is acceptable under the following conditions.

1. The gauged basin should have the same hydrometeorologic and physiographic characteristics as the drainage area for which the estimate is required.

2. The size of the drainage areas should be comparable. As a rule, they should not differ by more than 10 to 25 per cent.

3. The ratio of the calculated flood peaks at the two sites should be used to prorate the gauged values to the desired location by the following equation:

$$QTS = QTFG \cdot \frac{QTRS}{QTRG} \qquad (6.1)$$

where QTS = peak discharge desired for site(S) for return period (T);

QTFG = flood peak for return period (T) from the frequency curve (F) at the gauged location (G);

QTRS = flood peak for return period (T) from the regression equation (R) at the site(s) and;

QTRG = flood peak for return period (T) from the regression equation (R) at the location of the gauge.

This application fulfills both conditions 1 and 2; the increase in area to include Trout Brook is 16 percent, and to include Black Duck Brook is 19 percent. The flows at the tributary confluences were therefore estimated by direct proration of drainage areas for low flows, such as the calibration flow, while the contribution of the tributaries in each annual event was calculated as described above.

In addition, as a matter of general interest, a single station frequency analysis was carried out for Harry's River at the gauge site. However, the 1:20 and 1:100 year open water flood flows obtained in this analysis were not required for the overall determination of 1:20 and 1:100 year flood levels at any location on Harry's River. Results are presented in this report simply for purposes of documentation (See Section 6.3.3).

6.3.1 - Determination of Ratio of Upstream and Downstream Floods

The regression equations developed in the RFFA were used to estimate the 1:20 and 1:100 year recurrence interval flood flows for Harry's River at the gauge and at points downstream of the tributaries as outlined in the previous section. These estimates were used to obtain the ratio between peak flows at the gauge and flows downstream.

The physiographic and hydrometeorologic parameters, as determined from 1:50 000 scale National Topographic Series (NTS) mapping, are presented in Table 6.3. The estimate of mean annual runoff (MAR) was determined from RFFA Figure 3.2.2 (6). A comparison of the parameters listed in Table 6.3 against the permissible parameter ranges as given in the RFFA (6) indicates that all fall within the limits given.

The resulting 1:20 and 1:100 year recurrence interval flood flow estimates for each of the three sites are given in Table 6.4, along with the regression equations used. The ratio of the peak at the tributary confluences to the peak flows at the gauge are included. This table also includes the 95 per cent confidence limits for the estimated flows. These were determined using 1.96 times the standard errors given in Table 6.4 for each of the equations used.

6.3.2 - Estimation of Flood Flows at Confluences of Tributaries

The ratios of the RFFA flood peaks at each downstream site to the RFFA flood peak at the gauge were determined as described in Section 6.3.1. These ratios were then applied to the flood peaks at the gauge obtained from the single station frequency analysis for the same return periods. The peak flood flows at the tributary confluences were then estimated using equation 6.1.

The calculated flows at the tributary confluences for the 1:20 and 1:100 year open water flood flows at the gauge are given in Table 6.5. The ratios used to calculate each flow are also given. Using equation 6.1 is equivalent to applying an exponential factor to the ratio of drainage areas. From the regression analysis results, the average exponent is 0.84 (range 0.75 to

TABLE 6.3

PHYSIOGRAPHIC AND HYDROMETEOROLOGIC PARAMETERS FOR HARRY'S RIVER

Parameters*	Location: Harry's River at gauge	Harry's River at Trout Brook	Harry's River at Black Duck
DA (km^2)	640	742	763
MAR (mm)	1321	1321	1321
ACLS (%)	75	70	70
P (km)	164	188	192
SHAPE	1.81	1.93	1.95

*The following abbreviations are used:

DA = Drainage Area

MAR = Mean Annual Runoff

ACLS =Area Controlled by Lakes and Swamps

P = Perimeter of Drainage Basin

SHAPE = Shape Factor Defined by $(0.28 \times P)/(DA)^{0.5}$

TABLE 6.4

HARRY'S RIVER REGIONAL FLOOD FREQUENCY ANALYSIS - RESULTS

Location	1:20 Year Return Period QP_{20} (m^3/s)	1:20 Year Return Period Ratio to Peak Flow at Gauge	1:100 Year Return Period QP_{100} (m^3/s)	1:100 Year Return Period Ratio to Peak Flow at Gauge
Harry's River at Gauge (DA = 640 km^2)	520 (354,717)	1.0	633 (391,933)	1.0
Harry's River at Trout Brook Confluence (DA = 742 km^2)	590 (402,813)	1.13	729 (450,1075)	1.15
Harry's River at Black Duck Brook Confluence (DA = 763 km^2)	594 (404,819)	1.14	735 (454,1084)	1.16

The equations used are those for the South Region of the Island. These are:

1. $\log QP_{20} = -3.7581 + .8499 \log DA + 2.3604 \log MAR - 1.5248 \log ACLS - 1.6244 \log SHAPE$

   Standard error %: +19.3, -16.2

2. $\log QP_{100} = -3.5915 + .8141 \log DA + 2.4090 \log MAR - 1.6223 \log ACLS - 1.4277 \log SHA$

   Standard error %: +24.2, -19.5

Notes:
1. Values in parenthesis are the 95% confidence limits for the flow.
2. For this study, the above results were only used to obtain the ratio between floods at the gauge and floods downstream, at the confluences.

0.93 as shown in Table 6.5). These exponents are within the range expected, based on experience in other areas.

6.3.3 - Single Station Frequency Analysis Results

A single station frequency analysis was undertaken to estimate the open water peak flows for Harry's River at the gauge for the 1:20 and 1:100 year return periods. The maximum instantaneous flows in each year for the open water season, as given in Table 6.2, were input to the FDRPFFA Computer Program (10). The analysis results are presented in Appendix C. The coefficients of skew and kurtosis of -0.16 and 3.35 for the 3 PLN distribution compare well with theoretical values of 0.0 and 3.0, so the floods predicted from the 3 PLN distribution were used. The RFFA also reports that for almost all the rivers in the south region of Newfoundland, including Harry's River, a three parameter log normal (3 PLN) distribution provides the best fit to the data.

Table 6.6 presents the results of the single station frequency analysis and for the 3 PLN distribution and Figures 6.1 and 6.2 show the plots of the data and the fitted distributions for the maximum mean daily and maximum instantaneous series. The 1:20 and 1:100 recurrence interval flood flows of 632 m^3/s and 868 m^3/s compare to 617 m^3/s and 825 m^3/s as calculated in the RFFA using the same method, but without the last two years' data.

6.3.4 - Estimation of Flows for Annual Series

The annual series of flood flows along the river was then estimated, using the average factors of 1.14 and 1.15 respectively to calculate the flows at the Trout Brook and Black Duck Brook confluences. These flows are presented in Table 6.7 for each year of record as well as the 1:20 and 1:100 year return period flood flows at the tributary confluences, also estimated using the average factors of 1.14 and 1.15. In the case where the

TABLE 6.5

FLOOD FLOWS AT TRIBUTARY CONFLUENCES

Location	DA (km^2)	QP20 (m^3)	Ratio of Flows*	Equivalent Exponent of DA*	QP100 (m^3/s)	Ratio of Flows	Equivalent Exponent of DA
Harry's River at WSC Gauge	640	632	-	-	868	-	-
Harry's River at Trout Brook	742	714	1.13	0.84	998	1.15	0.94
Harry's River at Black Duck Brook	763	720	1.14	0.75	1007	1.16	0.83

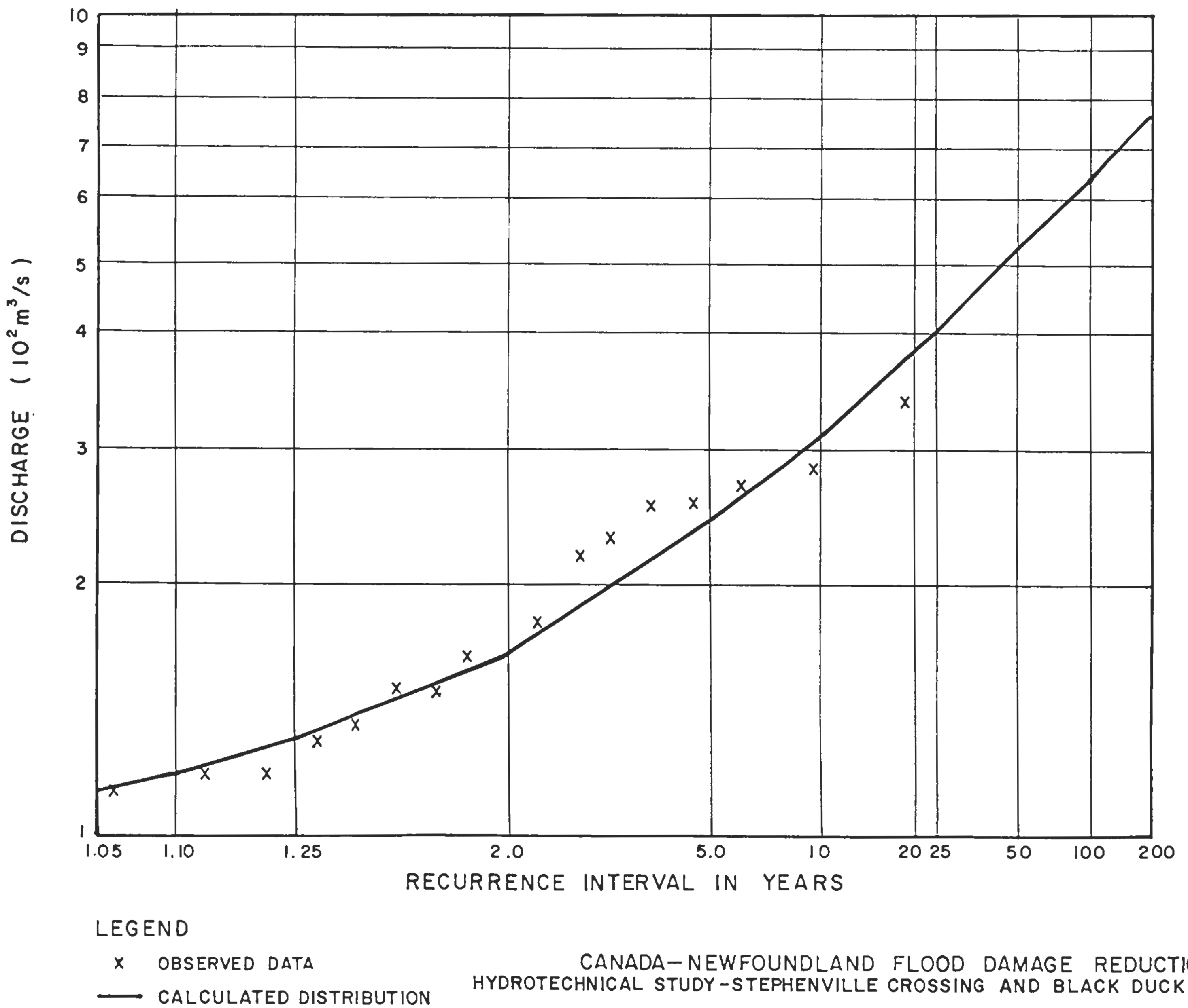
* obtained from RFFA analysis

TABLE 6.6

SINGLE STATION FREQUENCY ANALYSIS - RESULTS

Location	Series	1:20 Year Flow (m^3/s)	1:100 Year Flow (m^3/s)
At Harry's River Gauge	(1) Annual max. Instantaneous	632 (407,979)	868 (456,1653)
	(2) Annual max. Mean Daily	389 (228,664)	619 (277,1387)
	Ratio of (1) to (2)	1.6	1.4

Note: The values given in parenthesis are the upper and lower 95% confidence limits for flow.



CANADA–NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
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3 PLN DISTRIBUTION–MAXIMUM MEAN DAILY SERIES

FIG.6.1

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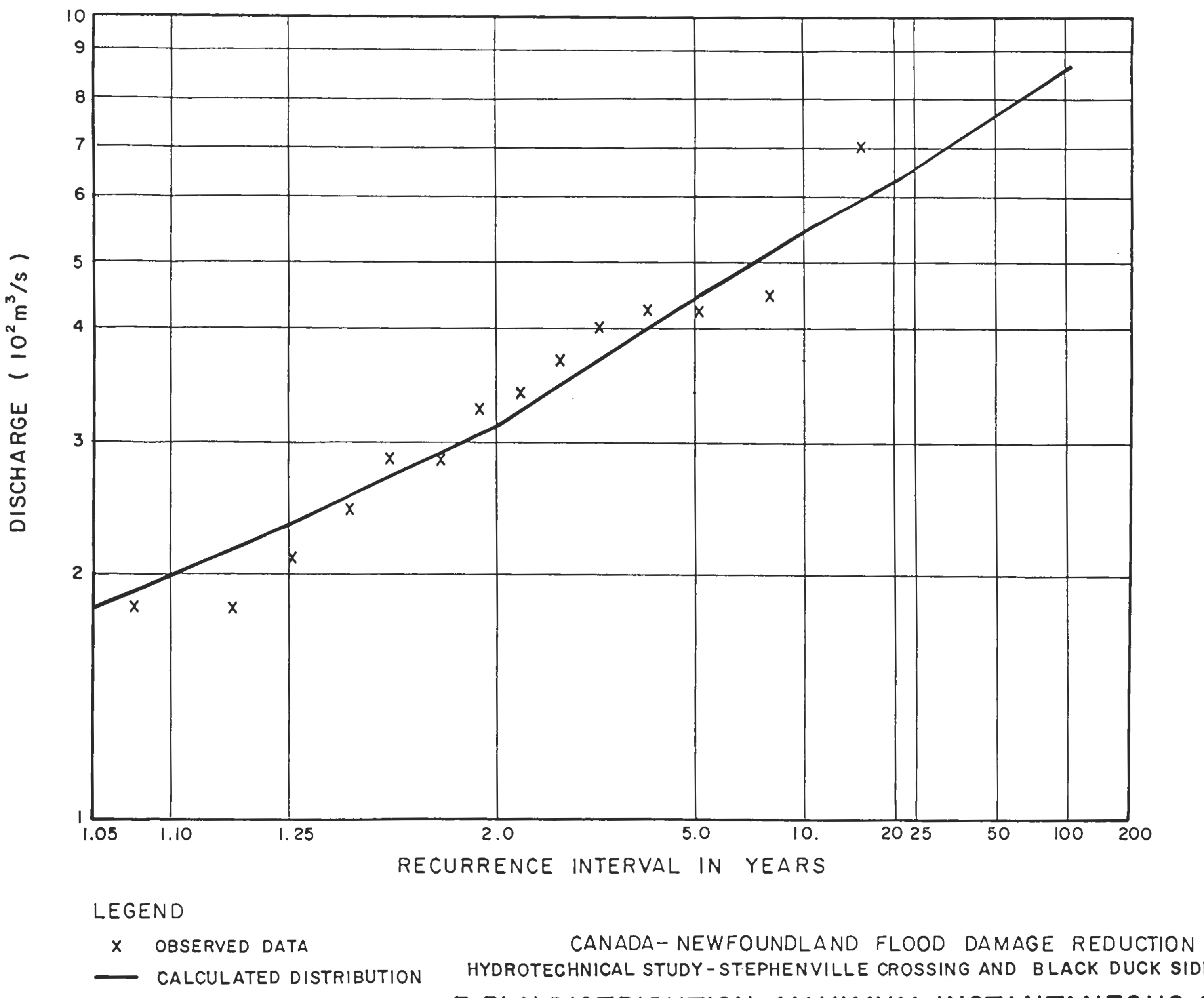

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HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

3 PLN DISTRIBUTION - MAXIMUM INSTANTANEOUS SERIES

FIG. 6.2

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TABLE 6.7

HARRY'S RIVER ANNUAL SERIES OF OPEN WATER FLOWS

Instantaneous Flow (m^3/s)

Year	Harry's River at Gauge	Harry's River** at Trout Brook	Harry's River** at Black Duck Brook
1969	436	497	501
1970	281	320	323
1971	183	209	210
1972	388*	442	446
1973	688	784	791
1974	326	372	375
1975	430	490	495
1976	379	432	436
1977	258*	294	297
1978	340	388	391
1979	244	278	281
1980	442	504	508
1981	187	213	215
1982	281	320	323
1983	197*	225	227
1984	208	237	239
1985	402	458	462
1:20 yr return period flood flow	632	720	727
1:100 yr return period flood flow	868	990	998

* Maximum instantaneous flows determined by applying a factor of 1.7 to the maximum mean daily flow.

**Flows determined by applying factors of 1.14 and 1.15 to the instantaneous flows recorded at the WSC gauge.

maximum instantaneous flow was not available for the year of record, a factor of 1.7, as determined in Table 6.2, was applied to the recorded maximum mean daily flow for the year.

This annual series of flow was then used to estimate the annual series of maximum water levels, using the stage discharge curves derived by the hydraulic analysis presented in Section 7.

7 - OPEN WATER HYDRAULIC ANALYSIS

7.1 - General

The purpose of the open water hydraulic analysis was to derive the open water surface profiles along the study reach, using the results of the field survey and the results of the hydrologic analysis presented in Section 6.

The HEC-2 computer model was selected for this analysis. This model was selected because it represents the state-of-the-art for the computation of water surface profiles for steady state conditions in open channels. It was developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Centre to compute water surface profiles for natural or man-made channels, assuming that the flow is steady and gradually varied. The effects of various hydraulic structures such as bridges, culverts, weirs, embankments and dams may be considered in the computations and hence the model can be easily used to evaluate possible remedial measures. The basic computational procedure involves the solution of the one-dimensional energy equation for the total energy at each cross-section and the application of Manning's formula for the friction head loss between cross-sections.

The HEC-2 model and its theoretical background are described in detail in the users' manual (15). The release used for this study was issued November 1974, updated May 1984.

The HEC-2 model was set-up using the pre-excavation cross sections obtained during the October, 1986 field survey. The model was calibrated using the water surface elevations measured during the fall field survey along with the observed floodplain and river characteristics. The calibrated model was modified to represent the changed conditions of the river since the recent excavation. It was then used to predict flood profiles resulting from maximum open water flows in each year of record. The calibrated HEC-2 model also provided the basis for the channel cross-section data used in ICESIM.

The following sections describe the set-up, calibration and application of the HEC-2 model for this study.

7.2 - HEC-2 Input Data and Model Set-up

The HEC-2 model was set-up for Harry's River using the river cross-section data obtained during the field survey. The sections were coded as if looking upstream along the watercourse, and were input starting at the downstream end of the study reach.

All field measured cross-sections were coded, using the same numbering as given in Figure 4.1. Distances between the cross-sections as measured in the field were input as well as the stations for left and right overbanks, where the natural channel ended and the treeline started. Abrupt changes between adjacent cross-sections and the bridge entrance and outlet were accounted for through adjustments in the expansion and contraction coefficients, using recommended values given in the HEC-2 users' manual.

At very high flows, the flood plain can extend for long distances over relatively flat terrain. During the field survey, cross-sections were extended to an identifiable landmark, so that cross-sections could be extended horizontally using 1:2500 scale mapping where necessary. In cases of extremely high flows and water levels some sections could not be extended horizontally to a sufficient distance within the study area to contain the depth of water calculated (Sections 2+492 - 2+917 at Hickey's Farm, Sections 4+712 - 5+419 at Dhoon Lodge). In these cases the model automatically extended the section vertically at the extreme edge of the extended overbank. At this point, the widths are so great and the flooded depths so low that errors introduced are very small.

7.3 - Model Calibration

The HEC-2 model as set-up for Harry's River was calibrated by modifying the channel and floodplain roughness coefficients (Manning's n) until acceptable simulation accuracy was achieved. This was accomplished by comparing the simulated water levels calculated by the HEC-2 model with the water levels that were measured during the field survey.

A flow of 32.1 m^3/s was used in the model for the calibration event as obtained from the WSC gauge 02YJ001 just below Harry's River Bridge. This was the flow at the time the water surface profile was measured during the fall field survey. Since this flow is only slightly higher than the mean annual flow, inflows at Trout Brook and Black Duck Brook were estimated by direct proration of drainage areas. The final flows used in the model for calibration were 32.1 m^3/s downstream from the bridge to Trout Brook, 37.2 m^3/s from Trout Brook to Black Duck Black Brook and 38.1 m^3/s from Black Duck Brook to the downstream end of the reach.

Estimates of Manning's n values were initially made for the channel and overbanks from the photographs and observations taken in the field. In cases where islands were visible, the horizontal variation in Manning's n across the section was input since the presence of the islands affects hydraulic characteristics at that section.

Section 0+204 was taken as the starting section for the HEC-2 modeling. This section was chosen since it was identified as a control section with the flow always passing through critical depth. The section is located at the end of a deep reach with very fast moving rapids downstream. The average slope downstream of the section is 0.0065 compared to an average slope of 0.003 in the river. For the HEC-2 runs, the starting depth was always set at critical depth by the model.

Calibration of the HEC-2 model was achieved by making adjustments to the Manning's n values for the channel and right and left overbanks until the simulated water levels were in close agreement with the observed. No fine tuning of the n values selected for the overbanks could be made since there is no water surface profile available for flood conditions in the overbanks.

The results of the calibration runs are presented in Table 7.1 along with the observed water levels and the calibrated n values determined for the channel. Figure 7.1 compares the measured and calibrated water surface profiles.

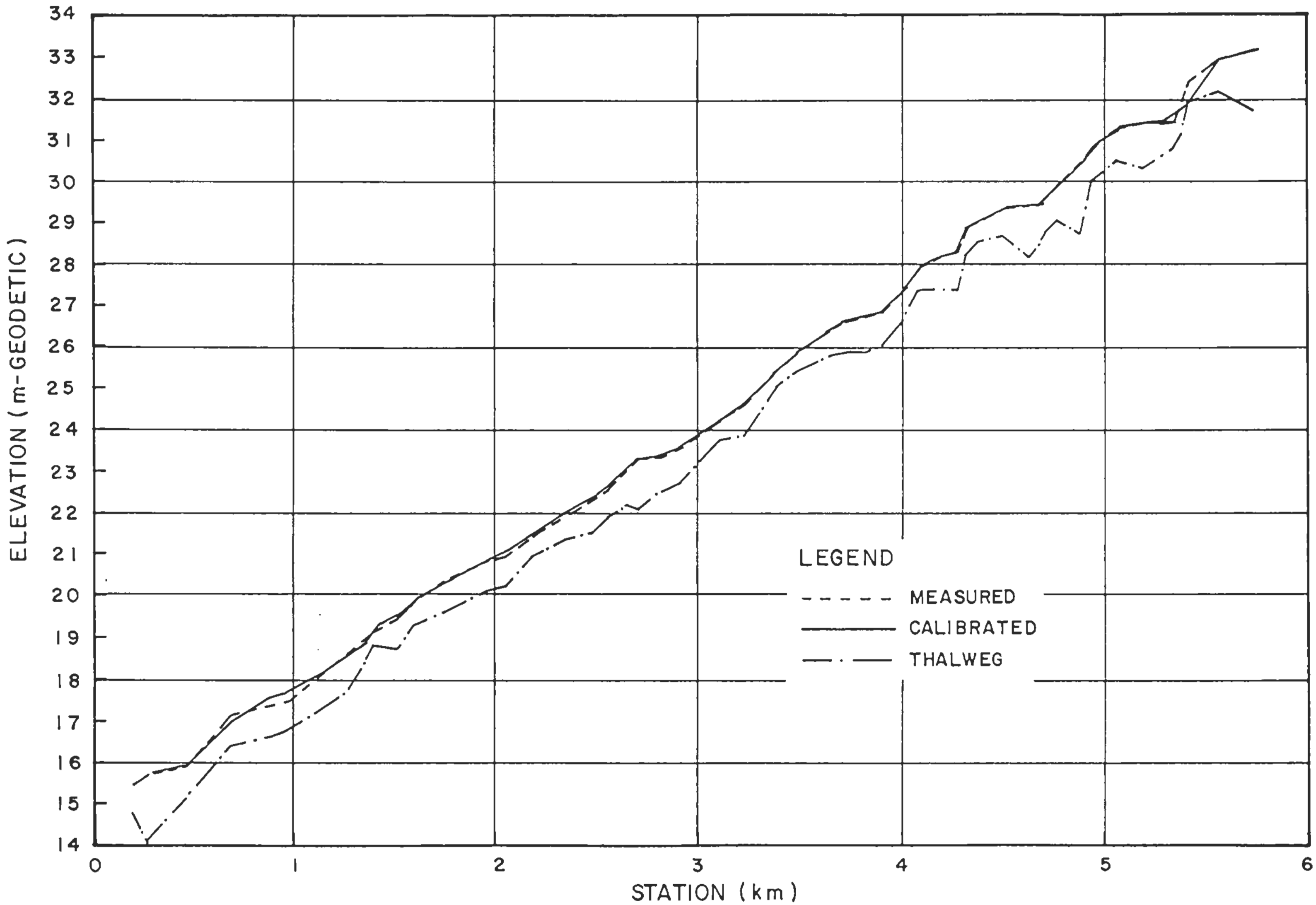
Comparison of the calibrated Manning's n values with those calculated for Blanche Brook, Stephenville in a previous hydrotechnical study (20) indicated that the values for Manning's n determined for Harry's River were generally lower than those found for Blanche Brook. Values of Manning's n for Blanche Brook range from 0.040 to 0.045, compared with 0.025 to 0.050 for Harry's River. This was expected since Harry's River is a larger river. As well, the calibrated Manning's n values were within the range referred to in the literature (e.g. Figure 5.5, photograph 12 in Chow(7) illustrates a cobble-bottom channel similar to Harry's River, with a Manning's n of 0.028). Chow(7) also recommends a minimum value for Manning's n of 0.025 for large streams were the top width at the flood stage is greater than 30 m, as is the case for Harry's River.

TABLE 7.1

MEASURED AND CALIBRATED HEC-2 WATER LEVELS - HARRY'S RIVER

STA	THALWEG ELEV (m)	MEASURED WATER EL. (m)	CALIBRATION RESULTS Ch n*	L/B n*	R/B n*	WATER EL (m)
5742-5382	31.75-31.22	33.21-31.83	0.025	0.10	0.10	33.23-31.82
5329-5060	30.83-30.55	31.47-31.27	0.025	0.08	0.10	31.58-31.31
4935-4883	30.00-28.75	30.83-30.42	0.050	0.08	0.10	30.80-30.49
4773	29.07	29.94	0.040	0.08	0.10	29.94
4712	28.77	29.69	0.040	0.10	0.10	29.74
4677-4367	28.50-28.51	29.50-28.96	0.030	0.10	0.10	29.50-28.99
4323-4227	28.22-27.39	28.84-28.34	0.045	0.10	0.10	28.79-28.34
4171-3232	27.36-23.91	28.17-24.63	0.025	0.10	0.10	28.16-24.63
3102	23.76	24.24	0.030	0.10	0.10	24.20
2917-2492	22.76-21.60	23.60-22.41	0.030	0.05	0.10	23.57-22.43
2332-1756	21.37-19.65	21.87-20.40	0.025	0.08	0.10	21.97-20.36
1591-1347	19.29-18.39	19.86-18.91	0.025	0.07	0.10	19.86-19.00
1254	17.68	18.58	0.030	0.07	0.10	18.53
1152	17.34	18.14	0.025	0.07	0.10	18.16
972-204	16.80-14.80	17.47-15.53	0.025	0.10	0.10	17.66-15.42

*L/B and R/B refers to the left and right overbanks, looking upstream. Ch refers to the Channel.



NOTE:

CALIBRATED FLOW 32.1 m^3/s
AT THE WSC GAUGE

FIG. 7.1

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

MEASURED AND CALIBRATED WATER SURFACE PROFILES

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7.4 - Model Verification and Sensitivity Testing

A water surface profile was not available for high flows in Harry's River. However, water level data are available at the WSC gauge. The maximum recorded water level was therefore compared with the predicted level to check that the model could be used to predict levels at high flows as well as low.

The maximum level recorded at the WSC gauge location was 30.44 m (geodetic), on May 1, 1982. The corresponding flow for this level is 198 m^3/s. The HEC-2 model predicted a water level of 30.47 m at the gauge location (Section 4 + 627) for this flow, indicating good agreement at high flows.

The sensitivity of the modeling results to changes in the roughness coefficients, the distance between cross sections, the computed peak discharge, and the contraction and expansion coefficients was tested. It was found that the modeling results are most sensitive to the peak discharge, with differences of -6% to +8% of the calibrated values for the -95% and +95% confidence level flows. Modeling results were least sensitive to the use of interpolated sections (maximum difference observed less than 1%), and variations in the expansion and contraction coefficients (no observed difference in levels). Variations in the channel and roughness coefficients were found to have some effect on the computed water surface profile, particularly at higher flows where there is inundation of water over the flood plain. The results of these sensitivity runs are detailed in Appendix D.

7.5 - Effect of Channel Modifications

The calibrated HEC-2 model was run using both pre- and post-excavated sections to determine the effect of the channel modifications on the open water surface profile.

At lower flows, the channel modifications result in lower water levels in the downstream reach. Table 7.2 presents the results of these runs for the calibration flow of 32.1 m^3/s at the gauge.

Figure 7.2 compares the open water surface profiles for the pre- and post-excavation cases, using the calibration flow of 32.1 m^3/s. The water levels at Hickey's farm (Sections 3+232 to 2+552) were lower for the post-excavation case, with the maximum difference in predicted water levels being 0.45 m at Section 2+802. Slightly lower water levels were also predicted immediately upstream of Tanglewood Ranch (Sections 1+152 to 1+401). The channel modifications have some effect on the water levels at high flows. For the 1:100 year flood flow of 868 m^3/s at the gauge, calculated water levels averaged 0.3 m lower in the area of Tanglewood Ranch and at Hickey's farm.

7.6 - Determination of Open Water Flood Profiles Using HEC-2.

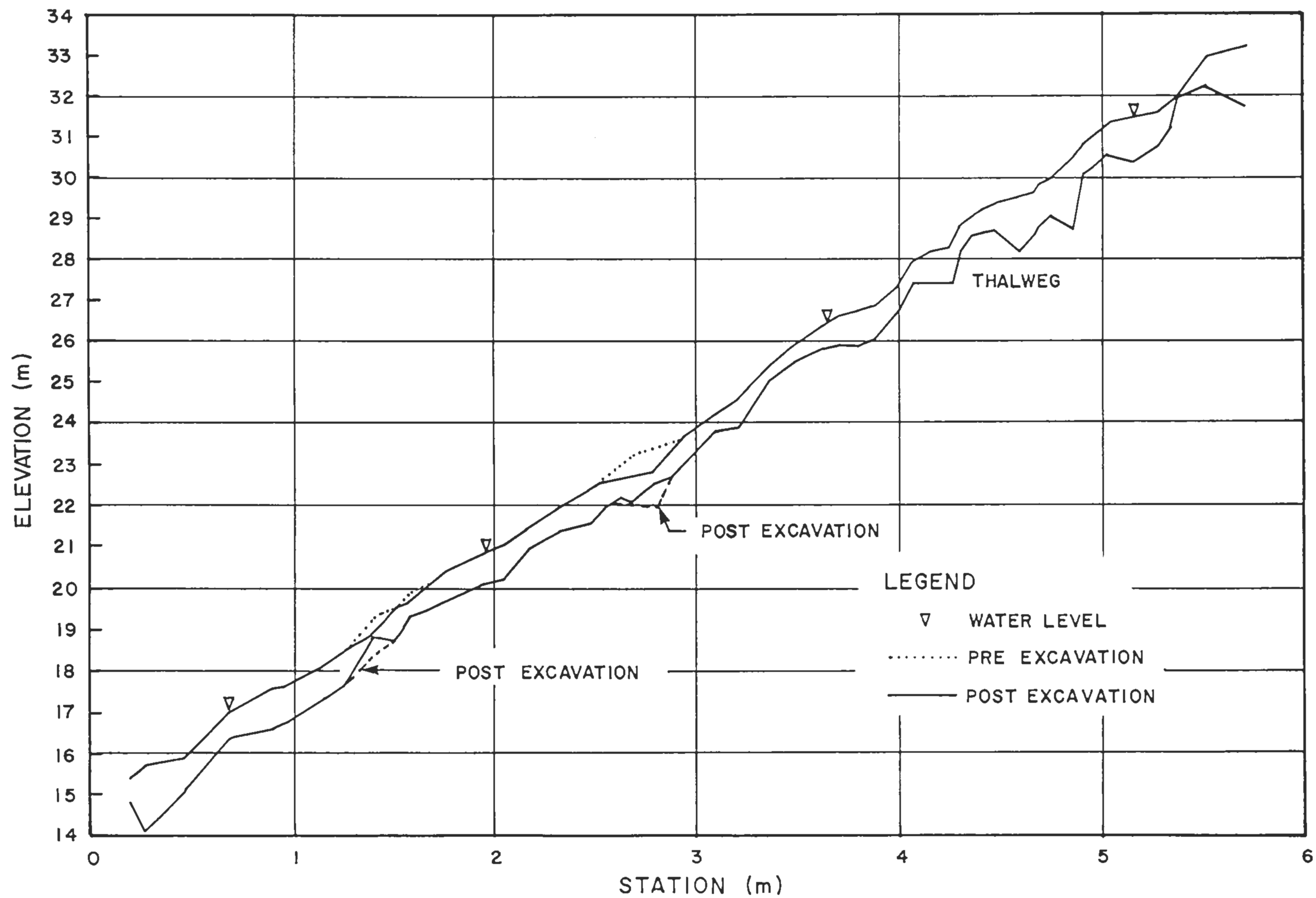
Once the HEC-2 model was calibrated and tested, it was used to generate open water stage-discharge curves at section locations along the river. The model was run for a range of flows, including the 1:20 year and 1:100 year return period flood peaks. All runs were done using the cross sections after channel modifications, to represent the existing condition of the river.

Key sections, which would allow for interpolation of water levels between sections were selected for the frequency analysis of water levels. The calculated stage discharge curves at the sections used in the frequency analysis are given in Appendix E. A summary of the predicted water levels for the annual flows given in Table 6.7 is presented in Table 7.3. The water levels calculated for the 1:20 and 1:100 year return period open water flood flows (632 m^3/s and 868 m^3/s respectively at the gauge) are also included in Table 7.3 for comparison.

TABLE 7.2

EFFECT OF CHANNEL MODIFICATIONS ON OPEN WATER SURFACE PROFILE AT CALIBRATION FLOW

STA	PRE-EXCAVATION WATER EL. (m)	POST EXCAVATION WATER EL. (m)
5742-5382	33.23-31.82	33.23-31.82
5329	31.58	31.59
5190	31.47	31.98
5060-4773	31.31-29.94	31.31-29.94
4712	29.74	29.76
4677	29.50	29.57
4627	29.45	29.54
4495-3232	29.33-24.63	29.33-24.62
3102	24.20	24.23
2917	23.57	23.51
2802	23.38	22.83
2702	23.24	22.73
2642	23.01	22.68
2552	22.63	22.57
2492-1950	22.43-20.86	22.43-20.84
1756	20.36	20.42
1591	19.86	19.75
1521	19.56	19.56
1401	19.30	18.92
1347	19.00	18.82
1254-204	18.53-15.42	18.52-15.42



NOTE :

CALIBRATED FLOW 32.1 m^3/s
AT THE WSC GAUGE

FIG. 7. 2

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

EFFECT OF CHANNEL MODIFICATIONS ON OPEN WATER SURFACE PROFILE

ACRES

TABLE 7.3

MAXIMUM OPEN WATER ELEVATIONS (m) 1969-1985*

	SECTION NUMBER**														
YEAR	0471	0873	1152	1347	1591	1950	2177	2552	2917	3488	4011	4627	4883	5060	5382
1969	17.55	19.27	19.78	20.75	21.40	22.58	23.28	24.41	24.96	26.89	29.09	31.10	33.23	33.27	33.80
1970	17.25	18.96	19.46	20.16	20.92	22.09	22.75	23.89	24.54	26.53	28.44	30.73	32.28	32.45	33.12
1971	16.90	18.56	19.10	19.72	20.55	21.70	22.34	23.47	24.22	26.36	28.04	30.41	31.63	32.02	32.63
1972	17.54	19.18	19.71	20.60	21.27	22.44	23.15	24.27	24.85	26.79	28.90	31.00	32.98	33.02	33.60
1973	18.41	19.89	20.52	20.72	22.07	23.27	23.93	25.11	25.81	27.65	29.93	31.54	34.55	34.54	34.81
1974	17.38	19.07	19.58	20.34	21.07	22.24	22.91	24.05	24.67	26.64	28.64	30.84	32.57	32.68	33.33
1975	17.65	19.25	19.77	20.73	21.38	22.56	23.26	24.39	24.94	26.87	29.07	31.09	33.19	33.24	33.77
1976	17.52	19.16	19.69	20.54	21.24	22.40	23.10	24.23	24.82	26.76	28.86	30.96	32.89	32.96	33.56
1977	17.19	18.90	19.40	20.08	20.86	22.02	22.68	23.81	24.50	26.49	28.36	30.68	32.17	32.36	33.03
1978	17.42	19.11	19.61	20.38	20.76	22.28	22.96	24.10	24.71	26.68	28.69	30.88	32.63	32.74	33.38
1979	17.13	18.83	19.34	20.01	20.80	21.96	22.61	23.74	24.43	26.46	28.29	30.62	32.03	32.27	32.94
1980	17.67	19.28	19.79	20.78	21.41	22.59	23.29	24.43	24.97	26.91	29.11	31.11	33.26	33.30	33.82
1981	16.92	18.58	19.12	19.75	20.57	21.73	22.36	23.50	24.25	26.38	28.06	30.43	31.65	32.05	32.66
1982	17.25	18.96	19.46	20.16	20.92	22.09	22.75	23.89	24.54	26.53	28.44	30.73	32.28	32.45	33.12
1983	16.96	18.62	19.17	19.80	20.63	21.78	22.41	23.55	24.29	26.39	28.10	30.47	31.72	32.07	32.71
1984	16.99	18.65	19.20	19.84	20.65	21.81	22.45	23.58	24.33	26.41	28.14	30.50	31.78	32.11	32.75
1985	17.58	19.19	19.72	20.63	21.31	22.47	23.17	24.30	24.87	26.81	28.96	31.03	33.03	33.09	33.66
1:20 YR	18.23	19.71	20.40	20.56	21.88	23.09	23.75	24.94	25.65	27.48	29.77	31.46	34.27	34.27	34.59
1:100 YR	18.79	20.27	20.78	21.09	22.51	23.68	24.25	25.51	26.33	28.15	30.43	31.91	35.38	35.36	35.54

*   Water levels obtained from stage-discharge curves (Appendix E) for annual flows given in Table 6.7.

** Sections can be located on Figure 4.1.

8 - ICE MECHANICS ANALYSIS

8.1 - General

In order to delineate flood plain boundaries along Harry's River, consideration must be given to the three types of events which cause extreme water levels to occur namely open water events and ice freeze-up and break-up processes. Sections 6 and 7 addressed the open water events; this section discusses the ice freeze-up and break-up events.

Ice-induced flooding in Harry's River has occurred during freeze-up in early winter and during mid-winter as a result of ice break-up. The flooding events since 1984 have been well documented, and these were used for model calibration and verification. Information on prior events is largely anecdotal. Detailed historic data are of limited value, however, since the water levels used for the frequency analysis cannot be based on historic events, due to the channel modifications carried out in 1986. The general methodology used to estimate the series of ice-induced flood levels is summarized as follows.

- Acres River Ice Simulation Model, ICESIM, was calibrated separately for freeze-up and break-up using documented events and river cross-sections before channel modifications were made.

- Criteria to identify historic freeze-up and break-up events were determined from flood documentation as well as available meteorological and streamflow records.

- Historical records were then searched to identify likely historical freeze-up and break-up events.

- The ICESIM model was used to estimate the flood water levels in the study area for each of the freeze-up and break-up events selected for analysis using the 1986 modified channel data.

This section of the report describes the ICESIM model and the analysis undertaken to evaluate flooding from ice freeze-up and break-up events. The results are discussed as well as the impact of the recent channel excavations on flooding in Harry's River.

8.2 - ICESIM Model

ICESIM is a river ice simulation model originally conceived as a tool in analyzing the numerous ice problems in the construction of the Limestone generating station on the Nelson River in Manitoba. It has since been used by Acres for over 15 years on a number of projects for a variety of purposes from ice management studies to floodplain studies. The model has been applied successfully to many Canadian rivers which vary dramatically in size, climate, and geography.

ICESIM is a time-step backwater model which simulates the major ice processes in a fragmented river ice cover during the formation/freeze-up period or the break-up period. The model is based on a number of empirical and physically based formulae which have been developed independently by various investigators to characterize different ice processes.

A more detailed description of the program is provided in the following section.

8.2.1 - Model Description

The model considers the various ice processes which affect the water profile along a river, namely

- rate of ice generation
- ice cover advancement by juxtaposition using a Froude number criteria
- ice deposition and transport
- ice erosion
- border ice growth
- ice retreat by shoving
- ice cover progression by packing.

In general, after a change in the ice regime, the backwater profile is recomputed to update the hydraulic conditions in the river.

A description of the calculations of ice processes and water surface backwater profiles and their assumptions follow.

<u>Rate of Ice Generation</u>

The first process which must be properly simulated is the volume of ice entering the reach under study with time. Daily volumes of ice can be input directly or calculated using heat transfer theory given mean daily temperatures. Newbury (19) and Michel (17,18) have studied the detailed heat balance processes prevalent in river ice generation. The thermal ice processes considered by Michel are summarized as follows.

- heat transfer by evaporation and convection when water temperature is approximately 0°C
- total solar radiation absorbed by water, considering direct and diffuse radiation gain from the sun
- long wave radiation exchange
- heat loss from snow precipitation
- heat gain from friction losses in a river
- heat gain from groundwater flow

An approximate formula which calculates the total heat loss per unit surface area was derived from this theory and is used in the model as a basis for calculating ice generation. The rate of ice generated per day is the heat loss per unit area divided by the average weight density of the ice pack and the latent heat of ice multiplied by the upstream area of open water.

Ice Cover Advancement by Juxtaposition

Incoming slush pans underturn and pile up under the cover causing a rise in stage until they are able to progress upstream at a determined thickness. In reaches of lower Froude numbers, a cover formed by incoming slush pans or solid floes, one layer thick, can progress upstream in a continuous manner. ICESIM does not explicitly consider this latter process in calculating the stable ice cover thickness of the juxtaposed floes. The point at which the ice cover begins to move upstream is defined by the critical Froude number, which is an input variable into the model. Typical freeze-up values for a rectangular prismatic channel range from 0.08 to 0.13. During break-up, the porosity is significantly less resulting in larger critical Froude numbers, with a maximum theoretical value of about 0.154 for rectangular sections. In practice, higher critical Froude numbers can exist at highly irregular sections or where the incoming ice arrives as a large assemblage of ice pieces. If the open water section has a Froude number less than the critical value, the cover will advance upstream at a thickness calculated using an equation derived by Michel (18).

$$Fr = \frac{V}{\sqrt{gH}} = \sqrt{2\,\frac{(\rho - \rho')}{\rho}\,(1-n)\;t/H\;(1-t/H)}$$

where

n = porosity (ratio of volume of voids to total volume of ice)
t = ice thickness (m)
H = mean hydraulic depth (m)
V = mean velocity (m/s)
ρ = water density (kg/m^3)
ρ' = density of ice pan (kg/m3)
g = acceleration due to gravity (m/s^2)

This equation is based on the Bernoulli equation, considering nonsubmersion of the frontal edge, and the continuity equation.

Ice Deposition and Transport

A simple limiting velocity criterion was proposed by Newbury (19). It was observed that frazil will be deposited at under-ice velocities between 0.8 and 1.5 m/s. Larger ice blocks encountered during break-up may be deposited at greater under ice velocities up to 2.0 m/s.

In this simulation program, when ice is available to be deposited under the ice cover, the volume of ice is "transported" down stream from cross-section to cross-section until a location is found where the velocity is less than the specified maximum which will permit deposition. If deposition of the entire volume of ice at a section would result in a velocity in excess of the maximum at this location, then an appropriate portion is again transported further downstream "in search of" low velocity areas. It is assumed that the entire volume of deposited ice can be translated downstream in the time period. If at the furthermost downstream section a volume of ice remains to be deposited, the elevation of the cover at that section can be increased by a prescribed amount if this is appropriate to the situation being studied.

Ice Erosion

A check for ice erosion is incorporated in the water surface profile subroutine. The program uses a single limiting velocity criterion. Experience indicates that unconsolidated slush will erode at velocities greater than 1.5 m/s, while consolidated ice may not erode until velocities in excess of 2 m/s are encountered.

If for any section, the velocity is less than or equal to a maximum non-eroding velocity then the ice cover will not erode. Otherwise, the volume of eroded ice is calculated knowing the average ice width and thickness at the actual section as well as the upstream and downstream sections. The incremental distances to the upstream and downstream sections are also considered. This ice volume is then deposited downstream.

Border Ice Growth

The calculation of border ice growth is contained in the backwater subroutine. Two options exist for calculating the rate of border ice growth.

One approach utilizes the empirical border ice growth equation derived by Newbury (19),

$$B = (M/V^{1.5})\ DD$$

where

B = width of border ice (m)
M = coefficient varying between 0.04 and 0.06 (usually 0.054)
V = mean velocity (m/s)
DD = degree-days of freezing (°C days).

The empirical equation is based on data from the Nelson River considering several years of varying severity.

For the second approach, a "border ice factor" is used to calculate border ice growth. This factor is a fraction of the total water area which remains open, and is input to the program as a function of degree-days of freezing. It is based on field observations or judgement. This approach may be advantageous where the reach under study is short and the amount of border ice growth is relatively consistent along the river. This concept was used in the early studies of the Nelson River because photographs of the river taken in recent years were a good source of data on the relationship between border ice growth and degree-days of freezing.

Ice Retreat by Shoving

As the ice cover progresses upstream, stresses in the ice cover increase. The forces which increase ice stress include the hydrodynamic shearing force of the flow under the cover, the shearing stress of wind on the cover, the weight of ice along the slope of the ice/water interface and the hydrodynamic thrust on the leading edge of the cover. These forces must be opposed by the internal resistance of the cover and the resistance of the banks, otherwise the cover will be unstable and a shove will occur. From Michel (17), the hydrodynamic force is defined as

$$F_T = \frac{\gamma}{2g} D(1-d/D)^2 \, Vu^2 \, W$$

where

F_T = hydrodynamic thrust of the flow (N)

D = depth of water upstream of leading edge (m)

d = depth of flow under the leading edge (m)

Vu = velocity under the leading edge (m/s)

W = width of ice cover (m)
γ = weight density of water (N/m^3)

The force on the ice cover from frictional drag on the cover is computed for each reach using the following equation.

$$F_D = \frac{\gamma}{2} \; \frac{dSn_i^{1.5}}{n_e^{1.5}} \cdot A$$

where

F_D = friction drag force (N)
S = slope of hydraulic grade line
n_i = Manning's roughness coefficient of the ice under surface
n_e = Manning's roughness coefficient of the composite cross-section
d = depth of flow under the ice cover (m)
A = under-surface area of ice exposed to flow (m^2).

The force from the weight of the ice cover was derived from Pariset, et. al. (22) and is based on a simple buoyancy criterion which is independent of porosity. It is calculated as

$$F_w = 9025 \cdot V \cdot S$$

where

F_w = gravitational force acting along the channel (N)
V = volume of ice cover (m^3)
S = slope of the hydraulic grade line

The force exerted on the cover due to the wind is calculated as follows

$$F_{WD} = P_W \cdot A$$

where

F_{WD} = wind drag force (N)
P_W = wind force per unit area of ice cover (Pa)
:positive if wind from upstream to downstream
A = ice surface area (m^2).

The wind force per unit area can be estimated from the Von Karman equation given in Acres (1).

Resisting forces include the cohesion of the ice cover to the riverbanks and friction of the ice cover against the riverbank. The cohesion expression (Pariset et. al. (22)) is given as follows

$$F_c = 2 \cdot C \cdot t \cdot L$$

where

F_c = force of cohesion of ice to two riverbanks (N)
C = cohesion per unit area of ice/bank interface (Pa)
t = average thickness of ice cover between cross-sections (m)
L = distance between cross-sections (m).

The cohesion is best derived by prototype measurements. Experience indicates a reasonable value at formation is about 1000 Pa with a range from 500 - 4000 Pa. At break-up, the cohesion is considerably less and is often considered negligible.

The hydraulic forces exerted on the ice cover in the streamwise direction create stresses in the ice which are spread laterally towards the riverbanks. The lateral stress results in a reaction of static friction at the riverbank which acts as a stabilizing influence on the cover. The following expression was derived by Pariset, et. al. (22) as

$F_F = 2 \cdot f \cdot t \cdot L \cdot k_1 \tan \phi$

where

F_F = friction force on the ice along the riverbank (N)

f = accumulative stress in the ice cover in the direction of flow (Pa)

t = ice thickness (m)

k_1 = a coefficient equal to the ratio of lateral stress to longitudinal stress in the ice cover (a ratio less than or equal to 1.0)

ϕ = angle of friction of ice.

The internal resistance of the ice cover (after Pariset et. al. (22)) is given as

$$F_{IR} = \rho' \left(1 - \frac{\rho'}{\rho}\right) . \frac{gt^2}{2} . K_2 W$$

where

F_{IR} = internal resistance of the fragmented ice cover (N)

t = ice thickness (m)

K_2 = a coefficient analogous to Rankine's passive coefficient in soils

ρ = water density (kg m^3)

ρ' = density of ice pan (kg/m^3)

W = river width (m).

The product $K_1 K_2 \tan \phi$, which is designated as the μ value, represents the ice-over-ice friction and the internal resistance of the ice cover (20). The values of μ have been fairly well defined, and there is reasonable agreement as to the value of this parameter. Beltaos (1983), Calkins (1984) and Shen and Yapa (1984) cite values ranging from 1.2 to 1.3 (3, 5, 24).

In ICESIM it is necessary to specify K_2 and $K_1 \tan \phi$ separately. In this study, calibrated values of K_2 and $K_1 \tan \phi$ were 8.7 and 0.15 respectively, to give $\mu = 1.13$. Although the individual values of K_1, K_2 and $\tan \phi$ have not been determined by prototype observations, comparative simulations with the mathematical model have indicated that the predictions of shoves and hence ice thicknesses are relatively insensitive to the choice of their individual values.

The total force at a cross-section is therefore estimated as

$$F = F_P + F_D + F_W + F_{WD} - F_C - F_F$$

where

F = total force

F_P = total force at previous cross-section (at the leading edge of the ice cover, this would be made up solely of the hydrodynamic thrust, FT)

F_D, F_W, F_{WD}, F_C, F_F = forces as defined above.

If this force, F, exceeds the internal resistance of the ice cover, F_{IR}, then a shove is assumed to occur to permit the ice cover to thicken to its internal equilibrium thickness for stability. When a shove occurs, the required stable thickness of the ice cover is computed as

$$t_{REQD} = (F/(361.k_2.\ W))^{0.5}$$

This process continues from upstream to downstream. Next, the volume of ice required to thicken all of the unstable cross-sections is calculated. Finally, the time to produce this volume of ice is calculated and updated. The model can also make allowance, if deemed appropriate, for a reduction in downstream forces due to the grounding of an ice cover or additional cohesion of ice to island banks.

Ice Cover Progression by Packing

Alternatively, staging to achieve leading edge Froude equilibrium may be achieved during breakup, by the influx of a large volume of ice to a given constricted river section. Acres refers to this process as packing.

Progression by packing can be invoked in river reaches where

- ice inflow to an ice front is much greater than the under ice transport

- when upstream progression is observed at a section with a Froude number greater than a theoretical limiting critical value. Very steep gradients and limited ice volume (compared to deposition) result.

The packing solution simply solves the Bernoulli energy equation across the head (downstream end) of an ice jam to the controlling Froude section, with the velocity under the jam head assumed to be less than or equal to the non-eroding velocity. It is not normal practice to invoke the packing methodology, since in using this method, the hydraulic force equilibrium criterion, which determines the internal equilibrium thickness of stable ice jams in a wide river, is suppressed.

It was not necessary to use the packing routine in the Harry's River analysis since the head losses simulated through the ice jam (based on the normal juxtaposition and subsequent thickening to satisfy force equilibrium) were representative of observed conditions.

Water Surface Backwater Profiles

Water surface profiles are based on the one dimensional Bernoulli energy equation. The backwater subroutine is intended to calculate water surface elevations for gradually varied sub-critical flow.

Profiles are calculated with and without an ice cover. All related hydraulic information, namely conveyances (with and without ice cover), widths, areas, and critical discharges are contained in tabular fashion as a function of water level elevations for each river station. Conveyance with an ice cover is based on a composite Manning's n roughness given by

$$n_c = \left(\frac{n_i^{3/2} + n_b^{3/2}}{2} \right)^{2/3}$$

where

n_i = Manning's n under ice cover roughness

n_b = Manning's n bed roughness

As shown by Michel (17), under ice cover roughness values can vary over a wide range from 0.01 to 0.10; however, in general, frazil ice has a lower roughness (0.01 - 0.05), while solid ice floes encountered at break-up have a larger roughness (0.04 - 0.1).

Area, head loss and the velocity distribution coefficients can be nonlinear and/or discontinuous functions. As a result, the equations are solved by an iterative procedure.

8.2.2 - Data Requirements

The model requires basic river geometry, hydraulic, ice, and meteorological data as input. The estimated values of the various ice parameters depend on the type of event being modeled (i.e., break-up or freeze-up). These parameters can be adjusted in the calibration of the model. The basic data requirements are summarized below:

Geometric Data

- distance between cross-sections
- cross-section (elevation, offset) data
- shifts in cross-section invert elevations, if appropriate
- area of open water upstream of modeling boundary (used for calculating ice production
- additional length of contact with ice (shoal length for grounded cover or perimeter of islands to increase hydraulic resistance to forces in ice cover)

The program calculates, for each section, a top width/flow area tables as a function of elevation for backwater computations.

The cross-section data can be received in HEC-2 format; however, the section numbers are defined by ICESIM in sequential order. Separate cross-section data files were derived for freeze-up and break-up considering both the pre and post-excavation conditions.

Hydraulic Data

- Manning's n bed and under ice roughness values at each cross-section
- variable discharge along river
- starting water level

The program calculates, for each section, open water conveyance and conveyance of ice covered channel tables as a function of elevation for backwater computations.

Ice Data

- deposition velocity
- erosion velocity
- cohesion of fragmented/ thermal ice to riverbank
- critical Froude number
- density of ice
- ice strength
- border ice growth coefficient

Meteorological Data

- degree day/fraction of open water area border ice relationship, if appropriate
- mean daily air temperature data, if appropriate
- mean daily ice volume, if appropriate

Specific data parameters adopted in this study are discussed in Section 8.3.2.

8.3 - Freeze-Up Analysis

Historical evidence indicates that the formation of the ice cover during freeze-up can result in flooding along Harry's River. The purpose of modeling the formation of the ice cover is therefore to estimate the magnitude of these events in order to combine the results with the maximum annual open water staging and ice break-up for the purposes of overall flood level frequency analysis.

The following methodology was adopted in conducting this analysis.

- The ice model was calibrated using documentation for the January 4-9, 1985 freeze-up event and verified using the January 18-20, 1986 freeze-up event.

- Criteria to identify a fragmented cover during freeze-up were determined from flood documentation as well as available meteorological and streamflow records.

- Historical records were then searched to determine the timing and likelihood of a fragmented cover progressing upstream to the study area during the freeze-up event.

- Finally, the model was used to estimate the flood water levels in the study area for each of the freeze-up events selected for analysis.

8.3.1 - Freeze-Up Ice Processes in Harry's River

The behavior of ice in a river is dictated by the hydraulics of the channel (river geometry, headpond level, discharge) and ice characteristics (strength, inflow rate, etc).

Although Harry's river is only 60 to 100 m wide, it is very shallow in certain areas and therefore categorized hydraulically as a wide river. This means that the ultimate thickness of the ice cover is governed in these locations by the accumulated forces in the cover. At any section, the net downstream force in the cover results from the hydrodynamic thrust on the ice cover front, the hydrodynamic drag force on the underside of the ice cover, the weight of the ice along the slope of the ice/water interface, and in some cases, the shearing stress of wind on the cover. Since Harry's River is a meandering river, sheltered by trees for the most part, the contribution of wind forces has been assumed as zero.

A known lodgement position was observed during the January 1987 field trip at lower White's farm, approximately 1.5 km downstream of the study area. In many rivers, lodgement is caused by a large volume of ice becoming compressed in a narrow section; however, in this case, the lodgement is believed to result from a very wide, long, reach having islands and a mild slope where the momentum of the ice is arrested. Local residents confirm that lodgement occurs each year at this location.

The ice cover requires some degree of freezing over time to progress upstream to the study reach. The rate of ice cover advancement depends on meteorological conditions which determine how much frazil ice will be generated in the river, and hence the inflow volume of ice. The analysis of the progression from White's farm to the study reach, and hence the development of freeze-up criteria, is described in the following section. The remaining sections describe the calibration and verification of the model for the freeze-up analysis.

8.3.2 - Model Calibration

The model freeze-up calibration was based on the observed January 4-9, 1985 event, as described previously in Section 3.2, since it was the best documented. Data used as a basis for the calibration included

- Harry's River Ice Monitoring notes for the 1985 winter (Newfoundland Department of Environment). This data provided a qualitative assessment of ice conditions and estimates of the ice thickness on a weekly basis. Approximate values of ice thickness were used to compare the simulation modeling results
- Newfoundland Department of Environment report entitled "Case Study of Ice Related Flooding of Harry's River at Black Duck Siding, Newfoundland" (13).
- Photographs

- Plan view of historical flood levels (Newfoundland Department of Environment).
- Harry's River WSC gauge data.

As stated previously, ICESIM requires input of the river cross-sections as used in the HEC-2 model, the river flow and the starting water surface elevation at the downstream end, as well as input of various ice parameters for the event being modeled. In most cases, these ice parameters were taken from the literature and/or from previous event modeling experience. A summary of the calibrated freeze-up model parameters is given in Table 8.1. The following notes provide additional explanation.

ICESIM Cross-Sections

River cross-sections used in the model are, for the most part, the same as those used for the HEC-2 program. Flood plain areas containing ice in storage with little or no conveyance of flow were removed for ICESIM modeling so that ice would only progress in the river reaches and not in the overbank areas.

Starting Downstream Water Level

Section 0+204 was taken as the starting section for the thermal cover. The water level and ice thickness at this section was calculated from appropriate modeling of ice processes in the downstream reach as described in detail in Section 8.3.4. The starting water elevation was calculated as 15.9 m and the starting ice thickness as 0.9 m for the January 1985 freezeup event.

Froude Number

The Froude number is an important ice parameter, as it, to a large degree, influences the type of processes and the peak water

TABLE 8.1

CALIBRATED ICESIM PARAMETERS: FREEZE-UP

Discharge at WSC Gauge	15.8 m^3/s
Downstream Water Level	15.9 m
Critical Froude Number	0.10
Deposition Velocity	1.0 m/s
Erosion Velocity	1.5 m/s
$K_1.K_2 \tan \phi$	1.3
Density	700 Kg/m^3
Coefficient in Border Ice Equation	0.023
Fragmented Ice Roughness	0.045
Fragmented Ice Cohesion	1 kPa
Thermal Ice Cohesion	40 kPa
Fragmented Ice Thickness (Downstream Boundary)	1.0 m

levels in certain reaches which occur in the river. A sensitivity analysis was done on this parameter, considering values from 0.08 to 0.12. Calibration data for the January 1985 freezeup event was used for the sensitivity analysis, keeping all other parameters as given in Table 8.1. The sensitivity analysis indicated that lower Froude numbers (0.08) resulted in more localized shoving (and higher water levels) in the area of Tanglewood Ranch but less shoving along the river reach than observed. Higher Froude numbers (0.12) resulted in considerably more shoving than observed along the entire reach and very high ice thicknesses. A critical Froude number of 0.10 resulted in the best fit to historic water levels and ice thicknesses and hence was adopted for the analysis (See Appendix F).

Inflow Volume of Ice

It is important to properly simulate the volume of ice entering the reach under study with time. The rate of ice accumulation in the formation process depends on the daily temperature and the area of open water available for frazil ice generation. ICESIM requires input of the area of open water upstream of the reach contributing to ice growth as well as the daily air temperatures for the period under study. It is assumed that the open water area upstream remains constant in time. In reality, this reach will be subject to normal ice processes and the open water area will decline as border ice or lodgements constrict the channel.

The open water area available within the reach is calculated by the model from the input data and extent of ice cover. The daily temperatures for the month of January 1985, were obtained from AES data. Different values for the area of open water upstream were tested until the rate of ice accumulation agreed with that observed. A final value of 100 000 m^2 was selected on the basis of the test runs. This was considered reasonable as it represented an open water length of 2 km to 10 km based on upstream river widths of 50 m to 10 m respectively.

Discharge

The average discharge at the gauge for the period being simulated was 13.3 m^3/s, which is considerably less than the mean flow. Prorated directly by drainage area because the flow is low, the discharge is 15.4 m^3/s downstream of Trout Brook, and 15.8 m^3/s downstream of Black Duck Brook. As with HEC-2, the discharge can be varied spatially but not temporally, so these discharges were assumed to be constant over the simulation period. The actual measured flows were quite stable over this period.

Deposition Velocity

Considerable judgement must be used in setting this value. It has generally been found that deposition velocities of 0.9 m/s to 1.1 m/s provide reasonable results for ice cover progression at freeze-up. A range of values were tested in the model (see Appendix F) for the January 1985 freeze-up event, keeping all other parameters constant. It was found that higher deposition velocities (1.2 and 1.5 m/s) resulted in higher ice thicknesses than observed in the area of Tanglewood Ranch and Hickey's farm although water levels remained essentially the same. A deposition velocity of 0.9 m/s resulted in the best agreement with observed water levels and ice thicknesses.

Ice Erosion Velocity

For the freeze-up event, a value of 1.5 m/s was selected, which is typical of that used in similar previous studies. No observations are available to substantiate the use of any particular value and sensitivity runs indicated that changes in the ice erosion velocity did not affect the ice processes and the calculated water levels and ice thicknesses (See Appendix F).

Ice Cohesion

ICESIM requires input of values for the cohesion of fragmented ice and of thermal ice. In this study, the most important of these parameters is the cohesion of fragmented ice to the shore ice along the river bank. Typical values for freeze-up are in the range of 0.5 to 3 kPa. A sensitivity analysis of this parameter indicated that higher values of cohesion resulted in lower water levels and ice thicknesses than observed (See Appendix F). The final calibrated value was selected as 1 kPa since this resulted in the closest agreement with observed and ice thicknesses. The cohesion of thermal ice to the river bank was taken to be much greater (40 kPa) to ensure that the thermal cover does not collapse.

Ice Friction Property (μ)

This value, corresponding to the product of K_1 K_2 tan ϕ in the model, was described in detail in Section 8.2.1. It was taken as a constant value of 1.3 in this study based on experience and literature sources (3, 5, 24).

Ice Roughness

The under ice roughness is specified in the input to the program and can be varied with each section. Initial values were selected based on previous studies which indicate that, for a cover of 0.5 to 1.0 m thick, the roughness coefficient will range from 0.02 to 0.04.

For the freeze-up calibration, ranges of 0.03 to 0.06 were tested for the entire reach. Final roughness coefficients were taken as

0.03 in areas where gentle progression is expected, and 0.045 in locations where shoving and thickening are expected.

Bed Roughness

The bed roughness (Manning's n) for each section was determined from the open water calibration (Section 7.3). ICESIM calculates the composite resistance at each section due to the ice cover and the bed.

Ice Density

This parameter has little impact on the modelling of ice processes using ICESIM and only affects the rate of progression of the ice cover. A comparison of the rate of progression of the ice cover shown by ICESIM for the January 1985 freeze-up event with the available documentation of the event indicated good agreement for an ice density of 700 kg/m^3.

Calibration Results

The calibrated river profile determined using ICESIM is shown in Figure 8.1, along with the observed water levels. Table 8.2 summarizes the results and also provides a comparison of the change in water level at the bridge as the ice cover advances.

The calculated water levels provide excellent agreement with those observed or estimated during the January 1985 event given the accuracy of the observed data. Calculated thicknesses seem somewhat large but reasonable, although it is difficult to compare ice thicknesses since observed thicknesses are spot values, whereas the model generates an average value across the section. There is not sufficient data, either along the river reach or across a section, to discuss whether the model is accurately predicting ice thicknesses. It is possible that,

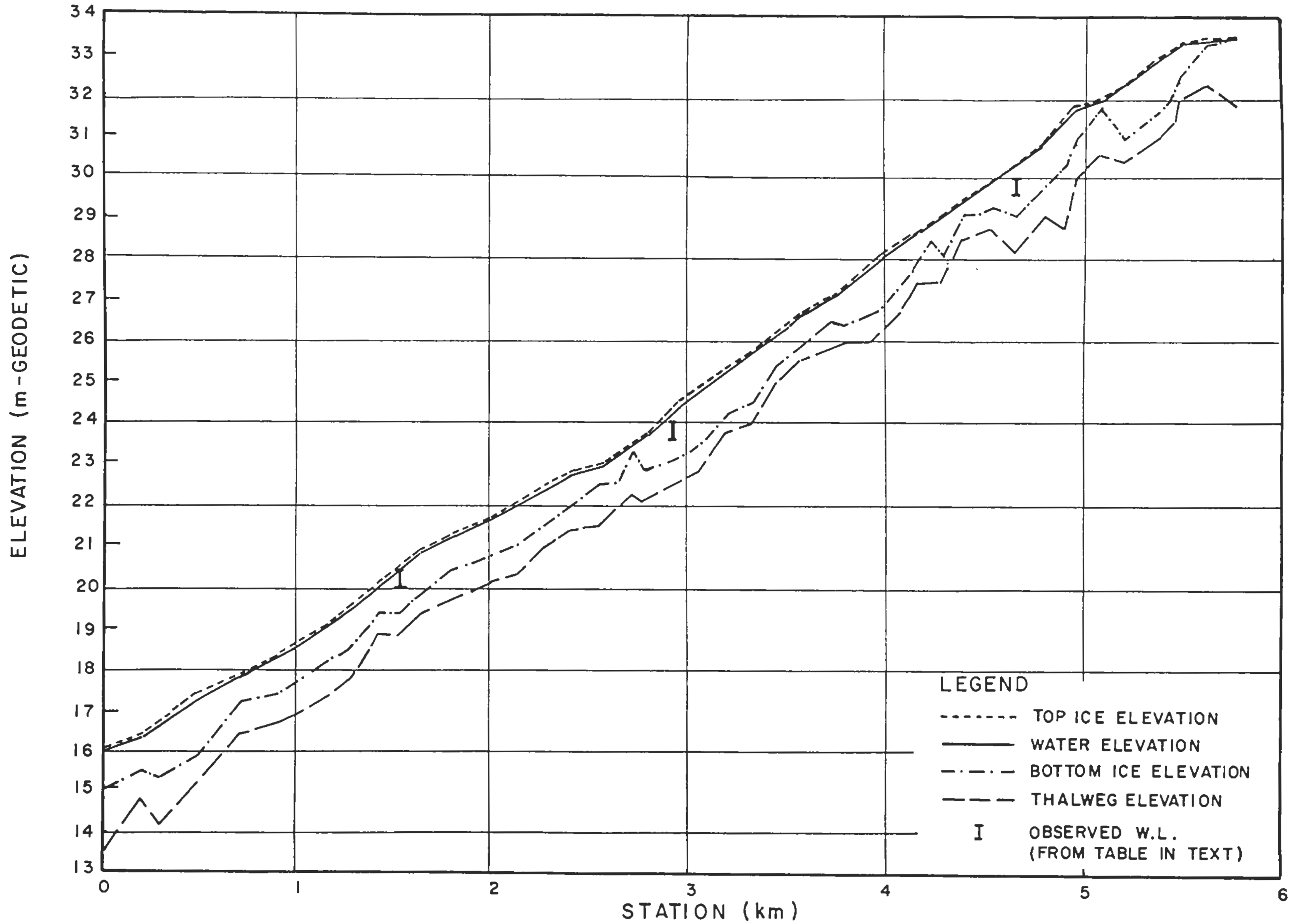


FIG. 8.1

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

JANUARY 1985 FREEZE-UP CALIBRATION

ACRES

TABLE 8.2

COMPARISON OF OBSERVED AND CALCULATED WATER LEVELS AND ICE THICKNESS FOR JAN 85 FREEZE-UP EVENT - MODEL CALIBRATION

ICESIM Section	Observed Ice Front Section	Water Levels (m) Observed*	Water Levels (m) Calibrated	Ice Thickness (m) Observed*	Ice Thickness (m) Calibrated
1+591	1+756 - 2+332	20-20.5	20.4	-	1.2
2+063	2+332 - 3+907	-	21.9	0.8	1.1
2+332	2+332 - 2+802	22-22.5	22.7	-	0.9
2+802	2+802 - 3+232	23.5-24	24.3	0.8	1.4
4+627	2+063**	29.4	29.3	-	-
	3+232**	29.8	29.3	-	-
	>4+627**	29.7	30.1	0.3	1.26

* Estimated from documentation provided by Newfoundland Department of Environment.

**These numbers show the water levels measured at the WSC gauge as the ice front moved upstream.

given further data, calibrated coefficients of ICESIM might need to be modified to change predicted ice thicknesses.

The areas of spatial flooding also match field observations well, i.e. the lower field of Tanglewood including the lodge and the lower field of Hickey's. A flood level of 26.1 m was predicted at Hobb's low berm (3+488) which is at an elevation of 26.4 m and a flood level of 31.9 m was predicted where the 32.0 m berm exists at Dhoon Lodge (5+060), indicating no flooding at either location.

The model predicted a rough fragmented cover, due to consolidation and thickening to achieve internal force equilibrium, in the following areas

- Sections 1+254 - 1+401
- Sections 2+063 - 2+802
- Sections 3+488 - 3+717
- Sections 4+421 - 4+627

These are all sections where photographs and reports indicated a very rough cover. The remainder of the reach is subject to a general deposition, Froude juxtaposition ice cover advancement process. This agrees well with documented evidence.

8.3.3 - <u>Model Verification</u>

The January 18-20, 1986 period was chosen to verify the calibrated ICESIM freeze-up parameters. The documentation and observations came from the same sources referred to in Section 8.3.2.

This period was characterized by a fairly constant discharge of 31 m3/s and a downstream water level of 16.5 m.

A plot illustrating the predicted river profile as well as the observed water levels is given in Figure 8.2. Table 8.3 summarizes the results of the model verification for the freeze-up event. In addition, a plan view indicating the spatial extent of the predicted flooding is given in Figure 8.3. The calculated water levels in Figure 8.2 show good agreement with the observed water levels. The flooded areas shown in Figure 8.3 match all the flooded areas identified by the Department of Environment in Figure 3.2. As well, there are some additional flooded areas identified from the modeling. This is not a point of disagreement, but rather information derived from the model about areas which are otherwise inaccessible.

8.3.4 - Criteria for the Selection of Historical Freeze-Up Events

In order to predict water levels resulting from the ice formation process in Harry's River, a means of establishing the steady state flow and rate of ice production for each year at the time of formation was required. In addition, it was necessary to determine conditions at the downstream boundary, namely the downstream water level and ice thickness. The known formation lodging point is, in fact, about 1.5 km downstream of the study reach, near lower White's farm. Since, in some years, the ice cover does not progress upstream into the study area, a cumulative degree day criteria was derived to allow the ice cover to progress to the downstream boundary of the study reach.

Initial attempts to correlate cumulative freezing degree days with discharge data marked with a "B" (indicating the influence of backwater at the gauge from ice in the river) in the records were unsuccessful. The "B" indicator does not appear to reliably identify the beginning of the freeze-up process in the study reach.

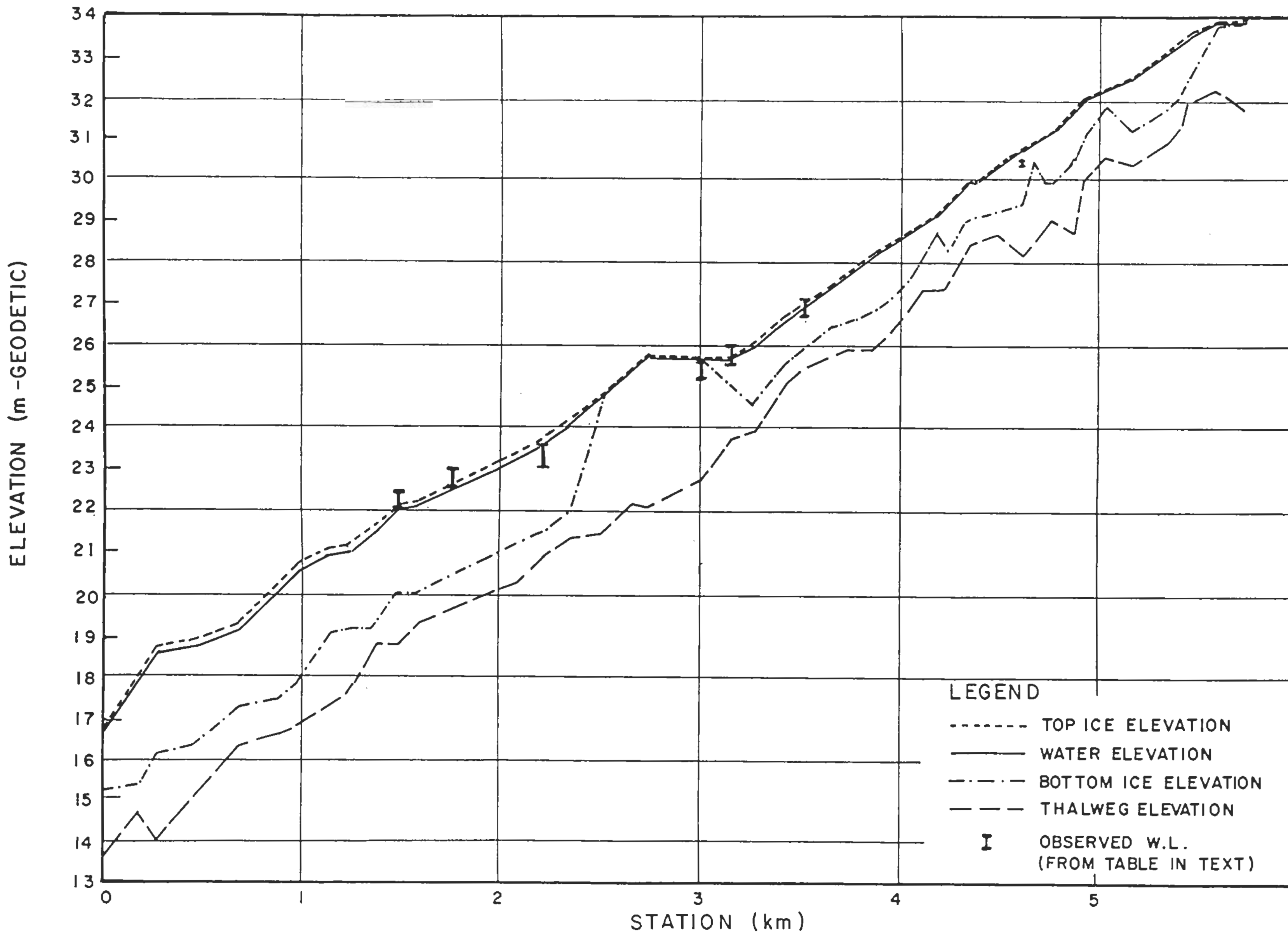


FIG.8.2

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS
JANUARY 1986 FREEZE-UP VERIFICATION

ACRES

TABLE 8.3

COMPARISON OF OBSERVED CALCULATED WATER LEVELS AND ICE THICKNESSES FOR JANUARY-FEBRUARY 1986 FREEZE-UP EVENT - MODEL VERIFICATION

ICESIM Section	Water Levels (m) Observed*	Water Levels (m) Calibrated	Ice Thickness (m) Observed**	Ice Thickness (m) Calibrated
1+591	22-22.5	22.1	-	2.3
1+756	22.5-23	22.5	-	2.1
2+177	23-23.7	23.6	-	2.2
3+232	25.6-26	25.7	-	0.7
3+488	26.5-27	26.8	-	1.0
4+627	30.5+	31.0	-	1.3

* Estimated from documentation of extent of flooding, provided by Newfoundland Department of Environment.

**No detailed observations on ice thicknesses are available but ice thicknesses were estimated to be in the order of 2-2.5 m (13).

\+ From WSC gauge records.

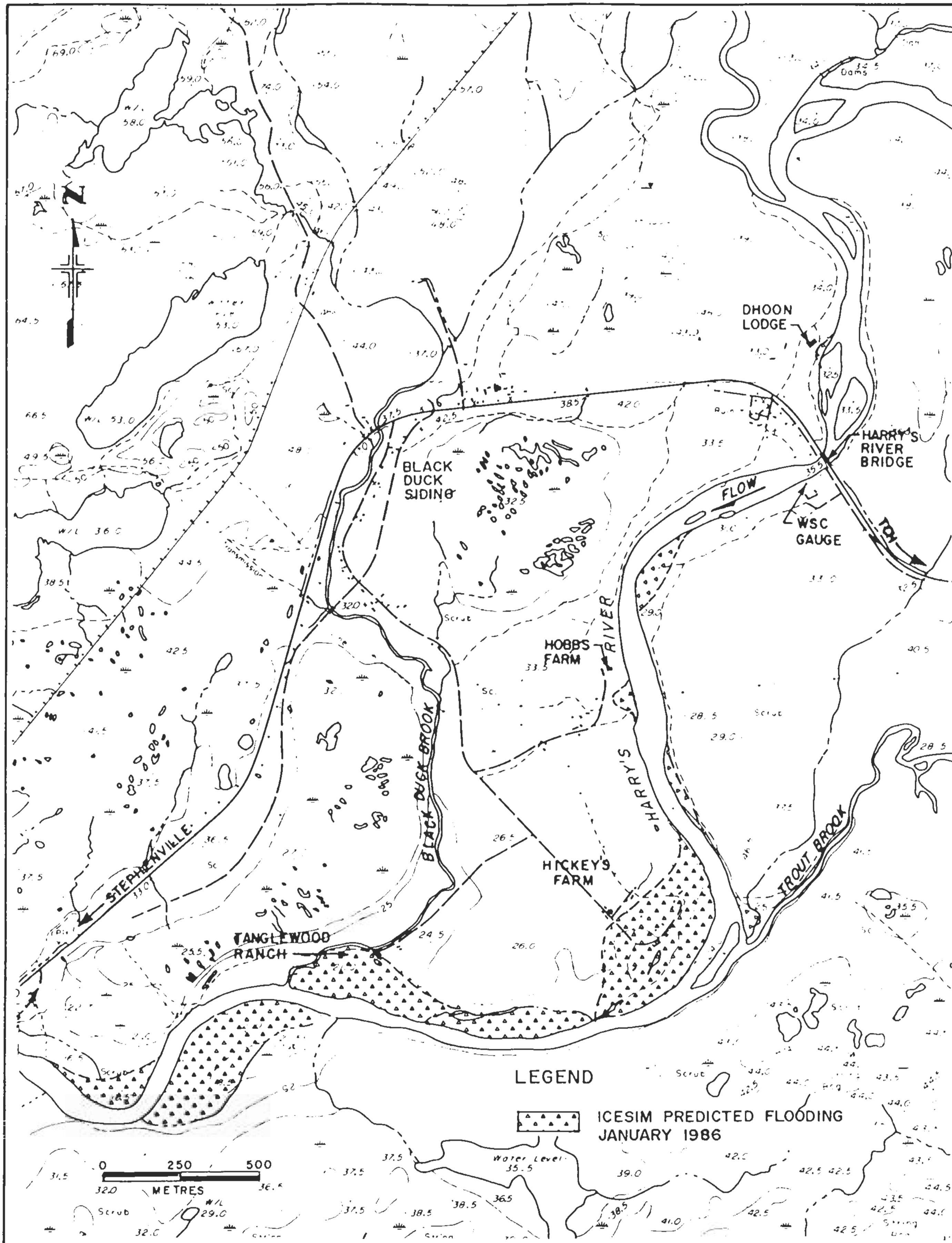


FIG. 8.3

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

EXTENT OF FLOODING (ICESIM) JANUARY 1986

ACRES

ICESIM was used as an alternative approach to model the upstream progression of a fragmented cover at freeze-up from lower White's farm to the study area, thereby establishing a cumulative degree day/discharge function given in Figure 8.4. The cumulative degree days refer to the sum of mean daily air temperatures, rather than days of freezing or thawing. The curve was calibrated using the known ice cover front locations for the 1984-1985 formation period. The timing of the 1985-1986 ice cover advancement was used to assess the adequacy of this methodology. Although the historical data were somewhat sketchy, the model predicted the presence of the ice cover front in the Hickey's/Hobb's reach in agreement with the observed data.

A uniform rectangular prismatic channel having a width of 60 m and a slope of 0.002 as determined from 1:50 000 mapping of the river was assumed in the model over this lower reach. This was felt to be representative of the downstream reach.

In addition, results from the model were used to derive an ice thickness/discharge function (Figure 8.5) and a mean hydraulic depth/discharge function (Figure 8.6) at the start of the study area. The boundary condition functions were corrected (parallel shift) to account for the irregular channel properties at the downstream limit of the modeling reach such that starting conditions of the observed 1984-1985 formation period were reproduced. Calculation of a standard shift to non-rectangular section (0+204) showed that the parallel shift in mean depth could not be in error by more than 0.3 m for an expected range in depths corresponding to historical data. Since the geometry of the actual controlling downstream section is not known, this rectangular section assumption is considered acceptable.

The criteria for determining the freeze-up dates, flows, and boundary conditions for each of the historical years was as follows.

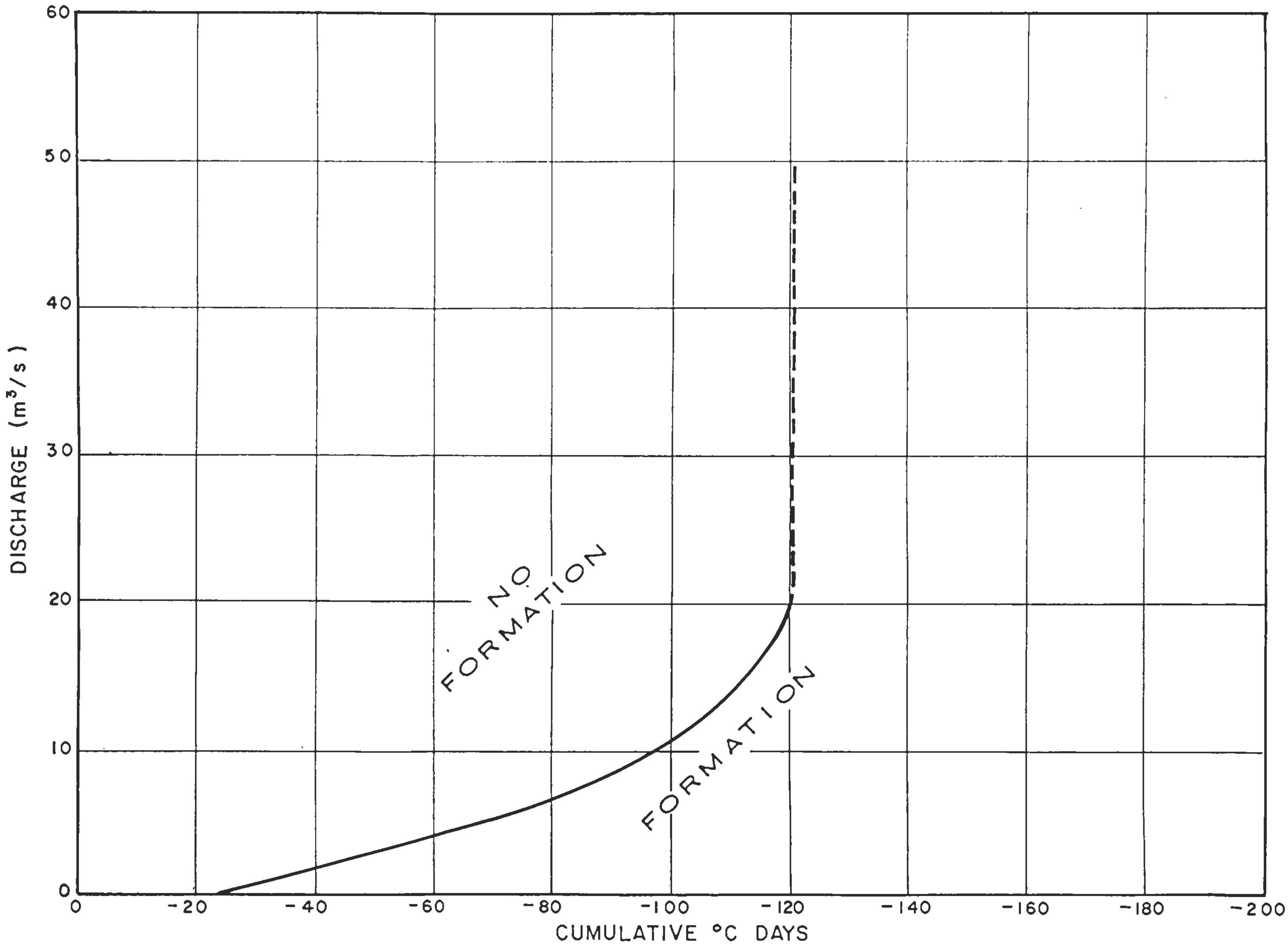


FIG. 8.4

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

FREEZE - UP EVENT IDENTIFICATION FUNCTION

ACRES

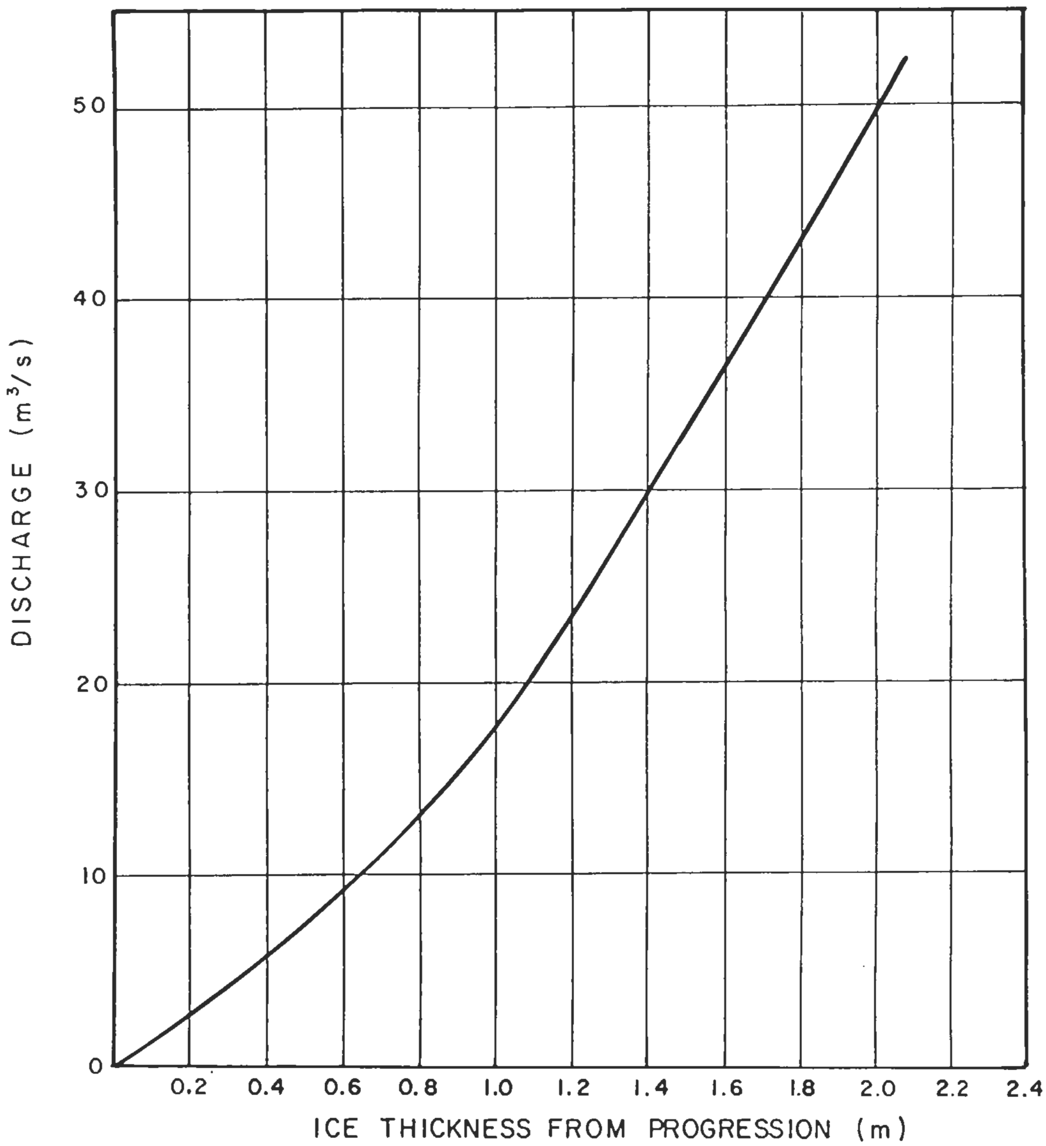


FIG. 8.5

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ICE THICKNESS BOUNDARY CONDITION FUNCTION

ACRES

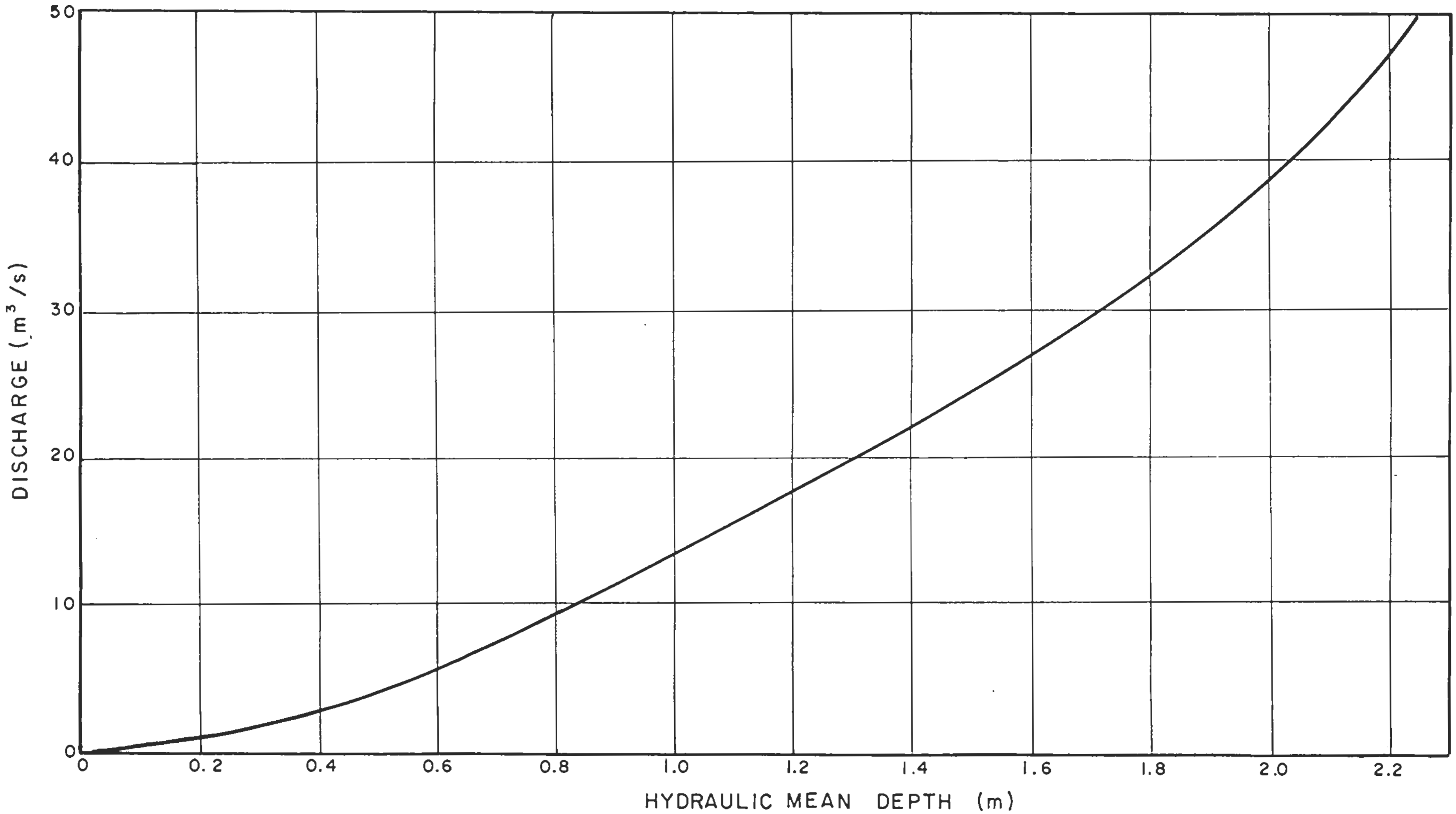


FIG. 8.6

CANADA–NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS
MEAN HYDRAULIC DEPTH BOUNDARY CONDITION FUNCTION

ACRES

- The beginning of winter was identified as the earliest time when freezing temperatures accumulated without a significant warm period in the record.

- The cumulative mean daily air temperatures were plotted versus a moving discharge average until the ice cover formation region of Figure 8.4 was encountered.

- Ice formation was assumed to start at the time the formation region of Figure 8.4 was encountered.

- The steady state discharge was taken as the average flow over the next seven days (corresponding approximately to the progression time of the ice front between Tanglewood and the highway bridge)

- The corrected boundary conditions given in Figures 8.5 and 8.6 were used as input to the model in the study reach.

Table 8.4 summarizes the freeze-up events. This table indicates the start of winter date and the calculated date of formation and flow in our study reach. Since the evolution of the ice cover for a particular event is considered steady state, an average 7 day flow over the expected period of formation within the study reach was selected.

8.3.5 - <u>Freeze-Up Flood Profiles</u>

The calibrated ICESIM model was used in conjunction with the boundary conditions derived from the freeze-up identification criteria explained in Section 8.3.4 to determine the freeze-up or formation water levels along the river for applicable winters. The modeling was done using the post excavation cross sections to reflect the impact on future flooding.

TABLE 8.4

ICE FREEZE-UP EVENTS

Year	Start of Winter	Start of Formation	7-Day Average Gauge Flow (m^3/s)*
1968-1969	Dec. 21	-	-
1969-1970	Dec. 31	Jan. 20	11.1
1970-1971	Dec. 6	Dec. 29	8.4
1971-1972	Dec. 14	Dec. 31	15.5
1972-1973	Dec. 10	Dec. 24	12.6
1973-1974	Dec. 31	Jan. 12	10.1
1974-1975	Dec. 14	Jan. 10	7.6
1975-1976	Dec. 28	Jan. 16	22.7
1976-1977	Jan. 9	Jan. 24	18.5
1977-1978	Jan. 17	Feb. 9	12.2
1978-1979	Nov. 20	Dec. 14	12.1
1979-1980	Dec. 30	Jan. 22	11.3
1980-1981	Dec. 10	Dec. 27	16.7
1981-1982	Dec. 18	Jan. 18	9.0
1982-1983	Dec. 30	-	-
1983-1984	Dec. 17	Jan. 9	19.7
1984-1985	Dec. 18	Jan. 5	13.3
1985-1986	Dec. 4	Dec. 27	25.7

* The 7 day average gauge flow is the average of the flows from the WSC records over the next 7 days after the start of formation.

A summary of the predicted water levels from freeze up events at these sections is provided in Table 8.5. In two years, 1969 and 1983, the calculations showed no ice cover formation events. The water levels are given for the sections used in the joint probability analysis. These key sections were selected as described in Section 7 at reasonable intervals to allow interpolation of water levels between sections.

TABLE 8.5

ICESIM FREEZE-UP WATER ELEVATIONS (m) 1969-1986

	SECTION NUMBER *														
YEAR	0471	0873	1152	1347	1591	1950	2177	2552	2917	3488	4011	4627	4883	5060	5382
1969**	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
1970	17.11	18.13	18.89	19.67	20.78	21.43	22.24	23.25	24.66	26.59	28.23	29.92	31.42	31.83	32.75
1971	17.03	18.06	18.81	19.55	20.69	21.29	22.15	23.16	24.65	26.47	28.21	29.96	31.32	31.63	32.69
1972	17.35	18.41	19.16	19.79	20.81	21.60	22.35	23.32	24.69	26.76	28.41	30.24	31.54	31.98	32.93
1973	17.17	18.19	18.93	19.74	20.83	21.50	22.28	23.27	24.72	26.62	28.27	30.10	31.49	31.95	32.86
1974	17.00	18.12	18.85	19.62	20.75	21.42	22.22	23.25	24.69	26.58	28.36	29.93	31.15	31.67	32.88
1975	16.94	18.07	18.78	19.64	20.46	21.31	21.98	23.02	24.73	26.44	28.14	29.96	31.32	31.59	32.90
1976	17.71	19.53	20.71	21.01	21.87	22.67	23.76	24.44	24.75	26.88	28.58	30.70	31.72	32.22	33.25
1977	17.51	18.48	19.24	19.79	20.84	21.71	22.45	23.41	24.85	26.80	28.49	30.85	31.52	32.05	32.93
1978	17.10	18.19	18.92	19.74	20.83	21.47	22.26	23.24	24.63	26.60	28.33	30.12	31.36	32.15	32.81
1979	17.09	18.18	18.91	19.55	20.55	21.49	22.26	23.27	24.71	26.62	28.25	30.10	31.50	31.96	32.83
1980	17.15	18.16	18.89	19.70	20.79	21.44	22.23	23.26	24.67	26.61	28.27	30.05	31.36	31.78	32.96
1981	17.32	18.44	19.19	19.80	20.83	21.68	22.43	23.41	24.80	26.78	28.45	30.46	31.48	31.97	32.74
1982	16.98	18.06	18.82	19.56	20.69	21.34	22.36	23.18	24.66	26.47	28.15	29.95	31.05	31.71	32.91
1983**	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1984	17.60	19.38	20.64	20.85	21.58	23.04	23.80	24.26	24.67	26.88	28.51	30.96	31.58	32.10	32.89
1985	17.23	18.30	18.99	19.69	20.74	21.52	22.29	23.26	24.63	26.55	28.38	30.22	31.52	31.93	32.73
1986	17.83	18.98	20.56	21.56	22.68	23.44	24.03	24.57	24.87	26.94	28.58	30.84	31.76	32.33	33.15

* Sections can be located on Figure 4.1.

** Years without formation event.

8.4 - Break-up Analysis

Several major floods have been observed in Harry's River to result from water backing up behind break up ice jams. The purpose of modeling ice break-up is therefore to estimate the magnitude of the flood levels resulting from these events. They can then be used with the maximum annual open water staging and annual staging from ice freeze-up for the purposes of flood frequency analysis.

The following methodology was adopted in conducting this analysis.

- The ICESIM model was calibrated using the February 5-12, 1984 break-up event.

- Criteria to identify whether enough ice cover from freeze-up was available for a break-up event were determined from flood documentation as well as available meteorological and streamflow records.

- Historical records were searched to determine the likelihood of a break-up event occurring.

- Finally, ICESIM was used to estimate the flood water levels in the study area for each of the break-up events selected for analysis.

The mechanisms of the break-up process are described in the following section. The remaining sections describe the technical analysis.

8.4.1 - Break-Up Ice Processes in Harry's River

At freeze-up, a cover forms from cohesive frazil slush ice. During the winter, gradual freezing of the ice generally takes place, depending on the severity of the winter. If the discharge increases due to rain and/or snowmelt, the ice cover will lift, breaking contact with the banks, resulting in a large supply of broken pieces of solid ice available to form ice jams.

Unlike the gradual formation process in Harry's River where the fragmented ice cover proceeds upstream from the known lodgement point at the lower White's farm, the break-up event is characterized by a sudden release of a large quantity of solid ice which may jam at a number of locations. Ice jam locations are typically characterized by sections having a sudden reduction in flow conveyance and/or large resistance downstream to the accumulated downstream forces (strong ice cover in a deep reach, islands, shallow grounded sections, etc).

Important elements in determining the flooding associated with a break-up event are therefore

- identifying ice jam location(s)
- determining the criteria which cause a break-up event to occur.

Three break-up ice jam locations were identified within the study area from documentation of the 1984 break-up event (14). Two of these jam positions are labelled IJ-1 and IJ-2 on Figure 3.2 and are located approximately at sections 2+063 and 2+702 respectively, and the third is at Harry's River Bridge. Section 2+063, which is just downstream of Hickey's farm, has an intermittent side channel which was most likely formed during ice-related flood events. Past flooding reports verify this since there has been considerable water flow observed through the natural side channel. Section 2+702 is just downstream of Hobbs' farm where the river experiences a sudden restriction. Both these sections are characterized by a significant reduction in flow conveyance, and Section 2+702 is situated at a bend where the momentum of the ice cover would be significantly reduced.

Local residents have identified Harry's River Bridge (Section 4+677) as a known ice jam location. The abutments of the bridge restrict the flow area and provide resistance to the downstream ice forces. This ice jam affects flood levels experienced at Dhoon Lodge.

8.4.2 - Model Calibration

The break-up calibration was based on the observed February, 1984 event as described previously in Section 6.2. Data used as a basis for the calibration included

- Report on the Flooding in Black Duck Feb. 5-12, 1984 (Newfoundland Department of Environment). This report provides a chronological description of the events which took place and spatial mapping showing jam locations and the areal extent of flooding as well as photographs of the ice cover.

- Water level and corrected flow measurements at the bridge (WSC).

As stated previously, the model requires input of the river cross-sections as used in the HEC-2 model (in this case, pre-excavation cross-sections were used for calibration), the river flow and the starting water surface elevation at the downstream end, as well as input of various ice parameters for the event being modeled.

In most cases, these ice parameters were taken from the literature and/or from previous event modeling experience. A summary of the calibrated break-up model parameters is given in Table 8.6. The following notes provide additional explanation.

Cross-Sections

River cross-sections used in the model are essentially the same as those described in Section 8.3.2. The flood plain areas from Hobb's farm to Tanglewood Ranch have been included for the break-up event modeling since ice has been observed to flow over these areas.

Starting Downstream Water Level

Section 1+347 was taken as the starting section for the calibration of the break-up event. This section was chosen since it was identified as a jam location in the downstream reach. The starting water level and ice thickness were estimated from documentation of the February 1984 event as 20.9 m and 1.7 m respectively. The initial ice thickness of 1.7 m is comparable to the mean thickness simulated through the jam.

Froude Number

As with the freeze-up analysis, a critical Froude number of 0.10 resulted in the best calibration and hence, was adopted for the analysis.

Inflow Volume of Ice

As arbitrary average daily inflow ice volume of 200,000 m^3 was assumed. This corresponds approximately to a reach length of 3 km.

TABLE 8.6

CALIBRATED ICESIM PARAMETERS: BREAK-UP

Discharge at WSC gauge	32 m^3/s
Downstream Water Level	20.85 m
Critical Froude Number	0.10
Deposition Velocity	1.0 m/s
Erosion Velocity	1.5 m/s
K_1 K_2 tan ϕ	1.3
Density	900 Kg/m^3
Coefficient in Border Ice Equation	0.023
Fragmented Ice Roughness	.045
Fragmented Ice Cohesion	0.2 kPa
Thermal Ice Cohesion	40 kPa
Fragmented Ice Thickness (Downstream Boundary)	1.7 m

Discharge

The average discharge at the gauge for the period being simulated was about 37 m^3/s. This discharge is assumed to be constant over the period.

Deposition Velocity

Considerable judgement must be used in setting this value. It has generally been found, from previous modeling experience, that deposition velocities of 1.0 m/s to 2.0 m/s provide reasonable results for break-up. A sensitivity analysis was done on this variable and it was found that best results, in terms of the basic ice processes involved, were obtained using a deposition velocity of 1 m/s.

Ice Erosion Velocity

For the break-up event, a value of 1.5 m/s was selected. This value is consistent with the freeze-up case and is typical of that used in similar previous studies.

Ice Cohesion

Typical values of cohesion for break-up accumulations are in the range of 0 to 1 kPa, with the final calibrated value selected as 0.2 kPa, which was considerably less than the freeze-up value of 1 kPa. Sensitivity analysis on this parameter indicated that a smaller cohesion did not result in noticeable differences, and greater values did not give a good calibration (see Appendix F).

The cohesion of thermal ice to the river bank was taken to be much greater to ensure that the thermal cover did not collapse.

Ice Friction Property (μ)

This value, corresponding to the product of K_1, K_2, tan ϕ in the ICESIM model was described in detail in Section 8.2.1. It was taken as a constant value of 1.3 in this study based on experience and literature sources (3, 5, 24).

Ice Roughness

As with the freeze-up calibration, ranges of 0.02 to 0.06 were tested for the entire reach with the calibrated roughness coefficients taken as 0.045 for the break-up event.

Calibration Results

The calibrated water levels determined using ICESIM is shown in Figure 8.7, along with the observed water levels. Table 8.7 summarizes the results of the model calibration for the break-up event. The results of the calibration runs indicate good correlation of calculated ice thicknesses and water levels with those observed and documented. The following comments are made.

- The ice jam between location IJ-1 and IJ-2 on Figure 3.2 had an observed thickness of 2.5 m. This corresponds reasonably well with Sections 1+591 to 2+177, resulting in an average ice thickness of 2.0 m with a maximum thickness of 2.3 m.

- A calculated water level of 23.4 m at Section 2+177 corresponds to the historical level of 23-23.5 m at location IJ-1 on Figure 3.2. Calculated water levels in the reach from Section 1+347 to Section 2+177 indicate that limited flooding would occur (approximately 60 m of flooding on the north bank, flooding up to front of Tanglewood Ranch).

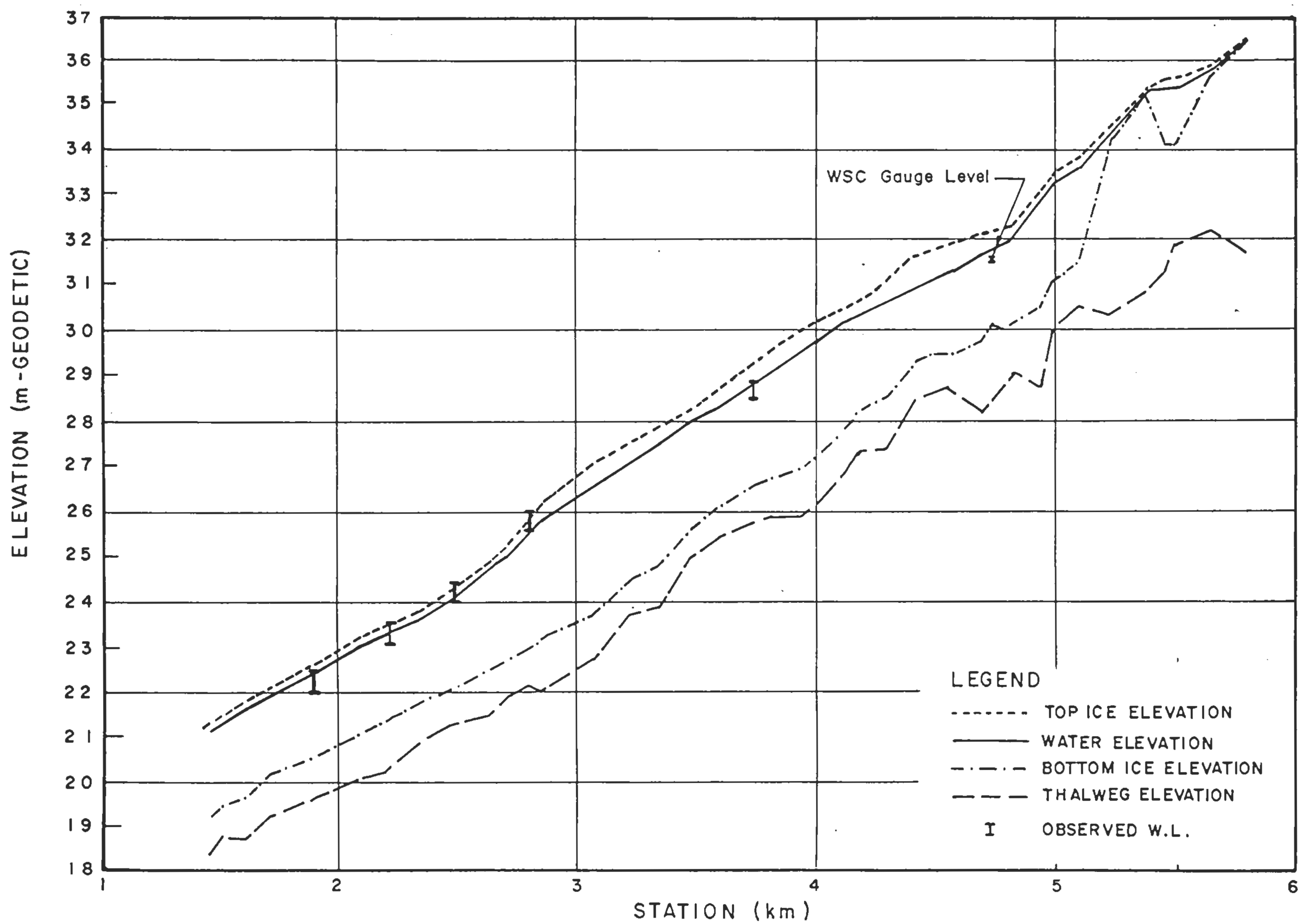


FIG. 8.7

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

FEBRUARY 1984 BREAK-UP CALIBRATION

ACRES

TABLE 8.7

COMPARISON OF OBSERVED AND CALCULATED WATER LEVELS AND ICE THICKNESSES FOR FEBRUARY 1984 BREAK-UP EVENT - MODEL CALIBRATION

ICESIM Section	Water Levels (m) Observed*	Water Levels (m) Calibrated	Ice Thicknesses (m) Observed**	Ice Thicknesses (m) Calibrated
1+950	22-22.5	22.6	2.5	2.1
2+177	23-23.5	23.2	2.5	2.1
2+492	24-24.5	24.2	-	2.2
2+802	25.5-26.0	25.5	3-3.5	2.9
3+717	28.5-29.	28.9	-	2.6
4+627	31.8+	31.9	-	1.7

*Estimated from documentation of February 1984 break-up event (14).

**No detailed observations on ice thicknesses are available but ice thicknesses estimated to be on the order of 2.5-3.5 m in jam locations (14).

+ From WSC gauge records.

- The February 1984 ice jam reached an observed height of 3.5 m in front of the island at Hickey's, and water levels were about 3 m above Hickey's property. This corresponds to elevations of 25.5-26.0 m on the topographic map. A water level of 25.5 m was calculated at Section 2+802. The calculated average ice thickness through the reach at Hickey's (Section 2+492 - 2+802) is 2.3 m, with water levels ranging from 24.2 m to 25.7 m.

- At the transmission line (Section 3+488), the calculated ice thickness is greater than 2.5 m and the water level is 28.4 m, indicating flooding over Hobbs' property which agrees with documented flood observations.

8.4.3 - Model Verification

Although break-up events have been observed in other years, such at 1979, sufficient data is not available to enable a true verification of the break-up model parameters. The only available information for model verification for the break-up event in 1979 is anecdotal evidence from local residents and some documentation by Newfoundland Department of Environment. This evidence suggests that flooding during this event was similar to that experienced in the February 1984 break-up event. The model was run for the 1979 event and the results indicated flooding at the same location on Hobb's farm as in the 1984 event, but slightly higher water levels and a wider flowpath. The model also predicted levels which would cause flow across Campbell's road, as was reported.

8.4.4 - Criteria for the Selection of Historical Break-Up Events

Ice cover break-up is generally caused by a combination of increased discharge, thermal ice weakening and melting. Break-up can occur from any one of these changes individually, as well. In fact, the most severe flooding events occur in mid-winter if there is a sudden increase of discharge and a strong cover in place.

Harry's River commonly experiences variable winter temperatures. During some years a cover does not even progress upstream to the study area, while in many other years the temperatures are not sufficient to adequately strengthen the ice cover to withstand larger forces associated with greater discharges. When subject to higher temperatures in the spring, the cover weakens and melts quickly causing the cover to move out of the reach without an ice jam forming. Residents report that this process frequently occurs; historical major break-up jams are reported in only four years, 1936, 1951, 1979, and 1984.

The screening methodology to identify potential break-up events was as follows.

1. Two mild winters 1969 and 1983, when the ice cover did not progress as far as the bottom of the study reach (based on the freeze-up analysis) were eliminated.

2. Meteorological and flow records were examined in detail to identify potential break-up events. Snowpack, rainfall, temperature and flow records were reviewed concurrently for the entire winter (usually December to March) for each year.

3. If any of these records indicated a change which could contribute to a break-up, the conditions at the same time in the other records were noted. These conditions included:

   - a sudden decrease in snowpack (eg. 30-40 cm in 2-3 days).

   - warm temperatures (e.g. 10-12 cumulative melting degree days in a 2-4 day period).

- sudden increase in flow (e.g. 30-50 m^3/s in a day), particularly in conjunction with disappearance of "B" indicator of ice conditions on WSC records.

- rainfall (eg. 30-40 mm over 2 days).

Twelve potential events were identified in this way and are listed in Appendix G.

4. These potential events were further screened by a more careful examination of the flow records and consideration of the freeze-up process in the particular year. The results of the freeze-up analysis indicated that for seven of the twelve events identified, the temperatures were not cold enough to enable the ice cover to strengthen sufficiently to be able to withstand larger forces associated with greater discharges. In the spring, these ice covers weaken and melt quickly, so that no ice jamming occurs. This additional screening left five events for detailed analysis -1970, 1972, 1976, 1979, 1984.

5. Chart records were requested from WSC archives for the relevant periods for the remaining 5 events. If the chart showed only a gradual increase in levels at the gauge, followed by a gradual decrease, (and continuous B's in the WSC record), the event was not considered to cause ice jams. The March 1972 potential event was eliminated in this way. For the January, 1976 potential event, the chart record was not available. After careful consideration, however, the event was eliminated for the following reasons

   - The freeze-up analysis showed that the ice cover reached the bottom of the study area about 10 days before the potential event. The cover then took an estimated 20-25 days to progress through the reach. There would therefore not be sufficient ice to form a jam at the time of runoff increase.

- The daily gauge record shows continuous ice conditions both during and after the increase in flow, probably indicating backwater effects occurring from slush and border ice.

6. The remaining 3 events were modeled in ICESIM. Two of these three are known jam events (1979, 1984). The remaining one is 1970, and residents report that a jam did occur in the early 1970's, although the date is uncertain. The post-excavation data sets were used to reflect the impact on future flooding. (Note that both the 1979 and 1984 events were independently modeled for the calibration and verification using the pre-excavation data sets).

8.4.5 - Break-Up Flood Profiles

Break-up flood profiles were determined by selecting the most likely jam locations and subsequently modeling at each jam location in each year. Four possible locations were modeled in the three years for a total of twelve potential break-up jam events.

Potential jam locations for the post excavation cases were identified by reviewing the conveyance at each section through the study reach. (Conveyance tables are part of the ICESIM output data.) Six locations showed a significant change in conveyance between sections upstream and downstream, suggesting the possibility of a jam. Three were at the same general locations as documented in the February, 1984 event and identified in the field program (Appendix B) namely Sections 2+063, 2+802, and 4+212. The other three included one section about halfway between the transmission line and the bridge (Section 4+227), the second at a narrowing of the river just upstream of the transmission line (Section 3+904), and the third was just upstream of Black Duck Brook at Tanglewood Ranch (Section 1+347). Further examination showed that the first two Sections (4+227 and 3+904) are located in steep reaches, where jams are very unlikely to form. In fact these sections were not changed in the recent channel modifications, and no jams have been reported at either of these locations, so these sections were not considered further.

The third potential site for ice accumulation (Section 1+347) is located at a bend in the river. At this location, the river is very shallow and subject to a sudden constriction. A deep reach immediately downstream provides a stable cover for resistance. Water levels have been observed in some years to flood adjacent properties upstream.

The lodgement location of lower White's farm (approximately 1.5 km downstream of the study reach) is the key jam location in the river. This jam will most likely be the most stable requiring the greatest discharge to wash it out. No flooding problems have been reported here, presumably since local flooding occurs so regularly that no development has taken place on the flood plain.

The calibrated ICESIM model was then used to determine the break-up water levels along the river due to jams at the four locations. The post-excavation cross section data was used. The ice jam at section 1+950 (identified as IJ-1 on Figure 3.2) was not stable using the calibrated ice parameters for the post-excavated data set. It was only marginally stable using a critical Froude number of 0.15 with larger deposition/erosion velocities. This implies that following the excavation of the island, the jam will probably not form here unless the river geometry changes. Therefore, this jam was not considered in evaluating flood levels. The jams were stable at the other three ice locations.

Key sections were selected for the frequency analysis of water levels. The highest level at each of these sections due to the effect of any of the three break-up jam locations was tabulated. These maximum water levels due to break-up events are given in Table 8.7.

8.5 - Effects of Channel Modifications

There are two main benefits likely to be realized from the recent channel modifications undertaken by the Newfoundland Department of Environment. These benefits assume that the channel is maintained in the post-excavation condition.

The first is that lower water levels will result upstream of the Tanglewood Ranch for any equivalent discharge at formation, and also at break-up, if the jam locations remain the same.

This effect can be shown in Figure 8.8 which compares the 1985 freeze-up calibration with a run having the post excavation cross sections. The flood levels will improve in the vicinity of Hickey's farm by about 0.5 m.

A second benefit is that ice jamming just upstream of Tanglewood (section 1+950) is not likely to occur in the future due to removal of the island as discussed in Section 8.4. The large downstream forces which used to be absorbed by the island across from Hickey's farm and also by shallow shoal areas will now be passed downstream. Previously the downstream cover was able to withstand the external forces and allow a jam to form. Now the forces will probably be too great to be supported by the downstream cover, and the jam will release. This will bring immediate relief from the water levels at Hickey's farm.

TABLE 8.7

ICESIM BREAK-UP WATER ELEVATIONS (m) 1969-1985

	SECTION NUMBER *											
YEAR	1347	1591	1950	2177	2552	2917	3488	4011	4627	4883	5060	5382
1970	22.50	22.67	23.73	24.57	25.96	27.20	28.72	30.26	32.95	33.89	33.96	35.81
1979	21.60	22.25	23.29	24.32	25.71	27.07	28.51	30.26	32.24	32.81	33.55	35.36
1984	21.45	22.29	23.24	24.38	25.74	27.11	28.55	30.29	32.01	32.83	33.57	35.45

* Sections can be located on Figure 4.1.

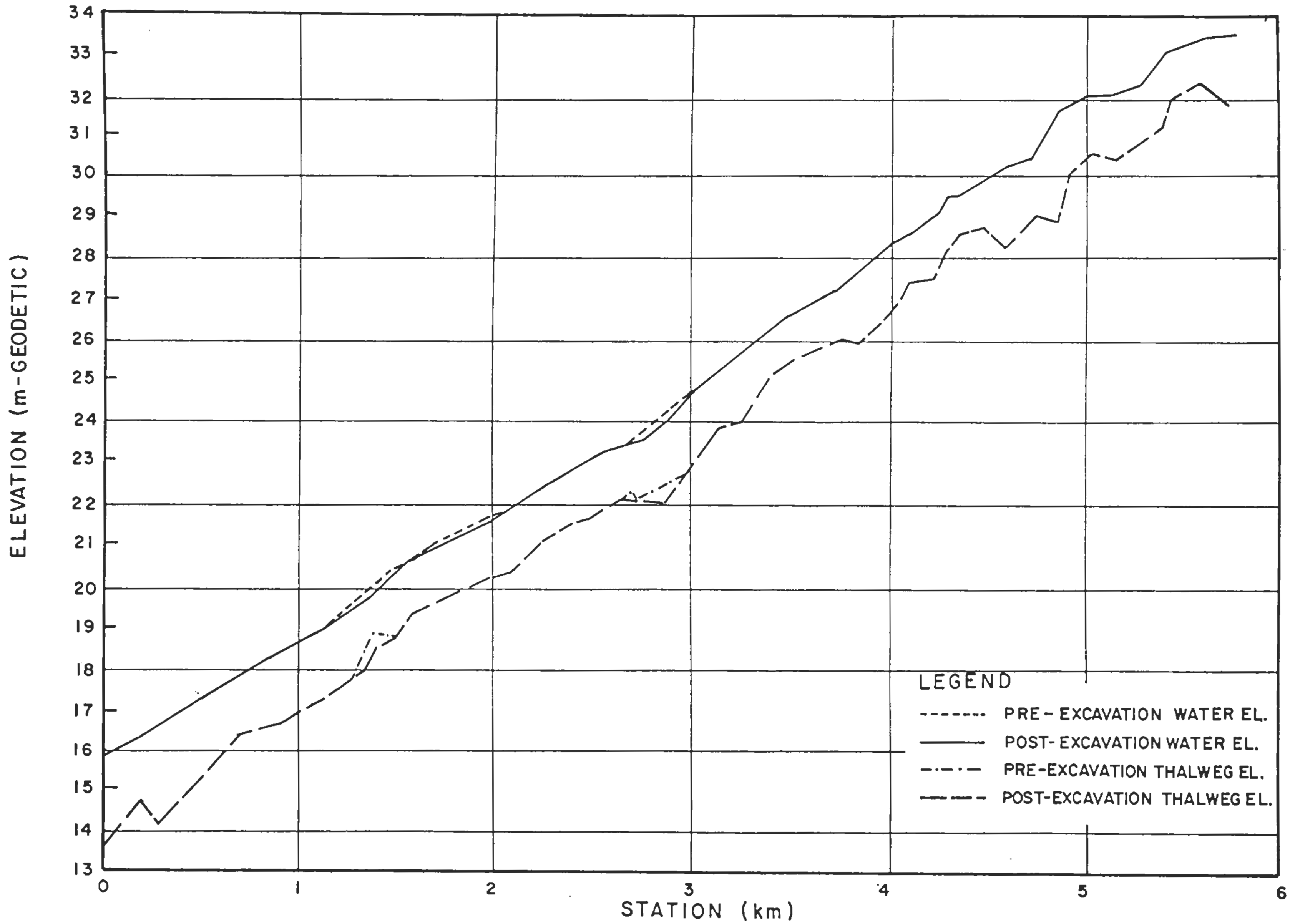


FIG. 8.8

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

JANUARY 1985 FREEZE-UP/CHANNEL MODIFICATIONS: COMPARISON OF WATER LEVELS

ACRES

9 - DETERMINATION OF 1:20 AND 1:100 YEAR FLOOD PROFILES

The frequency analysis for Harry's River is complicated by the fact that high water levels can be caused by any of 3 types of event, freeze-up, break-up and open water. A fundamental assumption of frequency analysis is that the sample set is homogeneous, and in the case of Harry's River, it clearly is not. Not only can all three types of events lead to high water levels at most cross sections, but also the dominant type of event can vary along the reach. Freeze-up events dominated at about a third of the cross-sections selected for detailed analysis, particularly in the downstream part of the study reach. Breakup events dominated at about half the cross-sections analyzed, chiefly in the middle part of the reach. In the remaining cases, either open water events dominated, or else two types of events both contributed about equally to the combined frequency curve.

As a result of these considerations, separate frequency curves were prepared for each type of event at each cross-section, and then mathematically combined.

The analysis proceeded as follows.

1. Select a number of representative cross-sections along the reach (14 sections were chosen, i.e. approximately every third or fourth cross-section).

2. Plot a frequency curve for each type of event (freeze-up, break-up, open water) for each section using the annual series of water levels at each section from the open water and ice simulations (described in Sections 7 and 8).

3. Select several trial water levels at each cross section. For each type of event (open water, freeze-up, and breakup) obtain the probability of occurrence from the frequency

analysis (i.e. 1 - cumulative probability) of the trial water levels. Multiply the probabilities obtained from the frequency analysis by the probability that an event of that type will occur in any year.

Freeze-up and open water frequency analyses were done using the Consolidated Frequency Analysis Package (CFA88) (23). Break-up analysis was done by manually plotting the 3 available points at return periods calculated using the Cunnane plotting position formula, and drawing a best-fit line.

4. Calculate the total probability of exceedence of each of the trial water levels by summing the probabilities of occurrence for each type of event. (The very small contributions of the joint probability terms in the probability relationship are neglected.)

5. Plot the sum of the probabilities of each of the trial water levels. From the resulting curve, obtain the water levels corresponding to return periods of 1:20 and 1:100 years.

All water levels used in the frequency analysis are for the present condition, i.e. with 1986 channel improvements.

Tables 9.1 and 9.2 and Figure 9.1 show a sample calculation for a cross-section located just downstream of the confluence with Trout Brook. The frequency curves have been adjusted to take account of the likelihood of occurrence of each type of event (18/18 for open water, 16/18 for freeze-up and 3/18 for break-up events). The detailed computer output used to prepare the plots is given in Appendix H.

Table 9.3 presents 1:20 and 1:100 year water levels estimated at each of the selected cross-sections. Figures 9.2 and 9.3 show the 1:20 and 1:100 year water surface profiles and Figures 9.4 and 9.5 show the resulting extent of flooding.

9.1 - Discussion

The 1:20 and 1:100 year flood levels were predicted using the following relationship from probability theory

$$P_t = P_o + P_f + P_b - (P_o*P_f) - (P_o*P_b) - (P_b*P_f) + (P_o*P_b*P_f)$$

where

P_t = annual total probability of given water level
P_o = annual probability of given water from open water event
P_f = annual probability of given water level from freeze up event
P_b = annual probability of given water level from breakup event
* = joint probability

The preparation of the frequency curves required an estimate of the likelihood of occurrence of each type of event. Open water events occur every year, and the probability that an open water event will occur is therefore 1.0. Calculations based on meteorological data indicated that freeze-up events would have occurred in 16 out of 18 years, for a corresponding probability of occurrence of 0.889. This is consistent with anecdotal reports. For the break-up events, only 3 events from the 18 year period were available for analysis. A check of meteorological data for the period before the establishment of the streamflow gauge was therefore carried out.

The available precipitation, degree-day and snow cover data for the previous 17 years (1951-1967) were examined, and about 3 periods were identified when conditions were conducive to break-up events with two additional marginal cases. No documentation is available. Even if water levels were recorded, they would probably not be directly comparable to more recent levels since changes have occurred in the overbanks (particularly clearing of land). The fact that meteorologic conditions likely to be associated with break-up events occurred with about the same frequency (3/18 years, compared with 3/19 analyzed) does lend support to the analysis, however.

It is also appropriate to note that in more typical flood frequency analyses, all types of storms, channel conditions and often ice are lumped together in the analysis, so that the extreme events are likely to be represented by a small population of the sample. In this work, these unusual but severe types of event have been separated out for analysis and the limitations identified. We would therefore anticipate the reliability to be similar to that where estimates have been generated from an 18 year record.

TABLE 9.1

MAXIMUM WATER LEVELS - SECTION 2552

Year	Maximum Water Level Due to Open Water	Freeze-Up	Break-up
1969	24.41	-	-
1970	23.89	23.25	25.96
1971	23.47	23.16	-
1972	24.27	23.32	-
1973	25.11	23.27	-
1974	24.05	23.25	-
1975	24.39	23.02	-
1976	24.23	24.44	-
1977	23.81	23.41	-
1978	24.10	23.24	-
1979	23.74	23.27	25.71
1980	24.43	23.26	-
1981	23.50	23.41	-
1982	23.89	23.18	-
1983	23.55	-	-
1984	23.58	24.26	25.74
1985	24.30	23.26	-
1986	-	24.57	-

TABLE 9.2

SAMPLE CALCULATIONS - SECTION 2552

P (WL = 26.3)

Open Water	(1 - 0.99985)	=	0.0002
Break-Up	(1 - 0.998)	=	0.0020
Freeze-Up	(1 - 0.9994)	=	0.0006
	P (26.3)	=	0.0028*
	T_r	=	360 yrs

P (WL = 26.0)

Open Water	(1 - 0.9995)	=	0.0005
Break-Up	(1 - 0.985)	=	0.0150
Freeze-Up	(1 - 0.9988)	=	0.0012
	P (26.0)	=	0.0167*
	T_r	=	60 yrs

P (WL = 25.7)

Open Water	(1 - 0.9982)	=	0.0018
Break-Up	(1 - 0.90)	=	0.1000
Freeze-Up	(1 - 0.9976)	=	0.0024
	P (25.7)	=	0.1042*
	T_r	=	10 yrs

Notes:

(*) These points are plotted on Figure 9.1, and used to draw the curve to obtain 1:20 and 1:100 year flood levels.

Tr = Return period in years
P(x) = Probability of water level reaching or exceeding x m above sea level in any one year.

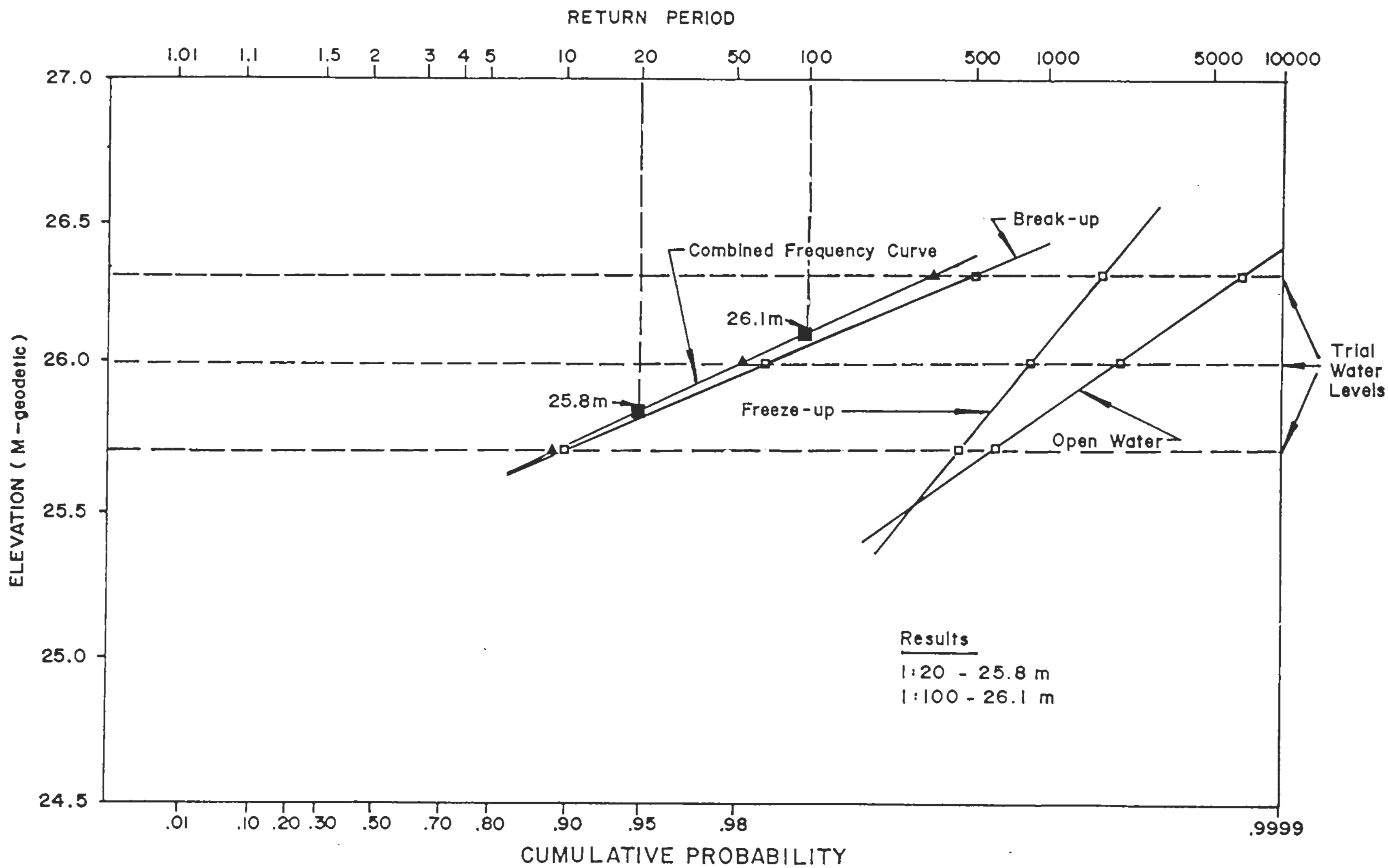


FIG. 9.1

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

FREQUENCY CURVES-SECTION 2552

ACRES

TABLE 9.3

PREDICTED WATER LEVELS - 1:20 AND 1:100 YEAR RETURN PERIODS

	Predicted Water Levels (m)	
Section Number	1:20 Year	1:100 Year
0471	18.1	18.4
0873	19.7	20.0
1152	20.7	21.6
1347	22.2	23.0
1591	22.7	23.2
1950	23.7	24.4
2177	24.8	25.8
2552	25.8	26.1
2917	27.2	27.3
3488	28.6	28.8
4011	30.3	30.4
4627	32.6	33.4
4883	34.2	35.0
5060	34.2	35.0
5382	35.8	36.1

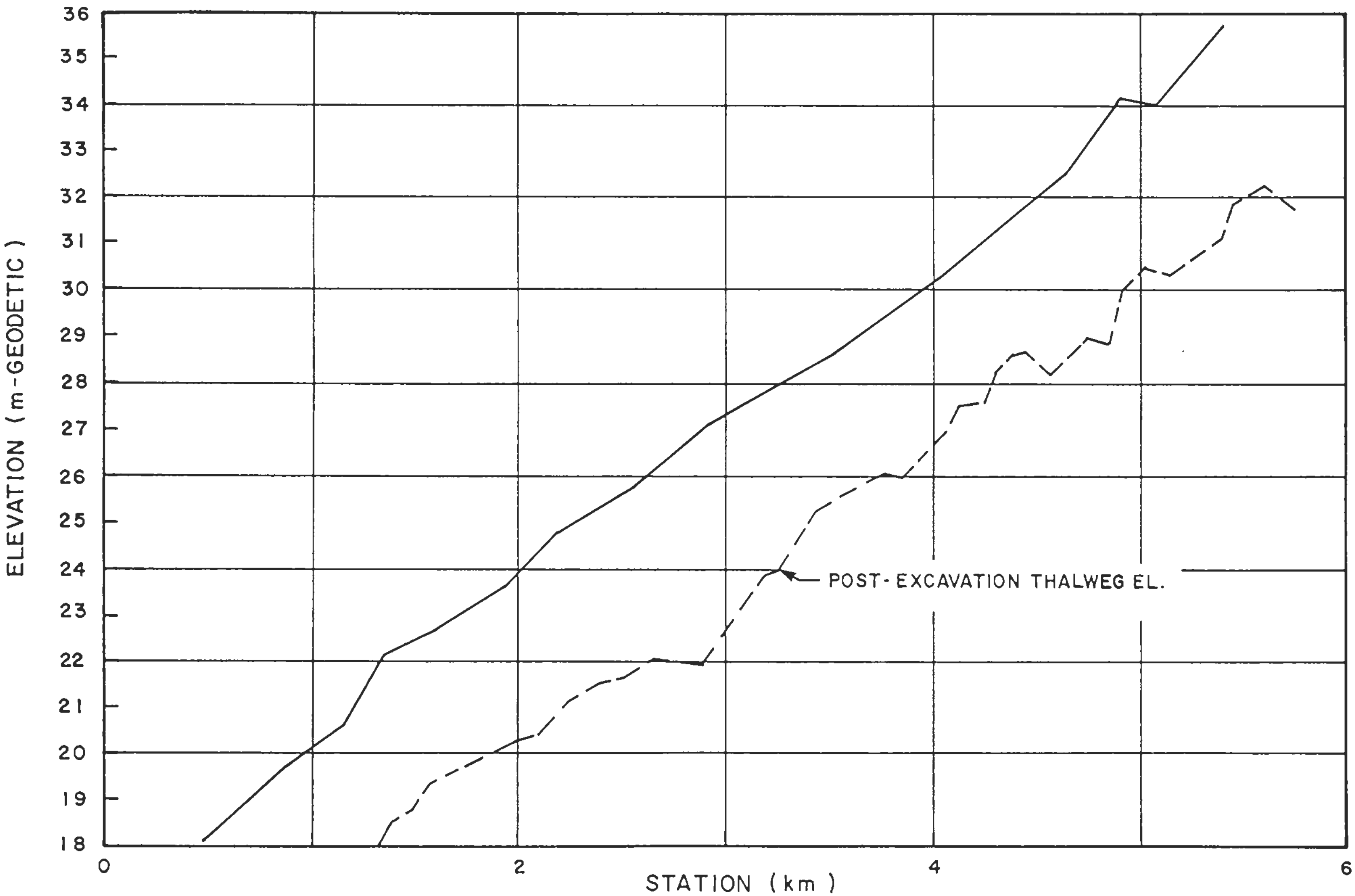


FIG. 9.2

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

FINAL WATER SURFACE PROFILE-1:20 YEAR RETURN PERIOD

ACRES

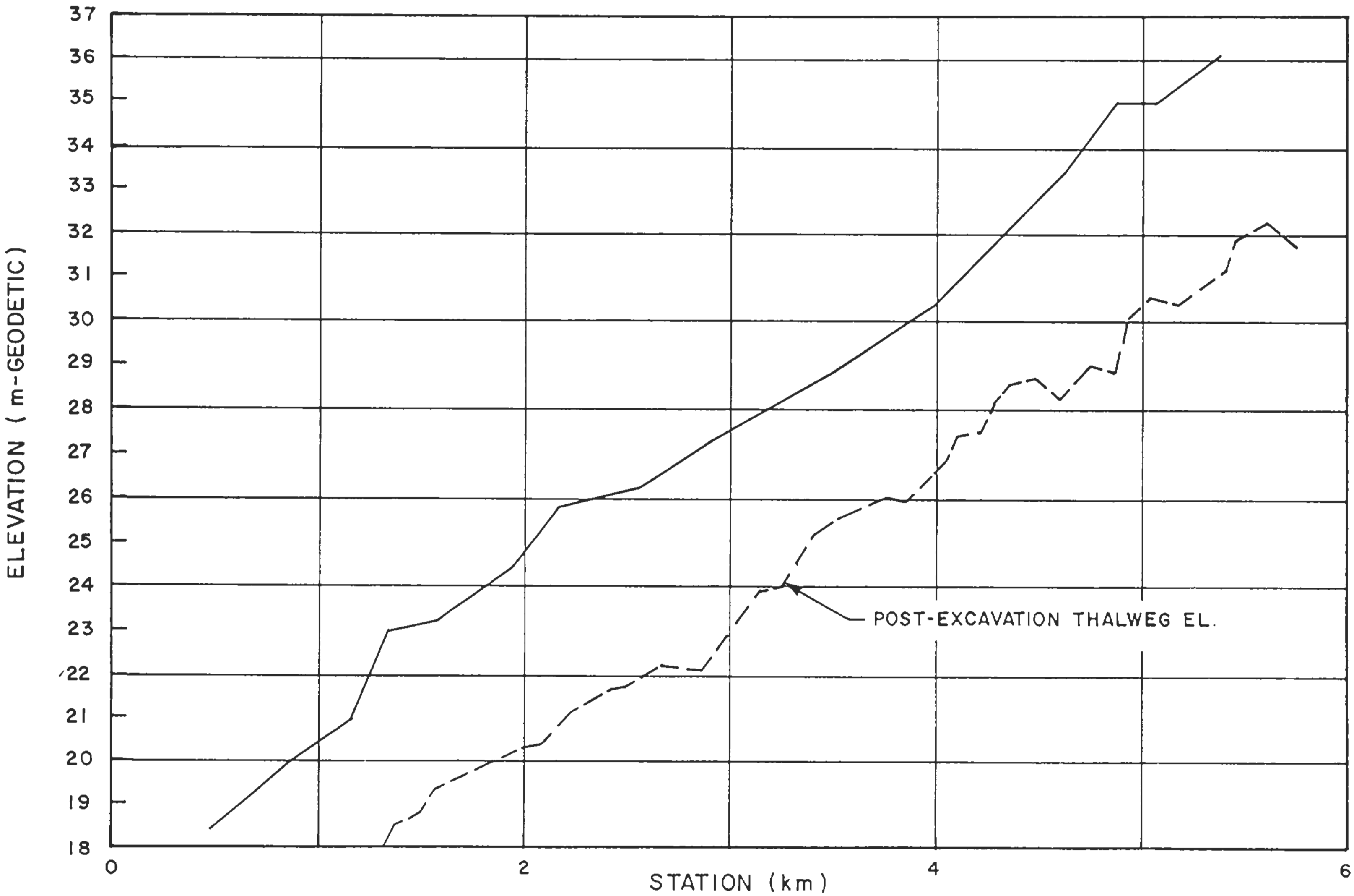


FIG. 9.3

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

FINAL WATER SURFACE PROFILE-1:100 YEAR RETURN PERIOD

ACRES

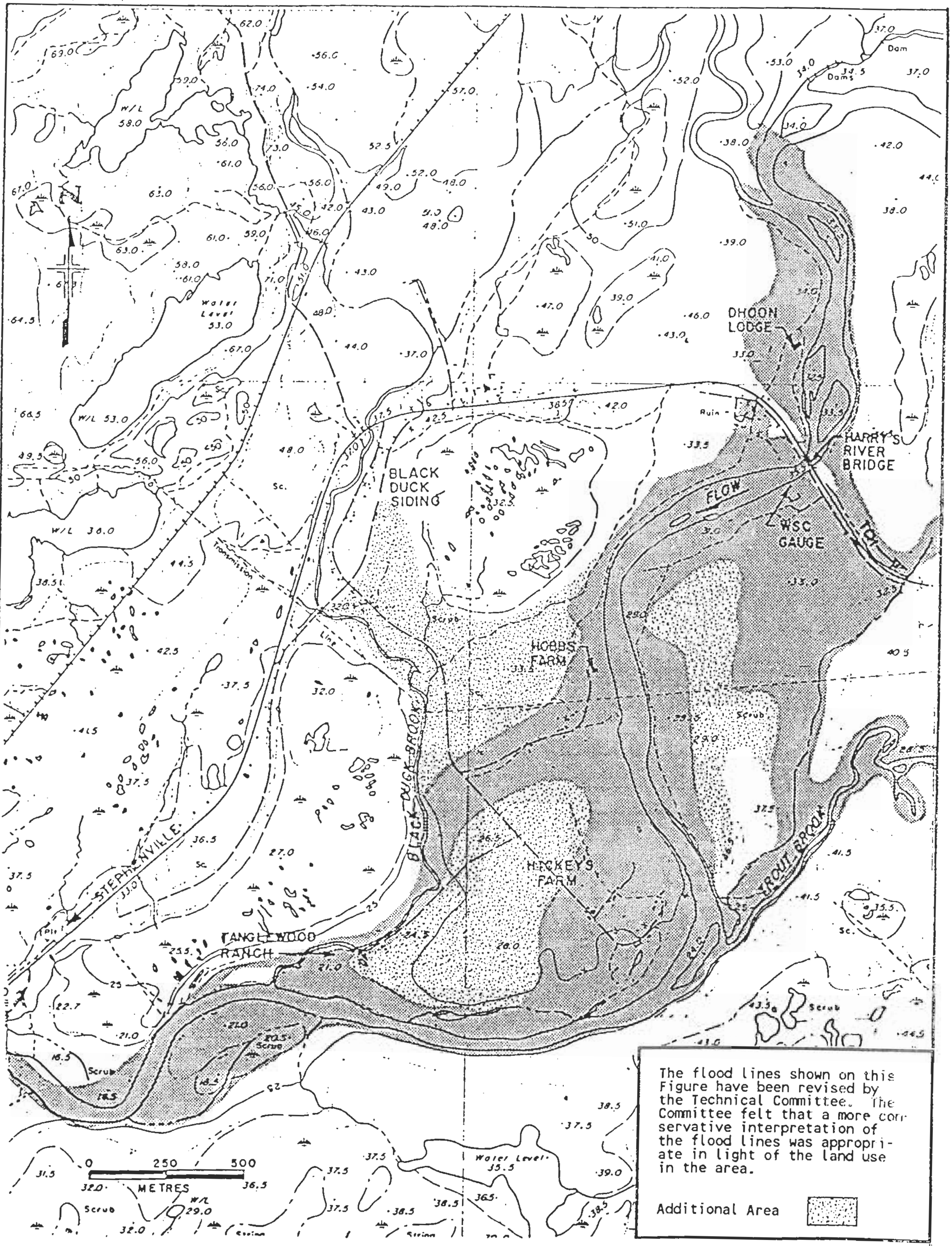


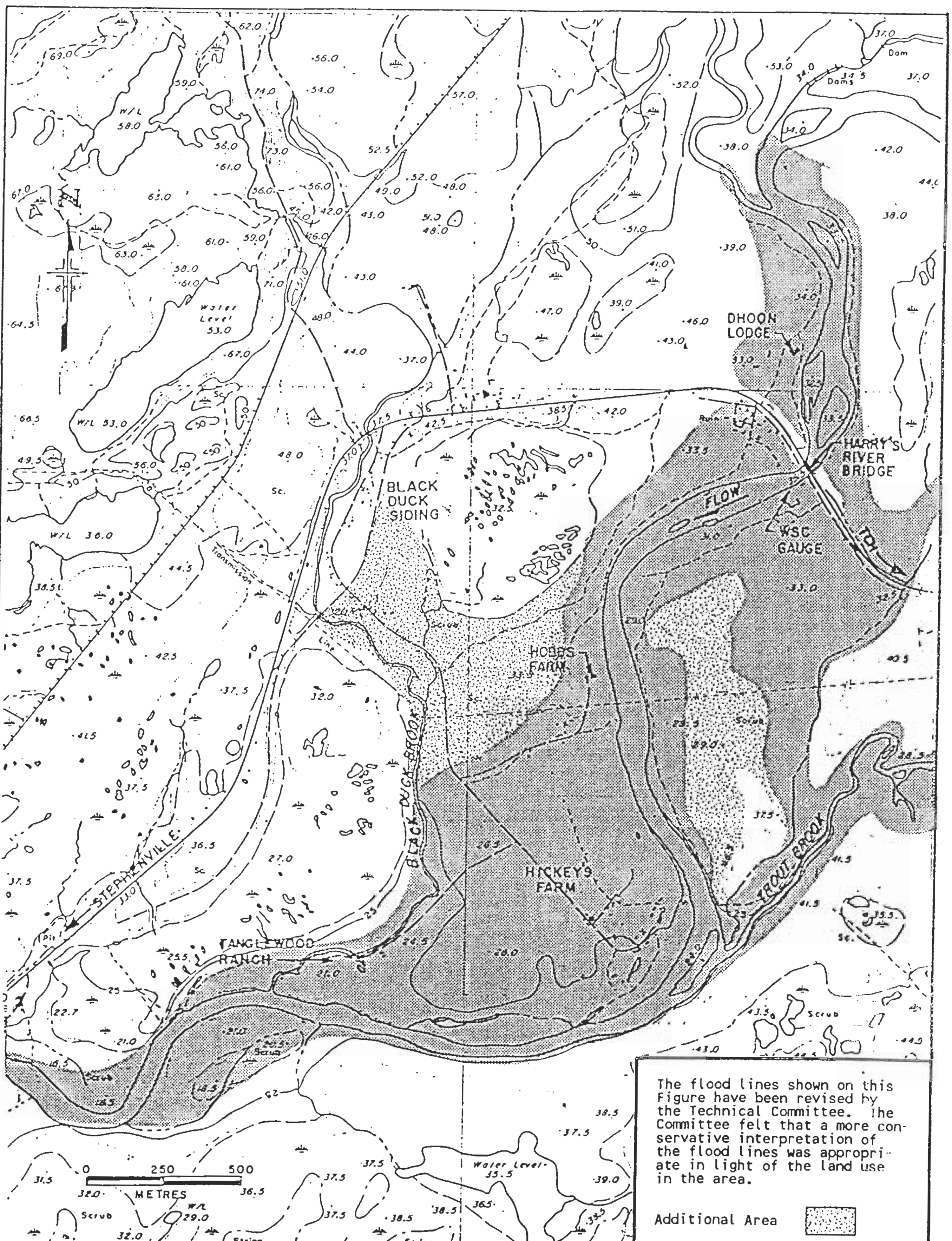
FIG. 9.4

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

BLACK DUCK SIDING - EXTENT OF FLOODING 1:20 YEAR LEVELS

ACRES



The flood lines shown on this Figure have been revised by the Technical Committee. The Committee felt that a more conservative interpretation of the flood lines was appropriate in light of the land use in the area.

Additional Area

FIG. 9.5

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

BLACK DUCK SIDING - EXTENT OF FLOODING 1:100 YEAR LEVELS

ACRES

PART B: STEPHENVILLE CROSSING

10 - BACKGROUND INFORMATION

10.1 - General

Stephenville Crossing, a town of approximately 1200 people, is located 12 km southeast of Stephenville. The community is built on the lowlands at the mouth of the St. George's River and adjacent to Rothesay Bay, as shown in Figure 10.1. The location of Stephenville Crossing makes it vulnerable to flooding from either high levels in St. George's River or overtopping of the beach in Rothesay Bay.

10.2 - Description of Historical Flooding

The two most significant recorded flooding events at Stephenville Crossing occurred December 18-19, 1951 and December 11-12, 1977. The more serious event occurred in 1951 when high seas overtopped the beach at Stephenville Crossing and St. George's River overflowed at the Gut. This flooding was thought to be caused by high winds and tides accompanied by rain and snow. Water washed in over the beach and flooded all the land from Seal Cove down to the Main Gut bridge. West Street was covered by 1.2 m of water. Pleasant Street and the surrounding area of Webb's field were hardest hit by the flood because of the low elevation. Over 600 people from this area were forced to evacuate their homes. The best estimate of the inundation level for the event was about +3.1 m (geodetic).

Stephenville Crossing sustained fairly extensive damage from this flood. All rail traffic, power and telephone services were disrupted and several homes sustained damage. Rail tracks were also washed out, farm animals were killed, drinking water was contaminated by sea water and telephone poles were knocked down.

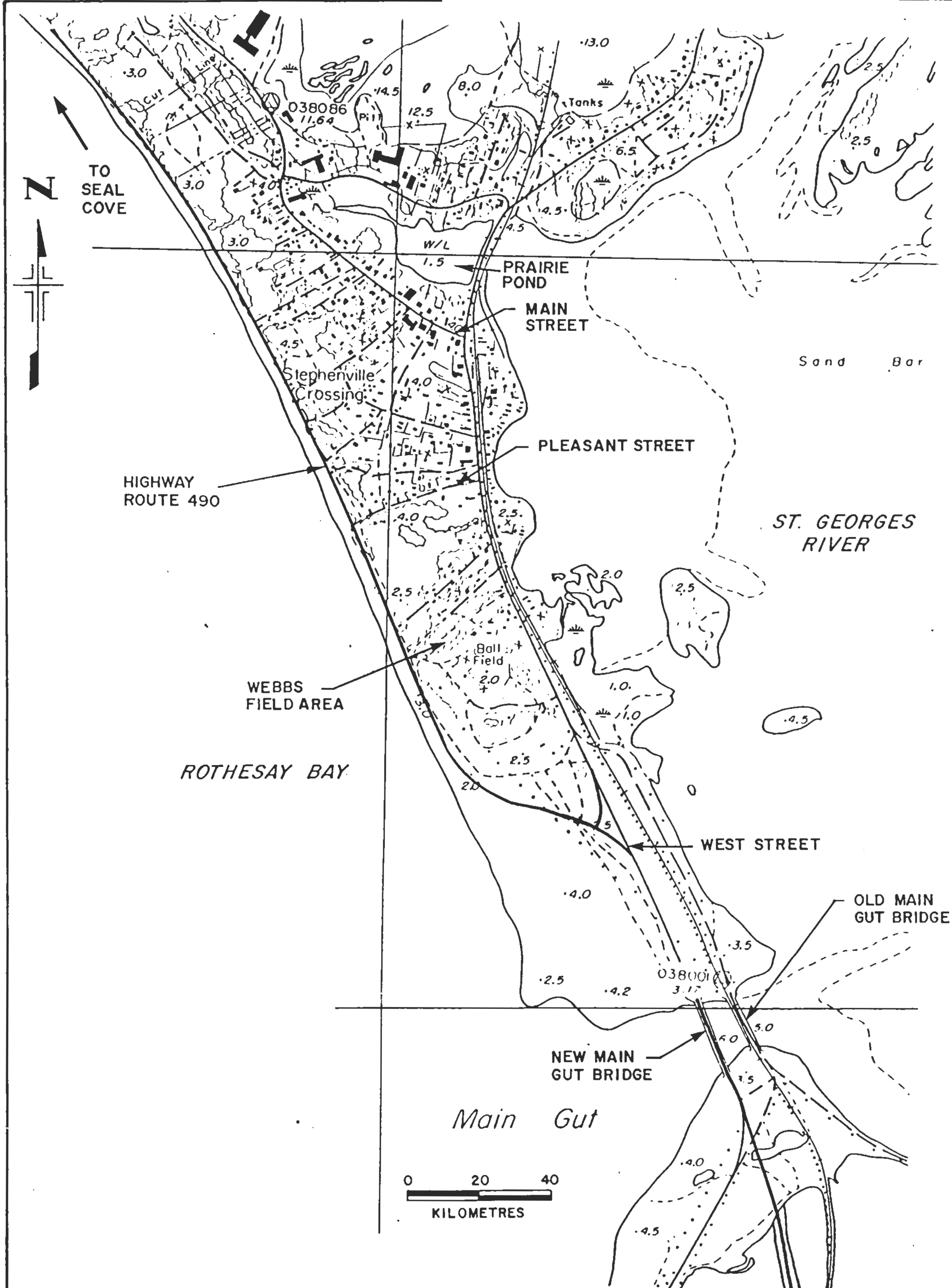


FIG. 10.1

CANADA - NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

STEPHENVILLE CROSSING STUDY PLAN

ACRES

While the flooding that occurred in 1977 was not as severe as the 1951 event, high tides and strong onshore winds were again believed to be the cause. Only Pleasant Street, which was covered by 0.5 m of water, was affected by this event. Five families were forced to evacuate this particular area, and one home was damaged.

Two major changes which could affect flooding have been made in the Stephenville Crossing flood plain since 1977. The first was the construction of a new bridge across Main Gut. The railway bridge remains in place, and the construction of the new bridge required the placement of several additional piers in the channel. The second was the construction of a new highway along the beach. While many of the local population believe this highway will act as an effective break-water, others have expressed a concern that the highway is not high enough to prevent the water from flooding the land. In addition to this, a concern has been expressed that fields along the highway are flooded during periods of heavy rain (from ponding). Subsurface drainage might also be affected.

The analysis for this study was based on the present configuration of the bridge and road.

11 - FIELD PROGRAM

The field program consisted of measuring beach profiles and installing tide gauges. Beach profiles were measured at two locations during the fall of 1986.

The installation of two Aanderra gauges was carried out to provide information on the exchange of water between St. George's River and the sea. The gauges were installed in early December 1986 in locations shown in Figure 11.1. The inside gauge was recovered on December 20 which provided 18 days of data. The outside gauge was never recovered despite repeated attempts by divers.

The Aanderra water level recorder measured pressure (water plus atmospheric pressure) twice hourly. The raw data were analyzed by the Canadian Hydrographic Service (CHS) at the Bedford Institute of Oceanography in Dartmouth, Nova Scotia.

Figure 11.2 shows the measured river level for the monitored period. A table of gauge pressures and corresponding water levels used to prepare Figure 11.2 is given in Appendix I. The hydrostatic pressure was converted to water level using the conversion 1 mb = 1 cm. A discussion on the measurement accuracy is also included in Appendix I.

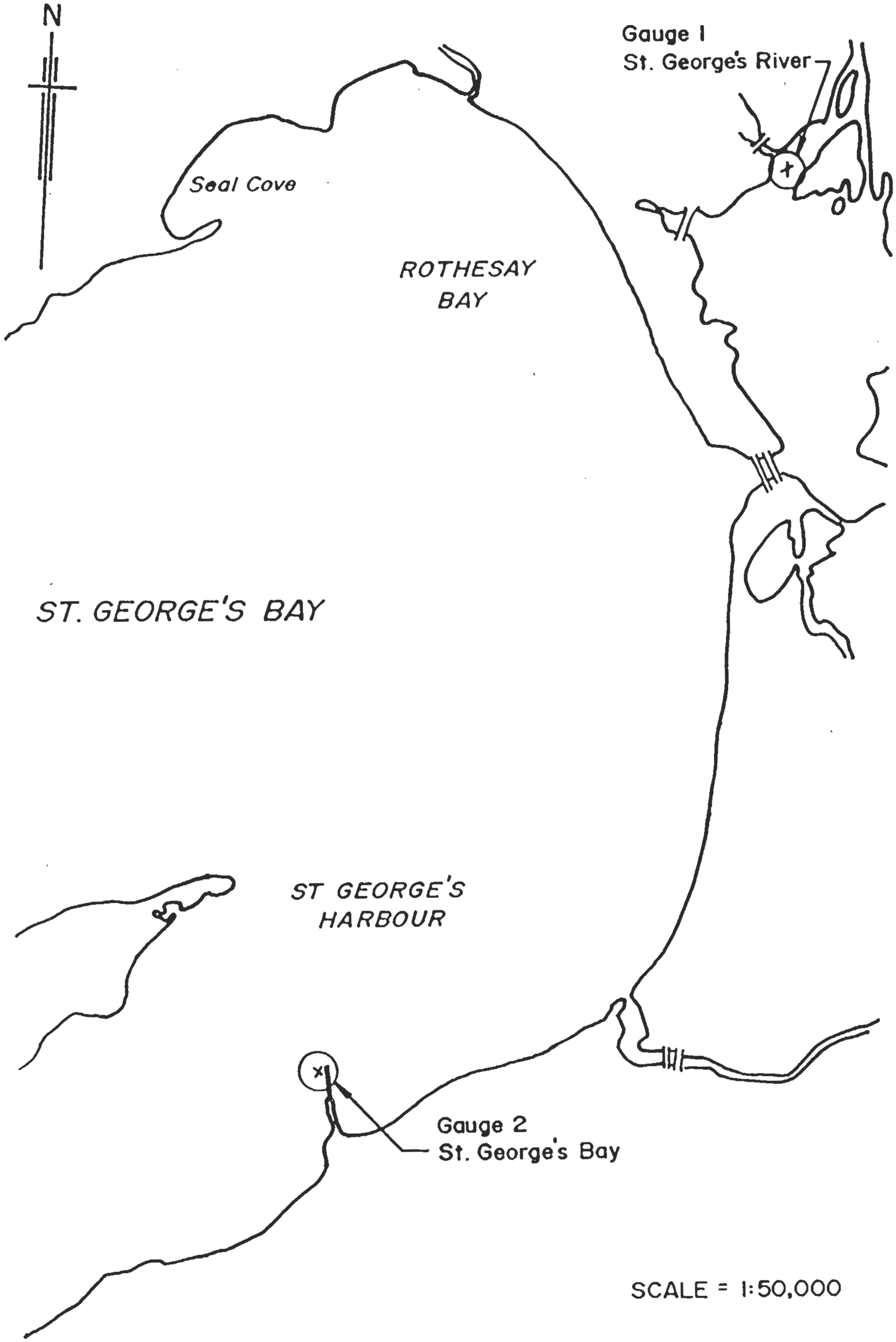


FIG. II.1

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

GAUGE LOCATIONS

ACRES

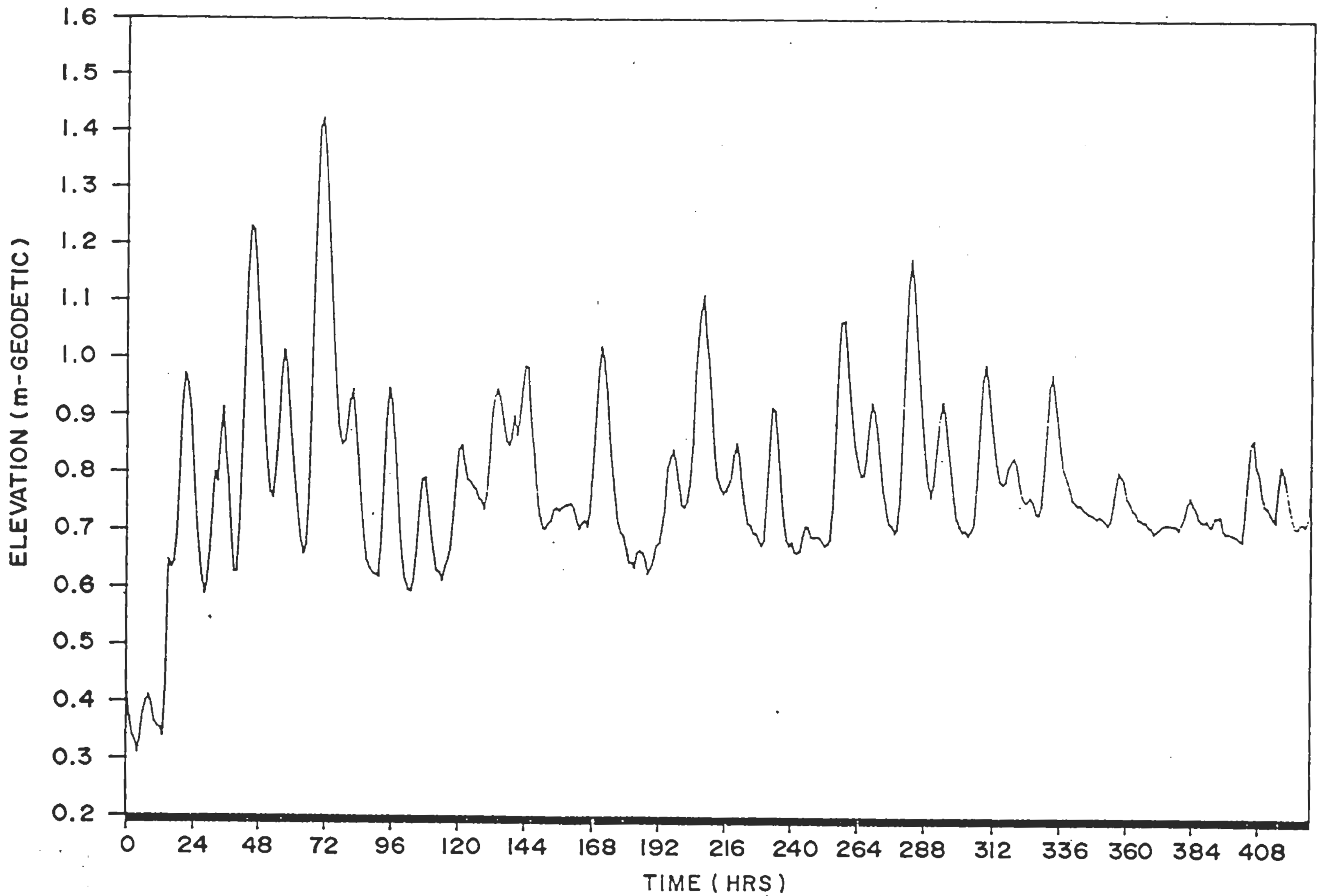


FIG. II.2

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ST. GEORGES RIVER MEASURED RIVER LEVELS

ACRES

12 - STEPHENVILLE CROSSING FLOODING: TECHNICAL ANALYSIS

Flooding at Stephenville Crossing occurs primarily as a result of surge and wave runup in combination with high tides. Surge and waves are caused by atmospheric forcing from prolonged winds from the west, while the tides are astronomically forced. The lee side of Stephenville Crossing (St. George's River) is subject to tide and surge levels similar to those on the side exposed to the Gulf of the St. Lawrence, but is protected from direct wave attack. However, flooding on the lee side can occur if levels on the windward side exceed the height of the land and overflow toward St. George's River. In addition, levels on the leeward side generated by surge and tide can be augmented by river flood flows into St. George's River. The analysis of the St. George's River side therefore required consideration of both surge and fresh water flow events.

Section 12.1 describes flooding arising from storm surge and tides in Rothesay Bay, and Section 12.2 the flooding generated by river flows.

12.1 - Calculation of Water Levels in Rothesay Bay

Raised water levels outside the Main Gut in Rothesay Bay are the result of the combination of tides and wind generated waves and surge. Winds from the westerly quadrant cause an overall super-elevation of static water levels in Rothesay Bay (surge), as well as large waves with the potential for runup.

The procedure used to calculate the combined frequency distribution of water levels due to both atmospheric and astronomic components is summarized as follows.

- Obtain a frequency distribution of water levels due to atmospheric forcing (surge plus runup).

- Prepare a probability density function of water levels due to astronomic tides.

- Mathematically convolve the two to obtain a combined frequency distribution.

The assessment of each component is described in the following sections.

12.1.1 - Tide Levels

The primary tide gauge operated by the Canadian Hydrographic Service (CHS) in the study area is Harrington Harbour in eastern Quebec. The tide levels at St. George's are derived by using published CHS correction factors. The high water elevations for Harrington Harbour are adjusted by subtracting 0.33 m (for a neap tide), 0.46 m (for a mean tide) or 0.61 m (for a spring tide), to yield the St. George's levels referenced to the chart datum. Levels can be converted to geodetic datum at St. George's by subtracting 0.92 m.

Recorded tides for St. George's were obtained from the Marine Environmental Data Service (MEDS) for the 28-day period August-September 1967; and predicted tides for December 1951, 1977 and December, 1986. Figure 12.1 illustrates the tides for the three periods.

A probability density function was derived for the St. George's data using all calculated astronomical high-water levels over a 1-yr period. Only high tide levels were used since the duration of the wave and surge events exceeds a single tidal cycle and coincidence of wind events with high water is very likely. Figure 12.2 illustrates the probability density function. Table 12.1 provides the data used to plot this function.

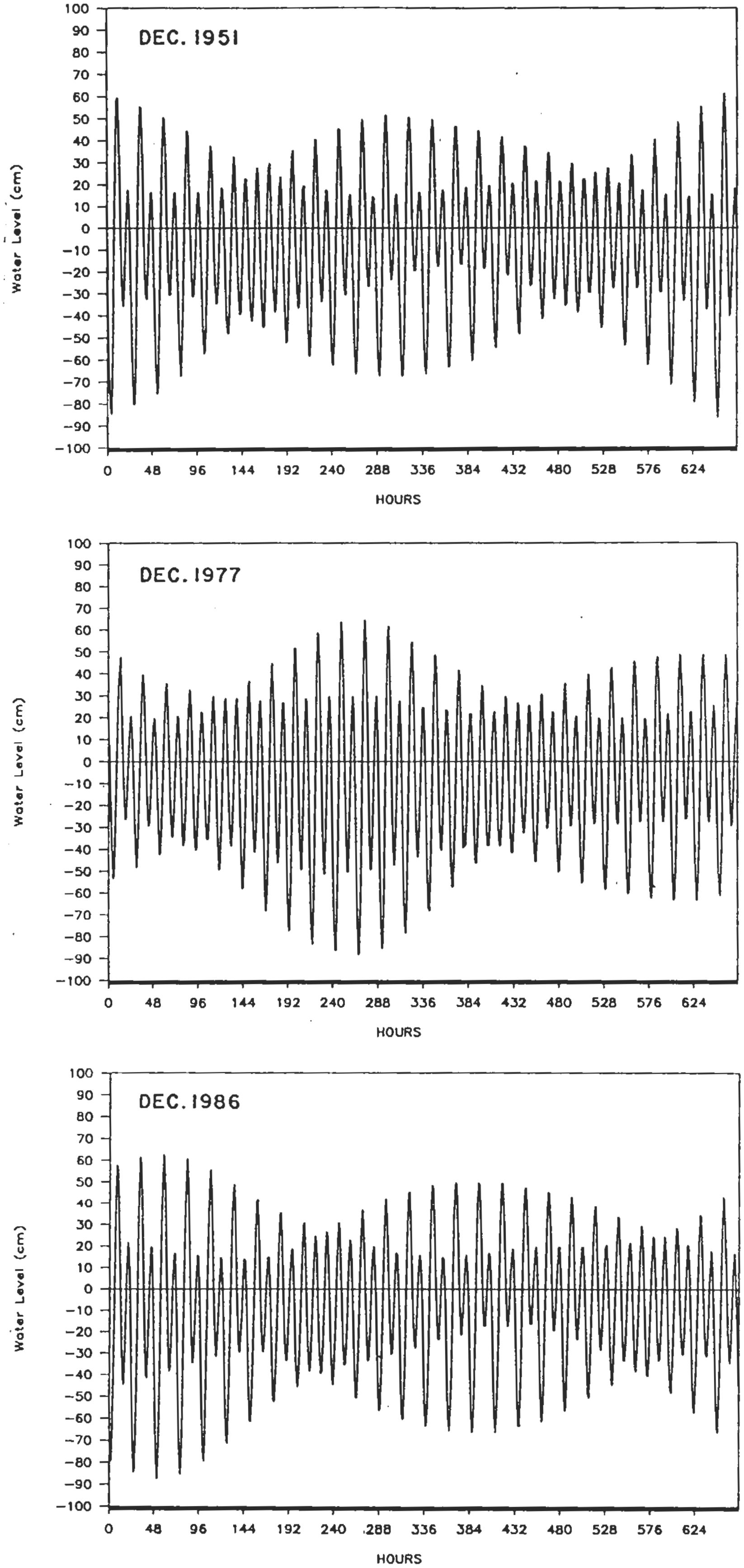
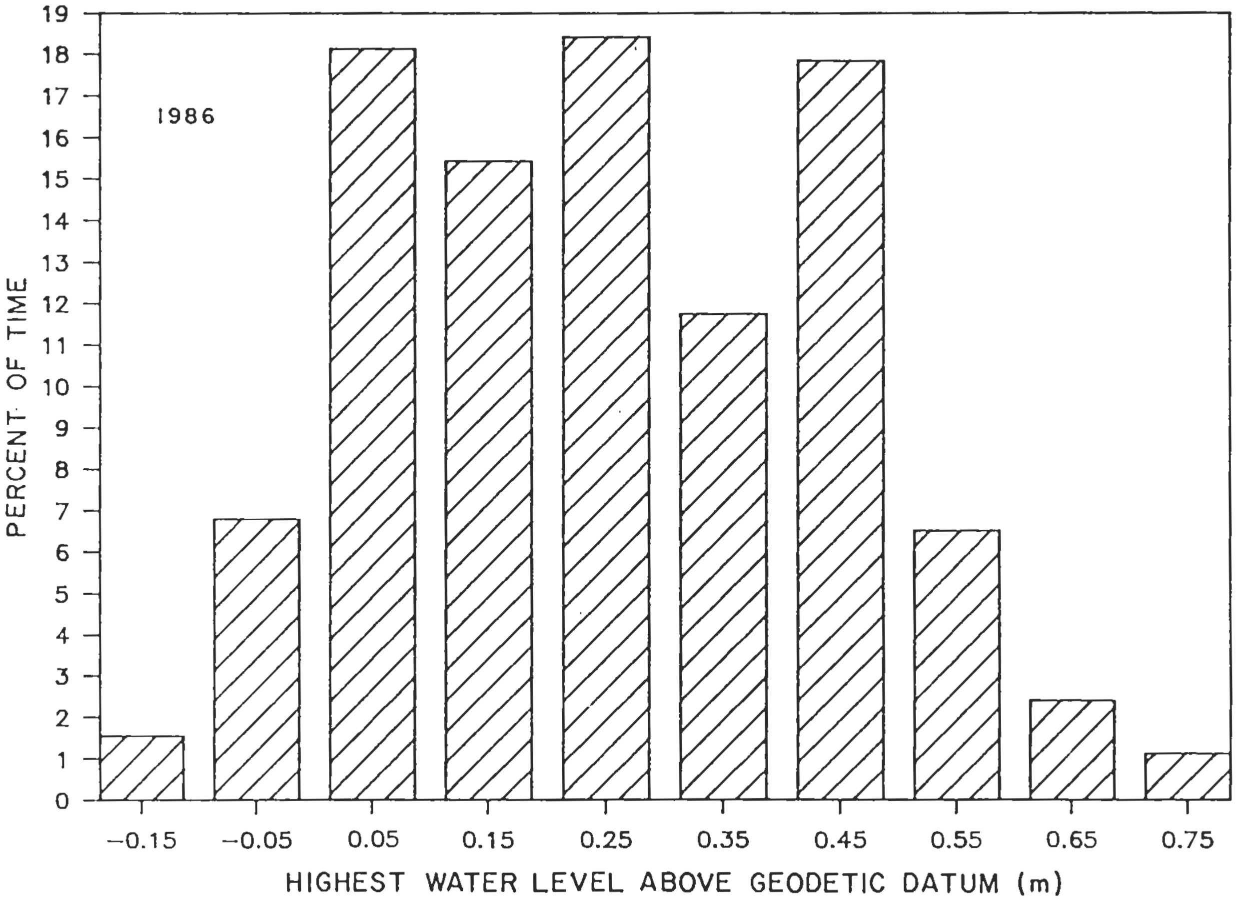


FIG. 12.1

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

TIDE DURING STORM EVENTS-STEPHENVILLE POND

ACRES



SOURCE: ADJUSTED HARRINGTON HARBOUR TIDES
CANADIAN HYDROGRAPHIC SERVICE

FIG.12.2

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS
PROBABILITY DENSITY FUNCTION OF TIDES AT STEPHENVILLE CROSSING

ACRES

TABLE 12.1

PROBABILITY DENSITY FUNCTION: TABLE OF VALUES

Water Level Range[1] (m) From	To	Number of Occurrences	% of Time
-0.2	-0.1	11	1.56
-0.1	-0.0	48	6.80
0.0	0.1	128	18.13
0.1	0.2	109	15.44
0.2	0.3	130	18.41
0.3	0.4	83	11.76
0.4	0.5	126	17.85
0.5	0.6	46	6.52
0.6	0.7	17	2.41
0.7	0.8	8	1.13
SUM		706	100.00

[1] Tide levels at St. George's corrected from Harrington Harbour and adjusted to Geodetic Datum.

12.1.2 - Surge

The magnitude and corresponding probability of occurrence of surge events at Stephenville Crossing were obtained by computing residuals between predicted tide levels and observed levels for the MEDS records at Lark Harbour. It is the nearest gauge with a sufficiently long record to permit a frequency analysis. Harrington Harbour, in Quebec, is the primary gauge in the Gulf of St. Lawrence and is consequently specified by CHS for predicting tide levels at other locations (Section 12.1.1). Lark Harbour is on the west coast of Newfoundland, and is expected to experience superelevation of water level due to surge similar to Rothesay Bay. The locations of the two stations are shown in Figure 1.1. Coincident residual values recorded at St. George's during a 28-day period in September 1967 confirmed that Lark Harbour values are reasonably representative of Rothesay Bay. The plots of residuals for the 2 stations are shown in Figure 12.3.

Hourly data for Lark Harbour were screened from 1963 to 1982 to isolate extreme residual events. A typical case is shown in Figure 12.4(a). The annual maxima were subjected to a frequency analysis as shown in Figure 12.4(b). Characteristic surge events are as follows

1:2 year	0.21 m
1:20 year	0.28 m
1:35 year	0.29 m
1:100 year	0.31 m

12.1.3 - Wave Attack

The water level superelevation caused by waves was determined by the following analysis.

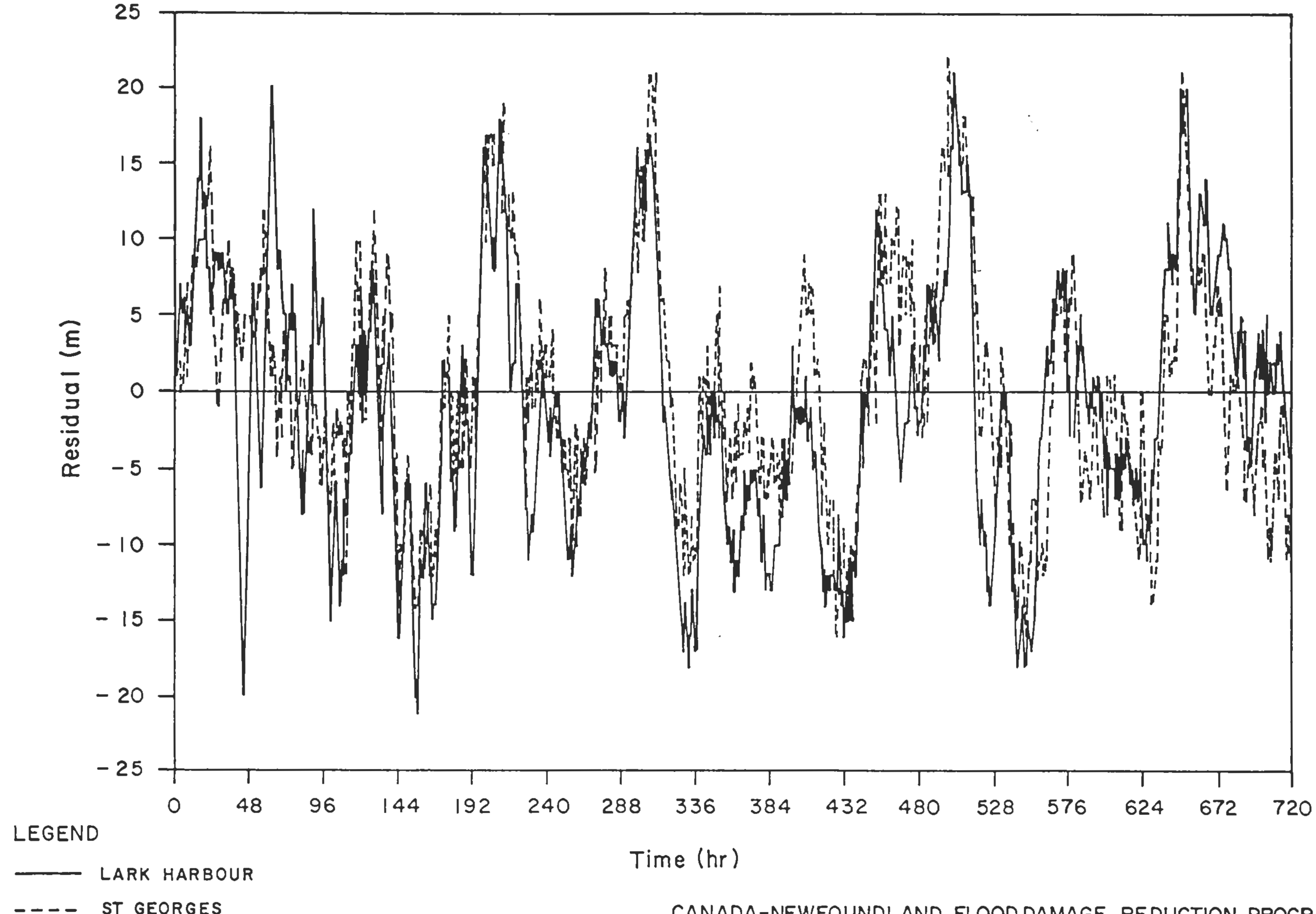
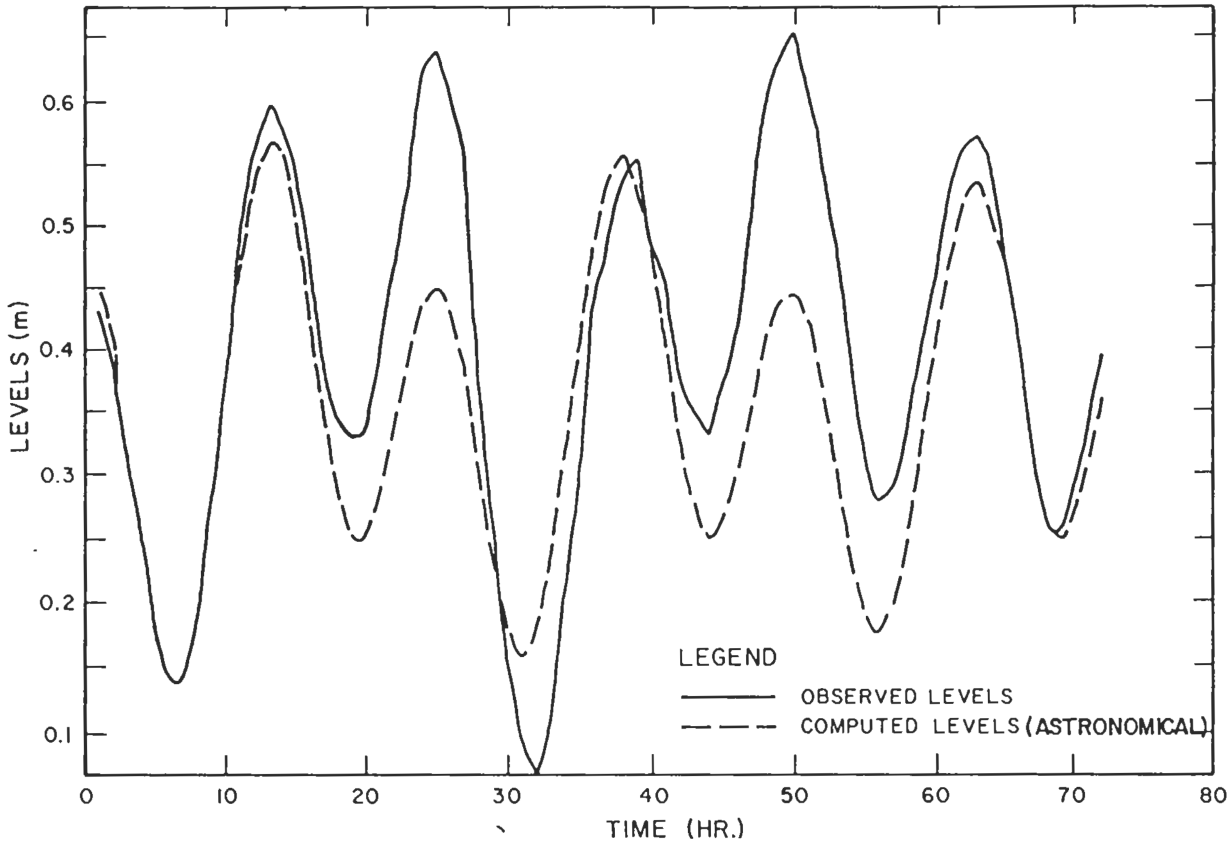


FIG. 12.3

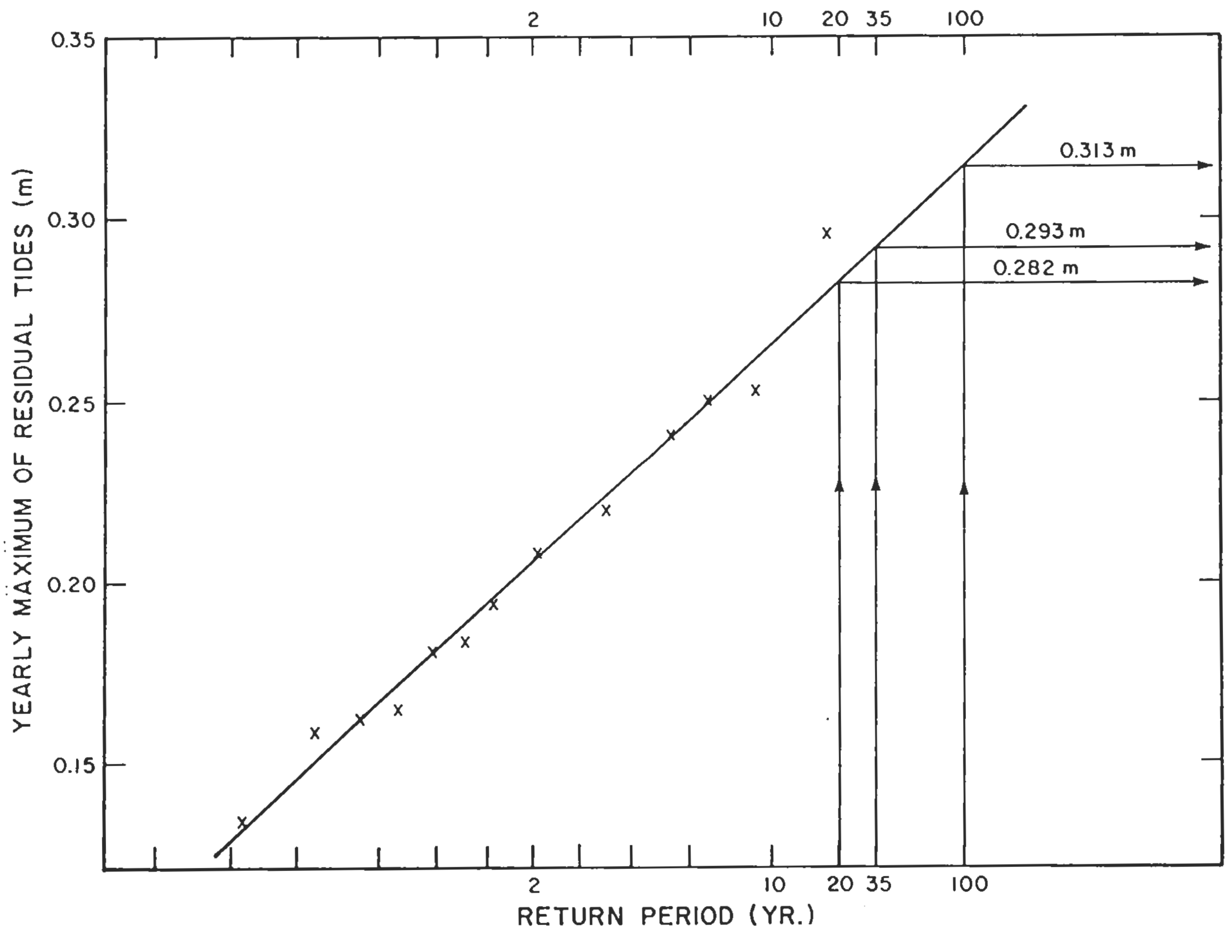
CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

WATER LEVEL RESIDUALS AT LARK HARBOUR AND ST GEORGES

ACRES



a) PREDICTED TIDE AND RECORDED LEVELS



b) PROBABILITY OF OCCURRENCE OF WIND SET-UP

FIG. 12.4

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

LARK HARBOUR-PREDICTED TIDE AND RECORDED LEVELS

ACRES

- hindcast waves at the mouth of St. George's Bay and carry out frequency analysis of significant wave heights at this offshore location

- define the 2 dimensional offshore energy spectrum in terms of both wave frequency and direction

- transform the offshore spectra to an inshore spectra just outside the breaker zone at Stephenville Crossing. This defines significant inshore wave heights

- determine wave setup and runup at Stephenville Crossing

- combine components to establish flood levels

The following sections discuss each of these tasks.

(a) <u>Hindcast Waves</u>

The first step in calculating runup is to hindcast wave heights from storm events. The following formulae as adopted from the Shore Protection Manual (SPM) (9) were used

$$T_c = 61\ F^{0.667}/U_W^{0.41} \qquad (12.1)$$

where T_c = minimum time required (seconds) for maximum wave formation in a specified fetch

F = Fetch (m): maximum available fetch of 427 000 m

U_w = wind stress factor (km/h)

$$F_e = 0.0021\ U_w^{0.615}\quad T_a^{1.5} \qquad (12.2)$$

where F_e = effective fetch when actual duration is less than calculated duration (m)

T_a = actual duration time (seconds)

$$H_w = 0.000075\ U_w^{1.23}\ F_e^{0.5} \qquad (12.3)$$

where H_w = height of wave (m)

F_e = fetch (may be either the available value of 427 000 m or the effective value calculated in equation 12.2, depending on the duration).

Deep water wave heights were calculated for periods of high wind with directions ranging from approximately 200 deg to 300 deg during the 1953 - 1982 period of wind record.

Winds at Grindstone Island were employed since they were deemed to be more representative of over water conditions than the records at Stephenville Airport. A frequency analysis of the annual maxima yielded the following results.

Return Period (yr)	Significant Wave Height (m)	Peak Spectral Period (s)
20	13.8	16.5
35	15.0	17.2
100	16.7	17.7

(b) Offshore Wave Spectra

The wave spectra for the 20-, 35- and 100-year wave events have been estimated with the use of JONSWAP spectrum as follows:

$$S(f) = \frac{\alpha g^2}{(2\pi)^4 f^5} \exp\left[-\frac{5}{4}\left(\frac{f}{fp}\right)^{-4}\right] \cdot \gamma^{\exp\left[\frac{-(f-fp)^2}{2\lambda^2 \ fp^2}\right]} \tag{12.4}$$

where S(f) is the spectral energy density
f is the wave frequency (1/T)
fp is the peak frequency of the spectrum

and

$\alpha = 0.008$

$\gamma = 3.3$

$\lambda = 0.07, \ f < fp$

$\quad\ \ 0.09, \ f > fp$

This yields the distribution of offshore energy as a function of wave frequency (i.e., wave period). The distribution of energy as a function of wave direction was determined with the use of a power cosine spreading function as follows

$$\frac{E\theta_{jk}}{E\theta_{tot}} = \frac{\int_{\theta_j}^{\theta_k} \cos^n (\theta_i - \theta_p) \, d\theta_i}{\int_{-\pi/2}^{\pi/2} \cos^n (\theta_i - \theta_p) \, d\theta_i} \tag{12.5}$$

where $\frac{E\theta_{jk}}{E\theta_{tot}}$ = ratio of energy between θj and θk and total energy

$\theta j, \ \theta k$ = wave direction

θp = central wave direction

n = power of cosine function

The JONSWAP spectrum for the 20-yr event is shown in Figure 12.5(a) and the cosine spreading function with the power 5 is shown in Figure 12.5(b).

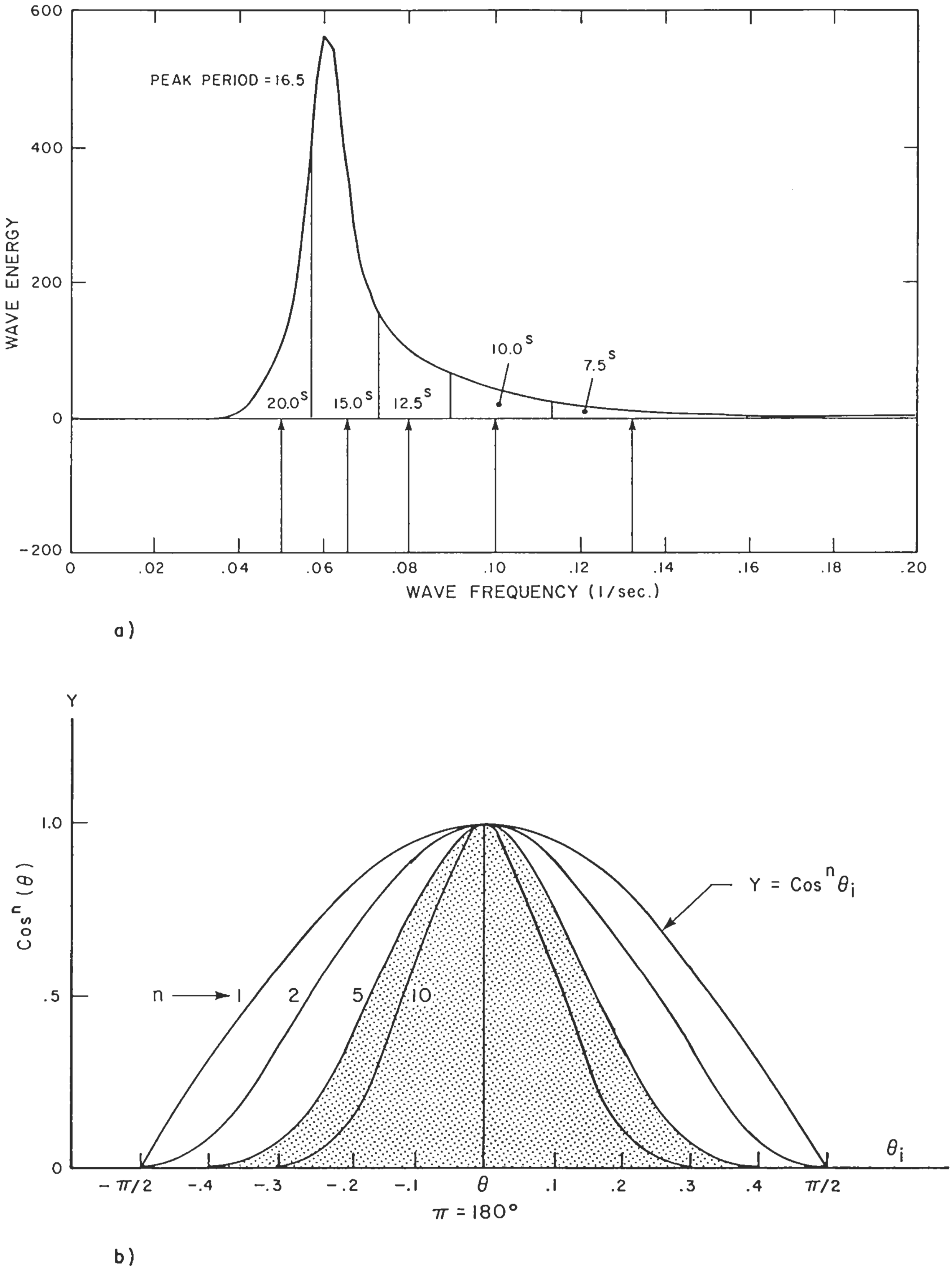


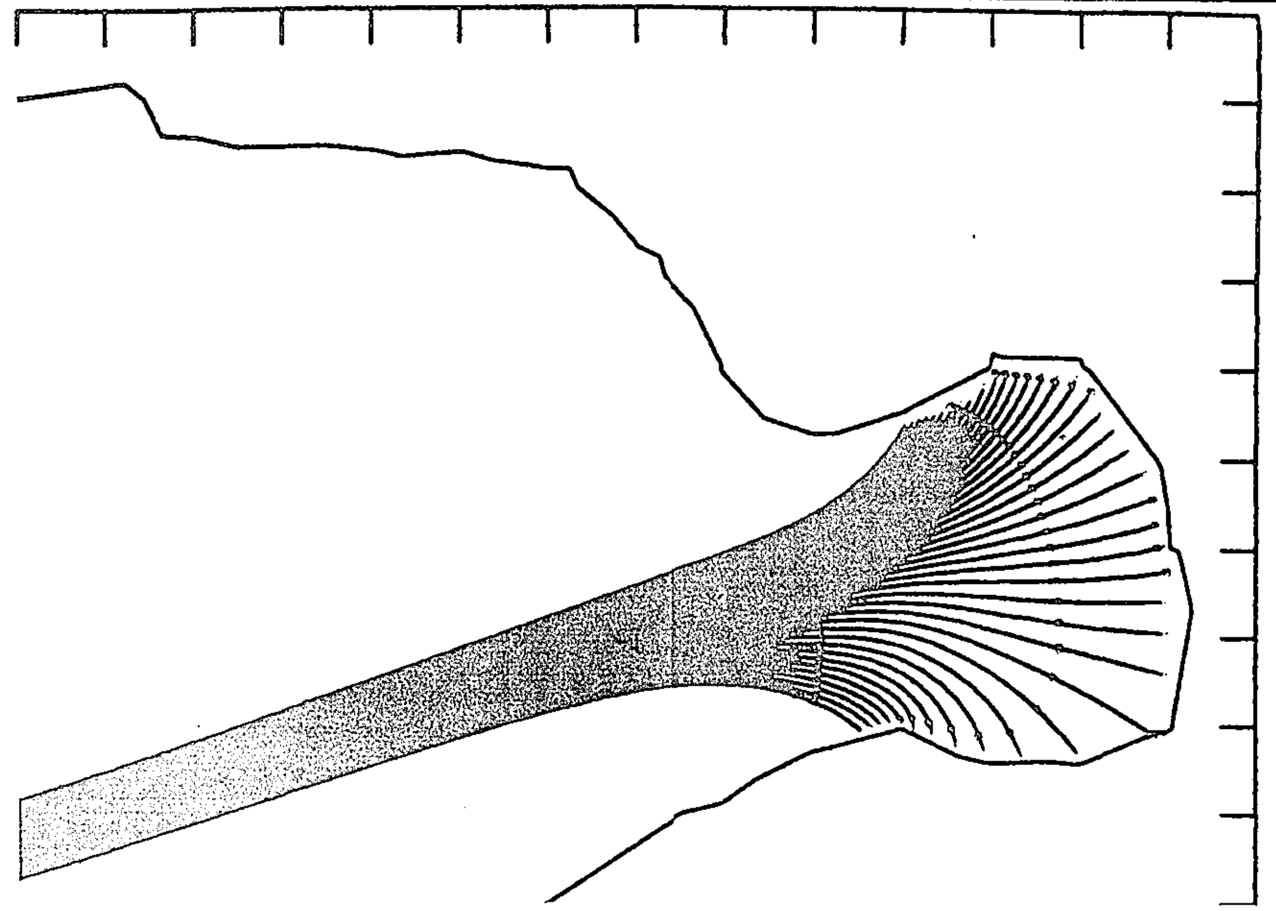
FIG. 12.5

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

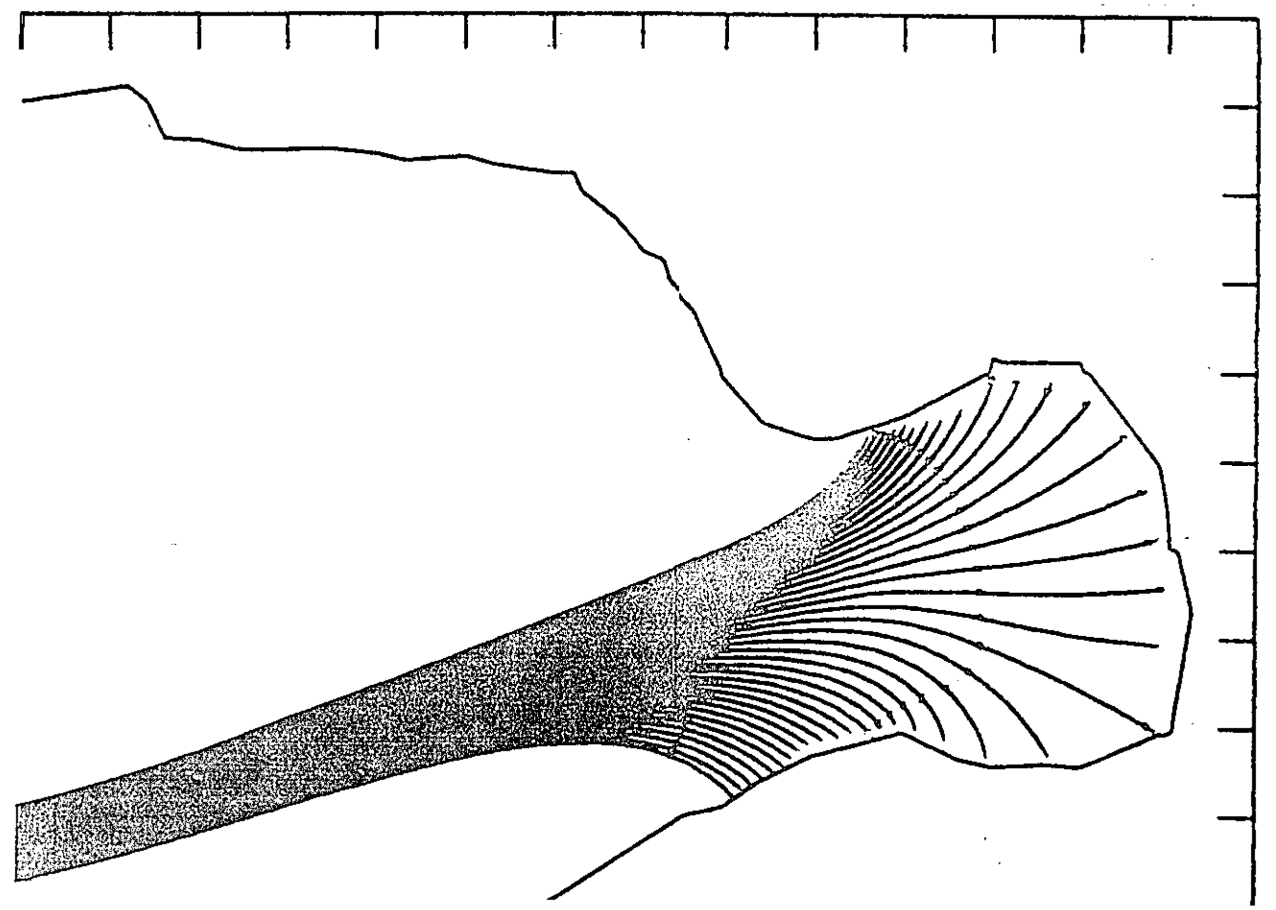
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

WAVE ENERGY SPECTRUM - OFFSHORE ST. GEORGES BAY

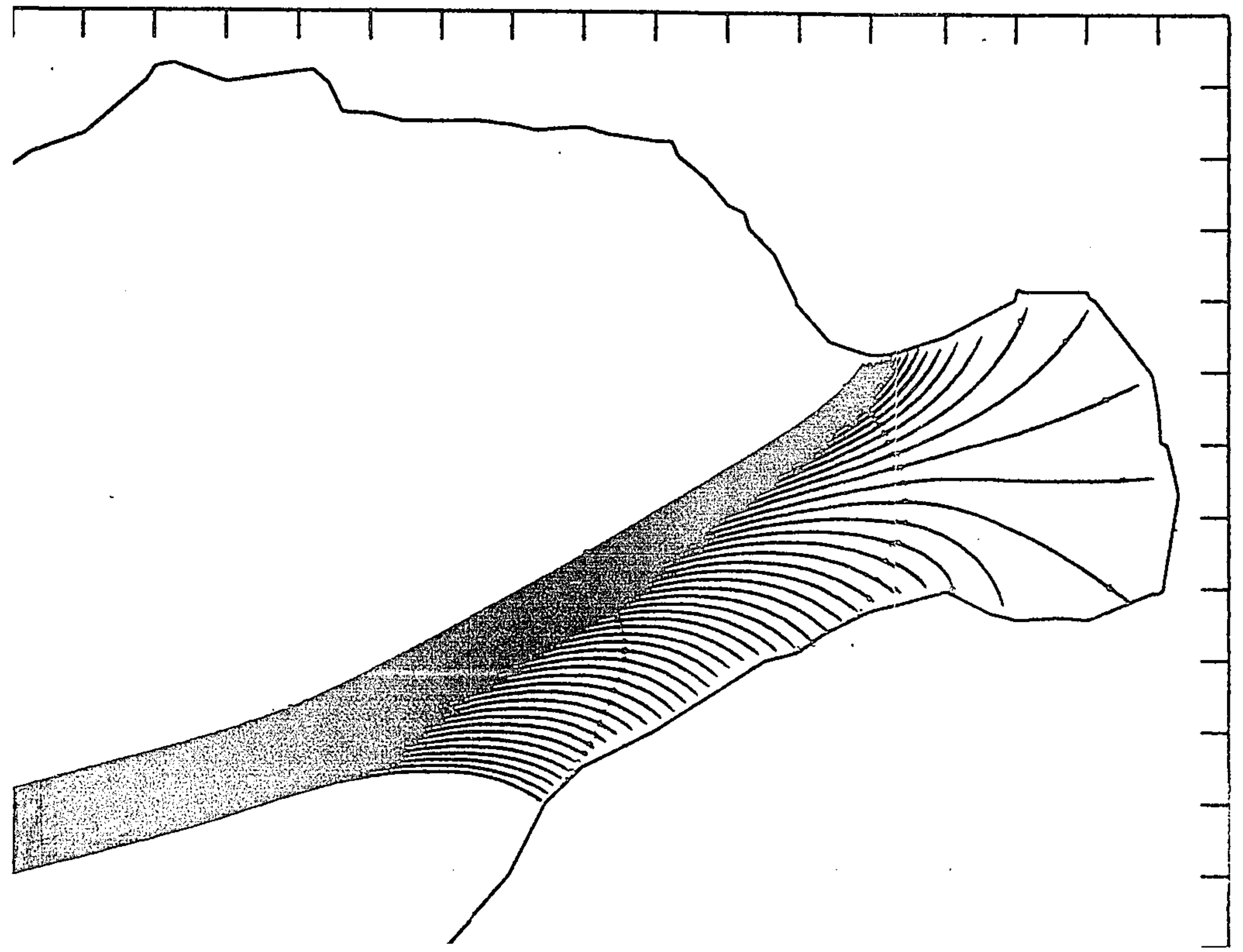
ACRES



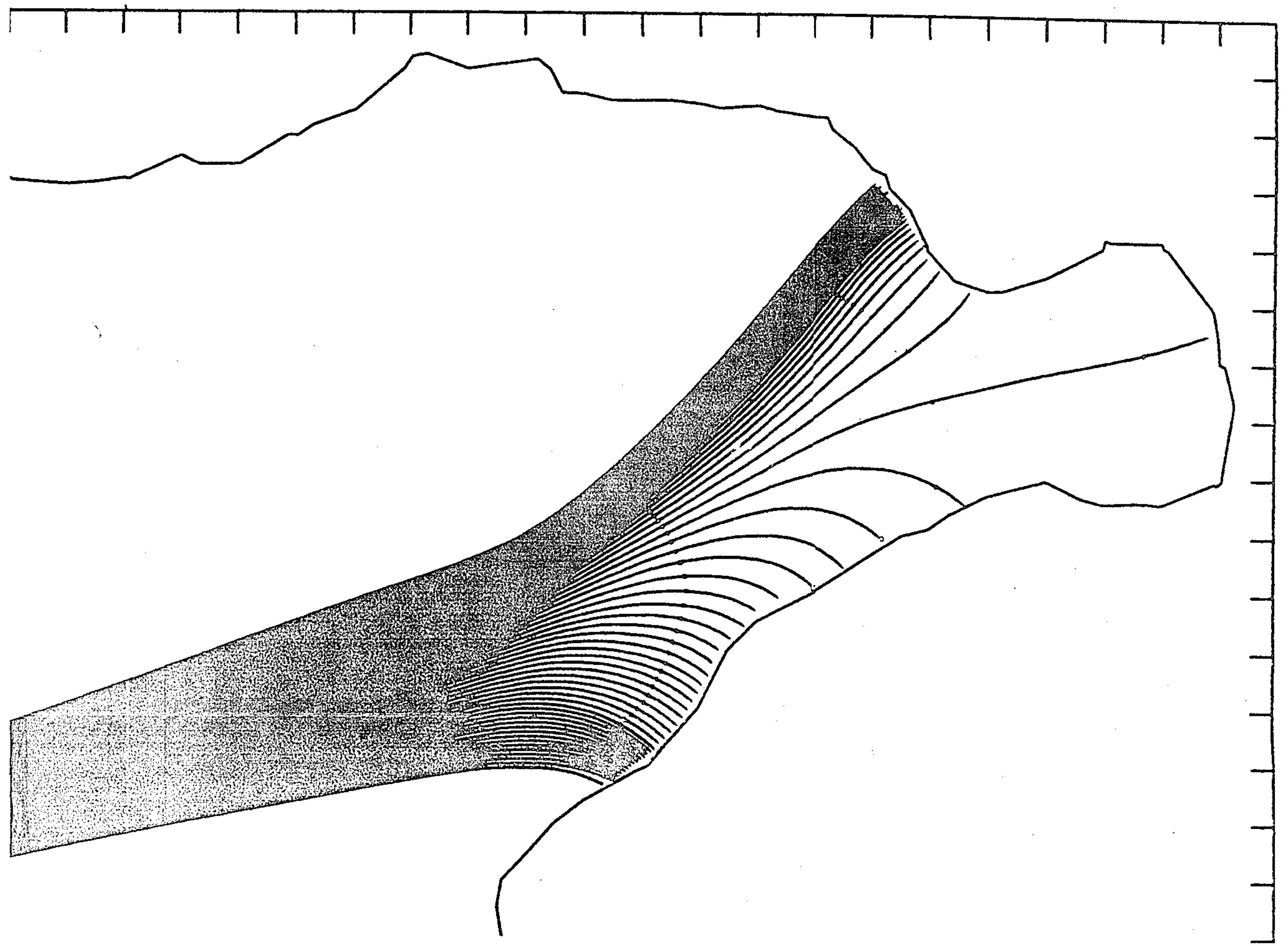
WAVE PERIOD = 7.5 SECONDS



WAVE PERIOD = 10 SECONDS



WAVE PERIOD = 12.5 SECONDS



WAVE PERIOD = 15 SECONDS

FIG.12.6

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY- STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

WAVE REFRACTION IN ST. GEORGES BAY

ACRES

(c) Wave Transformation

A refraction analysis for St. George's Bay was carried out to determine the transformation of the offshore spectrum to the inshore location near Stephenville Crossing.

The refraction grid was made up of computational elements 1500 m square over the entire Rothesay Bay (42 x 35 elements). A series of refraction cases were studied, encompassing wave periods of 7.5, 10.0, 12.5, 15.0 and 20.0 seconds. Forward tracking cases were conducted for visualization purposes and backward tracking cases were conducted to derive the directional transformation matrix.

Forward tracking cases are illustrated in Figure 12.6 for a variety of wave periods. As can be seen in Figure 12.6, the wave periods in excess of 10 seconds do not penetrate the bay in a significant manner.

In the backward tracking cases, a series of points of interest along the Stephenville Crossing area were established and rays were radiated in the offshore direction in 1 deg increments. The analysis demonstrated that at Stephenville Crossing only incoming waves confined to a very narrow directional band will reach the beach. The offshore wave direction limits are as follows:

Wave Period (seconds)	Offshore Directional Range (degrees)	Offshore Limits[1] (degrees)
7.5	29	-24 to +5
10.0	38	-32 to +6
12.5	18	-6 to +12
15.0	10	-6 to +4
20.0	0	0

[1]Zero degrees refers to central wave direction

Using a Cosine spreading function of power 5 described earlier, the proportion of the energy spectrum that will be transmitted will be as follows.

Spectrum Band (second)	Proportion Transmitted
7.5	0.440
10.0	0.540
12.5	0.310
15.0	0.058
20.0	0.000

The transmission coefficients were applied to the distribution of energy [as illustrated in Figure 12.5(a)] and the resultant inshore spectrum for the more extreme events had an energy only about 12% of that in the offshore spectrum. Since the wave height is proportional to the square root of the energy, the resultant inshore significant wave heights were reduced to:

Return Period (yr)	Wave Height (m)	Inshore Peak Period (s)
20	5.24	9.0
35	5.40	9.2
100	5.68	9.5

(d) Wave Setup and Runup

For the offshore profiles representative of Stephenville conditions, and for wave characteristics as quoted above, the setup of the water level due to the random wave field is about 4% of the wave height (SPM Figure 3.53 (9)), and the runup is about 0.40 times the wave height (SPM Section 7-II, Figures 7.8 to 7.20 (9)). This yields the following results.

Return Period	Setup	Runup	Surge	Total
20	0.21	2.10	0.28	2.59
35	0.22	2.16	0.29	2.67
100	0.23	2.27	0.31	2.81

Also shown above is the surge component (which is assumed to be fully additive to the other atmospheric-induced level changes) and the total superelevation caused by atmospheric affects.

Note that although the runup estimation is subject to considerable uncertainty, the estimate of the 1:35-year runup recurrence (2.16 m) agrees with reported observations from the largest historic storm event (1951).

12.2 - 1:20 and 1:100 Year Flood Levels, Rothesay Bay Side

The probability density function (pdf) of high water levels of the tides at Stephenville Crossing is given in Figure 12.2 and the frequency curve of wave components (wave setup, surge and runup) is shown in Figure 12.7. These probability relationships were combined using the following relationship.

$$\text{Prob } (WL \geq L) = \int_{i=1}^{NT} (\phi_{ti} dt) \; . \; \text{Prob } [W \; (L - T_i)] \qquad (12.6)$$

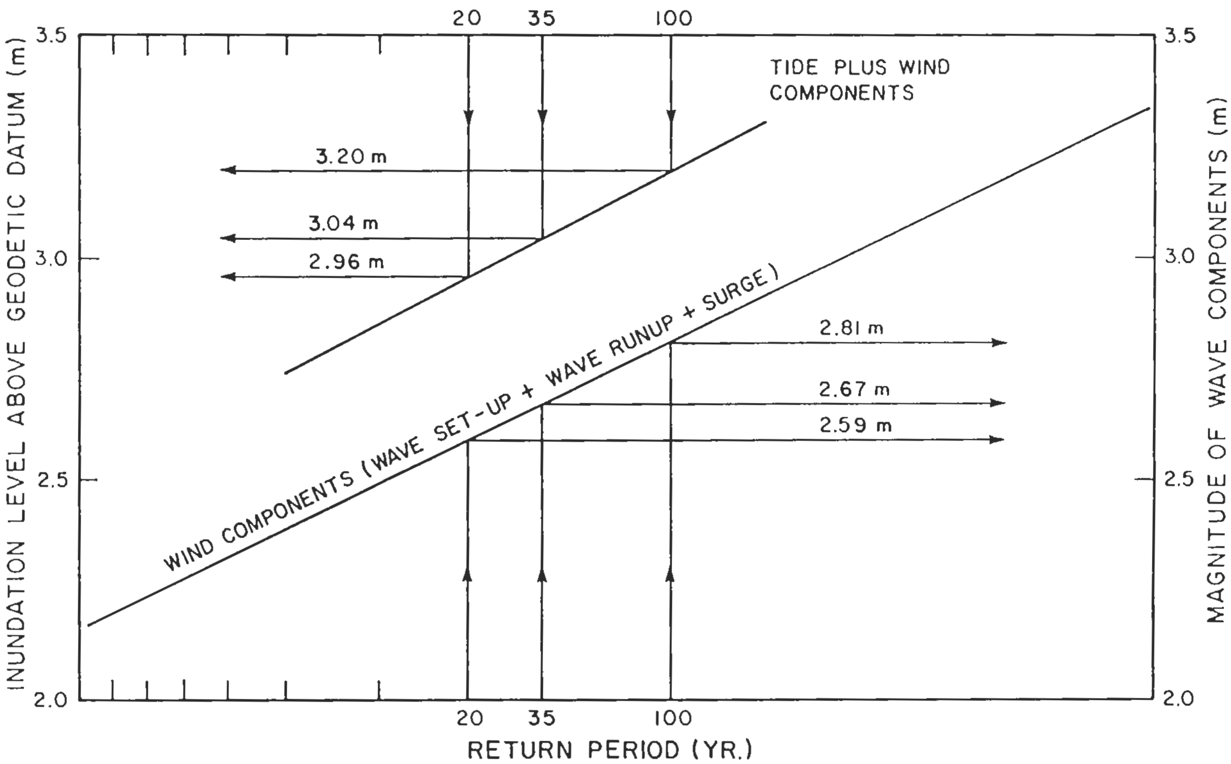


FIG. 12.7

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

FREQUENCY CURVES FOR WAVE COMPONENTS & TOTAL INUNDATION LEVELS

ACRES

where WL = water level

L = trial value of WL

i = segment of tidal pdf

N_T = number of segments in tidal pdf

T_i = magnitude of tide high water level for segment i

$(\phi_{ti} dt)$ - probability of tide being in segment i

Prob $[W \geq (L-T_i)]$ - cumulative probability of wave components exceeding the value $(L-T_i)$

The above convolution equation was applied for several values of WL to derive the combined frequency relationship. Figure 12.7 also shows this curve. The resulting levels are

Return Period (yr)	Total Water Level (m)
20	2.96
35	3.04
100	3.20

The observed maximum inundation level in the worst storm on record was about 3.0 to 3.1 m which is consistent with the magnitude of the 35-yr event.

The water levels indicated above include the runup component. In certain locations, particularly in the low area just north of the Main Gut, the highway embankment would be overtopped in these events. It is assumed that water from Rothesay Bay would run over/through the embankment, and that the land behind would be flooded to the level of Rothesay Bay.

This Rothesay Bay level was determined in exactly the same way as the total water level, i.e. by convolving the tidal pdf with the frequency curve of surge/setup, but excluding the runup component. The resulting water levels are

Return Period (yr)	Water Level (m)
1:20	1.0
1:100	1.1

12.3 - Flooding in St. George's River

Flood levels in St. George's River are influenced by three major factors

- tide levels in Rothesay Bay;
- superelevation of water levels in Rothesay Bay due to surge and wave setup during major wind events;
- high fresh water inflows.

The water level in St. George's River is dependent on the conveyance of the Main Gut Channel and on the water level in Rothesay Bay. In order to determine the 1:20 and 1:100 year water levels in St. George's River, i.e. inside the Main Gut, a routing model was developed and calibrated for this project to assess the interchange of water through the Main Gut.

The model was a standard hydraulic routing model using an interactive water balance approach. Channel conveyance parameters for the Gut as well as a volume-elevation curve for St. George's River are given data. The tide level outside and the fresh water inflow are read in for each 1 hour time step. The model estimates the level in St. George's River based on the previous water level and the fresh water inflow. The flow through the channel is calculated as a function of the water surface slope and the channel conveyance. The level in St.

George's River is then re-estimated. The procedure is repeated using the previous estimate of Gut flow until the difference between 2 successive estimates of volume in St. George's River is less than 0.01 Mm^3 (equivalent to about 0.5 mm in elevation). Details on the model are provided in Appendix 1, at the end of this volume.

Various combinations of events were considered to determine the most significant event.

Data from two events, each about 2 to 3 days in length, were used, one to calibrate and one to validate the model. The principal requirement for selecting suitable events was a clear response in the water level record to tides. A secondary consideration was wind strength and direction. The period chosen for calibration was December 13-14, 1986, when winds dropped to less than 10 kts from the east before rising to a relatively light 15 kts from the south west. A second period, December 4-6, 1986, was chosen for validation because it represents a period with more extreme tides, and because the water levels in St. George's River show a clear response (e.g. no obstruction by ice). Some strong winds did occur during this period, first from the southeast and then from the west.

12.3.1 - Model Calibration

The model was calibrated using the December 1986 event which was monitored in the field program. The main parameter varied in the calibration procedure was the conveyance of the Main Gut. The fresh water inflow and tide levels in Rothesay Bay were obtained as described below. The variables affecting the channel conveyance were adjusted until predicted levels of St. George's River matched the measured values.

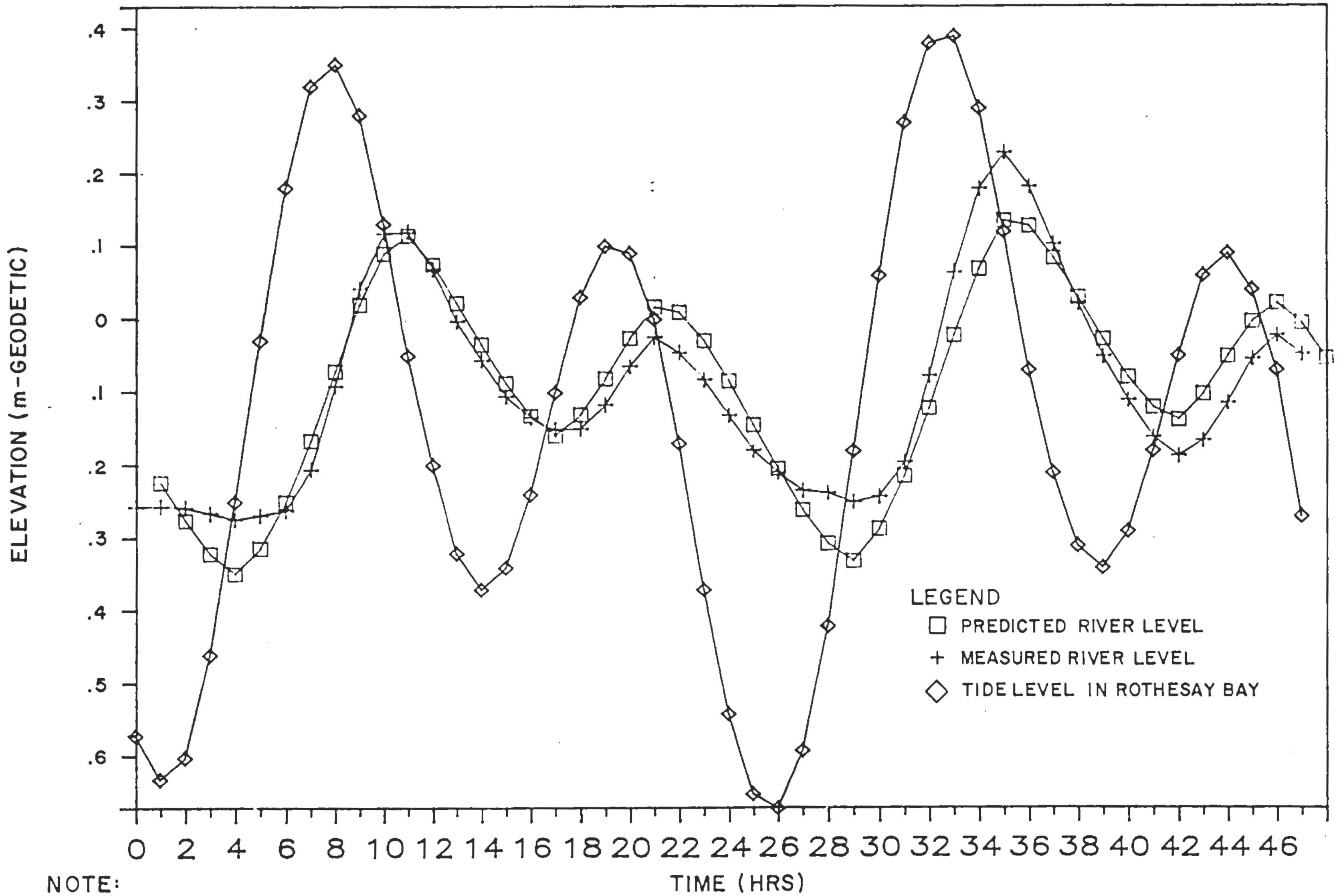
The fresh water flow into St. George's River was determined by direct proration of measured flows for Harry's River at the Water Survey of Canada gauge. The St. George's River catchment is drained by three major watercourses, Harry's River, Bottom Brook, and Southwest Brook, with drainage areas of 820 km^2, 215 km^2, and 580 km^2, respectively. The drainage area upstream of the flow gauge on Harry's River is 640 km^2. Assuming the three basins have similar runoff characteristics, the total fresh water flow into St. George's River is estimated by proration to be 2.52 times the measured flow on Harry's River.

In the absence of measured water levels outside the Gut, due to the loss of the tide gauge, predicted tide levels were obtained from CHS. These levels were adjusted to account for barometric pressure changes during the period. The model predicted water levels in St. George's River. These were then compared with measured levels from the field program. The measured and predicted levels are given in Figure 12.8 and Table 12.2.

The sensitivity of the results to the physical variables affecting conveyance is discussed in Appendix 1.

12.3.2 - <u>Model Validation</u>

After the model was calibrated, it was run for another period in December 1986 in which an event was measured. As can be seen in Figure 12.9 and Table 12.3, both measured and calculated values generally agree except for 2 time periods-hours 16-24 and 33-42. During both these periods, at least part of the discrepancy may be attributable to surge effects. During hours 16-24, strong southeasterly winds (35-55 km/h) with an 8-to 10-h duration may have resulted in reduction in the static water level in Rothesay Bay thus lowering St. George's River levels.



NOTE:

CALIBRATION PERIOD
DEC. 13-14, 1986

FIG. 12.8

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ST. GEORGES RIVER-LEVELS FOR ROUTING MODEL CALIBRATION

TABLE 12.2

WATER LEVELS - CALIBRATION EVENT

Period (h)	Tide Level (m-geo)	Predicted River Level (m-geo)	Measured River Level (m-geo)
0	0.39	0.722	0.690
1	0.33	0.672	0.690
2	0.36	0.629	0.689
3	0.49	0.601	0.682
4	0.70	0.635	0.674
5	0.91	0.696	0.679
6	1.11	0.777	0.686
7	1.25	0.869	0.740
8	1.28	0.957	0.850
9	1.21	1.025	0.980
10	1.06	1.049	1.052
11	0.89	1.011	1.054
12	0.74	0.960	1.005
13	0.63	0.905	0.936
14	0.58	0.854	0.884
15	0.61	0.810	0.836
16	0.71	0.784	0.808
17	0.84	0.812	0.792
18	0.97	0.860	0.794
19	1.03	0.913	0.826
20	1.03	0.954	0.876
21	0.94	0.948	0.915
22	0.77	0.910	0.894
23	0.58	0.857	0.859
24	0.42	0.799	0.811
25	0.31	0.741	0.765
26	0.29	0.687	0.736
27	0.37	0.642	0.712
28	0.53	0.619	0.709
29	0.76	0.661	0.696
30	1.00	0.731	0.704
31	1.20	0.821	0.749
32	1.31	0.917	0.864
33	1.32	1.005	1.001
34	1.22	1.069	1.112
35	1.05	1.062	1.159
36	0.87	1.019	1.114
37	0.73	0.967	1.038
38	0.64	0.912	0.959
39	0.61	0.862	0.889
40	0.66	0.822	0.832
41	0.76	0.806	0.782
42	0.89	0.839	0.757
43	1.00	0.889	0.778
44	1.03	0.935	0.828
45	0.98	0.959	0.886
46	0.87	0.933	0.917
47	0.68	0.887	0.892

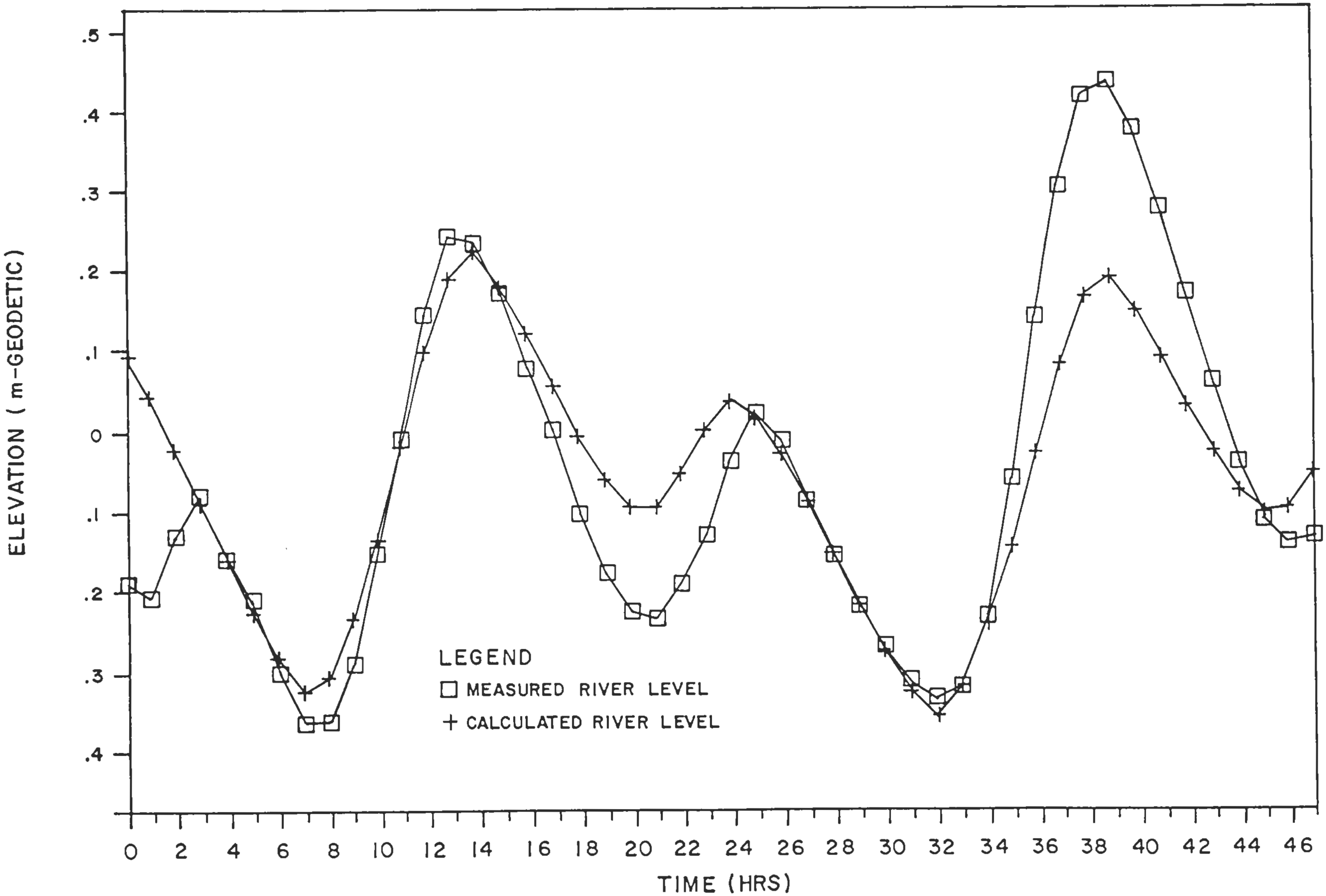


FIG. 12.9

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ST. GEORGES RIVER - MEASURED AND CALCULATED LEVELS

ACRES

TABLE 12.3

WATER LEVELS - VALIDATION EVENT

Period (h)	Tide Level (m-geo)	Predicted River Level (m-geo)	Measured River Level (m-geo)
0	0.96	1.067	0.781
1	0.75	1.015	0.764
2	0.48	0.949	0.842
3	0.25	0.880	0.893
4	0.10	0.811	0.812
5	0.05	0.747	0.762
6	0.13	0.690	0.671
7	0.36	0.648	0.608
8	0.67	0.666	0.609
9	0.98	0.738	0.681
10	1.25	0.837	0.817
11	1.44	0.953	0.963
12	1.51	1.069	1.114
13	1.42	1.161	1.212
14	1.22	1.195	1.205
15	0.99	1.151	1.142
16	0.79	1.092	1.048
17	0.64	1.028	0.973
18	0.56	0.966	0.867
19	0.60	0.913	0.796
20	0.72	0.877	0.746
21	0.87	0.878	0.737
22	0.98	0.921	0.780
23	1.06	0.972	0.841
24	1.05	1.010	0.934
25	0.93	0.990	0.994
26	0.71	0.943	0.960
27	0.46	0.884	0.884
28	0.26	0.820	0.814
29	0.13	0.757	0.753
30	0.11	0.698	0.703
31	0.21	0.648	0.661
32	0.44	0.618	0.639
33	0.72	0.657	0.654
34	1.00	0.732	0.738
35	1.24	0.830	0.910
36	1.41	0.943	1.108
37	1.46	1.053	1.271
38	1.36	1.137	1.387
39	1.17	1.162	1.403
40	0.96	1.119	1.343
41	0.78	1.062	1.246
42	0.65	1.002	1.138
43	0.59	0.945	1.029
44	0.64	0.897	0.930
45	0.76	0.869	0.857
46	0.88	0.875	0.829
47	0.98	0.919	0.837

In the second time period, when the measured water levels were higher than the calculated, strong westerly winds (35 km/hr) with an 18- to 20-h duration were recorded. These onshore winds likely caused higher water levels in Rothesay Bay than predicted thus increasing St. George's River levels.

12.4 - 1:20 and 1:100 Year Flood Levels, St. George's River Side

When any one of several conditions can produce a flood event, the probability of that event is approximately the sum of the individual probabilities, assuming the events are independent and those individual probabilities are small. The three major flood producing events in Stephenville Crossing are tides, surge/setup due to wind, and high freshwater inflows. It was assumed that a high tide could coincide with either a surge or a freshwater flow event.

The procedure adopted was as follows.

1. Estimate 1:20 and 1:100 year surge/setup events, and 1:20 and 1:100 year freshwater flow events.

2. Combine each of these with an average condition for the other, to maintain approximately the same level of probability, i.e. combine 1:20 or 1:100 year surge/setup with average inflow conditions, and vice versa.

3. For each of these combinations, assume that the tide is high. Since the storm period generally exceeded a tidal cycle, it was assumed that a high tide could occur during any wind or flood event.

4. In addition, check the high inflow events in a moderate tide.

12.4.1 - Independence of Flood Producing Events

Tides are non-random events, independent of wind and flood events. It was assumed here that a high tide could occur with any wind or flood event. Of more concern was the possible correlation of high freshwater flows and surge/setup events. For this study, they were assumed to be independent, for the reasons discussed below.

High surge/set-up is caused by very strong westerly winds; high fresh water floods are caused by heavy precipitation and/or snowmelt. Both of these are caused by weather systems moving across the Gulf of St. Lawrence, but the types of systems producing heavy precipitation are different from those producing strong westerly winds. The typical condition causing an extended period of heavy rain is a slow moving low pressure frontal system; winds are usually northeast to southeast during the rain. As the low passes, the winds move to the west, but they are not unusually strong. In contrast, the systems producing the strongest westerly winds are fast-moving, with less rain. Brief periods of heavy rain can occur during strong westerly winds in other more unusual meteorological circumstances, but the rain is not likely to be sufficient to result in a flood.

Similarly, periods of low pressures, producing higher water levels, are not associated with the strongest westerly winds. The strongest winds tend to occur in rising or high pressure conditions.

Freshwater flow events might also be caused by snowmelt in the late winter or early spring. Although warm temperatures might occur with westerly winds late in the spring, causing a snowmelt event, the frequency of strong winds decreases substantially in the spring.

The same conditions that bring heavy rain also bring an extended period of warm temperatures; cool temperatures return with the westerly winds. In January, for example, winds from the SSW to W occur nearly 30 percent of the time, with an average speed of nearly 26 km/h. In May, the frequency of winds from those directions is similar (32 percent) but the average windspeed is 15 km/h.

As confirmation of these assumptions, the record of actual extreme events was examined. During the 18 year period of the gauge record, the maximum daily flow never coincided with, nor was followed by, very strong westerly winds, as Table 12.4 shows. The highest westerly winds following a flood event were about 35-55 km/h.

Because of this lack of correlation based on physical considerations and observations, storm surge and runoff induced flooding were considered to be independent at the return periods of interest.

12.4.2 - Selection of 1:20 and 1:100 Surge/Setup Event

The 1:20 and 1:100 year superelevations of water levels in St. George's Bay due to surge and setup during wind events were obtained as described in Section 12.2, and are as follows.

Return Period	Surge (m)	Setup (m)	Total Superelevation[1]
1:20	0.28	0.21	0.49
1.100	0.31	0.23	0.54

[1] Since surge and set-up arise from the same wind event, their levels are added.

TABLE 12.4

WIND SPEED AND DIRECTION AT TIME OF ANNUAL MAXIMUM DAILY FLOW

Y/M/D	MAXIMUM DAILY DISCHARGE (m^3/s)		WIND SPEED (km/h)	AVERAGE DAILY WIND SPEED (km/h)	DIRECTION
1969 May 21	333	am	19-35	25	SW-NW
		pm	10-40		
1970 May 27	184	am	0-26	11	S-SW
		pm	0-36		
1971 Apr 16	121	am	16-48	16	E
		pm	0-7		
1972 May 16	228	am	0-16	12	NE-SE
		pm	8-23		
1973 Aug 3	260	am	10-32	20	SW
		pm	16-32		
1974 Nov 1	121	am	0-30	20	S-W
		pm	19-32		
1975 Dec 23	261	am	5-35	19	E-S
		pm	20-26		
1976 Nov 1	166	am	15-26	24	SE-SW
		pm	16-37		
1977 Dec 27	207	am	11-28	20	W-NW
		pm	9-28		
1978 June 9	186	am	15	11	S-SW
		pm	10		
1979 Mar 8	165	am	0-20	20	NE
		pm	28-33		
1980 Nov 6	219	am	22-46	33	W
		pm	28-37		
1981 Apr 30	131	am	22-30	25	E-SE
		pm	20-30		
1982 Apr 30	267	am	30-46	30	E-SE
		pm	10-30		
1983 Jan 13	192	am	10-20	16	E-S
		pm	10-25		
1984 May 15	137	am	15-28	18	SE-S
		pm	20		
1985 June 7	279	am	25	26	E-SE
		pm	25		
1986 Apr 17	142	am	n/a	n/a	n/a
		pm	n/a		

A nominal superelevation of 0.10 m due to surge/setup was used for the average surge condition. It is probably a slightly conservative assumption; an examination of the residuals for the September 1967 period of record for the St. George's gauge indicated that they are more often negative than positive during periods of moderate westerly winds (probably due to high barometric pressure). Occasionally, however, the residual reached about +0.2 m for several hours during westerly winds, so a nominal value of 0.10 m was used.

12.4.3 - Selection of 1:20 and 1:100 Year Freshwater Flow Event

Flood events on Harry's River are frequently very flashy and can cause increased levels in St. George's River. The flashiness of the floods can be seen in Figure 12.10 which illustrates two annual hydrographs.

A frequency analysis of flood volumes for the 1968-1985 period of record was carried out for 1-, 2- and 3-day duration floods on the Harry's River. The procedure used was as follows.

1. 1972 and 1973 hydrographs were plotted. A base flow of 10 m^3/s was estimated by inspection of the hydrographs.

2. The daily flow records were examined and the maximum 1 day, 2 day and 3 day inflows in each year were extracted from the record. These were tabulated to provide 3 annual series of maximum events, one for each duration with the base flow of 10 m^3/s-day subtracted. These annual series are presented in Table 12.5. From a frequency plot, 1:20 and 1:100 year return periods were obtained as follows:

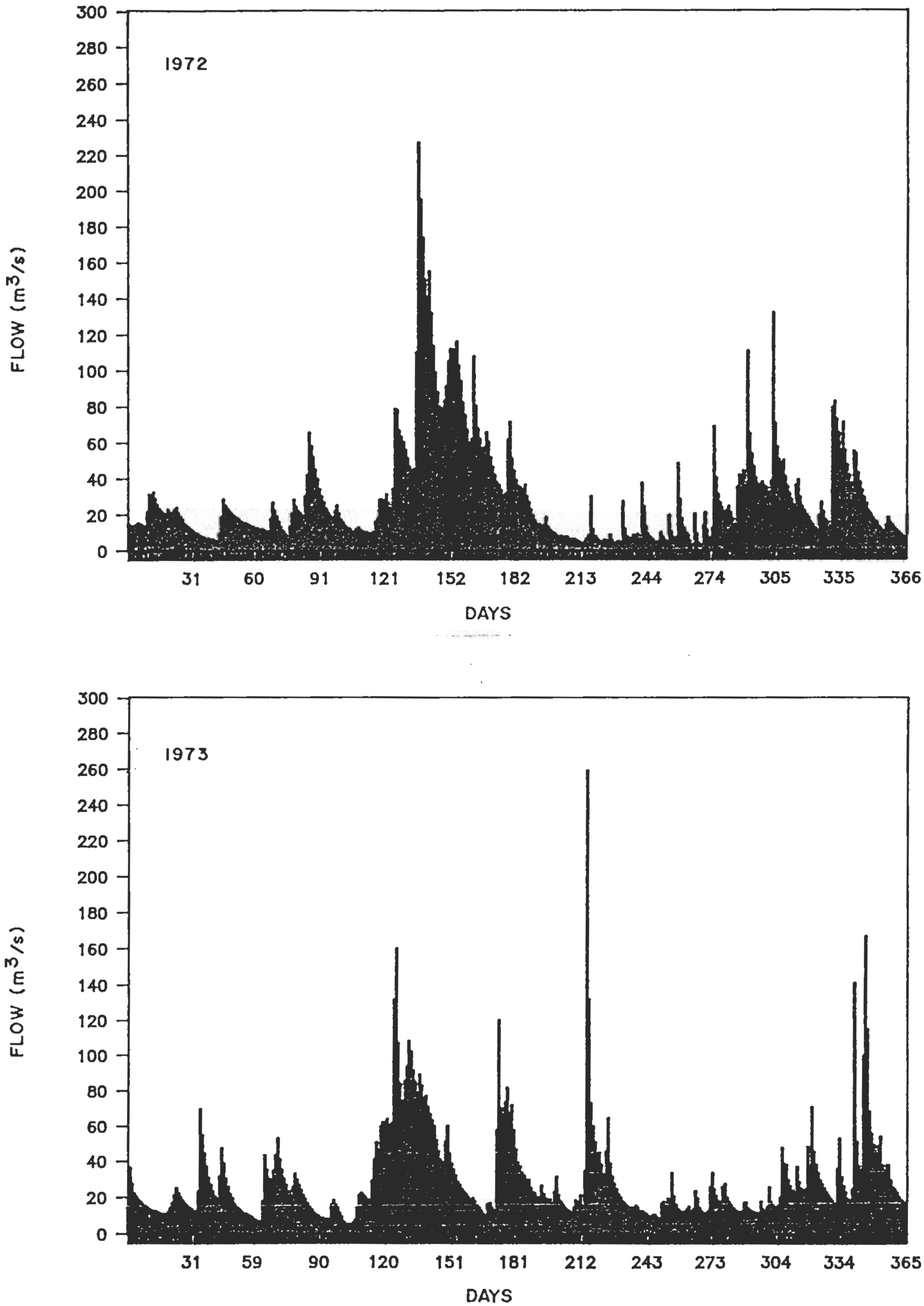


FIG. 12.10

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

HARRY'S RIVER HYDROGRAPHS

ACRES

TABLE 12.5

ANNUAL SERIES OF MAXIMUM 1, 2, AND 3 DAY INFLOWS

Year	Flow (m^3/s-days) 1 day	2 day	3 day
1968	132	232	307
1969	323	555	704
1970	174	266	334
1971	111	205	306
1972	218	404	569
1973	250	373	437
1974	111	199	286
1975	251	368	440
1976	156	274	344
1977	197	283	351
1978	176	324	479
1979	155	275	378
1980	209	328	398
1981	121	237	348
1982	257	487	686
1983	182	314	395
1984	127	247	331
1985	269	388	471

Event	Flow (m^3/s-days) 1 day	2 day	3 day
1:20 year	300	520	695
1:100	390	700	1030

(These are in line with results obtained in the Harry's River analysis.)

3. A base flow of 10 m^3/s-day was added to each estimate.

4. Total inflow for all 3 drainage areas was obtained by direct proration using the drainage area ratio of 2.52.

5. The 1-, 2-, and 3-day events were routed to determine which produces the highest water level in St. George's Bay. Both triangular and rectangular shapes were used. The 1-day triangular shaped hydrograph, with the peak staggered about 2 hours before the tidal peak, produced the highest levels in St. George's River.

The 1:20 and 1:100 year 1-day triangular hydrographs are given in Table 12.6.

12.4.4 - <u>Selection of Tide Sequences</u>

A typical monthly tide sequence is shown in Figure 12.11. The highest levels in St. George's River from surge/setup events occur when the surge/setup coincides with the highest tide. For the freshwater flow events, however, the highest tides are followed by very low tides, as shown in Figure 12.11. The head differential created through the Gut allows a large outflow. The freshwater flows were thus also combined with a low tidal range (Figure 12.11) to allow for the possibility that a low tidal range might produce higher water levels than a high tidal range when combined with a fresh water flow event.

TABLE 12.6

INFLOW HYDROGRAPHS FOR ST. GEORGE'S RIVER, 1:20 AND 1:100 YEAR EVENTS

1:100 year event		1:20 year event	
Hour	Qfresh Mm^3/hr	Hour	Qfresh Mm^3/hr
1	0.098	1	0.098
2	0.518	2	0.700
3	1.168	3	1.080
4	1.754	4	1.460
5	2.341	5	1.840
6	2.927	6	2.220
7	3.514	7	2.600
8	4.101	8	2.980
9	4.687	9	3.360
10	5.274	10	3.740
11	5.86	11	4.120
12	6.447	12	4.500
13	7.033	13	4.880
14	7.224	14	5.260
15	6.637	15	4.880
16	6.05	16	4.500
17	5.464	17	4.120
18	4.877	18	3.740
19	4.291	19	3.360
20	3.704	20	2.980
21	3.118	21	2.600
22	2.531	22	2.220
23	1.944	23	1.840
24	1.358	24	1.460
25	0.771	25	1.080
26	0.098	26	0.700

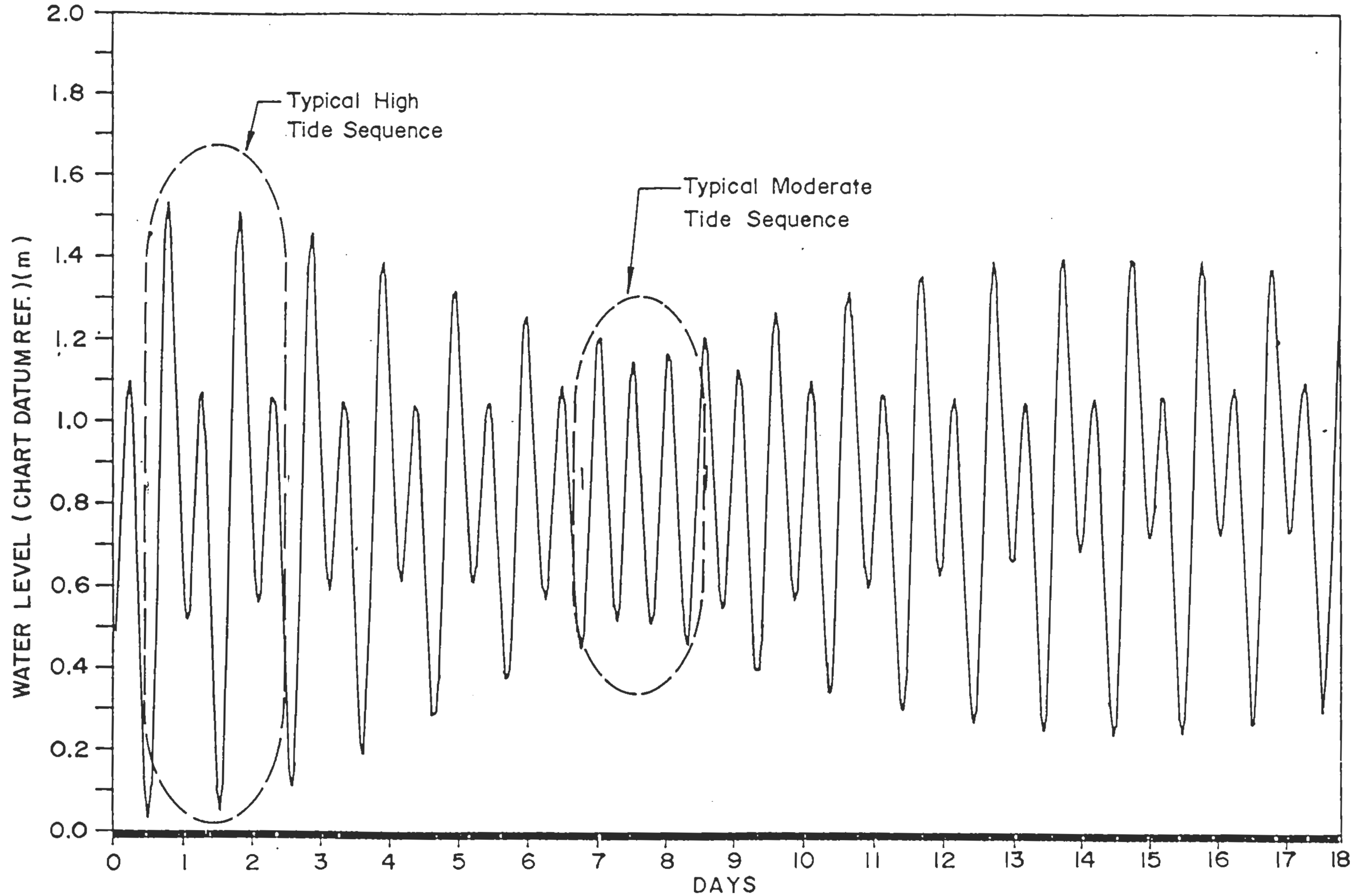


FIG.12.11

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

MONTHLY TIDAL SEQUENCE-ROTHESAY BAY

ACRES

12.4.5 - Results

The levels in St. George's River resulting from various combinations of events are shown in Table 12.7. This table shows that the highest levels result from large freshwater flow events. For these high inflows, the levels are only slightly higher in the high tidal range than in the low tidal range.

Because surge/setup and freshwater flood events are independent in extreme events (e.g. 1:20, 1:100), the probabilities of a given water level are additive. The two probability curves were therefore plotted and added to produce the combined curve. These plots are shown in Figure 12.12. As this figure shows, the water levels for the 1:20 and 1:100 year return periods are dominated entirely by the fresh water flood.

12.5 - 1:20 and 1:100 Year Flood Levels at Stephenville Crossing

Table 12.8 shows the recommended 1:20 and 1:100 year water levels in Stephenville Crossing resulting from the wave/surge/tide event in Rothesay Bay described in Sections 12.1 and 12.2, and the freshwater flood/surge/tide event in St. George's River described in Sections 12.3 and 12.4. The maximum water level in Rothesay Bay including the wave runup component is also given.

Figure 12.13 shows the extent of flooding due to the 1:100 year event (the 1:20 year levels would appear very similar on a map of this scale; both 1:20 and 1:100 year flood lines are plotted on the accompanying large scale mapping. The maximum water levels on the St. George's River side, are shown, and the approximate areas which would be affected by overtopping are also indicated.

The floodlines produced in this study do not apply if the highway embankment fails.

TABLE 12.7

WATER LEVELS IN ST. GEORGE'S RIVER RESULTING FROM VARIOUS COMBINATIONS OF EVENTS

Event Combination			
Surge	Qfresh	Tide	Resulting River Level (Geodetic) (m)
1:20	average	high	0.82
1:100	average	high	0.87
average	1:20	high	1.11
		moderate	1.06
average	1:100	high	1.05
		moderate	1.48

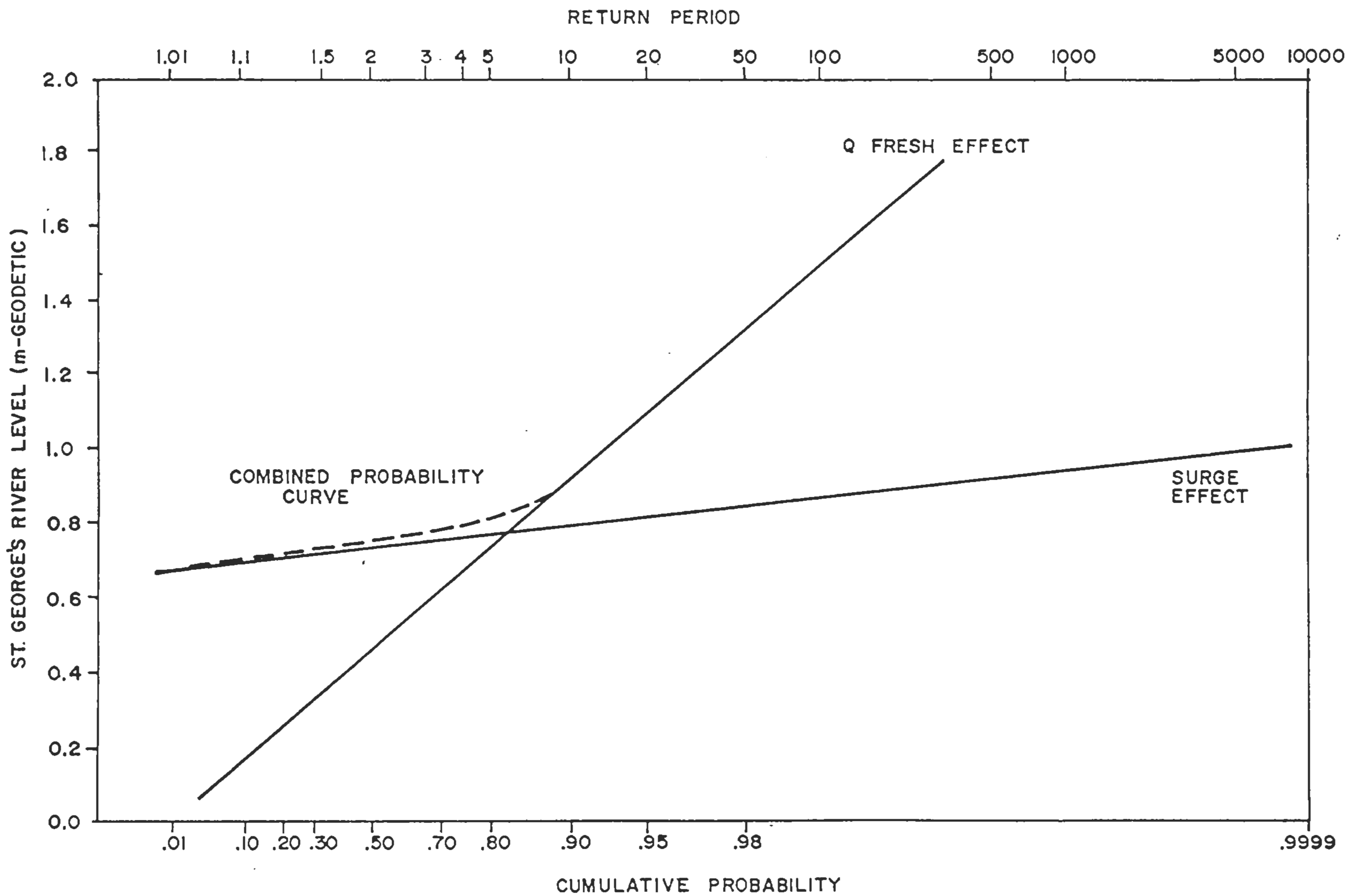


FIG. 12.12

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

PROBABILITY CURVES DUE TO SURGE/SET-UP EVENTS AND Q FRESH FLOW

ACRES

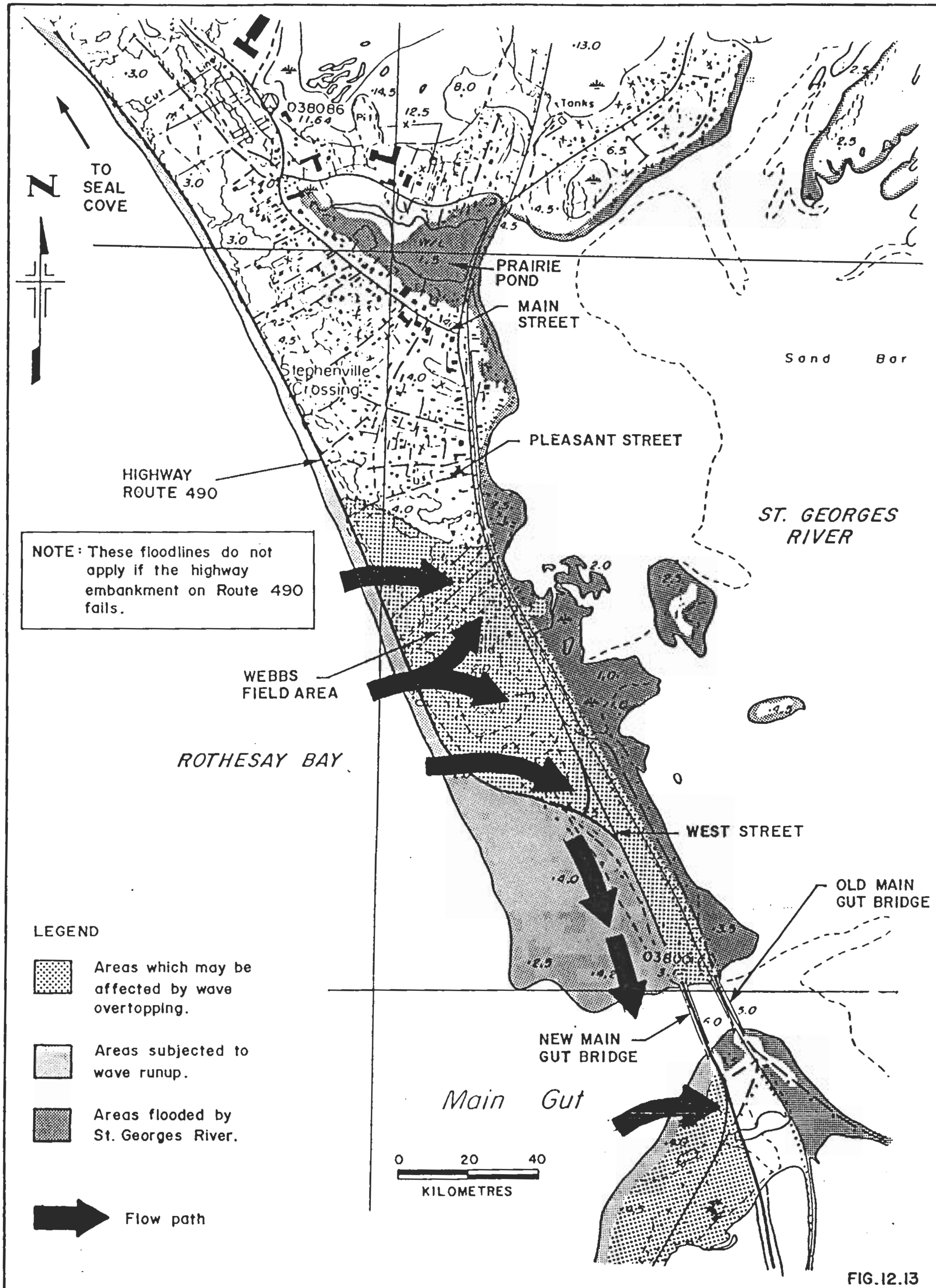


FIG.12.13

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

APPROXIMATE EXTENT OF FLOODING 1:100 YEAR EVENT

ACRES

As this figure shows, the beach and highway embankment on the Rothesay Bay side are above elev 3.0 m in the most heavily populated areas, and can be expected to prevent flooding from extreme events in Rothesay Bay. The two areas which will experience overtopping are a small area in the northwest corner of the study area and a larger area between the Webb's Field and the Gut. In these areas, the waves will overtop and likely breakthrough the highway embankment. The area will be flooded to and, in addition, depressions will be filled and a flow path will likely develop from the overtopped areas towards the Gut. Thus, the most extensive flooding in the area will result from wave breakthrough of the beach.

TABLE 12.8

ESTIMATED 1:20 AND 1:100 YEAR FLOOD LEVELS IN STEPHENVILLE CROSSING

Return Period	Water Level, St. George's River (m)	Static Water Level, Rothesay Bay (m)	Maximum Water Level* Rothesay Bay (m)
1:20	1.1	0.49	2.96
1:100	1.5	0.39	3.20

*Static water level plus runup component.

It may be noted that the surface area of St. George's River is about 20 km^2 and extends well beyond the boundaries of the study area. Similar water levels are likely to occur along much of the shoreline.

13 - REMEDIAL MEASURES

Several alternative remedial measures have been identified to reduce the flood hazard in the flood-prone areas.

The basic elements for a flood damage reduction plan can be classified into two categories:

1. The first category includes structural and non-structural measures to provide protection against potential flooding events. Examples of the measures are

   i) setting up a flood warning system which will provide advance warning of impending flooding so that appropriate precautions can be taken.
   ii) construction of a system of berms and/or dikes along flood prone areas to protect existing buildings against flooding.
   iii) floodproofing existing buildings in the flood risk zone.

   This category may also include measures that serve to mitigate against future flood damages or losses by reducing the potential for continued development in flood prone lands.

2. The second category includes measures that directly affect the flood characteristics by reducing or removing the factors that cause the flooding or ice accumulation problem. Examples of these types of measures include

   i) replacement of existing structures that may contribute or cause flooding, such as a culvert or bridge at which ice jams form.

   ii) channelization
   iii) ice retention structures (ice boom or ice dam).

A detailed analysis of the proposed remedial measures did not form part of this study. However, several possible alternatives were identified that could be considered in any future examination of remedial actions. The following sections describe the possible remedial measures for both Black Duck Siding and Stephenville Crossing. No evaluation of the technical or economic feasibility of these various options has been carried out.

13.1 - Black Duck Siding

The most severe flooding in Black Duck Siding is caused by ice accumulation during freeze-up and ice jamming during break-up of the ice cover. As shown in Figures 9.2 and 9.3, the flood risk areas determined for Black Duck Siding are extensive and include most of the presently developed area. In identifying the possible remedial measures in this area, alleviating the effect of ice accumulations was considered to be most important. The remedial measures identified are discussed below.

Floodproofing/Building Restrictions

The existing buildings in the flood risk area could be floodproofed and any future development in the area confined to areas outside the flood prone area.

Construction of Berms or Dykes

The berms that have recently been constructed at Harry's River as part of the channel modifications provide some relief from flood damages by preventing the ice from flowing in over the land. However, flooding will still occur from high water levels since there is still a path for the water to flow around the upstream ends of the berms. An alternative is to extend the existing berms to run the length of the river bank from Hobbs farm to downstream of Tanglewood Ranch. To contain the flood levels, the berms would have to be built to an approximate elevation of 30 m (geodetic) at Hobbs farm and 24 m at Tanglewood.

Flooding at Dhoon Lodge has been due to ice jamming at the highway bridge and from high open water flows. The construction of a berm along the bank of the river down to the highway embankment would serve to confine flood water to the channel. It is anticipated that a berm 900 m in length would have to be constructed.

Trees as an Ice Barrier

Since much of the damage in the downstream reaches has been caused by moving ice, a second alternative is to plant trees along the river bank where clearing has taken place. These trees would serve to act as a barrier to moving ice, limiting the damage to that from overland flow of water. Further clearing of trees along the river bank should be kept to a minimum.

Flood Warning

The development of an effective flood forecasting system that could provide adequate warning of impending floods is also identified as an important step in reducing potential flood losses. The river ice conditions during the winter should be monitored along with the meteorological forecast (rainfall and temperature) and flows measured at the WSC gauge. Given warning of an impending flood, appropriate precautions could be taken such as elevation or removal of contents above expected flood levels, or evacuation of buildings in the flood prone areas.

Ice Retention Structures

The intent of an ice retention structure is to control the location of ice accumulation to an area where the impact of flooding is minimal. One potential location for such a structure is just upstream of Section 5+742, since there are no developed areas upstream which would be affected by future flooding. The structure would encourage the formation of a thermal ice cover upstream and reduce the volume of frazil ice passed downstream to the study reach. This could reduce the volume of ice available for ice accumulation at freeze-up. As well, ice that would normally be released to pass downstream during break-up should be delayed by the structure and possibly thaw in place.

Possible engineering options for an ice retention structure include weirs and booms or an ice retention dam. The choice or configuration of any particular structure was not evaluated as part of this study. The cost of installing such a retention structure would be substantially more than any likely flood control benefits.

13.2 - Stephenville Crossing

Elevated water levels at Stephenville Crossing can result from both high water levels in St. George's River and from combined high water levels and wave action on the Rothesay Bay side of the community. This flooding from the Rothesay Bay side causes much more extensive flooding than does St. George's River. Both structural and non-structural measures could be considered in order to alleviate both sources of flooding.

In neither case, however, is there a practical opportunity to modify the water level or wave action. The high water levels in St. George's River are caused by high tides and high freshwater flows. Modifying the conveyance of the Gut cannot have a significant beneficial effect on St. George's River water levels, and upstream storage is not a practical option, particularly in view of the limited benefits.

13.2.1 - Flood Forecasting

Potential damage could be reduced by flood forecasting, allowing residents to evacuate their properties and remove portable items. A forecasting program would require monitoring meteorological forecasts of temperature and rainfall and flows measured at the WSC gauge on Harry's River as well as prediction of possible ranges in local sea level.

Although flood forecasting can reduce losses by allowing evacuation, it should be noted that forecasting is an inexact art and some cost and inconvenience would result from unnecessary evacuations.

13.2.2 - Floodproofing/Land Use Restrictions

Protection of individual buildings could be a cost-effective solution in the case of flooding from St. George's River, because only a small part of the community is affected. In the case of flooding due to waves breaking through the beach on the Rothesay Bay side, a much larger proportion of the community is affected, and floodproofing would be a relatively expensive alternative.

In both cases, land use restrictions within the flood prone areas could prevent additional damages.

13.2.3 - Construction of Berms

St. George's River: A berm could be constructed to an elevation of about 1.6 m to protect against high water levels in St. George's River. In part of the study area, the railway embankment already appears to serve this purpose. A berm about 1 km long extending from the Prairie Pond area north of Main Street to a point along the railway line near Webb's Field would be required. Consideration would have to be given to drainage of water trapped behind the berm by controlled outlets. Properties on the islands would still be subject to flooding.

Rothesay Bay: The new highway is high enough over part of its length not to be overtopped by waves from Rothesay Bay. If the embankment were raised to an elevation of 3.2 m along the entire length of the beach (a maximum increase of about 0.5 m) all the buildings in Stephenville Crossing would be protected. The details of the construction of the existing embankment and of its suitability as a wave barrier were not assessed in this report; and the embankment and existing beach in front of the embankment should be examined to determine its stability under storm attack.

About 1 km of highway embankment would have to be raised from the intersection with West Street near the Main Gut Bridge to near the west end of Pleasant Street. A small section of the road to Seal Cove would also need to be raised.

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APPENDIX 1
ROUTING MODEL

APPENDIX 1

ROUTING MODEL

In order to predict water levels in St. George's River and subsequently the flood risk areas of the 1:20 and 1:100-year events, a routing model was developed. The model was calibrated using measured levels of St. George's River during the field program to obtain an estimate of the conveyance characteristics through the Main Gut. The effects of both high tide level and fresh water flow events were then assessed.

The model input includes hourly fresh water flows and tide levels and allows interactive input of number of hours to be simulated, gut dimensions, Manning's n and initial depth of the St. George's River. Mean sea level, 0.9 m referenced to chart datum, arbitrarily corresponds to a river storage of 1000 m^3. The results are not sensitive to this value.

Model Description

The model follows standard hydraulic routing procedures. The computational steps in the program are outlined below.

1. The number of hours of simulation, gut dimensions, Manning's n, and initial river level are input interactively.

2. The initial storage volume of St. George's River is calculated using the initial level.

3. Fresh water flow is added to the initial storage to estimate the storage at the end of the first hour.

4. The river level associated with this volume is then calculated. Using this new river level along with the tide level and Gut dimensions, the flow through the Gut is calculated using the following equation.

$$Q = \frac{A^{5/3} \quad S^{1/2}}{np2/3}$$

where Q = flow through the Gut, may be positive or negative
A = area of Gut
S = slope of water level through Gut
n = Manning's n for Gut
p = wetted perimeter.

The area and wetted perimeter are calculated using an average of inside and outside Gut depths, and the side slopes of the Gut. Both tide and river level are referenced to chart datum. The absolute depth and side slopes of the Gut are taken from a Government of Newfoundland and Labrador, Department of Highways cross-section. The hydraulic gradient is determined by subtracting the inside from the outside levels and dividing this difference by the Gut length. An initial value of Manning's n was estimated from the literature as 0.035.

5. The flow through the Gut is added to the river storage. The model adjusts Gut flow until the difference between calculated levels in successive iterations is less than 0.01.

6. A new tide level and fresh water flow is then read for the next time step. The initial storage for this time step is found by adding the fresh water flow to the final storage volume of the preceding time step. Steps 4 - 6 are then repeated.

7. Final output includes date of study, hourly fresh water flows and tide levels, river level, storage volume and Gut flow.

<u>Sensitivity Tests of Routing Model</u>

The following parameters were varied to determine the model's sensitivity; channel cross section, Manning's n, Gut length, fresh water flow and initial river level. These sensitivities were based on the predicted tide data without pressure correction, but several check runs indicated that results are similar to the corrected case.

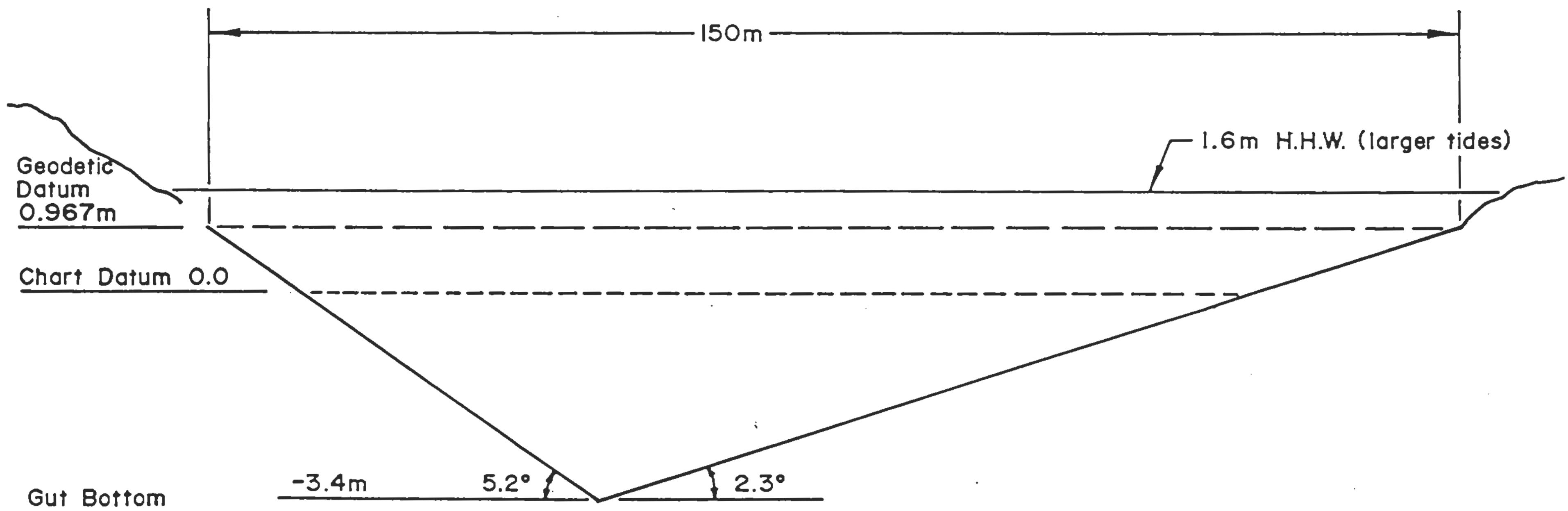
(a) <u>Channel Cross-Section</u>

The channel cross-section was initially based on a 1977 Department of Highways cross-section taken just outside the Gut. An initial value of depth was taken as 1.7 m below chart datum. This was subsequently narrowed and deepened to reflect the average Gut width, as shown in Figure 1.

The conveyance of the Gut is dependent on the area and wetted perimeter, and it was for this reason that the Gut depth was chosen as the cross sectional parameter to vary in the calibration. As seen in Figure 2, the river level is quite
sensitive to Gut depth.

(b) <u>Manning's n</u>

The value of Manning's n used in the model, 0.035, represents the initial value taken to describe the Gut's roughness. Figure 3 shows the variation in river level with Manning's n varied from 0.030 to 0.045. These values are within the range expected, based on experience.



Scale: Horiz. 1:750
Vert. 1:100

FIG. 1.

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY- STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

MAIN GUT SECTION

ACRES

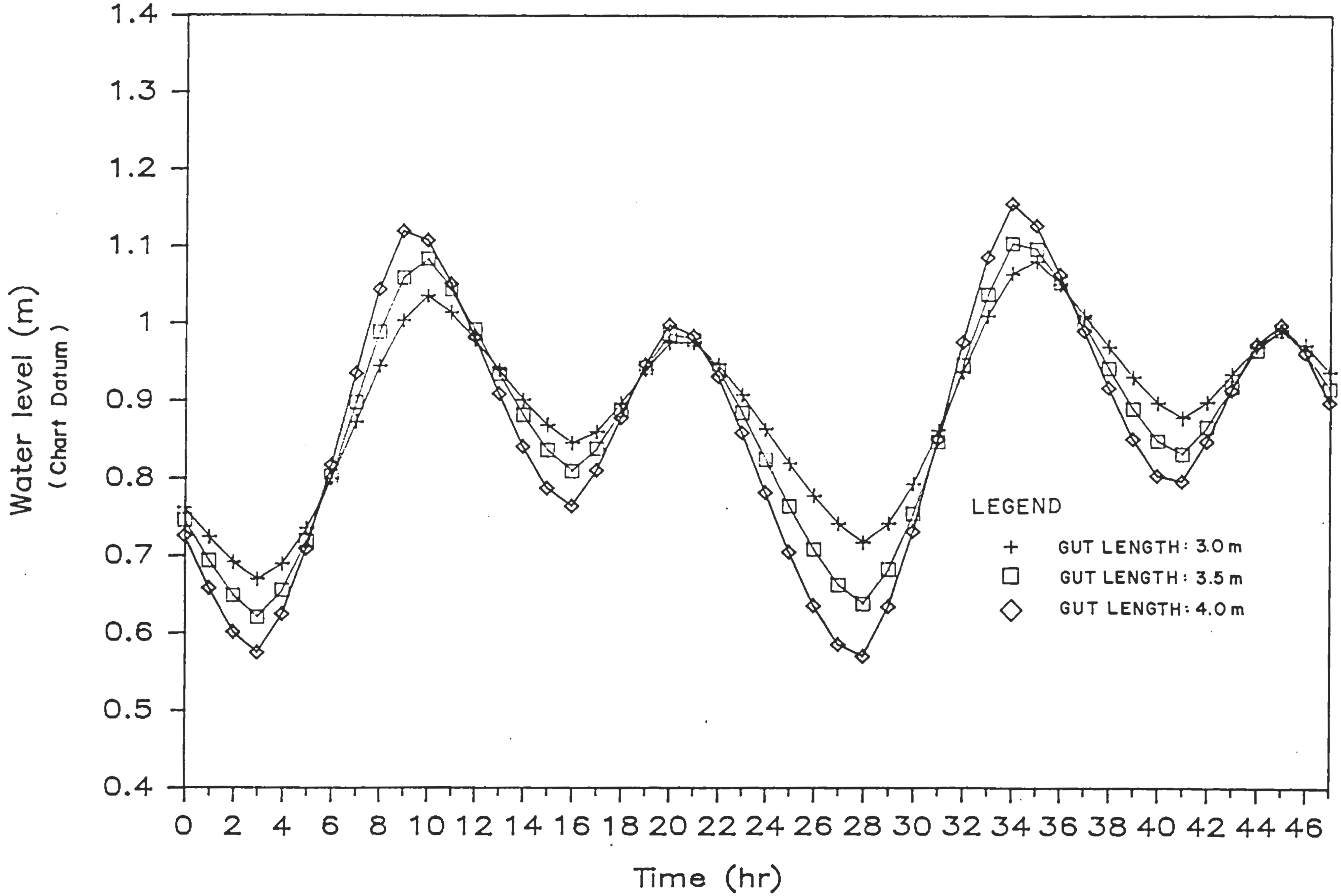


FIG. 2

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ST. GEORGES RIVER SENSITIVITY-GUT DEPTH

ACRES

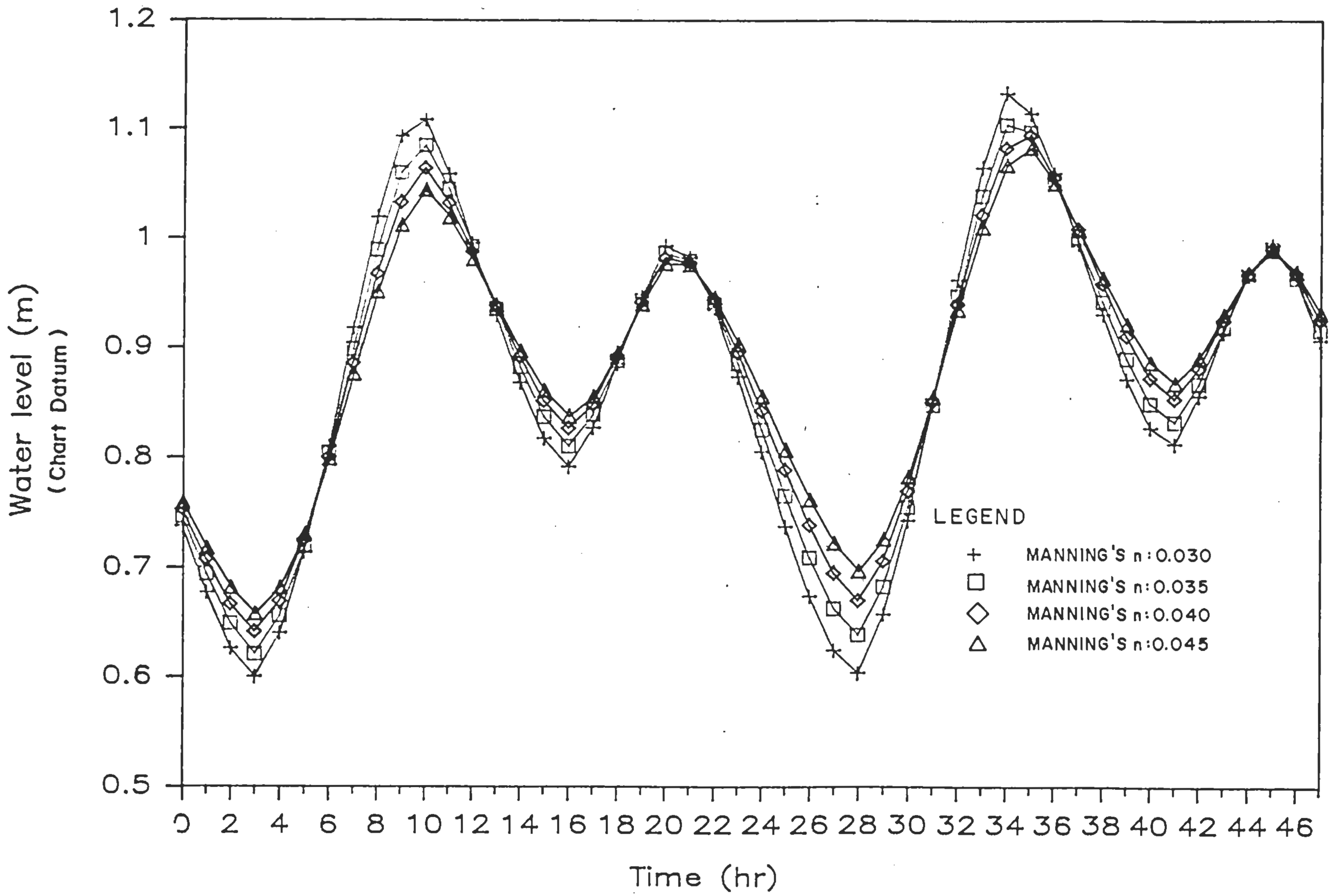


FIG. 3

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ST. GEORGES RIVER SENSITIVITY-MANNING'S n

ACRES

(c) Gut Length

The Gut length is used to calculate the water surface slope. The Gut length of 500 m was scaled from a 1:40 000 hydrographic map. Figure 4 shows the effect of variations in Gut length from 300 to 600 m. Figure 5 shows a plan view of the Gut, along with the various Gut lengths tested. The results using the selected length showed good agreement with observed data.

(d) Fresh Water Flow

Because the fresh water flow into St. George's River is prorated from the fresh water flows in Harry's River, any change in flow in Harry's River is assumed to result in a similar change in total flow into St. George's River. It was found that small changes in river flow, say from 25 m^3/s to 30 m^3/s had almost no effect on the water level in St. George's River.

Figure 6 shows the variation of levels with large variations in fresh water flows. A 15-m^3/s flow is half the average flow for December; 50 m^3/s is the highest monthly average flow for December; 100.0 m^3/s is the highest monthly average flow; and 200 m^3/s is a daily flow that occurred in December 1977, the month of the second flooding event in Stephenville Crossing. These flows were based on the Harry's River WSC gauge records. As seen in Figure 6, large fresh water flows have a significant effect on levels in St. George's River. No data are available for calibration or validation during periods of large fresh water flows.

After the design events were determined (1 day floods having return periods of 1:20 and 1:100 years, with a triangular unit hydrograph), the model was rerun for the design conditions to assess the sensitivity of the results to the

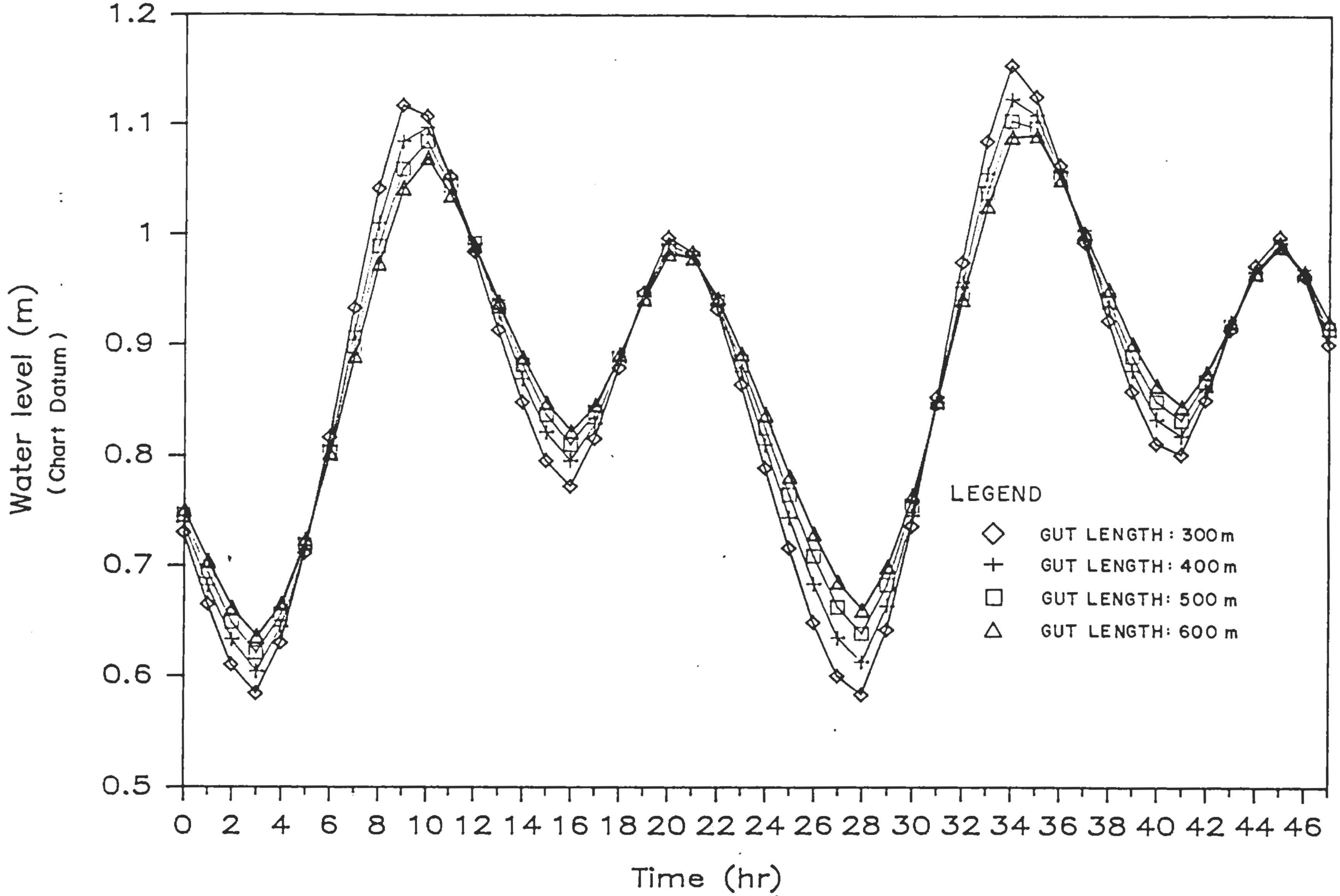


FIG. 4

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ST. GEORGES RIVER SENSITIVITY- GUT LENGTH

ACRES

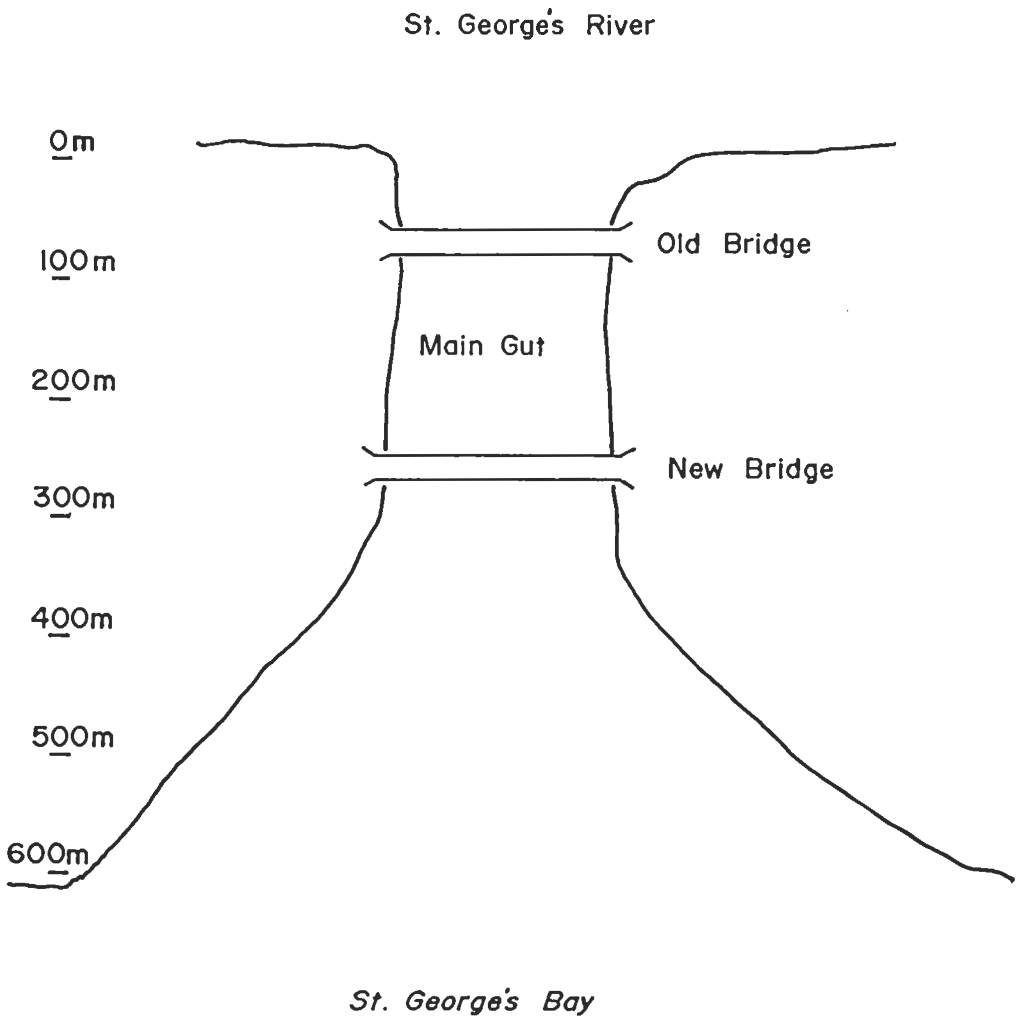


FIG. 5

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

MAIN GUT PLAN

ACRES

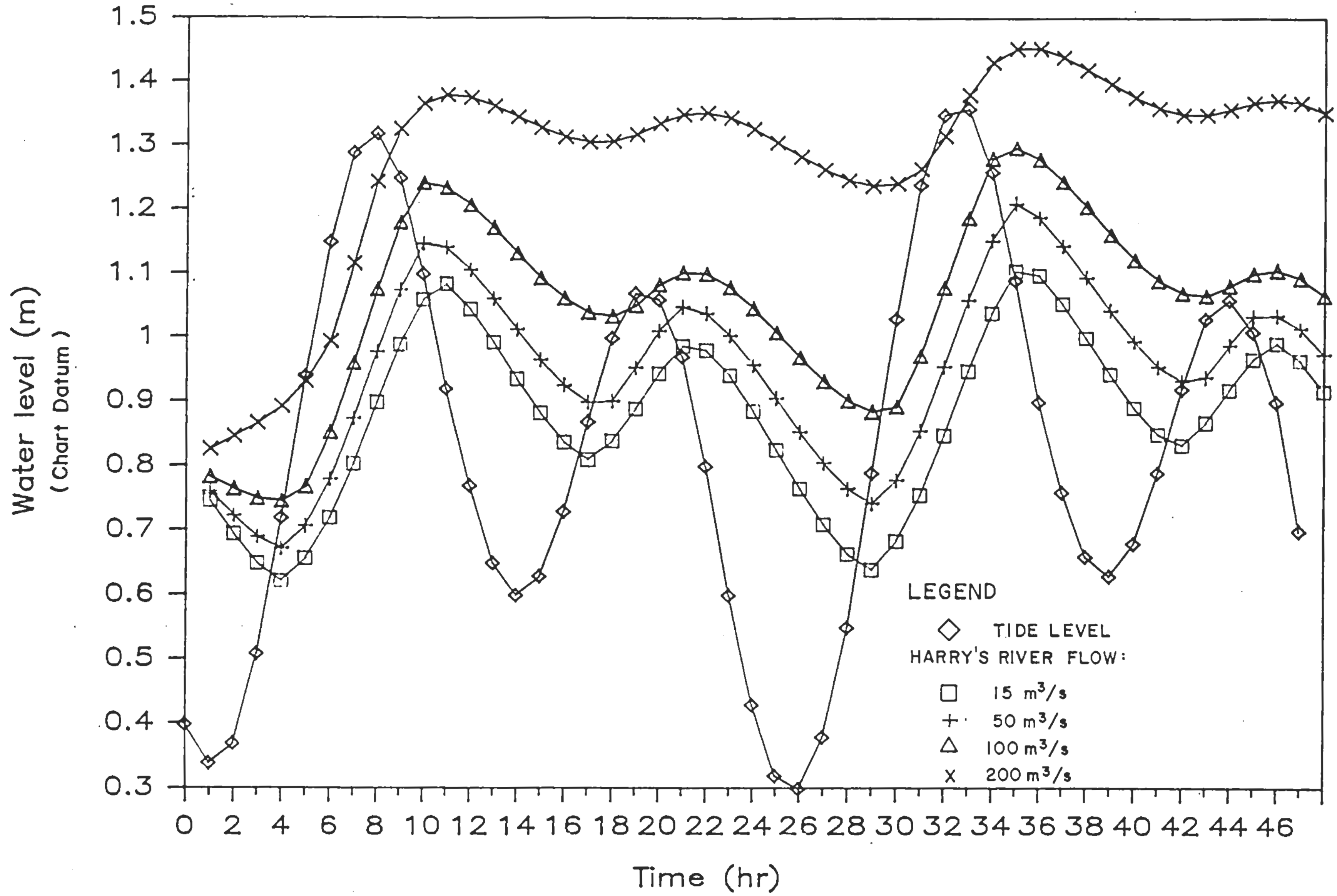


FIG. 6

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY-STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS
ST. GEORGES RIVER SENSITIVITY- HARRY'S RIVER FLOWS

ACRES

estimate of flood volume. A increase or decrease of 20 percent above the estimated 1:100 year flow in each hour of the simulation produced for the following results.

Inflow Condition	St. George's River Level (m - geodetic)
+20% (480 m^3/s)	1.80
Estimated 1:100 year daily flow (400 m^3/s)	1.50
-20% (320 m^3/s)	1.20

As expected, an increase in the estimate of the inflow flood has a significant effect on the levels in St. George's River. Until long-term flow records become available, however, the accuracy of these estimates cannot be assessed.

(e) Initial River Level

The initial river level must be estimated and, as seen in Figure 7, the value used significantly affects the results in the first 8 hours, with the effect decreasing with time. The lag between tide and river level shown in Figure 12.4 was used to determine the approximate difference between tide level and starting river level for any model run. If the model is started well in advance of the period for which the levels are required, the effect of inaccuracies in estimating initial water level should be negligible.

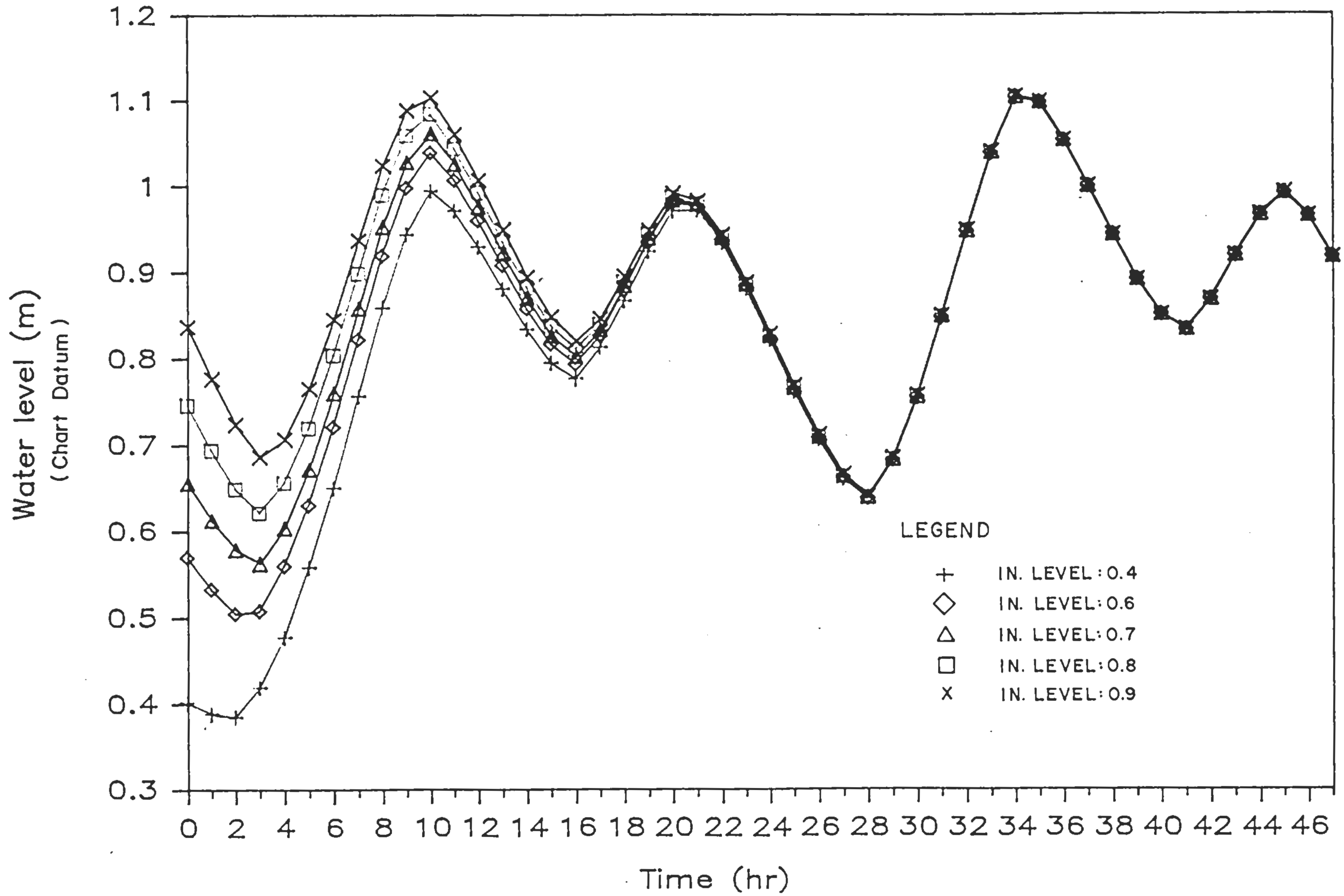


FIG. 7

CANADA-NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM
HYDROTECHNICAL STUDY - STEPHENVILLE CROSSING AND BLACK DUCK SIDING AREAS

ST. GEORGES RIVER SENSITIVITY - INITIAL RIVER LEVEL

ACRES